FLEXURAL BEHAVIOR OF CONCRETE-FILLED STEEL TUBES (CFSTs) USING RECYCLED AGGREGATE CONCRETE (RAC)

by

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Dedication

I dedicate this thesis to my family and friends for their endless love and encouragement...
Abstract

Concrete-filled steel tubes (CFSTs) can be considered as possible and promising alternative to conventional reinforced concrete members and steel structures since they have superior structural properties and economic advantages. In recent years, researchers have investigated the replacement of natural aggregate concrete (NAC) infill with recycled aggregate concrete (RAC) and utilizing the confinement provided by the steel tube in order to improve the mechanical properties of recycled aggregate concrete (RACFST). However, the research on RACFSTs has been limited to their behaviour under compression. The main goal of this research is to investigate the flexural response of RACFSTs experimentally and theoretically considering circular and rectangular cross-sections. For this purpose, a total of 12 circular and 8 rectangular CFST beams with different Diameter-to-thickness (D/t) and depth-to-thickness (h/t) ratios were tested under four-point bending. Concrete compressive strengths of 30 and 50 MPa and recycled coarse aggregate replacement percentages of 50 and 100% were used in the experimental investigation. The test results revealed promising outcomes on the feasibility of using RAC in CFST systems under flexure. The flexural behaviour of RACFSTs was found to be very similar to NACFSTs, and the change in the concrete compressive strength and the recycled coarse aggregate replacement percentage slightly affected the flexural behaviour of RACFSTs. In addition, the experimental flexural capacity of RACFST beams were compared to the theoretical nominal moments predicted by well-known design codes and methods including the AISC-LRFD, the Architectural Institute of Japan (AIJ), EuroCode4, the British Standard (BS) and Han’s method. The AIJ design code was the most accurate to predict the flexural capacity of RACFSTs followed by the EuroCode4 as they underestimated the flexural capacity of RACFST beams by an average of 5% and 12%, while the British Standard significantly underestimated the flexural capacity of the tested RACFST beams by an average of 34%.

**Keywords:** Concrete-filled steel tubes; Recycled aggregate concrete; Flexural behaviour; Design codes.
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List of Abbreviations

ξ  Confinement Factor

Ac  Area of Concrete

As  Area of Steel

b/t  Width to Thickness Ratio

CFST  Concrete-filled Steel Tube

D/t  Diameter to Thickness Ratio

f′ c  Compressive Strength of Concrete

f y  Yield Stress of Steel

h/t  Depth to Thickness Ratio

HSS  Hollow Steel Section

Mn  Nominal Flexural Capacity of RACFSTs

Mu  Ultimate Flexural Capacity of RACFSTs

My  Flexural Capacity at Steel Yield Stress of RACFSTs

NAC  Natural Aggregate Concrete

NACFST  Natural Aggregate Concrete-filled Steel Tube

R  Replacement Percentage of Recycled Coarse Aggregate

RAC  Recycled Aggregate Concrete

RCA  Recycled Coarse Aggregate

RACFST  Recycled Aggregate Concrete-filled Steel Tube
Chapter 1. Introduction

In this chapter, a general overview about Concrete-Filled Steel Tubes (CFSTs) and Recycled Aggregate Concrete (RAC) is provided. Then, the research significance, the problem statement, the research objectives as well as the research methodology are presented. At last, the general organization of this thesis is provided.

1.1. Overview

The growth in the world population together with the advancement in the construction industry have led to design and construction of high rise structures which requires high performance structural elements. The need to develop such high performance structural elements to go in line with the evolution in the construction industry has led to the use of composite members such as CFSTs. CFSTs were used in the industry since the beginning of the twentieth century as columns and bridge piers due to the many advantages of the combined steel tube and concrete infill. Nowadays, the applications of CFSTs have expanded to cover their use as main girders in bridges and buildings [3-5].

On the other hand, the depletion of the natural resources and the continuous deterioration of the environmental conditions have increased the need of sustainable development in many engineering disciplines. Civil engineering has a major impact on the depletion of the natural resources and the deterioration of the environmental problems as the construction sector is responsible for almost 33% of the world total CO₂ emission and buildings contribute up to 40% of the global energy consumption. Moreover, the construction sector uses concrete as a main building material and concrete contains about 55% to 80% of natural course aggregates by volume which means natural aggregates have a huge influence on the construction sector and the sustainability of structures. Therefore, this effect of the construction sector on the environmental degradation and resources depletion has motivated many researches to focus on the reuse of waste concrete as an alternative for natural aggregates in recent years. In addition to that, natural disasters (earthquakes and hurricanes) and demolishing of old structures to be replaced by new ones have resulted in a huge amount of waste concrete. All of this have encouraged the use of waste concrete as recycled aggregate all around the world. The reuse and recycling of waste concrete can contribute to achieve sustainable development in civil engineering, saving of natural
resources and reduce the solid waste at landfill sites. The recycled waste concrete is usually referred to as Recycled Aggregate Concrete (RAC) as it is used in place of the natural aggregate in the concrete mix. However, the use of the RAC in the industry is still questionable by many due to its weak properties compared to natural aggregate concrete [3, 6-8].

From all of the above, it can be concluded that the construction industry is in urgent need to develop high structural performance members with limited impact on the environmental and the natural resources. Unfortunately, the applications of RAC in the construction industry have been limited to low performance structures due to their weak properties as mentioned before. This problem can be solved using RAC as an infill concrete for the CFSTs as the confinement provided by the steel tube will help to overcome the disadvantages of RAC. Therefore, the use of RAC as infill concrete instead of NAC can be proposed as a very practical structural application with many economic and environmental advantages. This research will focus on the flexural behaviour of recycled aggregate concrete filled steel tubes (RACFSTs) compared to natural aggregate concrete filled steel tubes (NACFSTs) and to check whether the use of RACFSTs as flexural members is applicable or not.

1.2. Research Significance and Problem Statement

Although the use of CFSTs as composite structural members has taken place at the beginning of the 1900s, the research on CFSTs has started only in the 1960s. At first, the main focus was to study their behaviour under axial or combined (axial and eccentric) loads due to their large axial stiffness. Recently, CFSTs have been used as main girders of railway bridges to overcome the disadvantages of steel girders by reducing the vibration levels caused by trains. Moreover, the applications of CFSTs as flexural members have increased significantly in the last years. Therefore, the flexural stiffness and capacity of CFSTs have become critical parameters in many applications and their flexural behaviour has become very crucial to understand [9]. Thus, the attention of the research on CFSTs has extended to study their behaviour under pure bending. The research on CFSTs has also covered the behaviour of replacing the normal infill concrete with different types of concrete such as high strength concrete, self-compacted concrete, etc. Some of the previous studies investigated the effect of replacing the normal infill concrete with RAC. The main goal of using RAC in place
of normal concrete was to improve the mechanical properties of RAC due to the confinement effect provided by the steel tube which restrains the movement of the infill concrete. Further studies have been conducted to compare the behaviour of NACFSTs with RACFSTs, particularly their behaviour under axial loads. The main parameters on these studies included the use of different RAC replacement ratios, concrete compressive strengths and the steel to concrete area ratio (confinement index) by changing the dimensions and the shape of the steel tubes. However, there are very limited studies that covered the flexural behaviour of RACFSTs which makes the use of them in structural applications still questionable. Therefore, this research aims to understand the behaviour of RACFSTs under flexure loads compared with NACFSTs experimentally and theoretically. [4, 10].

1.3. Research Objectives

The main objective of this research study is to examine and proof the feasibility of using RAC in CFSTs to act as flexural members in structures. The detailed objectives can be summarized as follows:

1) To investigate the flexural behavior of RACFSTs beams by conducting a comprehensive experimental Program.
2) To compare the flexural properties (flexural stiffness, flexural strength and failure modes) of RACFSTs with NACFSTs and to examine the effect of the different parameters on both.
3) To check the applicability of using the methods provided by different design codes (AISC, AIJ, BS and EuroCode4) to calculate the flexural strength of RACFSTs.

1.4. Research Methodology

In order to fulfil the objectives of this research, the parameters that affect the performance of both NACFSTs and RACFSTs have been studied. At first, this research has addressed the background of CFSTs in terms of their composite behaviour, advantages and disadvantages, flexural behaviour and their applications in structures. This research also went through a brief background about recycled aggregate concrete considering their mechanical properties and their application in buildings. Furthermore, previous studies on the flexural behaviour of NACFSTs and RACFSTs were reviewed and all the findings of these researches were considered in this study. Then, an
experimental program considering the parameters that has a direct effect on the flexural behaviour of RACFSTs was conducted. The results of the experimental program were carefully evaluated and compared to design methods provided by several design codes such as: LRFD-AISC, AIJ, BS and EuroCode4. At last, guidelines and recommendations regarding the flexural behaviour of RACFSTs were provided based on the results that were obtained in this study.

1.5. Thesis Organization

In this chapter, a general overview about CSFTs and RAC is presented. Also, the rest of this chapter shows the research significance, objectives and methodology. The rest of the thesis is organized as follows: Chapter 2 provides background about CFSTs and RAC and presents a comprehensive literature review on the flexural behaviour of NACFSTs and RACFSTs. The experimental program including the material properties, the specimens’ details, the flexural test and the test setup and instrumentation are discussed in Chapter 3. The experimental results and discussions are presented in chapter 4. Chapter 5 compares the theoretical results obtained from design codes with the experimental results. Finally, Chapter 6 concludes the major findings of this research and outlines recommendations for the future work.
Chapter 2. Background and Literature Review

This chapter goes through the background of CFSTs and RAC showing their general properties and applications in structures. Also, this chapter shows the methods adopted by some design codes to estimate the flexural strength of CFSTs. Finally, this chapter covers the related research that has been done on the flexural behaviour of RACFSTs.

2.1. Concrete-filled steel tubes

CFST is a composite structural element developed by combining a hollow steel section (HSS) and infill concrete. The steel tube which can be in any shape (circular, rectangular, square, etc.) as shown in Figure 2.1 provides confinement to the concrete resulting in an increase in the compressive strength of concrete and enhances its inelastic behaviour, whereas the infill concrete improves the global and local buckling capacity of the steel tube [8, 11, 12].

The use of CFSTs has increased significantly in the recent years due to their structural and economic benefits. The steel tube works as a formwork and reinforcement to the concrete which means there is no need for shuttering during the construction phase of concrete and that will result on reducing the construction cost and time. Moreover, any type of concrete can be placed in the steel tube such as self-compacting concrete where no vibration is required. From the structural point of view, the composite behavior of the steel tube and the infill concrete increases the deformation capacity, also the global and local buckling resistance of the steel tube is increased because of the additional stiffness provided by the concrete. The CFSTs have been used mainly for impact resistance and seismic design due to their high stiffness, high strength, high ductility and large energy absorption capacity [2, 8, 13-15].

2.1.1. Composite action of CFSTs. The composite action between the steel tube and infill concrete is developed due to the shear stress transfer between the concrete and steel. The stress transfer can be attained by the natural bond between the steel tube and the infill concrete or by using shear connectors [3]. In general, the bond strength of CFSTs members is low. However, the circular cross sections provide higher bond strength and better confinement than the rectangular or square cross sections. Also, the local buckling will be prevented in the circular section and will be more likely
to occur in the rectangular or square shapes, hence the circular cross sections are preferable. In practice, the rectangular and square sections are used more than the circular sections due to the lack of design guidelines for the circular sections and the fact that the design of connections is easier for rectangular or square sections. The AISC (2005) and the ACI (318-08) provide design specifications for steel structures and reinforced concrete buildings respectively. However, there is no unified design code for composite members, while the AISC (2005) and the ACI (318-08) have different expressions for the strength and stiffness of CFSTs members [2, 8, 16].

The CFSTs members were developed to combine the prominent characteristics of the steel and concrete and to improve their weak characteristics. In CFSTs, the steel tube will confine the concrete which means increasing its inelastic behaviour and the infill concrete will support the steel tube resulting in improving the local buckling behaviour. Figure 2.2 presents the effect of the composite behaviour of CFSTs members in terms of axial load capacity [2]. As explained in the figure, the composite behavior of CFSTs enhances the axial load capacity of the steel tube and the reinforced concrete member combined. Moreover, the plastic behavior of the CFST member is also improved compared to the plastic behavior of the steel tube and reinforced concrete member combined. These results are attributed to the composite action developed between the steel tube and the reinforced concrete member.
2.1.2. Advantages and disadvantages of CFSTs. The CFSTs has many advantages over the use of bare steel or reinforced concrete members, these advantages can be summarized as follows [1]:

1- The use of CFSTs members in high rise buildings will result in a smaller cross sections compared with the very large cross sections of the reinforced concrete members that may be needed to fulfill the design requirements.

2- The infill concrete will improve the behavior of the hollow steel section in terms of local buckling capacity. Also, inward buckling of the steel tube will not occur due to the support of the infill concrete and thus, only outward local buckling can happen.

3- The construction of the CFSTs members is much easier as the steel tube will act as formwork for casting of the concrete eliminating the need for shuttering. This also means that the construction time and cost will be reduced.

In addition to the advantages of CFSTs, they have some disadvantages such as:

1- Since the steel tube is confining the concrete, the steel will be exposed to air and humidity which make the steel vulnerable to corrosion. Hence, the steel tube need to be painted and maintained continuously.

2- In CFSTs members, the steel is not protected from fire. Therefore, at elevated temperatures the steel strength will decrease significantly.
Another important disadvantage of CFSTs members is the lack of sufficient knowledge regarding the bond strength between the infill concrete and the hollow steel section.

2.1.3. Applications of CFSTs. The use of CFSTs in structures was introduced to the construction industry at the beginning of the 1900s. They were used as columns and piers in buildings and bridges due to their high axial load capacity. Nowadays, the use of CFSTs has extended worldwide, especially in Asia including their use as beams and columns in buildings and workshops and as towers, cables and truss members in bridges.

In Japan, the use of CFSTs in buildings for seismic resistance has increased in recent years. Therefore, research studies have been investigating and focusing on the behavior of CFSTs. They are also used as columns with rectangular or circular sections in moment frames. The diameter of tubes and the diameter-to-thickness (d/t) ratio used in Japan are less than 700 mm and 50 respectively. In China, the CFSTs were used since 1950 as columns in subway stations and power plants and their use is rapidly increasing. They are now used as compressive members in high rise buildings to reduce the large size required for steel columns. They are also used in bridges and other structures under different loading conditions. In the United States, CFSTs are also used as columns but their use in building are limited to structures with braced frames. The used diameters of CFSTs columns in the U.S are from 1 m up to 3 meters and the d/t ratio is about 100 and sometimes high strength concrete is used to increase the axial stiffness of the CFSTs members [2, 13]. Figure 2.3 shows the different applications of CFSTs.

![Figure 2.3: Applications of CFSTs](image)
2.1.4. Flexural behavior of CFSTs. Various design codes such as the AISC-LRFD [17], ACI [18], AIJ [19], BS and EuroCode4 [20, 21] give different specifications for the flexural strength of CFSTs, but all of these codes are taking the composite behavior of CFSTs into consideration. The AISC specifications provide two methods for estimating the flexural strength of CFSTs: the plastic stress distribution and the strain compatibility method. The first one assumes that the steel tube develops the yield stress in tension and compression whereas the concrete is subjected to a uniform compression of 0.85 $f'_c$ for rectangular and square sections and 0.95 $f'_c$ for circular sections which is larger than the 0.85 coefficient that is usually used for a Whitney stress block. This value of 0.95 $f'_c$ is intended to take into account the better confinement provided by the circular sections for the concrete. The second method uses the flexural strength calculation of a reinforced concrete section assuming linear strain distribution, a bilinear steel material curve and a parabolic unconfined concrete material curve. The second method is similar to the design procedure provided by the ACI code [8]. The two methods provided by the AISC and the method provided by the ACI code can be seen in Figure 2.4.

![Figure 2.4: Analysis of the CFST section according to a) AISC-PSDM b) AICS-SCM c) ACI-SC [8]](image)

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According to the AISC-LRFD specifications, CFSTs sections subjected to pure bending can be classified as compact, non-compact or slender sections. This classification depends on the local buckling of the steel tube, which affects the bending capacity of the CFST beams. The compact section develops the full plastic moment with the ability to form a plastic hinge, while the non-compact section can also develop the plastic moment but with limited rotation capacity. The slender section exposes to local buckling failure before reaching the elastic moment capacity. For example, the rectangular CFST section classification is controlled by the width-to-thickness (b/t) and the depth-to-thickness (h/t) ratios where the b and h are the inner width and depth of the welded cross section, whereas for circular sections the classification of the section is governed by d/t where d is the diameter of the cross section [22]. Table 2.1 shows the limiting values for composite members subjected to a flexural load with regard to the AISC specification:

Table 2.1: Classification of CFSTs beams according to AISC specifications

<table>
<thead>
<tr>
<th>Description of Element</th>
<th>Ratio</th>
<th>$\lambda_p$</th>
<th>$\lambda_r$</th>
<th>Maximum Permitted</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flanges of Rectangular HSS and Boxes of Uniform Thickness</td>
<td>b/t</td>
<td>2.26$\sqrt{Es/ fy}$</td>
<td>3.00$\sqrt{Es/ fy}$</td>
<td>5.00$\sqrt{Es/fy}$</td>
</tr>
<tr>
<td>webs of Rectangular HSS and Boxes of Uniform Thickness</td>
<td>h/t</td>
<td>3.00$\sqrt{Es/fy}$</td>
<td>5.70$\sqrt{Es/ fy}$</td>
<td>5.70$\sqrt{Es/ fy}$</td>
</tr>
<tr>
<td>Round HSS</td>
<td>d/t</td>
<td>$0.09\frac{Es}{fy}$</td>
<td>$0.31\frac{Es}{fy}$</td>
<td>$0.31\frac{Es}{fy}$</td>
</tr>
</tbody>
</table>

2.1.4.1. Flexural Stiffness of CFSTs. The effective flexural stiffness ($E_{I_{ef}}$) for CFSTs members is very critical to determine the buckling capacity and the deflection for the CFST member. However, the formula of calculating the effective stiffness is not unified as each design code provides different equation for the flexural stiffness of CFSTs. Table 2.2 presents the flexural stiffness for some design codes [8, 23]:

Table 2.2: Flexural stiffness of CFSTs for different design codes

<table>
<thead>
<tr>
<th>Design Code</th>
<th>Effective Stiffness ($E_{I_{ef}}$)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>AISC-LRFD</td>
<td>$E_iI_s + C_iE_iI_c$</td>
<td>$C_i=0.6+2(A_s/(A_s+ A_c))\leq0.90$</td>
</tr>
<tr>
<td>ACI</td>
<td>$E_iI_s + (0.2E_iI_c)/(1+\beta_d)$</td>
<td>$\beta_d = 1.0$</td>
</tr>
<tr>
<td>BS5400</td>
<td>$E_iI_s + E_cI_c$</td>
<td>-</td>
</tr>
<tr>
<td>EuroCode4</td>
<td>$E_iI_s + 0.6E_iI_c$</td>
<td>-</td>
</tr>
<tr>
<td>AIJ</td>
<td>$E_iI_s + 0.2E_iI_c$</td>
<td>-</td>
</tr>
</tbody>
</table>
2.1.4.2. Failure modes of CFSTs under bending. In general, CFSTs members subjected to flexural loads fail in a very ductile manner due to the increased stiffness of the member resulting from the confinement provided by the steel to concrete. When comparing the failure modes of CFSTs members under flexural loads to those of the reinforced concrete or hollow members, it was found that the behavior of CFSTs members is much better. The steel tube in the CFST member will only fail by outward local buckling of the compression flange and the local buckling capacity will be also increased due to the support of the concrete core, while the hollow steel section will be subjected to a series of inward and outward local buckling. Regarding the infill concrete of the CFST member, cracks will be developed in the area under tension. However, the crack width and distance between cracks are smaller than those of reinforced concrete members. Figure 2.5 is showing the differences in failure modes of CFSTs, reinforced concrete elements and hollow steel members when subjected to bending.

![Failure modes of CFST](image)

Figure 2.5: Failure modes of CFST, reinforced concrete and hollow steel members under bending [2]

2.2. Recycled Aggregate Concrete

Because of the increasing population and the construction of high-rise structures, the sustainability of structures has become very important demand in recent years because of the contribution of the construction industry to the natural resources deterioration as the concrete industry is a major consumer of natural resources. As a
Result, the concrete industry has started to use recycled coarse aggregate (RCA) instead of natural aggregate due to the availability of waste concrete from the demolishing of old structures and natural disasters, and also due to the cost reduction of acquiring natural aggregates. The recycled aggregates are produced by crushing and processing of waste concrete coming from demolished structural elements. After processing, the main difference between the recycled and natural coarse aggregate is the attached adhered cement on the core of recycled coarse aggregate which makes the mechanical properties of RAC quite complicated [24-29].

The mechanical properties of RAC can be summarized

1- Compressive Strength: the compressive strength of RAC depends on many factors such as the properties of recycled aggregate, w/c ratio and the mixing procedure. In general, at the same w/c ratio, the increase of recycled aggregate amount will lead to a decrease in the concrete compressive strength, up to 10% compared with NAC [30].

2- Flexural Strength: similar to the behavior of compressive strength, the flexural strength of RAC will decrease with increasing the recycled aggregate replacement ratio. For example, a study made by Bairagi et al. [30] observed that the flexural strength of RAC with 25% and 50% replacement ratio is lower by 6 to 13% respectively than the flexural strength of NAC.

3- Modulus of Elasticity: the effect of recycled aggregate on the modulus of elasticity is more obvious than the compressive and flexural strength for high replacement ratios. Kou et al. [31] reported that the modulus of elasticity declined by 12.6 and 25.2% for 50 and 100% replacement ratios respectively, while Pereira et al. [32] stated that the modulus of elasticity almost remained the same for replacement ratios less than 30%.

2.3. Recycled Aggregate Concrete-filled Steel Tubes

Many studies have been focusing to understand the mechanical properties of RAC. These studies found that the compressive strength and the modulus of elasticity of RAC are somewhat lower than those of natural aggregate concrete NAC. Furthermore, the strain developed due to the peak stress of RAC is about 30% higher when compared with NAC as well as the shrinkage and creep. In addition to this, producing of recycled aggregates involves high labor costs and large energy
consumption due to the crushing and demolition of buildings. These shortcomings have led to restricted and very limited applications of RAC in structures [5]. However, there are also a lot of advantages of RAC over NAC such as the higher water absorption capacity especially when the recycled aggregates are not pre-wetted, this means that the w/c ratio of the concrete will reduce due to the additional absorbing water of RAC resulting in increasing the strength of concrete [5, 33]. Additional advantages of RAC are the low thermal conductivity, low brittleness and low specific gravity which reduces the concrete self-weight [34].

In the past, the applications of recycled aggregate concrete RAC have been limited to low performance structures such as pavement base because of their low strength, high deformation and the unreliable test results of the compressive strength. The use of RAC as the infill concrete of the CFSTs is a widely accepted idea and can be considered as a practical and effective structural application of RAC that will help to overcome the drawbacks of RAC mainly. The steel tube will confine the infill concrete which results in increasing the compressive strength and the stiffness of the concrete core as seen in Figure 2.6. In addition, the confinement provided by the steel tube will improve the ductility of the infill concrete by enhancing its inelastic behavior. Moreover, the steel tube will help to control the scatter of the mechanical properties of the RAC infill, thereby controlling the cracks development as the steel tube will restrain the movement of infill RAC [4].

![Figure 2.6: Effect of the confinement on the stress-strain relationship of concrete [35]](image-url)
2.4. Literature Review

The literature review discusses the related work and previous studies related to this research. The literature review is divided into two parts, the first part goes through the previous research on the flexural behavior of CFSTs and the second part addresses the research on the flexural behavior of RACFSTs.

2.4.1. Related work on CFSTs. Furlong [36] was one of the first researchers to start working on CFSTs. Although his work was mainly focusing on CFSTs subjected to axial loads, he tested one square concrete filled still tube beam and he found that the flexural capacity of the beam is almost 50% higher than the capacity of the steel tube alone.

Charles W. Roeder et al. [5] evaluated the accuracy of using the methods provided by the design codes for determining the flexural strength of circular CFSTs. They collected the data of 122 circular CFSTs specimens, which were tested by previous researchers and compared the moment capacities of the collected data with the design provisions of AISC and ACI. They reported that the plastic stress distribution method given by the AISC specifications gives more reliable results than the strain compatibility method proposed by both the AISC and ACI specifications. They illustrated that the strain compatibility method significantly underestimates the moment resistance of circular CFSTs and the average ratio between the measured and predicted moment capacity was 1.65, while the average ratio for the plastic stress distribution method was 1.24.

Han [2] tested a total of 16 concrete filled square hollow sections (SHS) and rectangular hollow sections (RHS) specimens. The length of all specimens was 1100 mm, the depth over width (d/b) ratio was in the range from 1 to 2 and the depth to thickness (h/t) ratio was in the range from 20 to 50. The tubes were made of mild steel plates welded together into a square or rectangular shapes. The concrete used in this experiment was designed to give a compressive strength of 30 MPa at 28 days. From this experimental study, the author was able to determine the maximum moment capacity of the specimens and also investigate the failure pattern after passing the ultimate load. The researcher reported that the failure of the specimens was in a very ductile way. They explained this ductile behavior happens because of the establishment of the composite action between the steel tube and the infill concrete. He illustrated
from the moment vs. curvature diagrams that the behavior of the CFSTs is initially elastic, after that, an inelastic behavior occur and the stiffness of the section gradually decrease until the ultimate moment. He defined the ultimate moment capacity of the composite section as the moment corresponding to the maximum steel strain of 0.01 as the moment becomes stable after this value. He also showed that the moment vs. curvature relationship turns to the inelastic stage at 20% of the section's ultimate moment capacity. In addition to the experimental program, the researcher introduced a confinement factor (ξ) to develop a mechanical model that can predict the flexural strength of CFSTs. This factor is equal to $(\frac{A_s \times F_y}{A_c \times f'_c})$ and it was described by the author as a factor that represents the composite action of CFSTs. The author also compared the results of his experimental program with 4 different design codes and his proposed mechanical model. The design codes, which he used, are the AIJ, BS5400, LRFD-AISC and the EuroCode4. He concluded his results by showing that the design codes gave lower moment capacities than the test results by 10 to 20% and the EuroCode4 and his method were the best to predict the moment capacities.

A further study was made by Han [23] to investigate the flexural behavior of CFSTs using self-consolidating concrete (SCC) instead of normal concrete. They tested a total of 36 beams with circular and square sections, and the parameters they considered are the steel yielding stress (from 235 to 282 MPa), the tube diameter to thickness (d/t), and the shear span to depth ratio a/d (from 1.25 to 6). They stated that in general, the flexural behavior of self-consolidating CFSTs is very similar to those of normal CFSTs. Considering the effect of their parameters on the behavior of beams; they reported that the shear span to depth ratio has no obvious effect on the flexural behavior of CFSTs. They also compared their results with several design codes (AISC, BS5400, AIJ, and Eurocode4) and they found that the moment capacity and the flexural stiffness of self-consolidating CFSTs can be conservatively predicted by these codes.

Jiho Moon et al. [9] developed a nonlinear finite element analysis model that predict the behavior of CFSTs under bending. They reported that the model gave good estimations for the global and local behavior of CFSTs. They conducted parametric studies to select the angle of dilation and friction coefficient between the steel tube and the concrete infill. They recommended an angle of dilation of 20° and a friction coefficient of 0.47 to establish an accurate finite element model for circular CFSTs.
subjected to flexural loading. Using the FE model, they studied the effects of d/t and fy/fc' ratios on the flexural behavior of CFSTs. They stated that increasing the fy/fc' ratio improved the moment capacity of CFSTs and delayed the local buckling, while the effect of d/t is not significant to the flexural behavior of CFSTs. Furthermore, they compared their results with current design provisions. They indicated that the plastic stress distribution method gives reasonably conservative moment capacity of CFSTs, and the AISC design specifications provide reasonable values for the effective flexural stiffness $E_{I_{eff}}$.

Rui Wang et al. [6] summarized the flexural behavior of rectangular CFSTs according to previous studies. They mentioned that the parameters were considered are the d/b ratio, the concrete compressive strength, the steel yield stress and the shear span to depth (a/d) ratio. They concluded that the flexural strength of the rectangular CFST section is up to 50% higher than the strength of the bare steel tube section. They also added that the behavior of the rectangular CFST members is very ductile and their mode of failure include outward local buckling of the steel tube at or near the load point, while the infill concrete suffers from flexural cracks at the bottom of the section. They also found from previous studies that the shear span to depth ratio has no significant effect on the flexural behavior of rectangular CFSTs. In order to understand and explain the behavior of rectangular CFSTs, the authors developed a FE model and compared their results with the data of 70 beams tests from previous experimental programs. They found good agreement with the data obtained from tests results in term of the predicted load vs. deformation curves, failure modes and moment capacities. They explained this behavior of rectangular CFSTs by the composite action or the interaction between the steel tube and the infill concrete as this interaction causes stress redistribution in the steel and concrete which is the main reason of the high moment capacity and ductility.

2.4.2. Related work on RACFSTs. The first researchers to propose the use of recycled aggregate concrete in CFSTs were Konno et al. [10] and their objective was to improve the mechanical properties of RAC. After their research, many studies were conducted to investigate the efficiency of using RAC in different types of steel tubes such as carbon steel tubes, stainless steel tubes and also in carbon steel tubes strengthened with FRP sheets. The most important results that were reported from
previous studies are somewhat lower flexural capacity of RACFSTs compared to NACFSTs and the similarity of the failure modes for both types of CFSTs.

Yang and Han [37] investigated the behavior of RACFSTs under concentric and eccentric axial loads. In their study, they considered different shapes of steel tubes (circular and square) and replacement ratios of 25% and 50%. From the experimental program, they determined the axial load capacity and the failure pattern beyond the ultimate load. They reported that the typical failure modes for all specimens was overall buckling after yielding. They also stated that when the load reached 60 to 70% of the ultimate load, the deformation at middle height increased significantly. They concluded that the load vs displacement curves for all specimens are nearly the same, however, the ultimate loads of RACFSTs were somewhat lower than corresponding normal CFSTs. Zongping et al. [38] conducted another study on the axial behavior of RACFSTs. They tested 15 specimens with different replacement ratio, slenderness ratio and eccentricity. They achieved similar results to those obtained by Yang and Han, as they stated that the axial behavior of RACFSTs and NACFSTs are very similar in term of the failure process and the failure mode. However, they reported that the effect of the replacement ratio of RAC under eccentric loading is comparatively unobvious.

In addition of investigating the axial behavior of RACFSTs experimentally, some researchers such as Xiao et al. [39] and Yang [40] investigated their behavior using finite element analysis. Xiao et al. [39] developed a numerical model using the commercial software ANSYS to study the behavior of RACFSTs under pure compression. They found that the peak stress of RACFSTs increases with the increase in the thickness of the steel tube, but the plastic behavior is not affected by this increase. Yang [40] used finite element analysis to understand the behavior of RACFSTs under axial loads and cyclic flexural loads. The author stated that the behavior of RACFSTs under cyclic loading is very similar to that of NACFSTs.

Yang and Han [34] conducted an experimental study to investigate the effect of the replacement percentage of recycled coarse aggregate on the flexural behavior of CFSTs. They used a replacement percentage of 0%, 25% and 50%. They showed that the replacement percentage did not have any effect on the failure pattern of RACFSTs as all specimens failed by local buckling of the steel tubes near the loading area. However, they reported that the initial flexural stiffness and ultimate bending moment
of RACFSTs were somewhat lower than those of NACFSTs by 3.1 to 8.7% and 3.5 to 8.1% respectively.

J. Chen [8] investigated the behavior of RACFSTs under combined loading. They conducted an experimental program using 48 circular columns and 3 beams divided into 17 groups where each group has 3 identical specimens to check the scatter of the test results. Their study also included the effect of the replacement ratio and the steel to concrete area ratio. They concluded that for all identical specimens the results were consistent with differences less than 3% for each group. They attributed these results to the confinement effect by the steel tube. They also added that there is a small reduction of 4.5% to 11.2% in the maximum compressive load of RACFSTs when the replacement ratio is 100% compared to normal CFSTs. They also noted that the higher the steel to concrete area ratio the better the ductility of the specimen, while the replacement ratio has no significant effect on the ductility of the specimen. In addition to all of that, they compared their results with current CFSTs design provisions to check the applicability of these methods on RACFSTs. They recommended to uses the method proposed by EuroCode4 as it gave close prediction to the test results.

2.4.3 Bond strength of CFSTs. The bond strength or the bond stress capacity in CFSTs is defined as the stress at the interface when the infill concrete starts to slip. The importance of evaluating the bond stress capacity of CFSTs is very critical in order to enhance the composite action between the steel and concrete and thus improve the structural properties of CFSTs. The bond stress capacity is usually measured by the push-out test. Studies on bond strength using this test have been performed by Virdi and Dowling [41], Shakir Kalil [42] and Morishita and Tomii [43]. They used 104 circular sections with diameters less than 200 and d/t ratios from 15 to 35, they also used 49 rectangular sections. The results showed 3 general trends. First, Concrete strength is not related to the bond strength. Second, the rectangular CFT had lower bond strength than the circular CFT. Finally, the bond strength was decreased by the increase in the diameter and d/t ratio. These studies have agreed that the roughness of the inner surface of the steel tube increases the bond strength of CFSTs, while the use of shear connectors do not affect the bond strength and they only work after the slip of the infill concrete occurs.
According to a study made by Charles W. Roeder et al. [13] the bond strength in CFSTs depends on the radial displacements resulting from the pressure of wet concrete on the inner surface of the steel tube, the shrinkage of the concrete core and the roughness on the internal surface of the tube. The researchers explained that the bond between the steel tube and infill concrete relies on the interface condition. They mentioned three possible states at the interface, the first state that is called the chemical and mechanical bond is provided by the natural adhesion between the steel and concrete, the interface pressure and the friction coefficient. In the second state, separation between the steel and concrete occurs due to shrinkage of the concrete or the pushing of one material to the other when the load is applied. The third state, which is the common state, is an intermediate one between the first and second state where the adhesion between the steel and concrete is reduced as the separation in the second state starts to appear. The authors conducted an experimental program consists of 20 specimens in order to investigate the bond strength of circular CFSTs. the parameters of the study included the diameter of the concrete, the thickness of the steel tube and the shrinkage of concrete. The steel tubes diameters and the d/t ratio ranged from 250 to 650 mm and 20 to 110 respectively. The shrinkage of concrete was moderate for 8 specimens and was little for 12 specimens. Their results showed that the shrinkage of concrete is a main reason for reducing the bond strength of CFSTs; they also noticed that tubes with smaller diameters and d/t ratios develop larger bond strength. They recommended using steel tubes with inner irregularities as they significantly increase the bond stress capacity especially for tubes with small diameters and d/t ratios.

P.J. Nixon et al. [44], Chen et al. [45] and L. Butler et al. [46] studied the bond strength of RACFSTs without surface preparation. They achieved very similar and encouraging results. They reported that the bond strength increased with an increase in the replacement percentage of the recycled coarse aggregate, but they excluded cases with a replacement percentage of more than 75% from this general behavior. They attributed this larger bond strength than the NACFSTs to the higher strength of RACFSTs due to the water/cement ratio resulting from the high water absorption capacity of RACFSTs. They also investigated the bond strength for different shapes of RACFSTs; they concluded that circular steel tubes have better confinement than rectangular or square steel tubes under the same parametric conditions and hence higher bond strength.
Zhong Tao et al. [47] made an extensive research on the bond strength of CFSTs. They conducted a series of push-out tests on rectangular and circular CFSTs with different parameters. The main parameters considered in this study were cross-sectional dimension ranging from 120 to 600 mm, type of steel, type of concrete (normal concrete, recycled aggregate concrete and expansive concrete), and age of concrete and interface condition. They used shear studs and internal rings on the interface to investigate their effect on the bond strength. They indicated that the bond strength is higher in circular sections compared with rectangular ones. They also illustrated that the bond strength decreases remarkably with the increase in cross-sectional dimensions. As for the effect of the concrete type, they showed that in general the use of RAC instead of NAC will lead to an increase on the bond strength of CFSTs. They also reported that the bond strength will decrease with the increasing of concrete age. Finally, they added that welding internal rings in the inner surface of steel tubes is the most effective method to increase the bond strength followed by welding shear studs.

It is worth mentioning here that the push-out test has some limitations as only axial load is applied to the concrete core. Therefore, it is very important to study the importance of bond strength when bending moment is applied to the CFST as many researchers indicated that friction between the steel tube and the infill tube will increase under flexural loading.

2.4.4. Limitations of related work. Despite the research on the flexural behavior of CFSTs has increased in recent years, there are still very limited studies on the behavior of RACFSTs under bending as noticed from the reviewed literature. Many authors investigated the behavior of RACFSTs subjected to axial loads. In general, they showed the behavior of RACFSTs is very similar to normal CFSTs. Therefore, this research is taking a step forward towards thoroughly understanding the behavior of RACFSTs compared to NACFSTs by considering their flexural behavior. The parameters considered in this research are the shape of the steel tube (rectangular and circular), the steel tube cross-sectional area, the confinement factor (ξ), the concrete compressive strength (fc’) and the replacement ratio of the recycled aggregate (R).
Chapter 3. Experimental Program

The experimental program conducted in this research was designed in order to fulfil the research objectives. The experimental program aims to investigate the flexural response of RACFSTs and to check the feasibility of replacing the normal infill concrete with RAC. In this research, the following parameters are included in the experimental program:

- The shape and dimensions of the steel tube:
  For normal aggregate CFSTs, the moment capacity is significantly affected by the shape and dimensions of the cross-sectional area of the steel tube as the shape and dimensions of the tube considerably affect the confinement factor ($\xi$). The steel tube shapes to be included in this research are circular and rectangular sections with different D/t and h/t ratios. Table 3.1 provides the details of the hollow steel sections used in this study.

Table 3.1: Steel Tubes Details

<table>
<thead>
<tr>
<th>Section Label</th>
<th>Shape</th>
<th>Dimensions</th>
<th>d/t or h/t</th>
<th>AISC classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>Circular</td>
<td>114 mm Diameter Thickness = 3 mm</td>
<td>38</td>
<td>Compact section</td>
</tr>
<tr>
<td>C2</td>
<td>Circular</td>
<td>168 mm Diameter Thickness = 3 mm</td>
<td>56</td>
<td>Compact section</td>
</tr>
<tr>
<td>C3</td>
<td>Circular</td>
<td>168 mm Diameter Thickness = 4 mm</td>
<td>42</td>
<td>Compact section</td>
</tr>
<tr>
<td>R1</td>
<td>Rectangular</td>
<td>100x200 mm (hxb) Thickness = 3 mm</td>
<td>66.67</td>
<td>Compact section</td>
</tr>
<tr>
<td>R2</td>
<td>Rectangular</td>
<td>100x200 mm (hxb) Thickness = 4 mm</td>
<td>50</td>
<td>Compact section</td>
</tr>
</tbody>
</table>

- Compressive strength of Concrete:
  The flexural behavior of RACFSTs is not only a function of the cross-section shape and dimension of the steel tube. The material properties (steel and concrete) play an important role to determine the flexural behavior of RACFSTs. To date, researchers have not considered the effect of the compressive strength of concrete on the flexural behavior of RACFSTs.
Therefore, the change in the compressive strength of RAC is an important factor to consider, as the RAC is known for its weak mechanical properties and the scatter of the mechanical properties results. Although the steel tube may help to overcome these disadvantages, the effect of changing the RAC compressive strength on the flexural behavior of RAC compared to normal concrete must be taken into consideration. The concrete compressive strengths used in this study are low and medium strengths with 30 MPa and 50 MPa respectively.

- Replacement ratio of recycled coarse aggregate:
  According to the literature, the increase in the recycled coarse aggregate ratio will slightly affect the flexural strength of RAC. Therefore, it is important to know the effect of changing the replacement ratio along with other parameters. Replacement ratios of 50 and 100% are considered in this study.

### 3.1. Material Properties

#### 3.1.1. Concrete
In this study, two values of concrete compressive strength were used; the first mix has a compressive strength value of 30 MPa, while the second type has a compressive strength of 50 MPa. For the first type of concrete ($f'_c = 50 \text{ MPa}$), two concrete mixes were developed which are NAC and RAC with a replacement ratio of 100%. As for the second type of concrete ($f'_c = 30 \text{ MPa}$), RAC mixes with replacement ratios of 50% and 100% were developed. The details of each concrete mix are provided in Table 3.2. The mixing and casting procedure were performed in the AUS-CVE Construction Materials lab.

#### Table 3.2: Concrete mixes details

<table>
<thead>
<tr>
<th>Material (Kg)</th>
<th>Concrete Mix (0.1Kg/m3)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>50-N</td>
</tr>
<tr>
<td>Water</td>
<td>20</td>
</tr>
<tr>
<td>Cement (OPC)</td>
<td>50</td>
</tr>
<tr>
<td>NCA (10 mm)</td>
<td>110</td>
</tr>
<tr>
<td>NCA (20 mm)</td>
<td>-</td>
</tr>
<tr>
<td>RCA (15 mm)</td>
<td>-</td>
</tr>
<tr>
<td>RCA (10 mm)</td>
<td>-</td>
</tr>
<tr>
<td>Dune Sand</td>
<td>27.4</td>
</tr>
<tr>
<td>Crushed Sand</td>
<td>41.3</td>
</tr>
<tr>
<td>Super Plasticizer</td>
<td>600 ml</td>
</tr>
</tbody>
</table>
The recycled coarse aggregate (RCA) used in this study was locally produced by Beeah’s waste management plant in Sharjah. The average size of the RCA was 15 mm. The RAC mixes were prepared by directly replacing the NCA with the same weight of RCA. All other materials proportions were kept the same. Table 3.3 compares between the properties of the NCA and the RCA used in this study.

Table 3.3: Normal and Recycled Coarse Aggregate Properties

<table>
<thead>
<tr>
<th>Coarse Aggregate Type</th>
<th>Size (mm)</th>
<th>Apparent Specific Gravity</th>
<th>Bulk Specific Gravity</th>
<th>Absorption Ratio %</th>
<th>Moisture %</th>
<th>Crushing Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>NCA-20</td>
<td>20</td>
<td>2.66</td>
<td>2.59</td>
<td>1.69</td>
<td>0.78</td>
<td>19.8</td>
</tr>
<tr>
<td>NCA-10</td>
<td>10</td>
<td>2.74</td>
<td>2.70</td>
<td>1.13</td>
<td>1.28</td>
<td>19.1</td>
</tr>
<tr>
<td>RCA</td>
<td>15</td>
<td>2.72</td>
<td>2.38</td>
<td>5.75</td>
<td>1.51</td>
<td>17.9</td>
</tr>
</tbody>
</table>

For each concrete mix, 6 cubes with dimensions of 150x150x150 mm and 4 cylinders with diameter of 150 mm and height of 300 mm were cast to measure the compressive strength at 7 and 28 days for each mix. Figure 3.1 shows the cubes and cylinders that were cast for each concrete mix. The results of the compressive strength at 28 days for each mix are presented in table 3.4. The tested cubes and cylinders are illustrated in Figure 3.2 to show their failure modes after the test.

Table 3.4: Compressive Strengths of Concrete (a) Cubes (b) Cylinders

(a) Concrete Mix | Concrete Compressive Strength (MPa) | Cube 1 | Cube 2 | Cube 3 | Average |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>50-N</td>
<td></td>
<td>63.6</td>
<td>60.18</td>
<td>63.6</td>
<td>62.46</td>
</tr>
<tr>
<td>50-R100</td>
<td></td>
<td>57.16</td>
<td>56.75</td>
<td>53.5</td>
<td>55.80</td>
</tr>
<tr>
<td>30-R50</td>
<td></td>
<td>32.84</td>
<td>33.67</td>
<td>33.26</td>
<td>33.25</td>
</tr>
<tr>
<td>30-R100</td>
<td></td>
<td>31.07</td>
<td>31.56</td>
<td>28.71</td>
<td>30.44</td>
</tr>
</tbody>
</table>

(b) Concrete Mix | Concrete Compressive Strength (MPa) | Cylinder 1 | Cylinder 2 | Average |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>50-N</td>
<td></td>
<td>48.73</td>
<td>45.94</td>
<td>47.34</td>
</tr>
<tr>
<td>50-R100</td>
<td></td>
<td>43.19</td>
<td>45.36</td>
<td>44.28</td>
</tr>
<tr>
<td>30-R50</td>
<td></td>
<td>27.62</td>
<td>26.87</td>
<td>27.25</td>
</tr>
<tr>
<td>30-R100</td>
<td></td>
<td>24.46</td>
<td>22.79</td>
<td>23.63</td>
</tr>
</tbody>
</table>
Figure 3.1: Concrete cubes and cylinders to be tested for each concrete mix

Figure 3.2: Failure Mode of Concrete Cubes, (a) NAC Specimen (b) RAC Specimen
3.1.2 Steel. The steel tube beams used in this research were manufactured by a local supplier in Sharjah, UAE. Three coupon specimens were shaped from each tube section according to ASTM specifications as seen in Figure 3.4. The ASTM standards state that the tested specimen under tension should have a minimum total length of 203.3 mm, grip length and radius of filler of 50 mm and 12.7 mm respectively. Tensile tests using the INSTRON machine were conducted to obtain the properties of the steel.

![Figure 3.3: Coupon Specimens for each tube section](image)

The tensile test setup using the INSTRON machine is shown in Figure 3-4 (a) and the typical failure modes for coupon specimens is illustrated in Figure 3-4 (b). The stress-strain graphs for all coupon specimens are shown in Figure 3-5.

![Figure 3.4: Tensile Test, (a) Test Setup (b) Specimens Mode of Failure](image)
From Figure 3.5, the modulus of elasticity of all the tested coupon specimens is approximately the same and it was estimated around 200 GPa. However, it can also be observed from Figure 3-6 that the tested coupon specimens don’t have a well-defined yield point. Therefore, the 0.2% offset approach was adopted to find the yield strength.
and the yield strain of each tube section, the results obtained from the coupon tests including the ultimate tensile strength, the yield strength and the yield strain for each tube section are presented in Table 3.5.

Table 3.5: Steel Properties obtained from the tensile tests

<table>
<thead>
<tr>
<th>Section Label</th>
<th>Tensile Strength (MPa)</th>
<th>Yield Strength (MPa)</th>
<th>Yield Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>343.97</td>
<td>280.43</td>
<td>0.0024</td>
</tr>
<tr>
<td>C2</td>
<td>281.31</td>
<td>258.40</td>
<td>0.00231</td>
</tr>
<tr>
<td>C3</td>
<td>293.66</td>
<td>267.71</td>
<td>0.00235</td>
</tr>
<tr>
<td>R1</td>
<td>314.18</td>
<td>269.64</td>
<td>0.00232</td>
</tr>
<tr>
<td>R2</td>
<td>324.53</td>
<td>287.47</td>
<td>0.00238</td>
</tr>
</tbody>
</table>

3.2. Flexural Test

For this experimental program, a total of 20 CFST beams with 5 different steel tube sections (refer to Table 3.1), two different average compressive strengths of 30 and 50 MPa and two different replacement ratio of RAC (50 and 100%) were subjected to a 4-point flexural test using a universal testing machine (UTM) that has a static load capacity of 2000 KN. All the beam specimens were cast in the AUS-CVE lab as shown in Figure 3.6. The flexural test adopted on this research was designed mainly to compare the flexural behaviour between NACFSTs and RACFSTs. Also, this test aims to further the understanding of the flexural behaviour of RACFTs by investigating the effect of the different parameters mentioned above. Figure 3.7 and Table 3.6 provides a summary of the test matrix used in this research. The schematic diagram provided in Figure 3.7 also shows the labels of the beam specimens. The labels of the beam specimens indicate the steel tube section, the compressive strength of concrete, the type of the concrete mix (NAC or RAC) and the replacement ratio of the RAC. The length of all the beam specimens used in this research is 1.5 m. Every steel tube section has four beam specimens with different concrete mixes (50-N, 50-R100, 30-R50 and 30-R100). The details of the steel tubes sections and the concrete mixes were mentioned previously in the material properties section.
Figure 3.6: Casting of the Steel Tubes

Table 3.6: Experimental Program Test Specimens

<table>
<thead>
<tr>
<th>Concrete Compressive Strength (MPa)</th>
<th>RCA Ratio (%)</th>
<th>Specimens Labels (20 specimens)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C1</td>
<td>C2</td>
</tr>
<tr>
<td>30</td>
<td>50</td>
<td>C1-30-R50</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>C1-30-R100</td>
</tr>
<tr>
<td>50</td>
<td>100</td>
<td>C1-60-R100</td>
</tr>
<tr>
<td></td>
<td>N</td>
<td>C1-60-N</td>
</tr>
</tbody>
</table>

Figure 3.7: Test Matrix
CFST specimens used in this research were cast manually at the university lab and compacted carefully to ensure a uniform distribution of the concrete inside the steel tube. As mentioned above, all the tested specimens have a total length of 1.5 m and were tested over a 1300 mm centre-to-centre span with a constant moment region of 300 mm. The load induced by the universal testing machine was transferred to the tested specimens through two roller plates as seen in Figure 3.8 which shows the test setup arrangement. The rate of the load applied from the universal testing machine was 2mm/min. To prepare the specimens for testing, three strain gauges were attached and connected to each specimen. Before attaching the strain gauges, the surface of all beam specimens at the position of the strain gauges was cleaned from dust and scraps using a steel grinder as seen in Figure 3.9 (a) to ensure that the strain gauges are connected to the steel tube. The position of the attached strain gauges to the tested specimens was at the top, middle and bottom of the beam specimens at a distance of half of the length of the specimen as seen in Figure 3.8. Three linear variable displacement transducer (LVDTs) located at the specimen soffit were used to capture the in-plane displacement at the mid span and at a distance of quarter the length from each support. Special supports and loading equipment were used for circular sections as shown in 3.9 (b) and 3.9 (c). The special supports were used to hold the circular beams in place during the testing procedure as they provide good grip and prevent rotation. The loading equipment were used to ensure that the loading plates will not move or slip during the testing. The data of the load, strain readings and deflection measurements of the tested specimens were recorded during the test at each load increment using high speed data acquisition systems.

Figure 3.8: Typical Details of the Test Setup
Figure 3.9: Preparation of CFST specimens

Figure 3.10 shows a rectangular and circular CFST specimens under testing.

Figure 3.10: CFST Beam Specimens under testing
Chapter 4. Experimental Results and Discussion

In this chapter, the results obtained from the experimental program conducted in this research are presented. As stated in the previous chapter, testing of twenty beam specimens has been carried out. The results shown in this chapter are divided into four parts. The first part compares the flexural behaviour of NACFSTs and RACFSTs. The second part addresses the effect of the change in the confinement factor (ξ) on the flexural behaviour of RACFSTs. The third and fourth parts of this chapter consider the effect of the concrete compressive strength (f’c) and the RCA replacement ratio on the flexural behaviour of RACFSTs. The comparison between the tested specimens is based on the relationship between the moment versus mid span deflection, moment vs. strain, the moment at steel yield strain (My) and the ultimate moment capacity (Mu) of the tested beams. Moreover, the strain distribution at 0.5*Mu, My and Mu and the deflection shape of specimens at My and Mu were used to capture the flexural behaviour of the tested specimens throughout the test. The test results also show the deformation behaviour of the tested specimens beyond the yield point of the steel and their failure modes. The moment at yield (My) in this research is taken as the moment at the steel yielding strain (the yield strain of each steel tube section can be found in Table 3.5), whereas the ultimate moment capacity of the tested beams (Mu) was considered at a steel strain of 0.01. It was found that the moment of the tested CFST beams slightly increase after reaching the yield strain of the steel, and then the moment tends to stabilize around a steel strain value of 0.01. Also, the value of 0.01 strain was recommended by many researchers as a practical value for the ultimate moment capacity. Therefore, this value was considered as the steel strain value corresponding to the ultimate moment capacity of the tested beams.

4.1. Comparison of the Flexural Behaviour Between NACFSTs and RACFSTs

Four graphs have been developed to compare the flexural behaviour of NACFSTs and RACFSTs. Figure 4.1 compares the moment vs. mid span deflection between NACFST beams and RACFST beams with 100% replacement ratio of RAC. All the shown specimens in the graphs have a concrete compressive strength of 50 MPa. The moment vs. strain graphs for the same specimens are presented in Figure 4.2. The deflection shapes at My and Mu and the strain distribution at 0.5 Mu, My and Mu for the tested specimens are given in Figure 4.3 and 4.4 respectively.
Figure 4.1: Moment vs. Mid Span Deflection Relationship for NACFSTs & RACFSTs
Figure 4.2: Moment vs. Strain Relationship for NACFSTs and RACFSTs
Figure 4.3: Comparison of Strain Distribution at Different moment values between NACFSTs & RACFSTs
Figure 4.4: Comparison of Deflection Shapes at Yield & Ultimate Moments values between NACFSTs & RACFSTs
Table 4.1 summarizes the results of the yield moment (My) and the ultimate moment (Mu) as defined previously and their corresponding deformations for NACFST and RACFST beams.

Table 4.1: Ultimate and Yield Moments and their corresponding deformation for NACFSTs & RACFSTs

<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>Yield Moment (My)</th>
<th>Deformation at Yield Moment (mm)</th>
<th>Ultimate Moment (Mu)</th>
<th>Deformation at Ultimate Moment (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1-50-N</td>
<td>10.35</td>
<td>4.55</td>
<td>13.69</td>
<td>14.81</td>
</tr>
<tr>
<td>C1-50-R100</td>
<td>9.76</td>
<td>5.19</td>
<td>13.4</td>
<td>14.84</td>
</tr>
<tr>
<td>C2-50-N</td>
<td>21.14</td>
<td>3.37</td>
<td>30.38</td>
<td>10.08</td>
</tr>
<tr>
<td>C2-50-R100</td>
<td>21.27</td>
<td>3.53</td>
<td>30.21</td>
<td>10.16</td>
</tr>
<tr>
<td>C3-50-N</td>
<td>36.2</td>
<td>6.14</td>
<td>40.96</td>
<td>17.62</td>
</tr>
<tr>
<td>C3-50-R100</td>
<td>35.02</td>
<td>6.56</td>
<td>39.79</td>
<td>16.99</td>
</tr>
<tr>
<td>R1-50-N</td>
<td>45.54</td>
<td>4.41</td>
<td>59.57</td>
<td>7.83</td>
</tr>
<tr>
<td>R1-50-R100</td>
<td>43.25</td>
<td>3.86</td>
<td>54.39</td>
<td>7.11</td>
</tr>
<tr>
<td>R2-50-N</td>
<td>59.25</td>
<td>3.44</td>
<td>77.65</td>
<td>5.04</td>
</tr>
<tr>
<td>R2-50-R100</td>
<td>54.6</td>
<td>3.69</td>
<td>73.93</td>
<td>5.58</td>
</tr>
</tbody>
</table>

After careful examination of the results for the tested specimens of NACFSTs and RACFSTs, it was found that, in general, the flexural responses of NACFSTs and RACFSTs are very similar. The behaviour of NACFSTs and RACFSTs subjected to flexural loads was very ductile allowing the test to proceed in a very smooth and controlled way. This ductile behaviour of the tested specimens can be attributed to the infill concrete and the composite action established between the steel tube and concrete infill. The moment vs. mid span deflection and the moment vs. strain graphs presented in Figure 4.1 and 4.2 showed that the elastic and plastic behaviour for NACFSTs and RACFSTs is almost the same. However, the flexural strength at steel yield strain (My) of rectangular RACFSTs was slightly less than the flexural strength of NACFSTs. Moreover, the ultimate moment capacity (Mu) for rectangular RACFSTs decreased considerably compared to rectangular NACFSTs. This result was not noticed in circular specimens as both My and Mu were very similar for circular RACFSTs and NACFSTs due to the better confinement provided by the circular steel tube. Figure 4.3 showed that the strain distribution for RACFSTs and NACFSTs was similar throughout the test. Figure 4.4 also shows that the mid span deflection of Rectangular RACFSTs at My and
Mu increased slightly compared to rectangular NACFSTs. The failure mode of circular CFSTs differed from the failure mode of rectangular CFSTs. The rectangular RACFST and NACFST specimens suffered from outward local buckling near the loading points, however, the circular specimens did not show signs of local buckling. In general, the failure mode for both NACFSTs and RACFTs was very ductile and did not suffer from tensile fractures or brittle failure. The infill concrete suffered from crushing at the loaded zones and cracks at the tension zone. Figure 4.5 shows the failure mode for different NACFST and RACFST tested specimens after the test.

Figure 4.5: Failure Modes of NACFSTs and RACFSTs
(a) Circular Beams (b) Rectangular Beams
Figure 4.6 compares the values at of My and Mu between NACFSTs and RACFSTs.

![Figure 4.6: Comparison of Ultimate and Yield Moments between NACFSTs & RACFSTs](image)

4.2. **Effect of the Confinement Factor on the Flexural Behaviour of RACFSTs**

This section of the results addresses the effect of the Confinement factor ($\xi$) on the flexural behavior of RACFSTs. The tested specimens in this section have a concrete compressive strength of 50 and 30 MPa and RCA Replacement percentages of 50% and 100%. Two Circular steel tubes with D/t ratio of 42 and 56 and two rectangular steel tubes with h/t ratio of 50 and 66.67 were used to study the effect of the confinement factor on the flexural behaviour of RACFSTs. Figure 4.7 and 4.8 compares the moment vs. mid span deflection and moment vs. strain for RACFSTs with different confinement factor values. The strain distribution and the deflection curves at different moment values throughout the test are shown in Figure 4.9 and 4.10 respectively.
Figure 4.7: Moment vs. Mid Span Deflection Graphs for RACFSTs with different Confinement Factors
Figure 4.8: Moment vs. Strain Graphs for RACFSTs with different Confinement Factors
Figure 4.9: Comparison of Strain Distribution at Different Moment Values for RACFSTs with different Confinement Factors.
Figure 4.10: Comparison of Deflection Shapes at Yield & Ultimate Moments for RACFSTs with different Confinement Factors
Table 4.2 gives a summary of the results of the yield moment (My) and the ultimate moment (Mu) and their corresponding deformations for RACFST beams with the different D/t and h/t ratios.

Table 4.2: Ultimate and Yield Moments and their corresponding deformation for RACFSTs with different Confinement Factors

<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>Confinement Factor (ξ)</th>
<th>Yield Moment (My)</th>
<th>Deformation at Yield Moment (mm)</th>
<th>Ultimate Moment (Mu)</th>
<th>Deformation at Ultimate Moment (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C2-50-R100</td>
<td>0.39</td>
<td>21.27</td>
<td>3.53</td>
<td>30.21</td>
<td>10.16</td>
</tr>
<tr>
<td>C3-50-R100</td>
<td>0.55</td>
<td>35.02</td>
<td>6.56</td>
<td>39.79</td>
<td>16.99</td>
</tr>
<tr>
<td>C2-30-R100</td>
<td>0.65</td>
<td>20.96</td>
<td>3.66</td>
<td>29.19</td>
<td>10.97</td>
</tr>
<tr>
<td>C3-30-R100</td>
<td>0.91</td>
<td>31.8</td>
<td>4.90</td>
<td>37.94</td>
<td>11.37</td>
</tr>
<tr>
<td>C2-30-R50</td>
<td>0.65</td>
<td>20.09</td>
<td>3.82</td>
<td>30.05</td>
<td>10.50</td>
</tr>
<tr>
<td>C3-30-R50</td>
<td>0.91</td>
<td>33.43</td>
<td>4.84</td>
<td>38.72</td>
<td>12.23</td>
</tr>
<tr>
<td>R1-50-R100</td>
<td>0.52</td>
<td>43.25</td>
<td>3.86</td>
<td>54.39</td>
<td>6.11</td>
</tr>
<tr>
<td>R2-50-R100</td>
<td>0.76</td>
<td>54.6</td>
<td>3.69</td>
<td>73.93</td>
<td>5.85</td>
</tr>
<tr>
<td>R1-30-R100</td>
<td>0.87</td>
<td>41.64</td>
<td>4.41</td>
<td>56.05</td>
<td>7.73</td>
</tr>
<tr>
<td>R2-30-R100</td>
<td>1.27</td>
<td>50.05</td>
<td>3.56</td>
<td>70.51</td>
<td>6.33</td>
</tr>
<tr>
<td>R1-30-R50</td>
<td>0.87</td>
<td>42.71</td>
<td>4.63</td>
<td>55.36</td>
<td>7.80</td>
</tr>
<tr>
<td>R2-30-R50</td>
<td>1.27</td>
<td>50.54</td>
<td>3.87</td>
<td>69.86</td>
<td>6.88</td>
</tr>
</tbody>
</table>

Similar to the flexural behaviour of NACFSTs, the results of the tested RACFST beams showed that the increase in the confinement factor leads to a significant increase for circular beams and moderate increase for rectangular beams in the moment capacity and flexural stiffness of RACFSTs. This can be explained as a result of the increased area of steel when the D/t and h/t ratios decrease which helps in delaying the local buckling and allowing the tested beams to yield at higher loads. However, the plastic behaviour of all tested specimens with higher and lower confinement factors after yielding was not much affected as all specimens showed a very ductile behaviour. The results presented in Table 4.2 showed that decreasing the D/t ratio for circular RACFST beams from 56 to 42 led to an average increase of 62% and 40% of My and Mu respectively. As for rectangular specimens, the decrease of the h/t ratio from 66.6 to 50 led to an average increase of 22% and 29% of My and Mu respectively. The flexural strength for circular beams is considerably improved more than the flexural strength for
rectangular beams due to the important role played by the better confinement provided by the circular steel tubes. Figure 4.9 showed that the strain distribution for RACFST beams with different confinement factors at 0.5*Mu, My and Mu is almost identical. Figure 4.10 showed that the mid span deflection of rectangular RACFSTs with higher confinement factor at My and Mu decreased slightly compared to lower values. Figure 4.10 also showed that the mid span deflection of circular RACFSTs with higher confinement factor at My and Mu increased slightly compared to the lower confinement factors. The failure mode for RACFSTs with lower and higher confinement factors were very ductile and did not suffer from tensile fractures or brittle failure. Figure 4.11 shows the failure mode for RACFST specimens with different confinement ratios.

Figure 4.11: Failure Modes for RACFSTs with different Confinement Factors
(a) Circular Beams (b) Rectangular Beams
Figure 4.12 and 4.13 compares the results of My and Mu between RACFSTs with different D/t and h/t ratios for circular and rectangular beams respectively.

Figure 4.12: Comparison of Ultimate and Yield Moments for Circular CFSTs with different D/t ratios, (a) Yield Moment (b) Ultimate Moment

Figure 4.13: Comparison of Ultimate and Yield Moments for Rectangular CFSTs with different h/t ratios, (a) Yield Moment (b) Ultimate Moment

4.3. Effect of Concrete Strength on the Flexural Behaviour of RACFSTs

All tested specimens shown in this section have a RCA Replacement percentage of 100%. Two concrete compressive strength values of 30 and 50 MPa were used to study the effect of the concrete strength on the flexural behaviour of RACFSTs. Figure 4.14 and 4.15 compares the moment vs. mid span deflection and moment vs. strain for RACFSTs with the different concrete strengths. The strain distribution and deflection curves at different moment values are shown in Figure 4.16 and 4.17 respectively.
Figure 4.14: Moment vs. Mid Span Deflection Graphs for RACFST Beams with Different Concrete Compressive Strength Values
Figure 4.15: Moment vs. Strain Graphs for RACFST Beams with different Concrete Compressive Strength Values
Figure 4.16: Comparison of Strain Distribution for RACFST Beams with different Concrete Compressive Strength Values
Figure 4.17: Comparison of Deflection Shapes at Yield & Ultimate Moments for RACFST Beams with different Concrete Compressive Strength Values
Table 4.3 presents the results of the yield moment (My) and the ultimate moment (Mu) and their corresponding deformations for RACFST beams with the different concrete compressive strength values.

<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>Yield Moment (My)</th>
<th>Deformation at Yield Moment (mm)</th>
<th>Ultimate Moment (Mu)</th>
<th>Deformation at Ultimate Moment (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1-50-R100</td>
<td>9.76</td>
<td>5.19</td>
<td>13.4</td>
<td>14.84</td>
</tr>
<tr>
<td>C1-30-R100</td>
<td>9.81</td>
<td>6.01</td>
<td>12.79</td>
<td>18.56</td>
</tr>
<tr>
<td>C2-50-R100</td>
<td>21.27</td>
<td>3.53</td>
<td>30.21</td>
<td>10.16</td>
</tr>
<tr>
<td>C2-30-R100</td>
<td>20.96</td>
<td>3.66</td>
<td>29.19</td>
<td>10.97</td>
</tr>
<tr>
<td>C3-50-R100</td>
<td>35.02</td>
<td>6.56</td>
<td>39.79</td>
<td>16.99</td>
</tr>
<tr>
<td>C3-30-R100</td>
<td>31.8</td>
<td>4.90</td>
<td>37.94</td>
<td>11.37</td>
</tr>
<tr>
<td>R1-50-R100</td>
<td>43.25</td>
<td>3.86</td>
<td>54.39</td>
<td>7.11</td>
</tr>
<tr>
<td>R1-30-R100</td>
<td>41.64</td>
<td>4.41</td>
<td>56.05</td>
<td>7.73</td>
</tr>
<tr>
<td>R2-50-R100</td>
<td>54.6</td>
<td>3.69</td>
<td>73.93</td>
<td>5.58</td>
</tr>
<tr>
<td>R2-30-R100</td>
<td>50.05</td>
<td>3.56</td>
<td>70.51</td>
<td>6.33</td>
</tr>
</tbody>
</table>

Figure 4.14 and 4.15 together with Table 4.3 show that the increase in the concrete compressive strength did not affect the moment capacity at the steel yield strain (My) for circular RACFST beams, whereas the increase of the concrete compressive strength slightly improved the moment capacity (My) for rectangular beams. This means that the effect of the concrete compressive strength on the flexural behaviour of RACFSTs at the elastic stage becomes more negligible as the confinement of the concrete infill is improved. However, the increase in the concrete compressive strength has slightly enhanced the ultimate moment capacity (Mu) for both circular and rectangular beams. Furthermore, the change in the concrete compressive strength did not lead to any change in the ductile behaviour of RACFSTs post yielding. Figure 4.16 which captures the strain distribution for RACFST beams at 0.5*Mu, My and Mu showed that the behaviour of the tested beams with different concrete compressive strength values was similar throughout the test. Figure 4.17 showed that the mid span deflection of circular and rectangular RACFSTs at yielding moment (My) was not affected by the increase of the compressive strength. However, the increase of the
concrete compressive strength has slightly decreased the mid span deflection at the ultimate moment capacity (Mu) for circular and rectangular RACFSTs. The change in the concrete compressive had no influence on the failure mode of RACFST beams as all specimens showed a very ductile and did not suffer from tensile fractures or brittle failure. The failure mode of RACFST beams with different concrete compressive strength values can be seen in Figure 4.18.

Figure 4.18: Failure Modes for RACFSTs with different Concrete Compressive Strengths  
(a) Circular Beams (b) Rectangular Beams
The results of My and Mu for RACFST beams with different concrete compressive strengths are given in Figure 4.19.

Figure 4.19: Comparison of Ultimate and Yield Moments for RACFSTs with different Concrete Compressive Strengths, (a) Yield Moment (b) Ultimate Moment

4.4. **Effect of RCA Replacement Ratio on the Flexural Behaviour of RACFSTs**

The results presented on this section investigate the effect of the RCA replacement percentage on the flexural behaviour of RACFSTs. All tested specimens shown in this section have a concrete compressive strength of 30 MPa and RCA replacement ratios of 50% and 100%. Figure 4.20 and 4.21 compares the moment vs. mid span deflection and moment vs. strain for RACFSTs with the two different RCA replacement ratios. The strain distribution and deflection curves at different moment values during the test are captured in Figure 4.22 and 4.23 respectively.
Figure 4.20: Moment vs. Mid Span Deflection Graphs for RACFST Beams with Different RCA Replacement Ratios
Figure 4.21: Moment vs. Strain Deflection for RACFST Beams with Different RCA Replacement Ratios
Figure 4.22: Comparison of Strain Distribution for CFST Beams with different RCA Replacement Ratios
Figure 4.23: Comparison of Deflection Shapes at Yield & Ultimate Moments for RACFST Beams with different RCA Replacement Ratio

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The results presented in Table 4.4 show the yield moment (My) and the ultimate moment (Mu) and their corresponding deformations for RACFST beams with RCA replacement percentage of 50% and 100%.

Table 4.4: Ultimate and Yield Moments and their Corresponding Deformations for RACFSTs Beams with different RCA Replacement Ratio

<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>Yield Moment (My)</th>
<th>Deformation at Yield Moment (mm)</th>
<th>Ultimate Moment (Mu)</th>
<th>Deformation at Ultimate Moment (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1-30-R50</td>
<td>9.73</td>
<td>4.90</td>
<td>13.61</td>
<td>15.18</td>
</tr>
<tr>
<td>C1-30-R100</td>
<td>9.81</td>
<td>6.01</td>
<td>12.79</td>
<td>18.56</td>
</tr>
<tr>
<td>C2-30-R50</td>
<td>20.09</td>
<td>3.82</td>
<td>30.05</td>
<td>10.50</td>
</tr>
<tr>
<td>C2-30-R100</td>
<td>20.96</td>
<td>3.66</td>
<td>29.19</td>
<td>10.97</td>
</tr>
<tr>
<td>C3-30-R50</td>
<td>33.43</td>
<td>4.84</td>
<td>38.72</td>
<td>12.23</td>
</tr>
<tr>
<td>C3-30-R100</td>
<td>31.8</td>
<td>4.90</td>
<td>37.94</td>
<td>11.37</td>
</tr>
<tr>
<td>R1-30-R50</td>
<td>42.71</td>
<td>4.63</td>
<td>55.36</td>
<td>7.80</td>
</tr>
<tr>
<td>R1-30-R100</td>
<td>41.64</td>
<td>4.41</td>
<td>56.05</td>
<td>7.73</td>
</tr>
<tr>
<td>R2-30-R50</td>
<td>50.54</td>
<td>3.87</td>
<td>69.86</td>
<td>6.88</td>
</tr>
<tr>
<td>R2-30-R100</td>
<td>50.05</td>
<td>3.56</td>
<td>70.51</td>
<td>6.33</td>
</tr>
</tbody>
</table>

The moment vs. mid span deflection and the moment vs. strain graphs shown in Figures 4.20 and 4.21 display that the change of the RCA replacement percentage from 50% to 100% has no significant effect on the flexural behavior of RACFSTs for both circular and rectangular beams. However, the figures also show that the effect of the RCA replacement percentage is more noticeable for the specimens with higher D/t and h/t ratios than the specimens of lower ratios. Similar to the effect of the concrete compressive strength, the effect of the RCA replacement percentage on the flexural behavior of RACFSTs depends on the confinement provided by the steel tube to the concrete infill. The better confinement of the steel tube to the concrete infill means the effect of the RCA replacement percentage on the flexural behavior of RACFSTs becomes more ignorable. Moreover, all the specimens presented in this section failed in a ductile manner regardless of the RCA replacement percentage. The change in the RCA replacement percentage also did not impact the strain distribution for RACFST beams during the testing of specimens as shown in Figure 4.22. Figure 4.23 showed that the mid span deflection of circular and rectangular RACFSTs at yielding moment (My) and ultimate moment (Mu) was not affected by the change in RCA replacement.
percentage for specimens with lower D/t and h/t ratios, while the mid span deflection increased slightly for specimens with higher D/t and h/t ratios. The change in the RCA replacement ratio did not affect the ductile behaviour of the tested beams and the failure mode for all specimens was similar to the other specimens mentioned in the previous sections of this research. Figure 4.24 shows the failure mode of RACFST beams with different RCA replacement ratios.

Figure 4.24: Failure Modes for RACFSTs with different RCA Replacement Ratios
(a) Circular Beams (b) Rectangular Beams
Figure 4.25 reports the results of My and Mu for RACFST beams with different RCA replacement percentages.

![Graph (a)](image-a)

![Graph (b)](image-b)

Figure 4.25: Comparison of Ultimate and Yield Moments for RACFSTs with different RCA Replacement Ratio, (a) Yield Moment (b) Ultimate Moment
Chapter 5. Analytical Analysis

This chapter addresses the applicability of using the theoretical equations provided by well-known design codes to predict the flexural capacity of RACFSTs. The results presented in this chapter compare the moment capacities of circular RACFST beams obtained from the experimental program conducted on this research to the theoretical nominal moment capacities ($M_n$) predicted using five design methods. The theoretical methods used in this research to predict the moment capacities of RACFST beams include the AISC-LRFD method [17], the Architectural Institute of Japan (AIJ) method [19], the Euro Code 4 method [20], the British Standard (BS) method [21] and the analytical equation proposed by Han [11]. The theoretical moment capacity ($M_n$) for circular RACFST beams tested on this research are calculated using the different equations provided by the design methods mentioned before. Each one of these methods uses different assumptions and approaches to predict the flexural capacity of CFSTs as explained in the following sections.

5.1. Design Methods

This section gives a brief demonstration for the different design methods used in this research to predict the flexural strength of RACFSTs.

5.1.1. AISC-LRFD method. The AISC specifications use two approaches for predicting the flexural strength of CFSTs: the plastic stress distribution and the strain compatibility method. The first approach assumes that the steel tube develops the yield stress in tension and compression whereas the concrete is subjected to a uniform compression of $0.85 f'_c$ for rectangular and square sections and $0.95 f'_c$ for circular sections taking into account the better confinement developed by circular sections. The second method uses the flexural strength calculation of a reinforced concrete section assuming linear strain distribution, a bilinear steel material curve and a parabolic unconfined concrete material curve. The flexural capacity of CFSTs using the plastic distribution method can be calculated as follows:

$$ M = F Z + \frac{1}{n} 0.95 f'_c Z \quad \text{(for circular sections)} \quad (1) $$

$$ M = F Z + \frac{1}{n} 0.85 f'_c Z \quad \text{(for rectangular sections)} \quad (2) $$
It can be noted from the previous equations that the AISC-LRFD method does not take into account the effect of the confinement provided by the steel tube on the compressive strength of the concrete infill.

5.1.2. AIJ method. This method calculates the flexural capacity of a CFST by adding the flexural capacity of concrete ($M_{ci}^c$) and the flexural capacity of steel ($M_{si}^s$). The AIJ method calculates the flexural capacity of concrete considering into account the effect of steel confinement on the compressive strength of concrete. The confined compressive strength of concrete ($\sigma_{ci}^c$) can be calculated as follows:

$$\sigma_{ci}^c = r_c \times f'_c \times \frac{1.56 \times t_s \times \sigma'_s}{D - 2t_s}$$  \hspace{1cm} (3)$$

where $r'_c$ is a reduction factor used by the AIJ equals to 0.85, $t_s$ and $D$ the steel tube thickness and Diameter respectively and $\sigma'_s$ is the steel yield strength. The flexural capacity of concrete ($M_{ci}^c$) is calculated by considering the angular location of the neutral axis ($\theta_n$), infill concrete diameter ($D_c$) and the confined strength of concrete ($\sigma_{ci}^c$) as shown below:

$$M_{ci}^c = \sin^3 \theta_n \frac{D_c^3 \sigma_{ci}^c}{12}$$  \hspace{1cm} (4)$$

The flexural capacity of the steel tube ($M_{si}^s$) is calculated by differentiating between the steel yield strength in tension and compression, where a modification factor of $\beta_1 = 0.89$ is introduced to reduce the steel yield strength in compression, and a modification factor of $\beta_2 = 1.08$ is used to increase the steel yield strength in tension. The flexural capacity of the steel tube is calculated by the following equation:

$$M_{si}^s = (\beta_1 + \beta_2) \sin \theta_n \frac{(1 - t_s)^2}{D} \frac{D^2 t_s \sigma'_s}{2}$$  \hspace{1cm} (5)$$

As seen from the above equations, the AIJ method considers the concrete confining effect to calculate the flexural capacity of a CFST member. Therefore, this method is considered an accurate method by many researchers to estimate the flexural capacity of CFSTs.
### 5.1.3 EuroCode4 method.

The resistance moment \( (M_n) \) of a circular CFST beam according to this method can be expressed by the following equation:

\[
M_n = w_{ps} * f_y + 0.5w_{pc} * f_c - w_{psn} * f_y - 0.5w_{pcn} * f'c
\]  

(6)

The first and second terms of this equation represents the maximum moment of CFST, while the third and fourth terms of the equation represents the neutral moment of CFST. Where \( w_{pc} \) and \( w_{ps} \) are the plastic modulus for concrete infill and steel tube respectively, while \( w_{pcn} \) and \( w_{psn} \) are the plastic modulus for concrete infill and steel tube at twice the neutral axis \( 2h_n \).

\[
w_{pc} = \frac{(D-2t)^3}{6}, \quad w_{p} = \frac{D^3-(D-2t)^3}{6}
\]  

(7)

\[
w_{pcn} = (D - 2t)h^2, \quad w_{psn} = Dh^2 - w_{pcn}
\]  

(8)

The following equation is used to determine the location of the neutral axis \( h_n \):

\[
h_n = \frac{A_s f_y}{2D f_c + 4h(2f_y - f_c)}
\]  

(9)

### 5.1.4 BS method.

This method considers the steel section plastic modulus \( S \) to calculate the flexural capacity of a CFST beam. A factor \( m \) is introduced to include the effect of the concrete compressive strength and the D/t ratio, which can be determined from a given chart. The moment capacity \( (M_n) \) can be calculated as follows:

\[
M_n = 0.91 S f_y (1 + 0.01m)
\]  

(10)

### 5.1.5 Han’s analytical method.

Han considers the confinement factor \( \xi \), the yield strength of the composite section \( f_{cy} \) and the plastic modulus of the steel section \( W_{scm} \) to estimate the flexural capacity of CFSTs. He proposed the following equation to calculate \( (M_n) \):

\[
M_n = \gamma_m W_{scm} f_{cy}
\]  

(11)

The terms of this equation can be calculated by the following equations:

\[
W_{scm} = \pi D^3/32
\]  

(12)

\[
f_{cy} = (1.14 + 1.02 \xi) f_{ck}
\]  

(13)

\[
\gamma_m = 1.1 + 0.48 \ln(\xi + 0.1)
\]  

(14)
Analytical Results

5.2. Analytical Results Comparisons

This section provides a comparison between the theoretical moment capacities of RACFST beams calculated by the above mentioned methods and the moment capacities obtained experimentally.

5.2.1. AISC-LRFD method comparison. Table 5.1 presents the values of the theoretical moment capacities of RACFST beams calculated using the AISC-LRFD method and the ratio of the theoretical moment \((M_n)\) to the yield \((M_y)\) and ultimate \((M_u)\) moments for each specimen.

Table 5.1: Comparison of AISC Theoretical Moments to Experimental Moments

<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>Yield Moment ((M_y))</th>
<th>Ultimate Moment ((M_u))</th>
<th>AISC Theoretical Moment ((M_n))</th>
<th>Theoretical to yield moment ratio</th>
<th>Theoretical to ultimate moment ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1-50-R100</td>
<td>9.76</td>
<td>13.4</td>
<td>11.60</td>
<td>1.19</td>
<td>0.86</td>
</tr>
<tr>
<td>C1-30-R100</td>
<td>9.81</td>
<td>12.79</td>
<td>11.17</td>
<td>1.14</td>
<td>0.87</td>
</tr>
<tr>
<td>C2-50-R100</td>
<td>21.27</td>
<td>30.21</td>
<td>26.50</td>
<td>1.24</td>
<td>0.88</td>
</tr>
<tr>
<td>C2-30-R100</td>
<td>20.96</td>
<td>29.19</td>
<td>25.5</td>
<td>1.22</td>
<td>0.87</td>
</tr>
<tr>
<td>C3-50-R100</td>
<td>35.02</td>
<td>39.79</td>
<td>34.08</td>
<td>0.97</td>
<td>0.85</td>
</tr>
<tr>
<td>C3-30-R100</td>
<td>31.8</td>
<td>37.94</td>
<td>32.8</td>
<td>1.03</td>
<td>0.86</td>
</tr>
</tbody>
</table>

The results predicted by the AISC method overestimated the yield moment capacity of the specimens by an average of 20% except for the beams with the lowest \(D/t\) ratio, where the theoretical results were very close to the yield moment capacity. The AISC-LRFD results considerably underestimated the ultimate moment capacity of RACFST beams by an average of 14%. Figure 5.1 provides a comparison of the ratio between the AISC theoretical results and experimental moments for all specimens.
Figure 5.1: Comparison of moment capacities of RACFSTs with AISC method
(a) Yield Moment (b) Ultimate Moment

522 AIJ method comparison. The nominal moment capacities calculated using the AIJ approach and the comparison with the experimental results are given in Table 5.2.

Table 5.2: Comparison of AIJ Theoretical Moments to Experimental Moments

<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>Yield Moment (My)</th>
<th>Ultimate Moment (Mu)</th>
<th>AIJ Theoretical Moment (Mn)</th>
<th>Theoretical to yield moment ratio</th>
<th>Theoretical to ultimate moment ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1-50-R100</td>
<td>9.76</td>
<td>13.4</td>
<td>13.22</td>
<td>1.35</td>
<td>0.98</td>
</tr>
<tr>
<td>C1-30-R100</td>
<td>9.81</td>
<td>12.79</td>
<td>11.91</td>
<td>1.21</td>
<td>0.93</td>
</tr>
<tr>
<td>C2-50-R100</td>
<td>21.27</td>
<td>30.21</td>
<td>32.01</td>
<td>1.50</td>
<td>1.06</td>
</tr>
<tr>
<td>C2-30-R100</td>
<td>20.96</td>
<td>29.19</td>
<td>27.81</td>
<td>1.32</td>
<td>0.95</td>
</tr>
<tr>
<td>C3-50-R100</td>
<td>35.02</td>
<td>39.79</td>
<td>38.41</td>
<td>1.09</td>
<td>0.96</td>
</tr>
<tr>
<td>C3-30-R100</td>
<td>31.8</td>
<td>37.94</td>
<td>34.35</td>
<td>1.08</td>
<td>0.91</td>
</tr>
</tbody>
</table>

In general, the theoretical moment capacities predicted by the AIJ method slightly underestimated the ultimate moment capacities of the RACFST beams by an average of only 5%. The consideration of the confined concrete strength in this method led to very close predicted moment capacities to the experimental results.
5.2.3. **EuroCode4 method comparison.** Table 5.3 presents the nominal moment capacities for RACFST beams obtained using the EuroCode4 method and the theoretical to experimental ratios.

Table 5.3: Comparison of EuroCode4 Theoretical Moments to Experimental Moments

<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>Yield Moment (My)</th>
<th>Ultimate Moment (Mu)</th>
<th>EC4 Theoretical Moment (Mn)</th>
<th>Theoretical to yield moment ratio</th>
<th>Theoretical to ultimate moment ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1-50-R100</td>
<td>9.76</td>
<td>13.4</td>
<td>11.75</td>
<td>1.20</td>
<td>0.87</td>
</tr>
<tr>
<td>C1-30-R100</td>
<td>9.81</td>
<td>12.79</td>
<td>11.27</td>
<td>1.15</td>
<td>0.88</td>
</tr>
<tr>
<td>C2-50-R100</td>
<td>21.27</td>
<td>30.21</td>
<td>27.02</td>
<td>1.27</td>
<td>0.89</td>
</tr>
<tr>
<td>C2-30-R100</td>
<td>20.96</td>
<td>29.19</td>
<td>25.82</td>
<td>1.23</td>
<td>0.88</td>
</tr>
<tr>
<td>C3-50-R100</td>
<td>35.02</td>
<td>39.79</td>
<td>34.57</td>
<td>0.99</td>
<td>0.87</td>
</tr>
<tr>
<td>C3-30-R100</td>
<td>31.8</td>
<td>37.94</td>
<td>33.10</td>
<td>1.04</td>
<td>0.87</td>
</tr>
</tbody>
</table>

The results acquired by the EuroCode4 equations were very close to those obtained by the AISC method. The EC4 results underestimated the ultimate moment capacity of RACFST beams by an average of 12%. The comparison of the ratio between the EC4 theoretical results and experimental moments for all specimens are given in Figure 5.3.
524. BS method comparison. The theoretical moment capacities obtained using the British Standard code and the theoretical to experimental ratios are given in Table 5.4.

Table 5.4: Comparison of BS Theoretical Moments to Experimental Moments

<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>Yield Moment (My)</th>
<th>Ultimate Moment (Mu)</th>
<th>BS Theoretical Moment (Mn)</th>
<th>Theoretical to yield moment ratio</th>
<th>Theoretical to ultimate moment ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1-50-R100</td>
<td>9.76</td>
<td>13.4</td>
<td>9.36</td>
<td>0.96</td>
<td>0.70</td>
</tr>
<tr>
<td>C1-30-R100</td>
<td>9.81</td>
<td>12.79</td>
<td>9.36</td>
<td>0.95</td>
<td>0.73</td>
</tr>
<tr>
<td>C2-50-R100</td>
<td>21.27</td>
<td>30.21</td>
<td>18.42</td>
<td>0.87</td>
<td>0.61</td>
</tr>
<tr>
<td>C2-30-R100</td>
<td>20.96</td>
<td>29.19</td>
<td>18.42</td>
<td>0.88</td>
<td>0.63</td>
</tr>
<tr>
<td>C3-50-R100</td>
<td>35.02</td>
<td>39.79</td>
<td>25.35</td>
<td>0.72</td>
<td>0.64</td>
</tr>
<tr>
<td>C3-30-R100</td>
<td>31.8</td>
<td>37.94</td>
<td>25.35</td>
<td>0.80</td>
<td>0.67</td>
</tr>
</tbody>
</table>

In general, the BS method significantly underestimates the ultimate moment capacity of RACFST beams. This result was expected as the equation provided by the BS method does not account for the concrete infill nor the composite action of CFSTs. Figure 5.4 gives a comparison between the theoretical and experimental results calculated using the BS method.
5.2.5. Han’s method comparison. The moment capacities obtained from Han’s analytical method and the ratios of theoretical moment capacities to experimental results are shown in Table 5.5.

Table 5.5: Comparison of Han’s Theoretical Moments to Experimental Moments

<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>Yield Moment (My)</th>
<th>Ultimate Moment (Mu)</th>
<th>Han’s Theoretical Moment (Mn)</th>
<th>Theoretical to yield moment ratio</th>
<th>Theoretical to ultimate moment ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1-50-R100</td>
<td>9.76</td>
<td>13.4</td>
<td>11.40</td>
<td>1.17</td>
<td>0.85</td>
</tr>
<tr>
<td>C1-30-R100</td>
<td>9.81</td>
<td>12.79</td>
<td>10.38</td>
<td>1.06</td>
<td>0.81</td>
</tr>
<tr>
<td>C2-50-R100</td>
<td>21.27</td>
<td>30.21</td>
<td>26.87</td>
<td>1.26</td>
<td>0.89</td>
</tr>
<tr>
<td>C2-30-R100</td>
<td>20.96</td>
<td>29.19</td>
<td>24.00</td>
<td>1.14</td>
<td>0.82</td>
</tr>
<tr>
<td>C3-50-R100</td>
<td>35.02</td>
<td>39.79</td>
<td>33.68</td>
<td>0.96</td>
<td>0.85</td>
</tr>
<tr>
<td>C3-30-R100</td>
<td>31.8</td>
<td>37.94</td>
<td>30.47</td>
<td>0.96</td>
<td>0.80</td>
</tr>
</tbody>
</table>

The moment capacities predicted using Han’s equation underestimated the ultimate moment capacities obtained from the experimental results by an average of 16%. However, the theoretical results obtained for specimens with higher concrete compressive strength were slightly better than specimens with smaller concrete compressive strength.
Figure 5.5: Comparison of moment capacities of RACFSTs with Han’s method
(a) Yield Moment (b) Ultimate Moment
Chapter 6. Conclusion and Future Work

The research on concrete-filled steel tubes has noticeably increased in recent years due to their rapid use in various structures such as high-rise buildings and bridges. The CFST structural members can be considered as an alternative system to the steel or reinforced concrete structures. Recycled aggregate concrete (RAC) has been used by many researchers to replace the natural aggregate concrete (NAC) in CFST members in order to improve the structural properties of RAC benefitting from the composite action developed between the steel tube and concrete infill. Many of the previous research focused on the behavior of RACFSTs under axial or combined loads.

This research investigated the flexural behavior of RACFSTs experimentally and analytically. A total of five NACFST beams and fifteen RACFST beams were experimentally tested to investigate the flexural behavior of RACFSTs. A comparison was made between the behavior of NACFSTs and RACFSTs when subjected to pure bending. In addition, three parameters affecting the flexural behavior of RACFSTs were considered in this research, namely, the confinement factor, the concrete compressive strength and the replacement percentage of recycled coarse aggregate (RCA). The use of RAC to replace NAC in CFST beams has been evaluated based on the experimental results and proved to be very effective and can be used in structures.

An analytical investigation was conducted in order to check the applicability of using the design methods provided by well-known design codes to estimate the ultimate moment capacity of RACFSTs. These methods include the AISC-LRFD method, the architectural Institute of Japan (AIJ) method, the Euro Code 4 method, the British Standard (BS) method and the analytical equation proposed by Han. Based on the results found in this research the following observations and conclusions can be drawn:

- All the tested RACFST specimens behaved in a ductile mode and the testing procedure progressed in a very smooth and controlled way. The composite action developed between the steel tube and the concrete core is responsible for the enhanced structural performance of RACFST specimens. For rectangular specimens, outward local buckling near or at the loaded points was noticed after the test. This behaviour was not noticed for circular beams. However, for all the tested beams, no tensile fractures appeared on the tension zone. Moreover,
the infill concrete for both circular and rectangular beams suffered from tension cracks at the tension zone and crushing of the concrete at the compression zone.

- The Comparison of the experimental results between NACFSTs and RACFSTs showed that the flexural behaviour of RACFSTs is very similar to NACFSTs in terms of the strain distribution, the mid span deflection, the moment capacity at steel yield and the ultimate moment capacity. Also, both the elastic and plastic stage for NACFSTs and RACFSTs were very similar. The circular RACFSTs achieved better results than rectangular ones as the ultimate moment capacity for circular beams was very close to circular NACFSTs, while the ultimate moment capacity for rectangular beams was slightly decreased compared to rectangular NACFSTs. These results show that confinement provided by the steel tube to the recycled concrete plays an important role to determine the flexural capacity of RACFSTs.

- The flexural capacity of RACFST beams significantly increased with the increase in the confinement factor value. This result can be attributed to the increased steel area which helps in delaying the local buckling and allowing the RACFST beam to yield at higher loads. However, the change in the confinement factor did not affect the behaviour of the tested specimens after yielding as the response of all beams was very ductile. The decrease in the D/t and h/t ratios by 25% had improved the ultimate moment capacity of circular and rectangular beams by an average of 40% and 29% respectively. The moment capacity for circular beams was considerably improved more than the moment capacity for rectangular beams as circular steel tubes provide better confinement to the concrete infill than rectangular ones. The experimental results also showed that the mid span deflection at the ultimate moment capacity for RACFST beams was slightly decreased for lower D/t and h/t ratios. The change in confinement factor had no influence on the failure mode of RACFSTs.

- The increase in the RAC compressive strength from 30 MPa to 50 MPa resulted in a slight improvement on the ultimate flexural capacity of RACFSTs for both circular and rectangular beams. This may indicate that the confinement effect provided by the steel tube diminishes as the concrete strength increases. Furthermore, the increase of the concrete compressive strength had
insignificant effect to the ductile behaviour of RACFST beams. Also, the increase in the concrete compressive strength had slightly decreased the mid span deflection at the ultimate moment capacity. The general failure mode of RACFST beams was not affected by the change in the concrete compressive strength.

- In general, Increasing the RCA replacement ratio from 50% to 100% had insignificant contribution to the flexural behaviour of RACFSTs. However, the effect of changing the RCA replacement ratio was more noticeable for RACFST beams with higher D/t and h/t ratios than beams with lower ratios. The change in the RCA replacement ratio did not affect the ductile behaviour nor the failure mode of RACFST beams.

- The ultimate moment capacity of RACFST beams could be predicted conservatively by using the equations provided by AISC-LRFD, AIJ, EC4, BS and Han’s proposed model. Overall, the theoretical moment capacities calculated using the AIJ method were the closest to the test results, as AIJ method underestimated the ultimate moment capacity by an average of only 5%. The EC4, AISC-LRFD and Han’s methods gave a moment capacity about 12%, 14% and 16% lower than the test results respectively. BS method significantly underestimated the ultimate moment capacities of the tested RACFST beams by an average of 34%.

For future work, additional parameters can be considered to thoroughly understand the flexural behavior of RACFSTs. One important parameter to be studied is the effect of the bond stress between the concrete infill and the steel tube as the increase in the bond stress will lead to a better confinement which plays an important role to determine the flexural behavior of RACFSTs. More parameters with regard to the recycled aggregate concrete can be investigated such as the size of the recycled coarse aggregate and the age of the recycled concrete. Furthermore, Finite Element (FE) simulation can be applied to conduct a parametric study to further the flexural behavior of RACFSTs.
References


18] A. Committee, A. C. Institute, and I. O. f. Standardization, "Building code requirements for structural concrete (ACI 318-08) and commentary," 2008: American Concrete Institute.


Vita

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In August 2016, he joined the Civil Engineering master's program in the American University of Sharjah as a graduate teaching assistant. His research interests are in flexural behaviour of CFSTs.