AUGUST 2017 M.Sc. in Civil Engineering FARIS MOHAISEN OLEIWI ALRIKABI

UNIVERSITY OF GAZİANTEP

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PERFORMANCE EVALUATION OF REINFORCED LIGHTWEIGHT CONCRETE FILLED STEEL TUBULAR BEAMS

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IN

CIVIL ENGINEERING

BY

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Supervisor

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Faris Mohaisen Oleiwi ALRIKABI

ABSTRACT

PERFORMANCE EVALUATION OF REINFORCED LIGHTWEIGHT CONCRETE FILLED STEEL TUBULAR BEAMS

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This study presents an experimental research to evaluate the flexural performance of Reinforced Lightweight Concrete Filled Steel Tubular beams. 16 beam specimens were tested, 12 specimens as Reinforced Lightweight Concrete Filled Steel Tubular (RLWCFST) beams, two specimens as Lightweight Concrete Filled Steel Tubular (LWCFST) beams and two specimens as Square Hollow Steel Tubular (SHS) beams. The major experimental parameters in the study are: (1) ratio of longitudinal steel reinforcement (ρ) from 3% to 7%; (2) arrangement of longitudinal steel reinforcement (in two or three layers), and (3) the spacing of stirrups (6cm or 12 cm). Based on the test results, the flexural strength, failure modes, ductility, moment-deflection relationships and moment-strain relationships were studied and their impact on the flexural performance of LWCFST and RLWCFST beams was discussed. The results indicated that the longitudinal reinforcement has important effect on bending performance of RLWCFST beams and provided obvious increasing in bending strength, ductility and stiffness of LWCFST beam and helped the steel tube and concrete to improve their performance. The experimental ultimate moments are compared to theoretical moments calculated by two international design codes namely the AISC-2010 code and the EC4-2004. The two design codes predicted good estimate and safe design moment capacities for RLWCFST beams especially EC4 that limited the maximum ρ with 6%. However, test results showed that exceeding this ratio (6%) up to 7% is possible and EC4 design method is applicable, even if the limitation of ρ was increased up to 7%.

Keywords: Lightweight Concrete (LWC), Reinforced Lightweight Concrete Filled Steel Tubes (RLWCFST) beams, flexural strength, performance indices, ductility.

ÖZET

HAFIF DONATILI BETON DOLDURULMUŞ ÇELIK BORU KOMPOZIT KIRIŞLERIN PERFORMANSININ DEĞERLENDIRILMESI

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66 Sayfa

Bu çalışmada donatılı hafif beton ile doldurulmuş çelik tüp kirişlerin eğilme performansını değerlendirmek için deneysel bir araştırma sunmaktadır. 16 kiriş test edildi, 12 adet numune donatılı hafif beton ile doldurulmuş çelik tüp kiriş, iki adet numune hafif beton ile doldurulmuş çelik tüp kiriş ve iki adet içi boş kare kutu profil kiriştir. Çalışmadaki başlıca deneysel parametreler şunlardır: (1) boyuna donatı yüzdesi (ρ) %3'ten %7'ye; (2) boyuna donatının düzenlemesi (iki veya üç katmanlı) ve (3) etriye aralığı (6cm ve 12 cm). Test sonuçlarına dayanarak, eğilme dayanımı, göçme modları, süneklik, moment-sehim ilişkileri ve moment-gerinim ilişkileri incelenmiş ve hafif beton ile doldurulmuş çelik tüp kirişlerin ve donatılı hafif beton ile doldurulmuş çelik tüp kirişler kirişlerin eğilme performansı üzerine etkileri tartışılmıştır. Sonuçlar, boyuna donatı donatılı hafif beton ile doldurulmuş çelik tüp kirişlerin eğilme performansı üzerinde önemli etkiye sahip olduğunu ve eğilme mukavemetinde, süneklikte ve hafif beton ile doldurulmuş çelik tüp kirişlerin rijitliğinde belirgin bir artış sağladığı ve çelik tüpün ve betonun performansının artmasına yardımcı olduğunu göstermiştir. Deneysel kırılma momentleri AISC-2010 kodu ve EC4-2004 olmak üzere iki uluslararası tasarım koduyla hesaplanan teorik momentlerle karşılaştırılmıştır. İki tasarım kodu, maksimum ρ'yı % 6 ile sınırlayan EC4-2004 özellikle donatı donatılı hafif beton ile doldurulmuş çelik tüp kirişleri için iyi tahmin ve güvenli tasarım moment kapasiteleri hesaplanmıştır. Bununla birlikte, test sonuçları bu oranın (% 6) % 7'ye kadar çıkmasının mümkün olduğunu ve ρ'nun sınırlandırılmasının % 7'ye kadar arttırılmış olsa bile EC4-2004 tasarım metodunun uygulanabileceğini göstermiştir.

Anahtar Kelimeler Hafif Beton, donatılı hafif beton ile doldurulmuş çelik tüp kirişler, eğilme dayanımı, süneklik, performans indeksleri.

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TABLE OF CONTENTS

pages

LIST OF FIGURES

Pages

LIST OF TABLES

Pages

CHAPER 1

INTRODUCTION

1.1 General

Steel and concrete are the most important constructional materials with different characteristics. Steel is distinguished by high tensile and compressive strengths, perfect ductility and great modulus of elasticity, resulting in small size sections and a long clear span structures where buckling may be the most significant problem as well as fire resistance. Concrete is distinguished by relatively high compressive but weak tensile strength, high fire resistance and low economic cost, resulting in massive bulk members where brittle crushing and premature cracking may be the most critical problems. Steel and concrete can compensate with each other and work together to form composite structural members with expected characteristics.

Concrete filled steel tubes have various composite cross-sections. Circular, square and rectangular sections are commonly adopted while polygonal sections may be used in some special cases. Conventionally, only plain concrete is used to fill the hollow steel sections. In addition, the concrete core can be reinforced by fiberglass-reinforced plastics and steel fiber to improve the strength and ductility of concrete, especially for high strength concrete. It can also be reinforced by steel rebar to enhance the ductility and fire resistance. For convenience, the steel rebar can be replaced by an internal steel tube which can supply higher confinement effect on the concrete core. Other steel sections, such as solid steel bars and I-sections, can be encased in the concrete core to improve the resistance and reduce section sizes. For members mainly subjected to bending, concrete filled double-tube sections can be used to increase the stiffness but with a relatively smaller volume of materials.

1.2 Concrete-filled steel tubes

Concrete-filled steel tubes are composite members, in that two different materials work together to support loads and provide stiffness. Generally, a concrete-filled tube is either a circular, rectangular or square steel tube filled with concrete. The combination of steel and concrete in this manner provides excellent flexural and axial capacities, and buckling resistance. The steel tube also provides excellent confinement for the concrete which results in increased compressive strength and strain capacity of the concrete, which in turn increases the ductility of the member. Also, concrete core delays local buckling of steel tube. The total weight of material for conventional reinforced concrete structure can be significantly larger than the total weight of material in a concrete-filled steel tubes structure with the same resistance. As a result of a lower building weight, the lateral forces that need to be designed for will be lower. In addition to a reduced overall weight, another benefit is the speed and ease of construction. The steel tube and framing elements in a building can be erected several stories prior to casting the concrete.

1.3 Lightweight concrete (LWC)

Light weight concrete has been defined as a type of concrete with an expansion factor that increases the size of the mixture with additional qualities such as dead weight reduction. Light weight concrete is of paramount importance to the construction industry. Current concrete studies focus on high performance concrete, such as cost-effective materials that meet the required performance requirements, including durability. Lightweight concrete lighter than conventional concrete. Low density and thermal conductivity are the main specialties of lightweight concrete. Therefore, its features are that there are lower transportation and handling costs, lower dead load and faster construction.

Light weight concrete is a versatile material that consists mainly of cement mortar based on mixed with at least 20% of air volume. Lightweight concrete is being used in an increasing number of implementations, starting with one-step casting house to fill low-density vacuum. The average normal concrete density is 2,400 kg / m3, while the lightweight concrete density ranges from (1800, 1700, 1600) kg / m3 to

300 kg / m3. The compressive strength ranges from 40 MPa to nearly zero for really low intensity

1.4 Objectives of research

The objective of study is to investigate performance evaluation of reinforced lightweight concrete filled steel composite beams in small scale experiments. Therefore, more investigations should be conducted to evaluate the performance of concrete-filled steel tubes with high strength fiber composite materials and assess their potential applications in multi-story, high-rise or long span constructions.

Despite the excellent engineering properties of concrete-filled steel tubes, they are not as widely used as traditional structural steel and reinforced concrete members. Although much research has been performed on the topic, the amount of information regarding concrete-filled steel tubes is significantly less than that available for traditional steel or reinforced concrete members. As a result, the design procedures have been in part based on these traditional systems. In addition, Current design codes and standards provide small information on the flexural behavior of concrete filled steel tubes. Thus, the major targets of this study are three folds: 1) to study new test data on reinforced lightweight concrete-filled steel tubular beams, 2) to research the flexural behavior of reinforced lightweight concrete-filled steel tubular beams in terms of moment capacity, ductility and failure modes, 3) to compare the test results with that calculate by codes and try to push up the reinforcement ratio limits that stated in the codes.

1.5 Importance of research

At the present time, world requires high performance structures and structural units to satisfy demands of increasing population and propose cost effective solutions. From the beginning of 20th century concrete and steel have been the most popular structural materials to build residential or commercial structures. In addition, study of Reinforced Concrete Filled Steel Tubular beams is significant for another several reasons. First, current design codes and standards provide a little information on the flexural behavior of Reinforced Concrete Filled Steel Tubes. Second, there is a little research on Reinforced Concrete Filled Steel Tubular Beams. Third, according to the authors' knowledge the performance of reinforced lightweight concrete-filled steel tubular beams had not been reported yet and the research in this field is still in the primary stage.

This study presents an experimental research on reinforced lightweight concrete filled steel tubular beams with different size of steel bars (8 mm,10 mm and 12 mm nominal diameter) and stirrups of diameter (5 mm) at different spacing (6 cm and 12 cm), lightweight concrete filled steel tubular beams and hollow section beams as shown in Figure 1.1. The study consists of 16 beam specimens of square section (120 x 120mm nominal outer dimensions) and (700mm) in length that can be detailed as follows:

- Square Hollow Steel Tubular beams, (two specimens).
- Lightweight Concrete Filled Steel Tubular beams, (two specimens).
- Reinforced lightweight reinforced concrete filled steel tubular beam with different size of steel bars, different distribution of steel bars (two layers or three layers) and different spacing of stirrups (6 cm or 12 cm), (twelve specimens).

Figure 1.1 Sixteen beam specimens of square section for this study

1.6 Layout of thesis

This thesis summarizes the research performed on performance evaluation of reinforced lightweight concrete filled steel tubular beams. This report contains five chapters covering, introduction, previous research, assessment of design codes, experimental testing, data interpretation and conclusion.

Chapter 1 introduces the motive for the present study and clarifies the objectives to be followed up in this M.Sc. project.

Chapter 2 summarizes previous research studies on the topic of thesis in detail, like composite members, concrete-filled steel tubes, lightweight concrete, lightweight concrete-filled steel tubes, reinforced lightweight concrete-filled steel tubes and analysis for composite members.

Chapter 3 describes the current design specifications for concrete-filled steel tubes, according to American Concrete Institute's (ACI) code, the American Institute of Steel Construction (AISC), and (EC4) Euro code provisions.

Chapter 4 describes the experimental program. It details the material properties of specimens, the construction procedure, the test setup and discusses the observed performance of the experimental tests. Comparisons between specimens are also made.

Chapter 5 summarizes the main conclusions achieved in this study from experimental results with a brief summary of the research results.

CHAPTER 2

LITERATURE REVIEW

2.1 General

This chapter displays a review of studies carried out on concrete-filled steel tubular members. The discussion is fundamentally concentrated on flexural members at static loading condition, cyclic loading condition and analytical consideration. assurance is given on ultimate moment capacity, ductility, energy absorption and stiffness of the composite members.

The studies discussed in this chapter are divided mainly in six sections. The first section reviews the experimental and theoretical studies put through by many researchers on composite beam members. The second section reviews studies put through by many researchers on concrete-filled steel tubular sections. The third section reviews studies on lightweight concrete. The fourth section reviews studies on lightweight concrete-filled steel tubes. The fifth section includes the studies on reinforced concrete-filled tubes. Finally, the sixth reviews studies carried out by different researchers on analysis for concrete-filled steel tubes.

2.2 Composite beam members

2.2.1 General

Composite steel–concrete beams are the first form of the composite construction. steel sections encase by concrete were initially evolved to resolve the problem of fire resistance and to secure the stabilization of the steel section during loading. Steel section and concrete work together to withstand pivotal force and flexure. A composite member was evolved because they supplied lasting and complete formwork and were useful in decrease time of construction and thus costs.

2.2.2 Composite beams

Nguyen (1991) carried out a study on the feasibility of utilizing steel channels as reinforcement for concrete beams. The empirical program consists of thirty-two beams were made of thin-walled steel stiffened channels and concrete subjected to shear force and bending moment. It can be concluded that, the composite beams evolved the same strength in bending and shear force as traditional reinforced cement concrete beams with economy and quickly construction.

Oehlers (1993) presented a study on the strength of composite profiled beams by using the steel profiled sheets as lasting formwork to the bottom and sides of reinforced concrete beams. The study consists of six large-scale beams, three specimens of beams for flexural tests and three for shear tests. From the test results, it can be noticed that the profiled steel sheets increase the shear strengths and flexural without losing in ductility.

Nie et al. (2004) studied the behavior of composite beams under negative bending with various degrees of shear interaction. The experimental study consists of three simply supported composite beams, 4 m long each and I20 steel beam and a slab of 800 mm x 3110 mm with profiled sheeting. The results indicated that sliding exists for composite beams under negative bending and the sliding influences in an extra curvature of the bending in beam and minimizes the section stiffness in rate of 10% to 20% when compared with a beam without any sliding in serviceability condition.

Ipe et al. (2012) made an experimental study on flexural behavior of composite beams. The experimental program consists of simply supported beams with effective span of 1440 mm with composite box and channel sections under two-point loading. The results indicated that the strength to weight ratio of composite beams is much higher than reinforced concrete beams and ductility index is also more than reinforced concrete beams and empty beams.

Vinay et al. (2015) studied the flexural behavior of the steel-concrete composite beams experimentally. The authors carried out this experimental investigation on eight simply supported beam specimens to understand the flexural performance. Two beams were control beams and the remaining six beams were composite beams. Different configuration of the shear transfer mechanism provided to six composite beams. The span to depth ratio ranging from 6 to 9 and shear span to depth ratio ranging from 2.5 to 3. Also, f_c=30 MPa for concrete and f_y =415 MPa for steel. The authors used T-shear connectors to establish the connection between the steel and the concrete section with three different configurations. Researchers studied the cracking load, load-deflection behavior, ultimate load and mode of failure for beam specimens by subjecting them to two-point loading. The results indicated that, the load carrying capacity of the composite beams were increased about 38.09% to 214.28%., the span to depth ratio and shear span to depth ratio have an impact on the increase in the load carrying capacity of the composite beams. The deflection of mid-span for the composite beams at ultimate load were reduced by 50% when compared with control beams.

Kumar and Reddy (2016) studied the steel-concrete composite beams in bending experimentally. These researches were carried out to study the behavior of steel concrete composite beam with various spacing of shear connectors subjected to pure bending. The researchers analyzed the behavior of beams with various shear connector spacing, 75 mm, 100 mm, 125 mm and 150 mm under pure bending up to the failure. The researchers found that the beam having 125 mm spacing has higher ultimate bending moment, the value of flexural rigidity decreases with the increase in the moment. The enhancement in flexural stiffness because of the closely spaced shear connector which contributed additional confinement.

2.2.3 Lightweight concrete composite beams

Assi et al. (2002) studied the partially encased composite beams. The study was carried out both theoretically and experimentally. Twelve beams of four various sizes in length of 2.0m were tested. The beam specimens were loaded by two-point loading. Four of the simply supported beams were partly covered in LWC and four with normal concrete and three were tested as a hollow steel sections. The results of this study showed that, the utilize of normal mix concrete showed little enhancement to the flexural strength of the tested composite sections when compared to LWC.

Hossain (2003) studied the behavior of thin walled composite filled beams with normal and LWC as in-fill experimentally and theoretically. The strength and modes of failure for beams are found to rely on the interface connections. Also, the results of testing showed that the beam’s strength is limited by the compression buckling capacity of the steel plate at the top of the open box cross-section.

2.2.4 Conclusion

Previous studies indicate the development of the steel channel as part of the side moldings and the replacement of conventional steel bars. Then the shape became a profiled composite beam where the steel profiled sheets form a permanent working on the sides and bottom beams. It was observed that bending strength increases in these types of beam without loss of elasticity.

2.3 Concrete-filled steel tubes

2.3.1 General

A steel tube member filled with concrete is an innovative idea, where the steel element works together with a concrete component, so that both elements resist axial loads and flexural. These structures carry loads far in excess of their design loads, but are still in-service due to their conservative design of modern standards. This review aims to investigate the structural improvements of hollow steel tubes filled with concrete.

2.3.2 Bond strength

Prion and Boehme (1993) Conducted an investigation into concrete-filled steel tubes in bending. The results indicated that the specimen dissipated a large amount of energy with a small decrease in strength when the loading cycle progressed. The strength of steel concrete-filled tubes within subsequent cycles was not significantly affected by sliding between the two materials. The beam specimens showed a loss of hardness due to lack of bond and cracking of concrete after the first cycle.

Roeder et al. (1999) Studied composite work in steel tubes filled with concrete. Reference was made to the importance of bond stress and behavior of interface conditions. It has been shown that deflation can be very detrimental to the ability of bond stress and the importance of shrinkage depends on the properties of concrete, tube diameter and the surface condition of the inner tube. The bond capacity was smaller in large diameter tubes and large d / t ratios. Bond capacity was correlated with sliding on the surface of steel concrete.

Naguib and Mirmiran (2003) presented a study on the modeling of the crawling of concrete-filled steel tubes using the rate of the flow method and the double energy law of the basic creep of the concrete. The suggested model showed good compatibility with previous crawling tests on bonded and un-bonded tubes filled with concrete. For diameter to the thickness ratios of 40 or less, the bonded tubes are more coherent in cracking the crawling than those unbound equivalents of the same volume of continuous loads.

2.3.3 Slenderness limits

Elchalakani et al. (2002) studied the plastic slenderness limits for circular hollow sections and reviewed the existing data on plastic slenderness limits. Plastic bending tests were performed on various sizes of circular hollow sections with variable d/t from 13 to 39. The test results and design rules given in various codes were compared. The plastic slenderness limit of λ_p = 60 was acquired for cold-formed circular hollow sections under bending.

Elchalakani et al. (2002) reported a study on bending tests to determine the limits of slenderness for cold-formed circular hollow sections. A total of twelve bending tests were performed up to failure with variable diameter-to-thickness ratio d/t from 37 to 122. The authors compared the test results with other experimental data and the design rules given in various steel specifications and developed a design curve for cold-formed circular hollow sections under pure bending.

Elchalakani et al. (2004) studied the periodic inelastic flexural behavior for the cold-formed circular hollow section beams. The tests were conducted on various sizes of compact circular hollow sections with variable d/t value from 20 to 162. The circular hollow section beams indicated stabilized hysteresis behavior up to local buckling and then exhibited a significant deterioration in the strength and ductility depending upon the d/t ratio.

Lai et al. (2014) presented an experimental database, design, and analysis study on non-compact and slender rectangular concrete filled tubes. According to the slenderness ratio of the steel tube walls can be classified as compact, non-compact or slender in relying on the slenderness ratio of the steel tube walls. The authors observed that the AISC 360-10 equations were suitable for classing rectangular concrete filled tubes into compact, non-compact or slender sections for flexure and axial compression.

Lai and Varma (2014) carried out an experimental database, design, and analysis study on non-compact and slender circular concrete filled tubes. The researchers study the basis of the current AISC-360-10 specification for the design of non-compact and slender circular concrete filled tube members under axial compression, flexure, and combined axial and flexural loading. The researchers observed that the AISC 360-10 equations design can be utilized to compute the strengths of circular concrete-filled tubular steel members.

Guo et al. (2013) studied the behavior of thin-walled circular hollow section tubes subjected to bending experimentally. A total of 16 specimens were tested up to failure on various sizes of circular hollow section with D/t ratio ranges from 75 to 300. The researchers found that the specimens with small ratios D/t =75 failed by wide plastification on the central part of the tube and for specimens with D/t ratio over 200, the load carrying capacity decreased due to the initial geometrical imperfections.

2.3.4 Ductility, stiffness and energy absorption capacity

Elchalakani et al. (2001) carried out a series of test on concrete filled tubes subjected to bending where d/t ratio of the hollow steel section ranges from 12 to 110. This investigation compared the behavior of bare and void-filled, circular hollow sections under pure elastic bending. It was noticed that void fillings increased flexural strength, ductility and energy absorption capacity especially in thinner sections.

Gho and Liu (2004) conducted experiments on twelve, 1600 mm long rectangular hollow steel section filled with high strength concrete beams. The specimens subjected to pure bending up to failure. In this test, three various sizes of hollow steel sections, filled with high-strength concrete were used. From test results, the following conclusions were obtained, a good ductility performance of the specimens was noticed in all tests.

Arivalagan and Kandasamy (2009) studied the behavior of rectangular hollow sections unfilled and filled with concrete with respect to ductility, energy absorption under cyclic reversible loading. From the results of study, it can be noticed that void filling increases energy absorption capacity and the factor of ductility.

2.3.5 Moment capacity

Han (2004) conducted an empirical study on sixteen concrete filled steel rectangular and square section beam specimens. This experiment helps not only to determine ultimate moment capacity but, also to determine the failure mode beyond the yield load. From this test, the following conclusions are obtained, due to the infill of concrete, the steel rectangular and square section behave in a ductile manner. The in-filled concrete also increases the ultimate moment value.

Soundararajan and Shanmugasundaram (2008) studied 50 specimens of steel hollow sections as a simply supported beam filled with deferent types of concrete (normal mix, fly ash, quarry waste and low strength concrete) and had the same section dimensions. From the experimental results, it can be found that void filling with these types of concrete enhance the moment carrying capacity and increase the flexural capacity of steel hollow sections.

Arivalagan and Kandasamy (2010) presented a study on the testing of void-filled square section beams under periodic loading. Specimens of steel grade 310 with nominal yield stress equal to 345 MPa, depth/thickness ratios equal to 20.5, thin walled section with 3.2mm thickness and (normal mix concrete and fly ash concrete) as a filler material. The results of study showed that ultimate moment capacity of void filled square section beams depends fundamentally on the filler material strength.

Guler et al. (2012) studied six hollow beams and nine super high performance concrete beams under bending. According to these results, increase in moment capacity and corresponding curvature is much greater for thinner square hollow steel section beams than for thicker ones.

Ghannam (2016) studied flexural strength of concrete-filled steel tubular beam with partial replacement of coarse aggregate by granite. Six specimens with strength of concrete as 20 MPa were tested under bending action. The results indicated that beams filled with partial replacement of coarse aggregate by granite is almost the same as that of traditional concrete.

2.3.6 Conclusion

The concrete filled beam sections under loading increases ductility, stiffness and energy absorption capacity when compared to hollow steel beam sections. Also, concrete filling increases the strength of concrete filled beam members and prevents the local buckling. It is also noticed that the bond in concrete filled beam sections is greatly affected by the kind of concrete. It is also observed that the AISC 360-10 equations were suitable for categorizing rectangular concrete filled tube members into compact, non-compact or slender sections for flexure.

2.4 Lightweight concrete

2.4.1 General

Many studies are being conducted to study various properties of lightweight concrete. This section presents a review of the literature on the properties of lightweight concrete, lightweight aggregate and mixing ratio.

2.4.2 Lightweight aggregate

Chi et al. (2003) studied the influence of aggregate properties on the strength and hardness of light weight concrete. Three types of aggregates with different fly ash contents were studied and empirical data were statistically analyzed. Two important factors influencing the compressive strength and elastic modulus of the concrete were the properties of the light weight aggregate and the water / binder ratio.

Kan and Demirboga (2009) conducted an experimental study to evaluate the usability of modified waste expanded polystyrene aggregates as an aggregate for concrete and other secondary construction materials. The advantages of use this type of lightweight aggregate are: lowering in the cost of construction materials and cost of waste disposal.

Ramesan et al. (2015) presented a study on performance of lightweight concrete with plastic aggregate to research the appropriateness of recycled plastics as coarse aggregate in concrete by carried out different tests such as workability by slump test, compressive strength of cube and cylinder, splitting tensile strength test of cylinder. From results, it can be observed that plastic aggregate is a lightweight material with specific gravity 0.94 and the workability of concrete increased by 50% for a mix containing 40% plastic aggregate.

2.4.3 Lightweight concrete

Saritha and Chamundeeswari (2011) conducted an experimental study of lightweight concrete by partial replacement of coarse aggregate using pumice aggregate. The study was an attempt to compare between the conventional concrete and lightweight aggregate concrete using mix M25. Lightweight concrete is made by partial replacement of coarse aggregate with different ratios of pumice ranging from 50%, 60% and 70%. The researchers found that the maximum value of strength was obtained in 60% replacement of pumice with coarse aggregate.

Mohammed and Hamad (2014) presented a review of the classification of lightweight concrete. They classified the lightweight concrete into two types according to production methods and utilization purpose and focused on the materials used to obtain lightweight concrete. Also, they discussed the applications of lightweight concrete. They made a comparison between the two types of concrete (conventional concrete and lightweight concrete). The advantages of lightweight concrete were higher strength to weight ratio, reinforced in thermal and sound insulation.

Avinash and Dhinakaran (2015) presented a study on high performance lightweight concrete. This type of lightweight concrete was produced by two ways. First, by using air entraining agent, in which air entraining agent was added as additive in different percentages and the second by using lightweight aggregate, in which the coarse aggregate was partially replaced with different percentages of expanded clay as light weight aggregate. Eight different combinations of mixes at three different ages of concrete 7, 14 and 28 days were studied. It can be noticed that high performance lightweight concrete produced with air entraining agent was better than that of concrete produced with light weigh aggregates.

Pravallika and Rao (2016) studied the strength properties of lightweight concrete using lightweight aggregate. A comparison made between the properties of conventional M40 concrete with properties of concrete with lightweight aggregate, produced by replacing coarse aggregate with pumice stone by 0%,10%,20%,30%,40% and 50%. The researchers observed that the increasing percentage of pumice stones will show negative impact on strength of concrete and for replacement of 20% of pumice stone gives optimum value beyond 20% the compressive strength value decreases.

2.4.4 Conclusion

The different advantages of lightweight concrete and aggregate will pave the way for selection of a suitable alternate material for construction industry. Studies concentrate on high performance concrete, such as a cost of material that satisfies demanding performance requirements, including durability.

2.5 Lightweight concrete-filled steel tubes

2.5.1 General

Many studies showed that the concrete filled steel tubes are affected due to their concrete core. Use lightweight concrete as a filling material avoid problem of wastage and reduces the demand of natural aggregate. In addition, lightweight concrete has many additional advantages as mentioned above in section 2.4.

2.5.2 Lightweight concrete-filled steel tubular beams

Hunaiti (1997) reported a research on hollow steel sections filled with foamed and lightweight aggregate concrete. This study covers eight numbers of square and circular columns and eight numbers of square simply supported beams. It aimed to compute the ultimate moment capacity of filled specimens. Unfilled specimens of similar sections were also tested for comparison purpose. From this test, the beam specimens filled with foamed and lightweight aggregate concrete behave with flexural manner and developed excess moments of the theoretical values.

Zhao and Grzebieta (1999) conducted an experimental investigation of void-filled square section beams subjected to a large deformation periodic bending. The specimens used in the tests were of different d/t ratios. The filler materials were normal concrete, polyurethane and lightweight concrete. The results indicated that increase in the ultimate moment capacity fundamentally depended upon the strength of the filler material.

Ji et al. (2013) studied the stability behavior of light weight aggregate concrete filled steel tubes. The failure modes and deflection were studied and their influence on the flexural behavior of steel tubes filled with lightweight aggregate concrete was analyzed. Filling with lightweight concrete can clearly improve the flexural behavior of steel tube. The moment capacity of lightweight concrete filled steel tube increased as the steel ratio and lightweight concrete strength increased.

2.5.3 Conclusion

Lightweight concrete increases the moment capacity of steel tube as the steel ratio and lightweight concrete strength increased. It is also observed that the bond reduction due to age in normal concrete specimens is higher than that of lightweight aggregate concrete specimens.

2.6 Reinforced concrete-filled tubes

2.6.1 General

The research on reinforced concrete filled tube beams can be divided into two types. The first type is concrete-filled fibre reinforced polymer tubes with internal reinforcement of fiber reinforced plastic bars or steel bars and the second type of research was reinforced concrete filled tube beams.

2.6.2 Reinforced concrete-filled fibre-reinforced polymer tubes

Mohamed and Masmoudi (2010) presented an experimental and theoretically study consists of four beams reinforced with conventional steel bars while six reinforced with glass fiber reinforced plastic bars. The parameters of this study were the type of internal reinforcement, type of transverse reinforcement, thickness of fibre polymer tube and concrete compressive strength. The researchers showed that all the concrete-filled fibre reinforced polymer tubes beams failed in flexural, the strength, deflection and ductility were influenced significantly by the axial stiffness of the fibre reinforced polymer tubes.

Abouzied and Masmoudi (2017) conducted an experimental and analytical investigations on flexural behavior of rectangular fibre reinforced polymer tubes filled with reinforced concrete. Eight full-scale rectangular concrete-filled fiber-reinforced polymer beams and two reinforced concrete beams were tested under a four-point bending. The results indicated that wonderful performance of the concrete-filled fiber-reinforced polymer beams in terms of strength and ductility compared to the reinforced concrete beams.

2.6.3 Reinforced concrete-filled steel tubes

Xiamuxi et al. (2014) studied the effect of axial reinforcement on bending performance. The steel tube was with yield strength f_{sy} = 314 MPa and thickness t = 3.2 mm. Axial reinforcement with yield strength f_{sr} = 352 MPa and diameter ds = 6 mm. The ratio of axial reinforcement ρ for reinforced concrete filled steel tubular beams was on the range of $0.8\% \leq \rho \leq 6.0\%$. Transverse reinforcement was with yield strength f_{sl} = 304 MPa and diameter ds = 3 mm. The researchers observed that the axial reinforcement ratio of reinforced concrete filled steel tubular beams helped the steel tube and concrete to improve their performance.

Joseph et al. (2016) studied the flexural behavior of concrete filled steel tubular beams with and without reinforcement. It can be noticed that a concrete filled steel tubular beam with reinforcement resists tension, bending moments and also increases load carrying capacity when compared to a normal reinforced concrete and steel section of similar dimensions.

2.6.4 Conclusion

From the previous studies mentioned above on reinforced concrete-filled tubular beams it's quite clear that there is a lack in research about the flexural performance of reinforced concrete filled steel tubular beams. Also, to the authors' knowledge the performance of reinforced lightweight concrete-filled steel tubular beams had not been reported yet, therefore this study represents an attempt to investigate and understand the behaviour of reinforced lightweight concrete-filled steel tubular beams with different parameters.

2.7 Analysis for concrete-filled steel tubes

Al-Rodan and Al-Tarawnah (2003) studied finite element analysis of the flexural behavior of rectangular tubular sections filled with high-strength concrete as a three-dimensional, geometrically and materially nonlinear finite element model when used as a beam. It can be observed that methods of design for EC4 with high strength concrete beams appear generally on the safe side. The finite element load-displacement curves for various high strength concrete filled beams show satisfactory agreement with those of tests.

Arivalagan and Kandasamy (2010) presented a study on finite element analysis on the flexural behavior of concrete filled steel tube beams. Numerical analyses have shown that for square concrete-filled steel tube beam a good confining impact can be provided. This effect is enhanced especially by the filling. Finite element load-deflection curves for different mix concrete filled beams show satisfactory agreement with those of tests.

Moon et al. (2012) reported a study on analytical modeling of bending of circular concrete-filled steel tubes. The study developed a finite element model for concrete filled steel tube subjected to bending because the experimental results alone are not enough to backing the engineering of these components. According to study's results, it can be concluded that, plastic-stress distribution method provides rationally conservative moment capacity for concrete-filled steel tube beams under bending.

Vijay and Chitawadagi (2014) presented a study on finite element analysis of concrete filled steel tube subjected to flexure. The finite element model predicts moment capacity at ultimate point for circular, rectangular concrete filled steel tube beams fully agree with those determined from actual experiments. The predicted value moment capacities for circular and rectangular cross sections found to give good results without conducting experiments.

CHAPTER 3

DESIGN CODES AND SPECIFICATIONS FOR COMPOSITE MEMBERS

3.1 General

This chapter describes the current design methods available for concrete-filled steel tubes, especially the flexural strength. The current design specifications of concrete filled steel tubes, according to American Concrete Institute's (ACI 318-14) code, the American Institute of Steel Construction (AISC-360-10), and (EC4) Euro code provisions, these specifications will be compared with experimental results later in chapter 4.

3.2 American Concrete Institute (ACI)

The ACI building code was the only major reference in the United States for the design of composite members until the AISC-LRFD specification was published in 1986. The ACI code and AISC-LRFD specifications provide design rules with regard to composite members in the United States. The ACI building design code (ACI 318-14) provides only one method, it is the strain compatibility method (SCM), which is one of the two methods that AISC provides. In the circular section of composite members, The ACI method regards the concrete filled tube as regular reinforced concrete by replacing the steel tube with the same amount of reinforcement bars. Based on the assumption of 0.003 of concrete compressive strain, the moment capacity of the CFT beam is computed. Furthermore, according to ACI 318-14, the relation between concrete strain and concrete compressive stress distribution can be assumed in several shapes, such as rectangular or trapezoidal. Generally, the rectangular compressive stress distribution of concrete is the most common assumption. The ACI method supposes that the compressive uniform stress of concrete is $0.85f_{\grave{c}}$ as Whitney rectangular stress distribution, where $f_{\grave{c}}$ is the specified compressive strength of concrete.

3.3 American Institute of Steel Construction code formulae (AISC)

AISC-360-10 provides two methods for determining the flexural strength of a composite member. The plastic stress distribution method (PSDM) which gives a simple and comfortable calculation method for the most common design situations and the strain compatibility method provides a general calculation method. To determine the nominal strength of the composite sections, the plastic pressure distribution method is adopted. However, for special situations such as when a cross-sectional area changes the method of strain compatibility will be the best option. When using this method, the local buckling of the filled composite members should be considered. The AISC-360-10 determines that for (PSDM), the steel tube reaches yield and the strength of concrete infill is 0.85 $f_{\grave{c}}$.

3.3.1 Strain compatibility method (SCM)

The strain compatibility method (SCM) assumes a complete composite work between the steel and concrete sections such as the plastic stress distribution method. The stress at any point in a composite member must correspond to the strain at the point. The maximum strain is supposed to be at the extreme compression fibers of 0.003, and the compressive stress distribution of concrete is represented by the Whitney stress block. Similar to the plastic stress distribution method, the contribution of concrete tensile strength is deleted. Linear elastic is perfectly behavior for the steel section and longitudinal reinforcement. Hence, for the strain compatibility method we can summarize the following assumptions.

1. The distribution of strain across the section is linear.
2. The maximum flexural strength, M_n*(s.c.),* corresponds to a strain of 0.003 in./in. at the most extreme concrete compression fiber.
3. The steel and concrete stress distribution shall be obtained using stress-strain relationships from previous tests or published results.
4. The tensile capacity of the concrete is zero.

3.3.2 Plastic stress distribution method (PSDM)

The method of plastic stress distribution (PSDM) is based on the supposition of linear strain across the cross section and elastic-plastic behavior. The plastic stress distribution method assumes that the complete composite work is developed in a composite member, i.e., non-slip between the steel and concrete sections and the entire steel section and longitudinal reinforcement have reached their yield strength. It supposes that the concrete has reached its crushing strength in compression at a strain of 0.003 and a corresponding stress (typically $0.85f_{\grave{c}}$) on a rectangular stress block, and that the steel has overtaken its yield strain, taken as f_y/E_s. The actual interaction graph for moment and axial force for a composite section based on a plastic stress distribution is similar to that of a reinforced concrete section as shown in Figure 3.1.

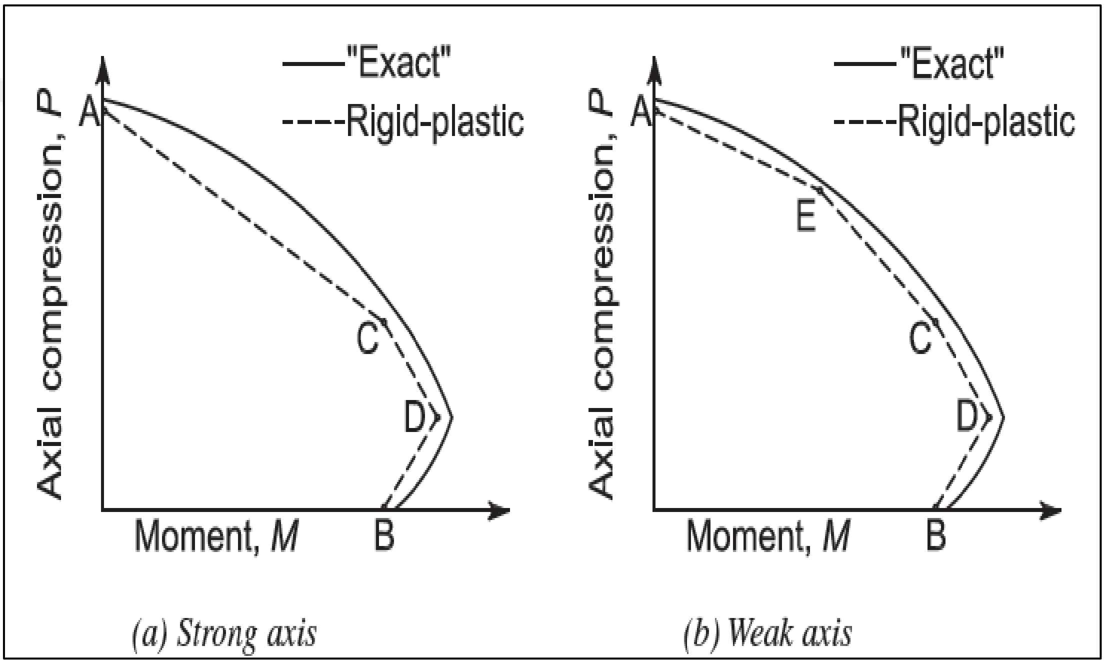


Figure 3.1 Exact and simplified moment-axial compressive force envelopes

The plastic stress approach of the compression members supposes that there is no slippage between the steel and concrete portions and that the required width to thickness ratios block the local buckling until some yielding and concrete crushing have occurred.

3.3.3 Material limitations

AISC imposes some material limitations on the structural steel and concrete infill. These restrictions must be met, unless such testing or analysis is justified.

1. For normal weight concrete, the $f_{\grave{c}}$ should not be less than 21 MPa and not be more than 70 MPa. A minimum limit of 21 MPa is specified for both normal weight and lightweight concrete and a maximum limit of 42 MPa is specified for lightweight concrete to promote the usage of good quality. In this study, these limitations have been met 21 MPa < $f_{\grave{c}}$ =34 MPa <42 MPa.

2. The yield stress of the used structural steel in the composite member should not be more than 525 MPa. This condition is available in this study because f_y = 324 MPa < 525 MPa.

3.The area of steel, A_s, throughout the cross section must be at least 1% of the total cross-sectional area which is defined as $A_s + A_c$, where A_c is the area of concrete. In this study the areas of steel, A_s , throughout the cross sections were (3%,5% and 7%).

3.3.4 Classification of filled composite sections for local buckling

Local buckling of the concrete filled steel tubes should be calculated by classifying these composite members into compact, non-compact or slender. The member is classified a compact when D/t ratio is less than λ_p, and a non-compact when D/t ratio is more than λ_p but less than λ_r. Furthermore, if D/t ratio of the section more than λ_r, then the member is classified as slender. Table 3.1 shows the limiting width-to-thickness ratios for compression steel elements in composite members subject to axial compression. Table 3.2 shows the limiting ratios of width/thickness for compression steel elements in composite members subject to flexure. The maximum allowed D/t ratio specified in Table 3.3 should not be overridden to make AISC's formulae workable. Table 3.3 shows the limits of D/t ratios for the members of the tube filled with concrete subject to axial compression and their computations for the classification of the local buckling. Based on these limits, all the specimens in this study are compact sections due to b/t ratios are less than 56.15.

Table 3.1 Limiting Width-to-Thickness Ratios for Compression Steel Elements in Composite Members Subject to Axial Compression (AISC-2010)

Descriptions of Elements	Width to Thickness Ratio	λ_p Compact / Noncompact	λ_r Noncompact / Slender	Maximum Permitted
Wall of Rectangular HSS and Boxes of Uniform Thickness	b/t	$2.26\sqrt{\frac{E}{f_y}}$	$3.00\sqrt{\frac{E}{f_y}}$	$5.00\sqrt{\frac{E}{f_y}}$
Round HSS	D/t	$\frac{0.15E}{f_y}$	$\frac{0.19E}{f_y}$	$\frac{0.31E}{f_y}$

Table 3.2 Limiting Width-to-Thickness Ratios for Compression Steel Elements in Composite Members Subject to Flexure (AISC-2010)

Descriptions of Elements	Width to Thickness Ratio	λ_p Compact / Noncompact	λ_r Noncompact / Slender	Maximum Permitted
Flanges of Rectangular HSS and Boxes of Uniform Thickness	b/t	$2.26\sqrt{\frac{E}{f_y}}$	$3.00\sqrt{\frac{E}{f_y}}$	$5.00\sqrt{\frac{E}{f_y}}$
Webs of Rectangular HSS and Boxes of Uniform Thickness	h/t	$3.00\sqrt{\frac{E}{f_y}}$	$5.70\sqrt{\frac{E}{f_y}}$	$5.70\sqrt{\frac{E}{f_y}}$
Round HSS	D/t	$\frac{0.09E}{f_y}$	$\frac{0.31E}{f_y}$	$\frac{0.31E}{f_y}$

Table 3.3 Limits of D⁄t ratios for concrete-filled steel tune members for local buckling classifications

Limits	λ_p compact/non-compact	λ_r non-compact/ slender	Maximum permitted
Code's expression	$2.26\sqrt{\frac{E}{f_y}}$	$3.00\sqrt{\frac{E}{f_y}}$	$5.00\sqrt{\frac{E}{f_y}}$
Calculated values	56.15	74.54	124.23

F_y specified minimum yield stress of steel section, E_s = modulus of elasticity of steel. In this study F_y=342 MPa and E_s=200000MPa.

3.3.5 Stiffness of the composite section

The effective stiffness of the composite section, EI_{eff},for all sections shall be:

$$EI_{eff} = E_s I_s + E_s I_{s\,r} + C_3 \quad (3.1)$$

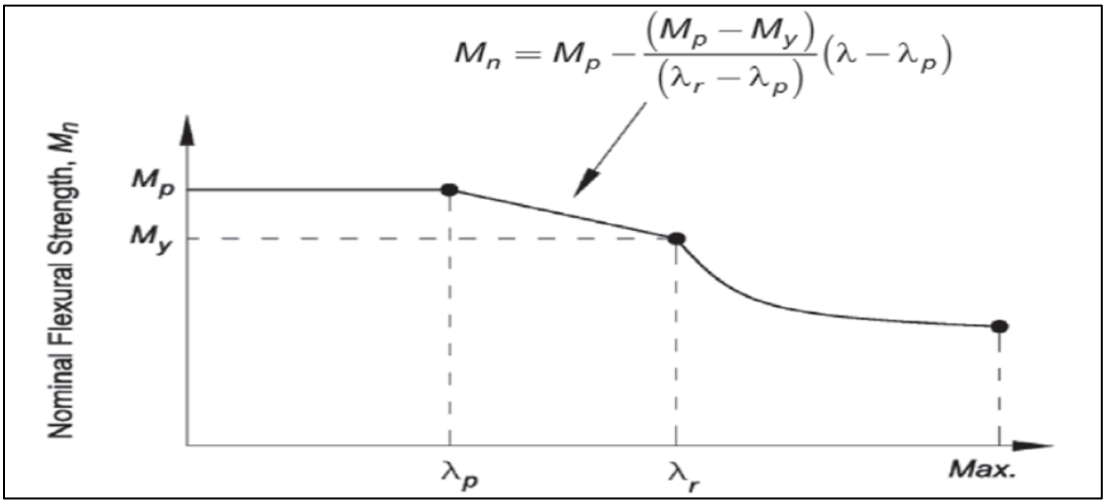
Where:

C_3 = coefficient for calculation of effective rigidity of filled composite compression member

$$C_3 = 0.6 + 2[A_s/A_s] \ll 0.9 \quad (3.2)$$

3.3.6 Flexural strength

Figure 3.2 shows the changes in the nominal bending strength (M_n) for the filled section vs. the hollow steel section slenderness and the compact sections can improve the full plastic strength (M_p) in bending. The nominal bending strength (M_n) of non-compact sections can be specified using a linear interpolation between the plastic strength (M_P) and the yield strength (M_y) vs. the hollow steel section slenderness.



Tube slenderness limits

Figure 3.2 Nominal flexural strength of filled beam vs. HSS slenderness

(M_n) of composite members can be computed as the plastic moment (M_p) strength of the cross-section by using the (PSDM) in AISC 360-10. This method supposes rigid-plastic behavior for the materials of steel and concrete with the steel yield stress equal to f_y in compression and tension, and the strength of concrete equal to $0.85f_{\acute{c}}$ in compression and zero in tension. The Eqs. (1) and (2) given below for computing a_p and M_p where a_p represents the distance of the neutral axis from the compression face and M_p the plastic moment strength. The variables in these equations are defined graphically in Figure 3.3.

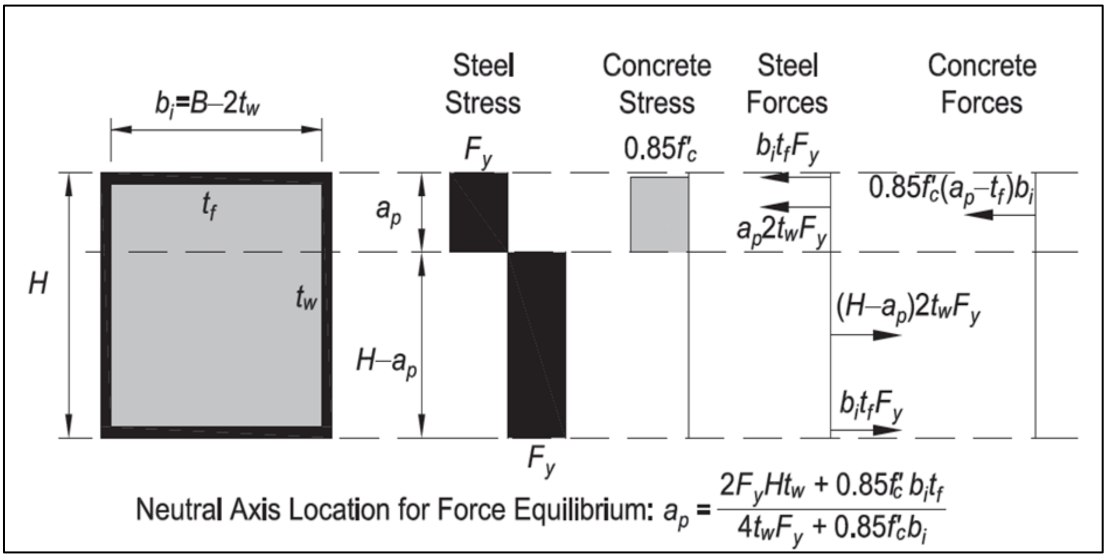


Figure 3.3 Compact section-stress blocks for calculating M_p

$$a_p = \frac{2f_y\, Ht_w + 0.85f_{\grave{c}}bt_f}{4f_y t_w + 0.85f_{\grave{c}}\, b} \tag{3.3}$$

If $\lambda \leq \lambda_p$

$$M_n = M_p = f_y\, bt_f\left(a_p - \frac{t_f}{2}\right) + f_y\, bt_f\left(H - a_p - \frac{t_f}{2}\right)$$

$$+f_y\, a_p\, 2t_w\left(\frac{a_p}{2}\right) + f_y\,(H - a_p)2t_w\left(\frac{H - a_p}{2}\right)$$

$$+0.85f_{\grave{c}}\,(a_p - \, t_f)b\left(\frac{a_{p-t_f}}{2}\right) \tag{3.4}$$

To estimate the minimum bound capacity of non-compact sections with tube slenderness (λ) = λ_r are shown in above Fig. 3.6. b. The steel tube is supposed to subject yielding and plastification in tension and reach the yield stress f_y in compression.The supposed maximum compressive stress of concrete is equal to $0.70f_{\grave{c}}$. The Eqs. (3) and (4) given below for computing a_y and M_y where a_y represents the distance of the neutral axis from the compression face and M_n (M_n=M_y) the moment capacity. The variables in these equations are defined graphically in Figure 3.4.

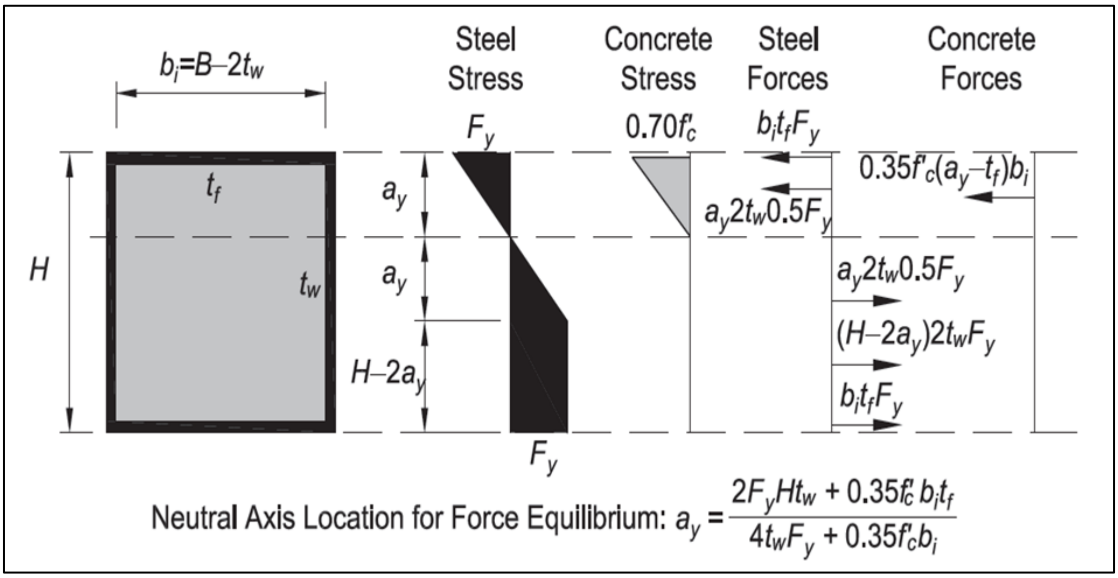


Figure 3.4 Non-compact section-stress blocks for calculating M_y

$$a_y = \frac{2f_y\, Ht_w + 0.35f_{\grave{c}}bt_f}{4f_y t_w + 0.35f_{\grave{c}}\, b} \tag{3.5}$$

If $\lambda = \lambda_p$

$$M_n = M_y = f_y\, bt_f \left(a_y - \frac{t_f}{2}\right) + f_y\, bt_f \left(H - a_y - \frac{t_f}{2}\right)$$

$$+0.5 f_y\, a_y\, 2t_w \left(\frac{2a_y}{3}\right) + f_y\,(H - 2a_y)2t_w \left(\frac{d}{2}\right)$$

$$+0.35 f_{\grave{c}}\,(a_y - t_f) b \left(\frac{2\left(a_{y-t_f}\right)}{3}\right) \tag{3.6}$$

For non-compact sections with tube slenderness (λ) in the range $\lambda_p < \lambda \leq \lambda_r$, the nominal moment capacity (M_n) can be computed by using Eq. (5), which supposes a linear variation between the moment capacities M_p corresponding to λ_p, and My corresponding to λ_r and tube slenderness.

If $\lambda_p < \lambda \leq \lambda_r$

$$M_n = M_p - \frac{(M_p - M_y)}{(\lambda_r - \lambda_p)}\,(\lambda - \lambda_p) \tag{3.7}$$

To estimate the minimum bound capacity of slender sections with tube slenderness $\lambda \geq \lambda_r$ are shown in Figure 3.5. The steel tube is assumed to just reach the yield stress f_y in tension and the critical buckling stress f_{cr} in compression. The supposed maximum compressive stress of concrete is equal to $0.70 f_{\grave{c}}$. The Eqs. (6) and (7) given below for computing (a_{cr}) and M_{cr} where (a_{cr}) represents the distance of the neutral axis from the compression face and M_{cr} (M_n=M_{cr}) the moment capacity. The variables in these equations are defined graphically in Figure 3.5.

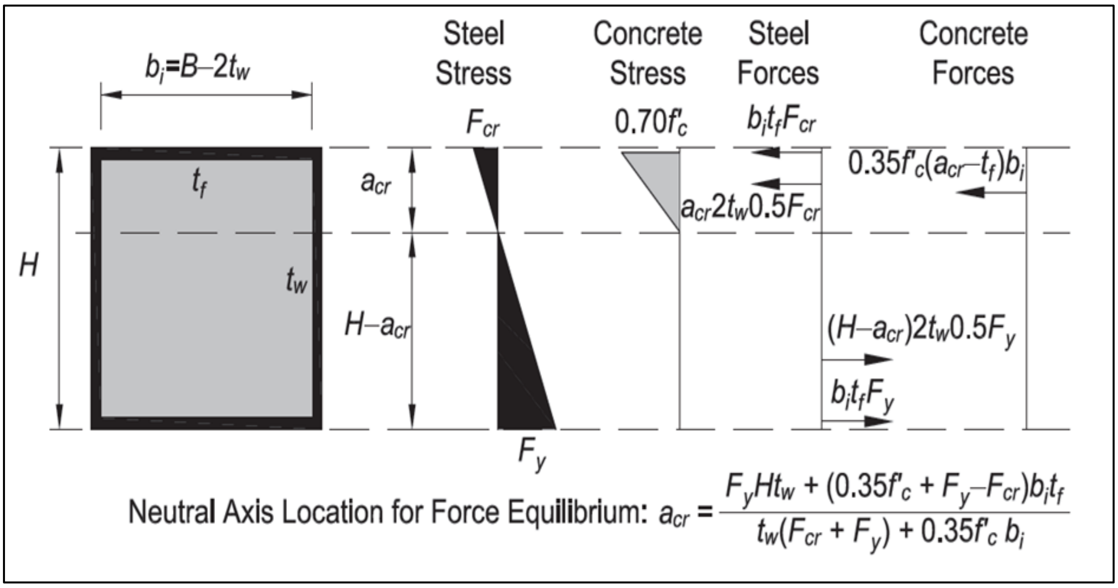


Figure 3.5 Slender section-stress blocks for calculating first yield moment, M_{cr}

$$a_{cr} = \frac{f_y\, H t_w + \left(0.35 f_{\grave{c}} + f_y - f_{cr}\right) b t_f}{t_w\left(f_{cr} + f_y\right) + 0.35 f_{\grave{c}}\, b} \tag{3.8}$$

If $\lambda \geq \lambda_r$

$$M_n = M_{cr} = f_{cr}\, b t_f \left(a_{cr} - \frac{t_f}{2}\right) + f_y\, b t_f \left(d - a_{cr} - \frac{t_f}{2}\right)$$

$$+0.5 f_{cr}\, a_{cr}\, 2t_w \left(\frac{2a_{cr}}{3}\right) + 0.5 f_y\ \left(d - a_{cr}\right) 2t_w \left(\frac{2(d - a_{cr})}{3}\right)$$

$$+0.35 f_{\grave{c}}\, \left(a_{cr} - \ t_f\right) b \left(\frac{2\left(a_{cr - t_f}\right)}{3}\right) \tag{3.9}$$

The specimens in this study met the required limits by AISC-360-10.

3.4 (EC4) Euro code 4 formulae

Eurocode 4 applies to the design of composite structures for buildings and civil engineering works. It is subdivided in various sections, Part 1-1: General rules and rules for buildings, Part 1-2: Structural fire design and Part 2: General rules and rules for bridges. There are two methods adopted by the Eurocode 4 for composite member, the general method and the simplified method.

3.4.1 General method of design

In this method, the effects of the second order including residual stresses, cracking of concrete, local instability, creep and shrinkage of concrete, yielding of structural steel, yielding of reinforcement and defects of the compression members are taken into regard explicitly.

The important design issues which should be considered using the general method, are as follows:

1.Non-linearity for geometrical and material.
2.The effects of the second order (on slender members).
3.Shrinkage and creep of the concrete under long-term loading.
4.Contribution of tensile strength to concrete between cracks.
5.Disadvantages for calculating moments around both axes and internal forces.
6.Distribution of moments and internal forces between the concrete and steel section through the load path is clearly defined.

3.4.2 Simplified method of design

The simplified method is limited to symmetrical cross-sectional members and a uniform cross-section along the length of the member with cold-formed, rolled or welded steel sections. If the component of structural steel consists of two or more unconnected sections, the simplified method is not applicable. The imperfections of element are implicitly taken into account.

we can summarize the following calculation procedure which should be considered using the simplified method, are as follows:

1. Make sure that the boundaries of the simplified design method.
2. Compute cross section properties.
3. Compute the buckling resistance for member.
4. Make sure whether second order effects should be considered.
5. Compute the impact of interaction between axial load and bending.
6. Compute the longitudinal and transverse shear.

3.4.3 Limitations

1. Longitudinal reinforcement that can be used in the calculation shall not exceed 6% of the concrete area. This study exceeded that ratio (ratios were 3%,5% and 7%).
2. The ratio of the cross-section's depth / width of the composite section should be within the limits 0.2 and 5.0. In this study, the ratio equal to (1).
3. The maximum non-dimensional slenderness ratio of the composite members $\bar{\lambda}$ is limited to 2.0.

$\bar{\lambda} \leq 2{,}0$

$$\bar{\lambda} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}} \qquad \text{EC4} - \text{Part} - 2\ (6.39)$$

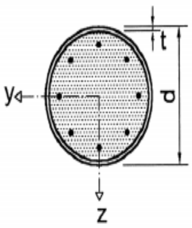
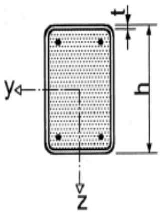
where:

$N_{pl,Rk}$ is the distinctive value of the plastic resistance to compression.

N_{cr} is the elastic critical normal force for the relevant buckling mode.

4. The impact of local buckling of the steel section on the resistance shall be considered in design shown table below.

Table 3.4 Maximum values (d/t), (h/t) and (b/t_f) with f_y in N/mm2

Cross-section		max (*d*/*t*), max (*h*/*t*) and max (*b*/*t*)
Circular hollow steel sections		$\max\ (d/t) = 90\dfrac{235}{f_y}$
Rectangular hollow steel sections		$\max\ (h/t) = 52\sqrt{\dfrac{235}{f_y}}$

5. Partial safety factors.

Values of partial safety factors for loads and materials and replace them with "boxed" values in the Euro codes. The boxed values in the table 3.7.

Table 3.5 Values for partial safety factors for loads and materials

Loads		EC4-1-1 (boxed – Values)	
Imposed (variable) load, γ_Q Dead (permanent) load, γ_G		1.50 1.35	
Materials:			
Steel	γ_a	1.10	$f_{ad} = f_y/\gamma_a$
Concrete	γ_c	1.50	$f_{cd} = f_{ck}/\gamma_c$
Reinforcement	γ_s	1.15	$f_{sd} = f_{sk}/\gamma_s$

3.4.4 The plastic moment resistance

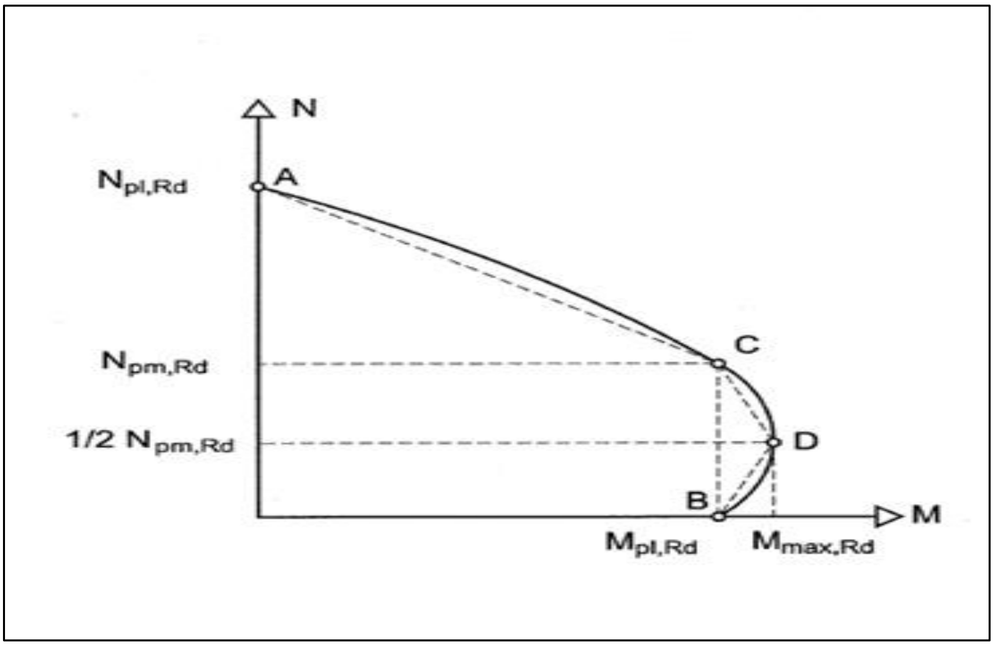


Figure 3.6 Simplified interaction curve and corresponding stress distributions

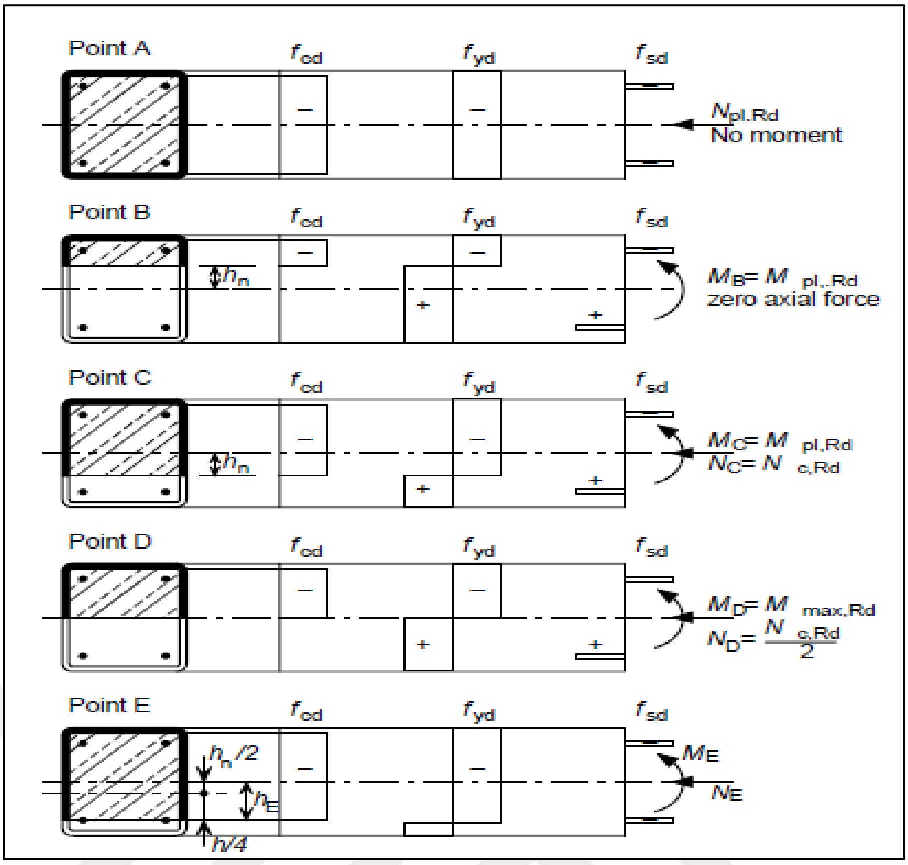


Figure 3.7 Stress distributions for the points on the interaction curve for concrete filled hollow sections, according to EC4.

The plastic moment resistance of a concrete filled hollow section calculated following equations.

$$M_{pl,Rd} = f_{yd}(W_{pa} - W_{pan})$$

$$+0.5 f_{cd}(W_{pc} - W_{pcn})$$

$$+f_{sd}(W_{ps} - W_{psn}) \tag{3.10}$$

Where:

f_{yd}, f_{sd}, f_{cd} are $\frac{f_y}{\gamma_a}$, $\frac{f_{sk}}{\gamma_s}$, $\frac{f_{ck}}{\gamma_c}$ respectively

W_{pa}, W_{pc}, W_{ps}

represented the plastic modulus of the steel section, concrete of the composite cross-section and the reinforcement respectively.

$$W_{pan}, W_{pcn}, W_{psn}$$

are the plastic modulus of the corresponding components within the region of 2hn from the center-line of the composite cross-section.

The values of the relevant parameters in the above equation for concrete filled hollow sections are:

For rectangular hollow sections:

$$W_{pc} = \frac{(b-2t)(h-2t)^2}{4} - \frac{2}{3}r^3$$

$$-r^2(4-\pi)\left[\frac{h}{2} - t - r\right] - W_{ps} \tag{3.11}$$

Where: r is the internal radius of the corners to the hollow section.

For circular hollow sections:

$$W_{pc} = \frac{(d-2t)^3}{6} - W_{ps} \tag{3.12}$$

In general, for both types of section:

$$h_n = \frac{A_C\, f_{cd} - A_{sn}(2f_{sd} - f_{cd})}{2bf_{cd} + 4t\left(2f_{yd} - f_{cd}\right)} \tag{3.13}$$

$W_{ps} = A_s$ times the distance from midline.

where: A_{sn} is the area of reinforcing bars within the region of 2 h_n from the center-line of the composite cross-section.

For rectangular hollow sections:

$$W_{pan} = 2t{h_n}^2 \tag{3.14}$$

$$W_{pcn} = (b-2t){h_n}^2 - W_{psn} \tag{3.15}$$

CHAPTER 4

EXPERIMENTAL PROGRAM AND RESULTS DISCUSSION

4.1 Experimental Program

4.1.1 Material properties

The experimental program in this study consists of four materials. These materials are lightweight concrete, longitudinal reinforcement bars, transverse reinforcement bars and square steel tubes. The following sections provide detailed description of these materials.

4.1.1.1 Lightweight concrete (LWC)

The design of lightweight concrete mixture has been based upon the experience and taking the recommendation that obtained from past researcher's studies of lightweight mixes design. The procedure of mixing for LWC by used different types of materials such as (cement, water, Silica fume, super plasticizer, coarse lightweight pumice aggregate and crushed stone) with different percentages as shown in the Table 4.1.

Table 4.1 Properties for LWC mixes

<table>
<tr><th>Mix proportions</th><th>C</th><th>SF</th><th>W</th><th>SP</th><th>Coarse LWA</th><th>CS</th></tr>
<tr><td>Kg/m^3</td><td>750.00</td><td>187.50</td><td>234.375</td><td>14.00</td><td>300.00</td><td>444.00</td></tr>
<tr><td>Discretions:</td><td colspan="3">C: Cement
SF: Silica fume
W: Water
SP: Super Plasticizer</td><td colspan="3">Coarse LWA: Coarse lightweight pumice aggregate
CS: Crushed stone</td></tr>
</table>

Ordinary Portland cement manufactured by Limak Gaziantep cement (Turkish cement) was used in all mixes throughout this study. The cement had a 28-day compressive strength of 42.5 MPa. The specific gravity of the cement was 3.15

g/cm3. Coarse lightweight pumice aggregate with 10 mm size was used. The density and specific gravity of lightweight pumice aggregate are 0.25 g/cm3 and 0.85 respectively. The method of the mix was, firstly, coarse lightweight pumice aggregate was mixed with some of the percentages of water to let the lightweight pumice aggregate absorb the water as shown in Figure 4.1.

Figure 4.1 Coarse lightweight pumice aggregate

After that, the dry contents such as (crushed stone, cement, and fly ash) were mixed slowly in the concrete mixer with the coarse lightweight pumice aggregate. Super plasticizer and the extra water were premixed completely and then added to the dry mixture.

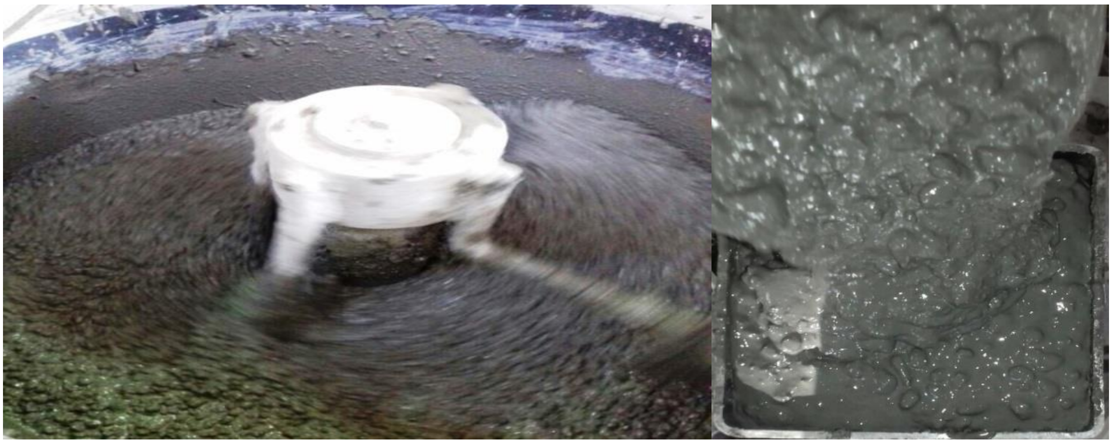


Figure 4.2 Final stages for lightweight concrete mixture

Finally, mixing is done continuously until desired consistency and homogeneous is obtained. Chemical admixture such as (super plasticizer) was added to the mix to get on better homogeneity with the high performance of lightweight concrete mixture as shown in Figure 4.2. Four batches from lightweight concrete were made. Three batches for reinforced lightweight concrete-filled steel tubular beams (one batch for four specimens) and one batch for lightweight concrete-filled steel tubular beams (two specimens). Three lightweight concrete cylinders (100x200mm) were prepared from each concrete batch and cured in water tanks with a 28-day and tested at the same day of the corresponding beam specimens as shown in Figure 4.3. The average cylinders' strength of the four batches and density as shown in Table 4.2.

lightweight concrete cylinders (100x200mm)	**Before testing**	**After testing**

Figure 4.3 Casting and testing the Cylinders

Table 4.2 Test results for LWC Cylinders

Item	Cylinder strength 28d f_{cu} (MPa)	Bulk density (kg/m3)
Mix 1	35.4	1787.5
Mix 2	31.0	1779.4
Mix 3	35.6	1828.8
Mix4	34.5	1789.4
Average value	**34.0**	**1796.3**

4.1.1.2 Steel bars (longitudinal and transverse)

Three different steel bar diameters were used as a longitudinal steel reinforcing bars (8mm ,10mm and 12mm nominal diameter) as ratios of reinforcement approximately 3%, 5% and 7% respectively. Mild steel bars of 5 mm diameter were used as transverse reinforcement at two different spacing (6 and 12 cm) as shown in Figure 4.4 and Table 4.3. The average values of yield tensile strength, f_y, were 534.8 MPa, 438.4 MPa, 472.2 MPa and 439.4 MPa for steel bars (8 mm, 10 mm, 12 mm and 5 mm) diameter, respectively with ultimate tensile strength, f_{su}, 645.4 MPa, 548.4 MPa, 586.4 MPa and 549.7 MPa respectively.

(a) Ø8 (b) Ø10 (c) Ø12

Figure 4.4 Arrangements of longitudinal and transverse reinforcement

4.1.1.3 Square steel tubes

The study consists of 16 beam specimens of square hollow steel section. All the steel tubes were made from two cold-formed square hollow sections. Three coupons from each tube were cut from three faces of the steel hollow sections to find the actual material properties. The nominal outer dimensions of the all square hollow steel sections were 120 mm × 120 mm with nominal steel tube thickness 3 mm. The specimen’s actual dimensions were listed in the Table 4.3. The width-to-thickness ratio b/t of the specimens ranging from 43 to 44.7 (compact section) according to the actual dimensions for each specimen. The average yield and ultimate tensile stress for each steel tube were 324 MPa ,475 MPa respectively.

Table 4.3 Dimensions of specimens and reinforcement details

Beam. No.	Lab symbol	Beam Type	B × H × t in (mm)	b/t	Top bars	Middle bars	Bottom bars	Stirrups
Hollow Section Beam								
B1	1-3	SHS	120.6×120.5×2.8	43	-----	-----	-----	-----
B2	1-4	SHS	120.4×120.4×2.8	43	-----	-----	-----	-----
Control Beam								
B3	2-3	LWCFST	120.3×120.6×2.8	43	-----	-----	-----	-----
B4	2-4	LWCFST	120.6×120.4×2.7	44.7	-----	-----	-----	-----
RCFST (ρ= 3%)								
B5	3-3	RLWCFST	120.8×120.4×2.7	44.7	3Ø8	2Ø8	3Ø8	Ø5@6cm
B6	3-4	RLWCFST	120.6×120.4×2.8	43	3Ø8	2Ø8	3Ø8	Ø5@12cm
B7	6-3	RLWCFST	120.6×120.5×2.7	44.7	4Ø8	-----	4Ø8	Ø5@6cm
B8	6-4	RLWCFST	120.8×120.4×2.8	43	4Ø8	-----	4Ø8	Ø5@12cm
RCFST (ρ= 5%)								
B9	4-3	RLWCFST	120.7×120.4×2.7	44.7	3Ø10	2Ø10	3Ø10	Ø5@6cm
B10	4-4	RLWCFST	120.6×120.6×2.8	43	3Ø10	2Ø10	3Ø10	Ø5@12cm
B11	7-3	RLWCFST	120.7×120.5×2.8	43	4Ø10	-----	4Ø10	Ø5@6cm
B12	7-4	RLWCFST	120.4×120.5×2.8	43	4Ø10	-----	4Ø10	Ø5@12cm
RCFST (ρ= 7%)								
B13	5-3	RLWCFST	120.4×120.5×2.7	44.6	3Ø12	2Ø12	3Ø12	Ø5@6cm
B14	5-4	RLWCFST	120.8×120.5×2.8	43	3Ø12	2Ø12	3Ø12	Ø5@12cm
B15	8-3	RLWCFST	120.6×120.6×2.8	43	4Ø12	-----	4Ø12	Ø5@6cm
B16	8-4	RLWCFST	120.3×120.5×2.8	43	4Ø12	-----	4Ø12	Ø5@12cm
Discretions:			SHS: square hollow section beam. LWCFST: Lightweight concrete filled steel tube beam. RLWCFST: Reinforced lightweight concrete filled steel tube beam.					

4.1.2 Specimens preparation

The two long cold-formed steel tubes of six meters long have been cut and shaped to the required length, ensuring that no form of irregularities as shown in Figure 4.5. and Figure 4.6.

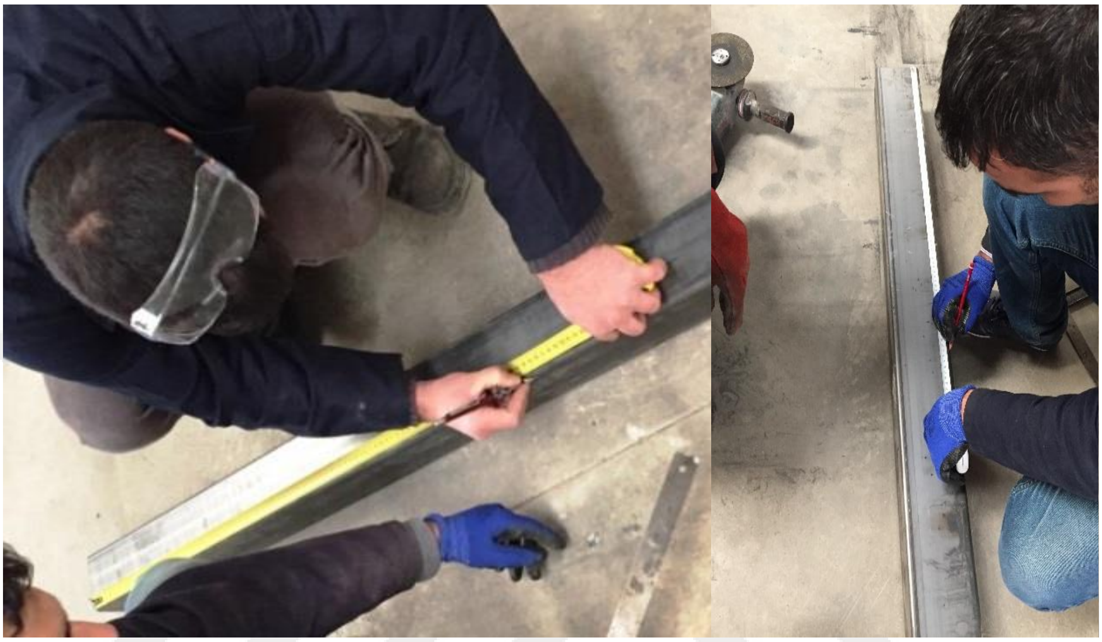


Figure 4.5 Measuring the required length for specimens

Figure 4.6 Cutting the specimens

The inside faces of sixteen beam specimens were brushed and cleaned**.** This study consists of 16 beam specimens of square section as shown in Figure 4.7 and can be detailed as follows: hollow steel tubular beams (two specimens) and concrete filled steel tubular beams with lightweight concrete (two specimens). 12 specimens of reinforced lightweight concrete filled steel tubular beam with longitudinal steel bars of (8Ø8, 8Ø10, 8Ø12) at two different distribution method of steel bars and two different spacing of stirrups (6 and 12 cm). All the details of internal longitudinal reinforcement bars and stirrups for the specimens were listed in Table 4.3. For the 12-reinforced lightweight concrete filled steel tubular beam specimens, the first type of distribution consists of three bars at top, three bars at bottom and two bars at the middle of the tube. While the second type of distribution, the longitudinal bars were four bars at top and four bars at bottom of the tube. The transverse reinforcement for the two types of distribution were at different spacing (6 and 12 cm) with diameter of 5 mm. The cover between the transverse reinforcement bars and inside of the tubes was 10 mm.

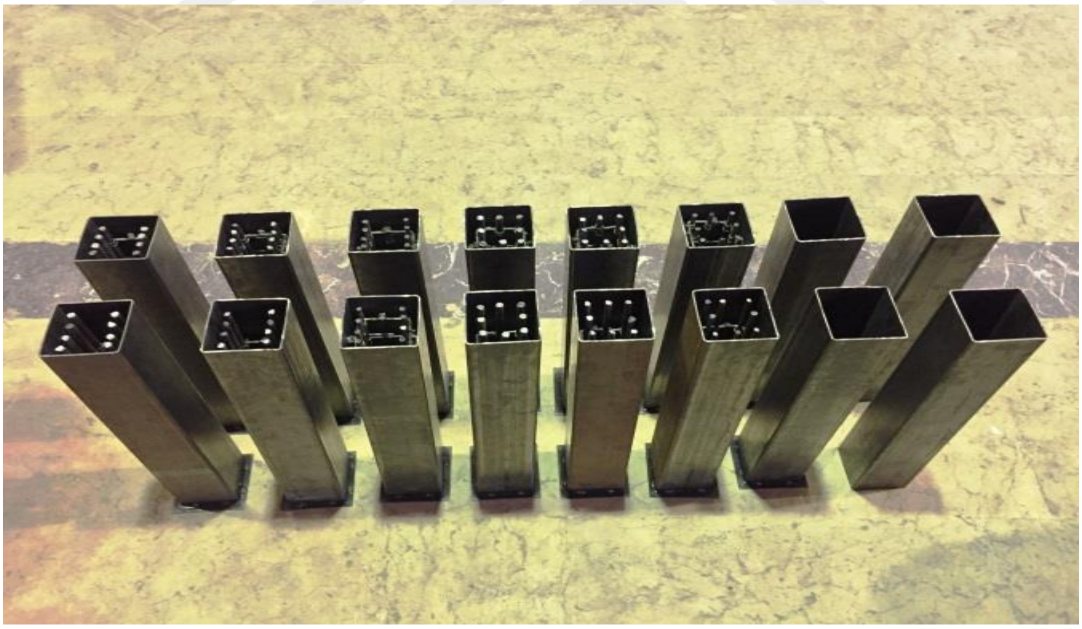


Figure 4.7 Sixteen specimens of study

The bottom ends of the tubes were capped with square steel base plate as shown in Figure 4.8 and the concrete is poured from the top and the steel tubes were kept in a vertical position as shown in Figure 4.9. The specimens were filled with concrete in about four equal layers. Also, each layer was compacted with 25 blows from a steel rod to avert any voids that happen inside the specimens. The top of the concrete was trimmed off using a trowel and the concrete-filled steel tubes were kept under the wet surface for 28 days. After 28 days, the other end was polished with epoxy coating, and welded with a square steel base plate.

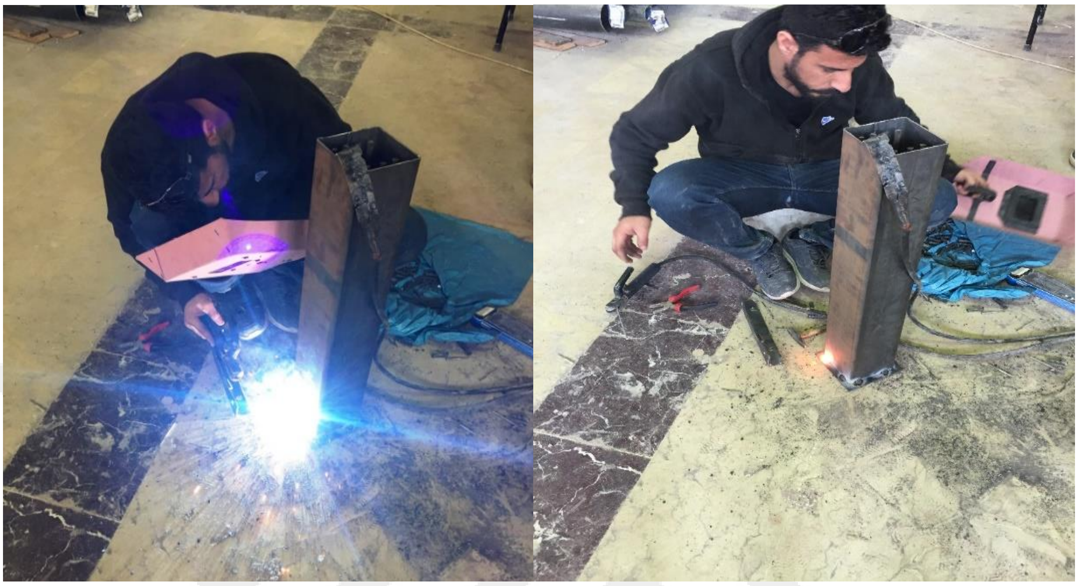


Figure 4.8 Welding the bottom of specimens

Figure 4.9 Casting the specimens

4.1.3 Test set up and procedure.

All the specimens were test under pure bending moment up to failure. The specimens were of an effective span of 600 mm and put them in simply supported by 40 mm diameter steel rods shown in Figure 4.10 and figure 4.11. A 500 KN test instrument was used to conduct the test. The beams were tested under four points loading method providing constant bending zone of 200 mm. Force was applied by the four-point loading method at one-third distance of the effective span of the specimen. Deflections of the beam specimens were measured by three LVDTs (Linear Variable Displacement Transducer). One placed in the middle of the specimen, placed two others under concentrated loads by shifting at 50mm from the left and right of the concentrated load positions. Three gauges of strain were bonded in the specimen and used to determine the maximum compressive and tensile strains, two strain gauges at the top and bottom flange center of specimen and one at the web center as shown in Figure 4.11. From the strain values and deflection, moment vs. strain and moment vs. deflection behavior are studied and discussed. The diagrammatic sketch of the beam setup and schematic diagram of tested beam as shown in Figure 4.10 and Figure 4.11 respectively.

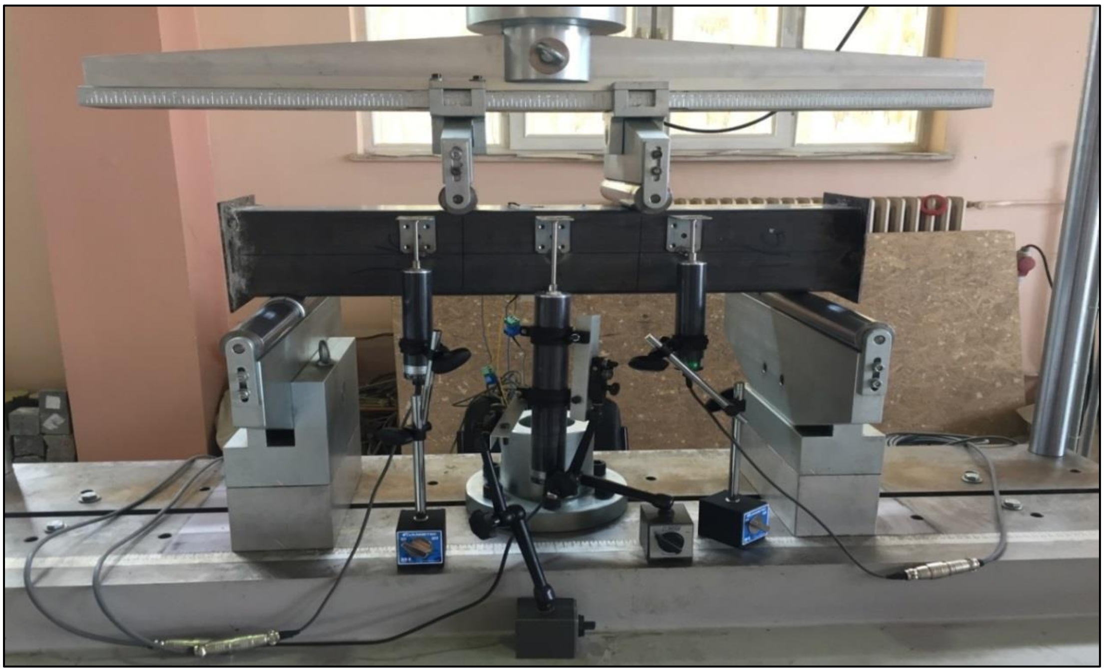


Figure 4.10 The diagrammatic sketch of the beam setup

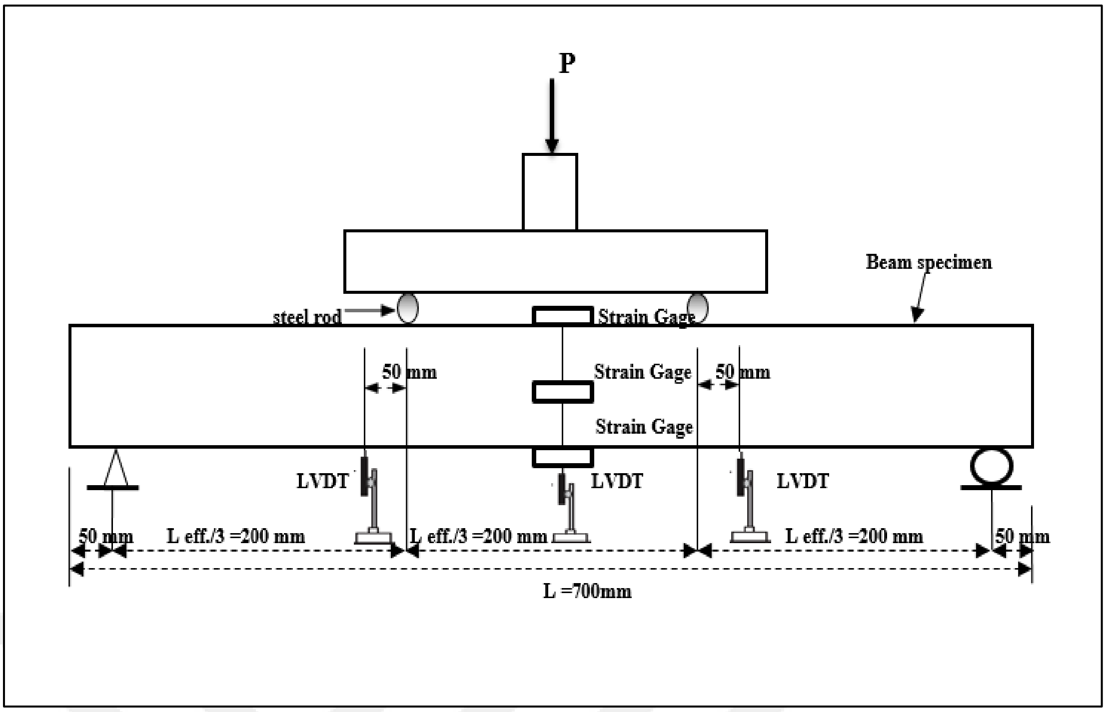


Figure 4.11 Schematic diagram of tested beam

4.2 Results and Discussion

4.2.1 Failure modes

Typical local failure modes of the tested specimens are shown in Figures 4.12-21. In general, for the two hollow section beams (B1 and B2) in Figure 4.12, a double outward local buckling happened in the front and the back views of the two specimens and the inward local buckling at the points of loading locations appeared, without a crack in the tension zone was observed.

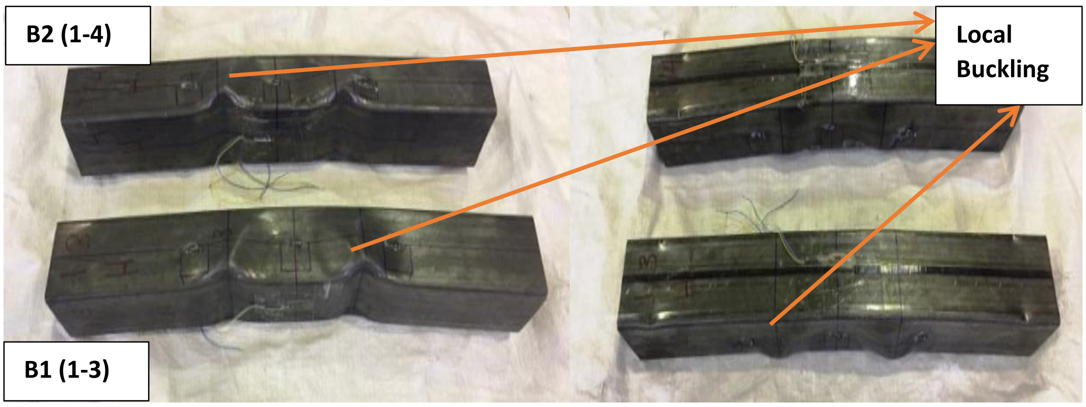


Figure 4.12 Local backing for hollow section beams

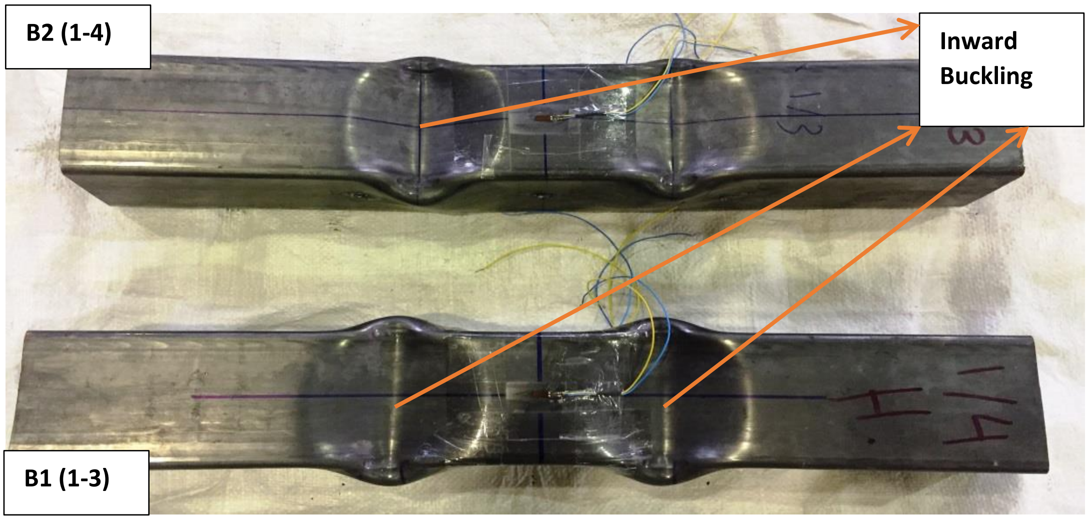


Figure 4.13 Inward local backing for hollow section beams

The specimens failed due to local collapse of the section under the points of loading locations. When the two specimens reached the peak load, the inward local buckling (inward bend) under the points of loading was obvious, accompanied by sudden drop in load capacity and increased vertical displacement as shown in Figure 4.13**.**

The Figures 4.14-15 show the failure modes for the two control beams (B3 and B4), LWCFST beams. At load-carrying capacity equal to 240 KN a single outward local buckling near the left point loading begun to appear in the top flange in compression zone. A single outward local buckling happened in the two sides of the specimens. Also, a single outward local buckling occurred near the left point loading in the top flange in compression zone as shown in Figure 4.14.

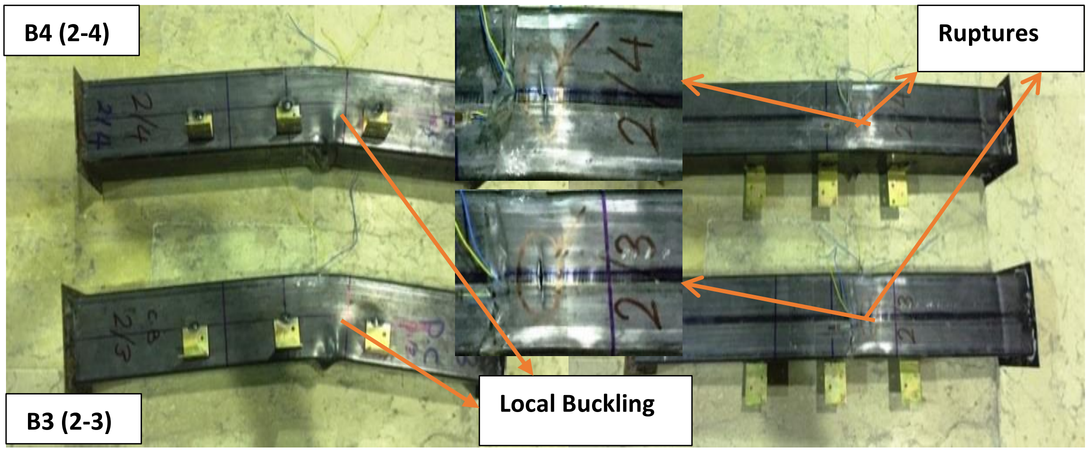


Figure 4.14 Local backing and ruptures for control beams

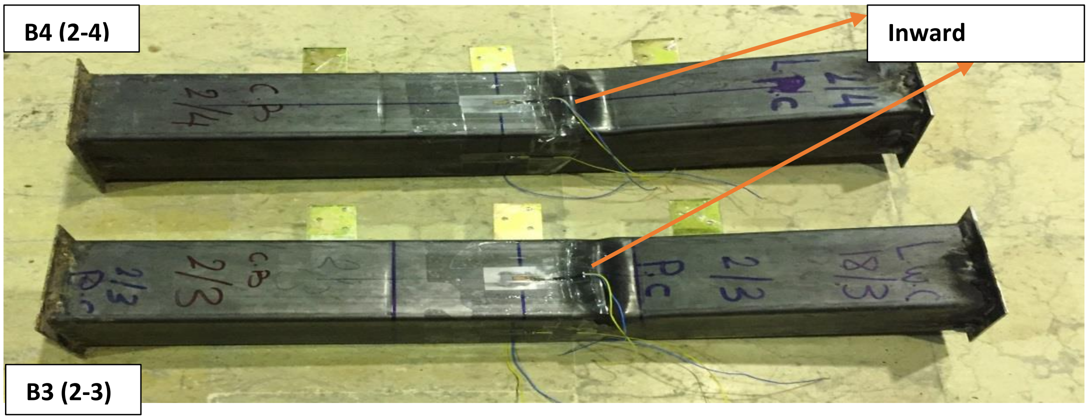


Figure 4.15 Inward local backing for control beams

A single obvious transverse rupture occurred at the bottom of the specimens in the tension zone under the left point loading at testing end as shown in Figure 4.14. It can be seen that the LWC core change the failure mode by preventing the development of local buckling. This failure mode was different from that of specimens without filling with LWC. Compared with the failure mode of specimens without filling with LWC (in Figures 4.12-13), the outward buckling deformation under point loads is somewhat smaller as shown in Figure 4.15. After the peak load was reached, the load-carrying capacity declined progressively with increasing in the vertical displacement.

The Figures 4.16-17 represents the failure modes for the four reinforced LWCFST beams (B5, B6, B7, B8) with ratio equal to 3% of longitudinal reinforcement with two arrangement methods and transverse reinforcement with two different spacing as listed in Table 4.3.

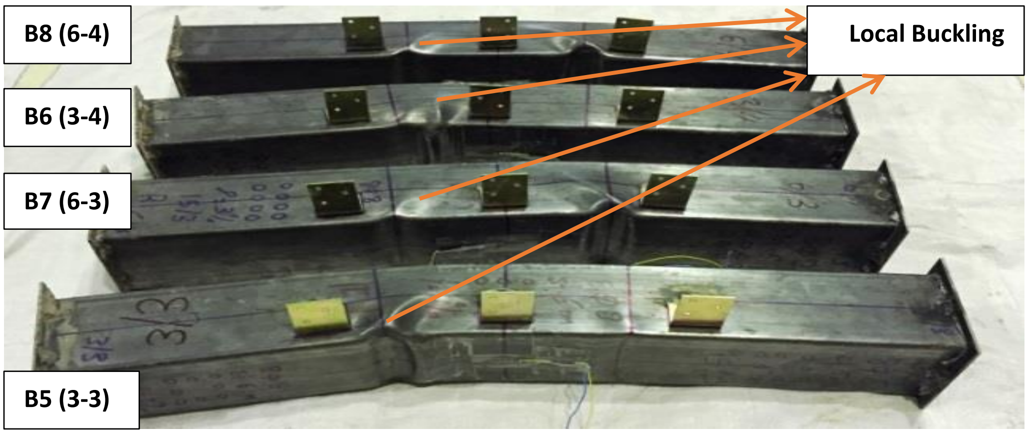


Figure 4.16 Local backing for RCFST (ρ= 3%) beams

For the B5 and B6 have the same shapes and location of local buckling where a single outward local buckling near the right point loading begun to appear in the top flange in compression zone. Also, a single outward local buckling happened in the two sides of the two specimens under the right point loading as shown in Figure 4.16. Also, another different in mode failure where in B5, two obvious longitudinal ruptures occurred at the bottom of the specimen in the tension zone under the right point loading at testing end as shown in Figure 4.17.

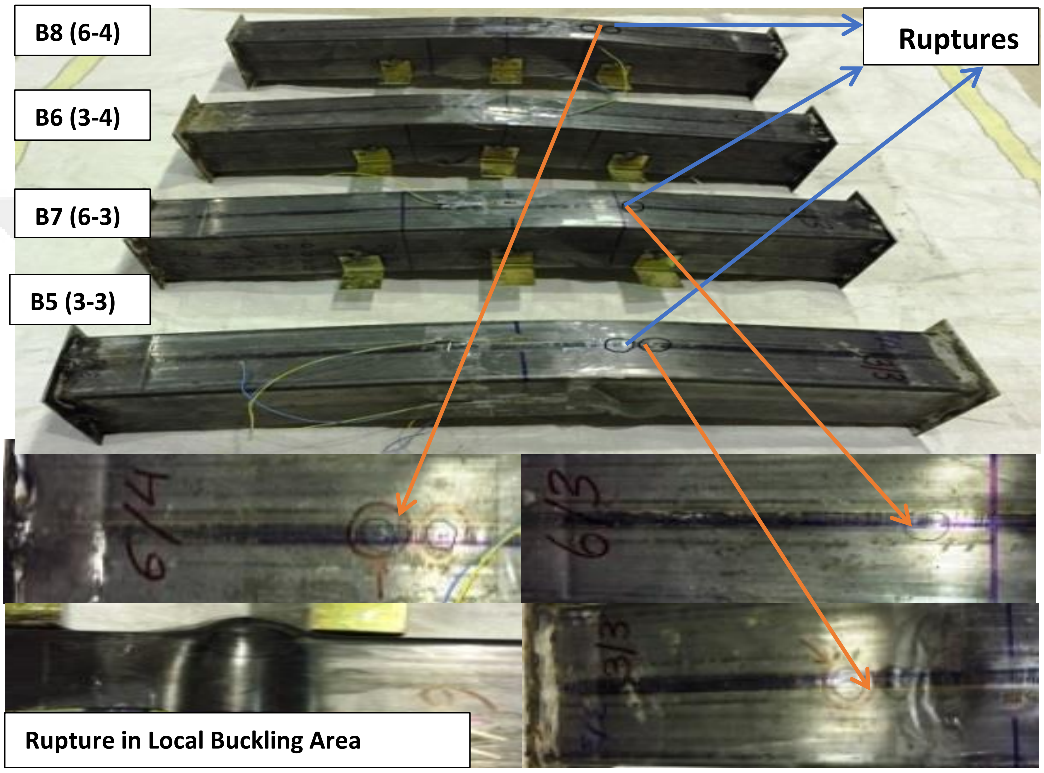


Figure 4.17 Ruptures for RCFST (ρ= 3%) beams

The B7 and B8 have the same shapes and location of local buckling where a single outward local buckling occurred in the two sides of the two specimens under the points of loading. Also, a single outward local buckling near the left point loading in the compression zone. The two beams have obvious longitudinal ruptures happened at the bottom of the specimen in the tension zone under the left point loading one rupture in B7 and two ruptures in B8 at testing end. B8 has a new failure mode not appeared in the failure modes of the all specimens, where B8 has obvious rupture in the local buckling area at the top corner of the beam specimen under the left point loading as shown in Figure 4.17.

Figures 4.18-19 represents the failure modes for the four reinforced LWCFST beams (B9, B10, B11, B12) with ratio equal to 5% of longitudinal reinforcement. The same failure modes of reinforced LWCFST beams (B5, B6, B7, B8) occurred in the reinforced LWCFST beams (B9, B10, B11, B12). Two obvious longitudinal ruptures occurred at the bottom of the specimen in the tension zone under the left point loading in the specimen B10 at testing end as shown in Figure 4.9.

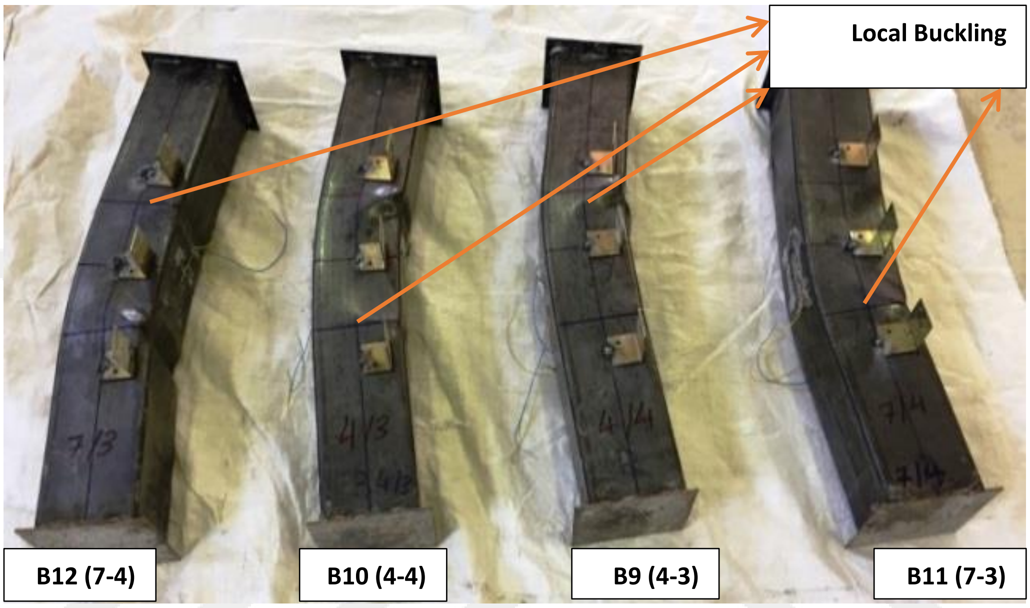


Figure 4.18 Local backing for RCFST (ρ= 5%) beams

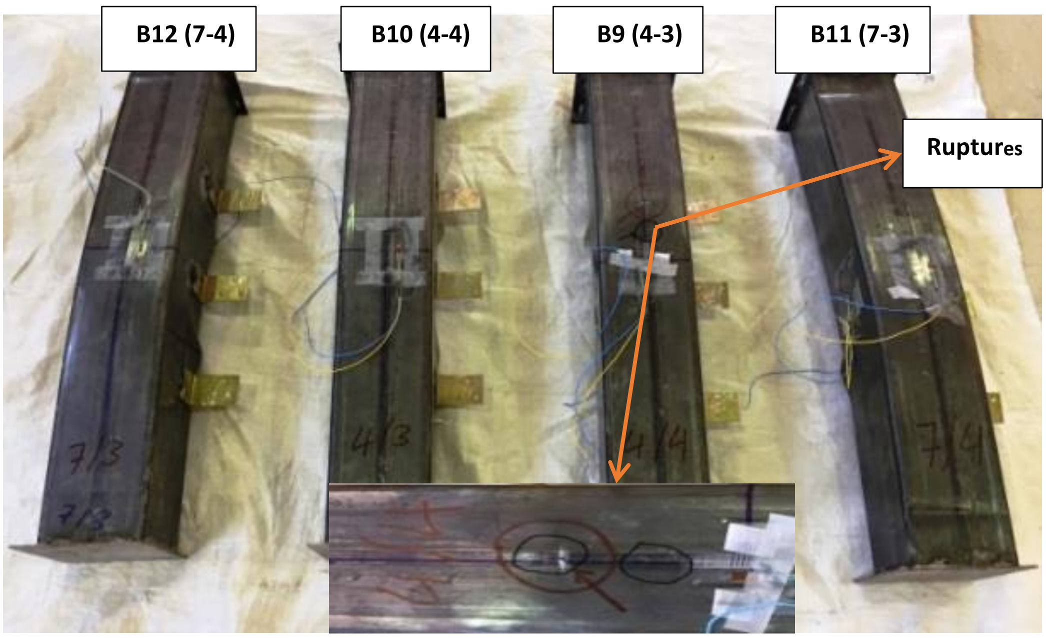


Figure 4.19 Ruptures for RCFST (ρ= 5%) beams

Finally, the Figures 4.20-21 represents the failure modes for the four reinforced LWCFST beams (B13, B14, B15, B16) with ratio equal to 7% of longitudinal reinforcement. The same failure modes of reinforced LWCFST beams (B9, B10, B11, B12) occurred in the reinforced LWCFST beams (B13, B14, B15, B16). Also, obvious longitudinal ruptures occurred at the bottom of the specimen in the tension zone under the points loading in the specimen B13, B14, B15 at testing end as shown in Figure 4.10.

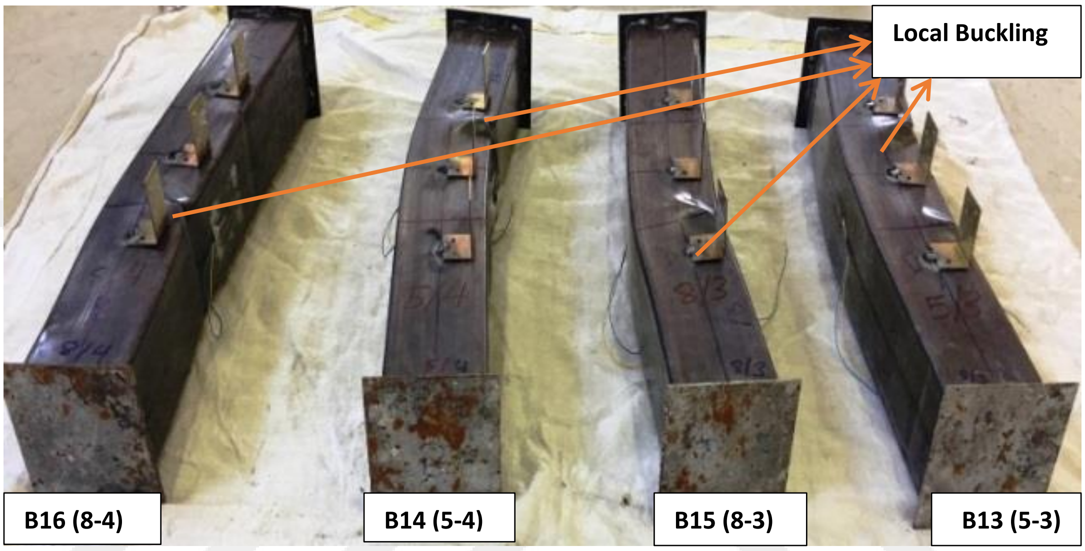


Figure 4.20 Local backing for RCFST (ρ= 7%) beams

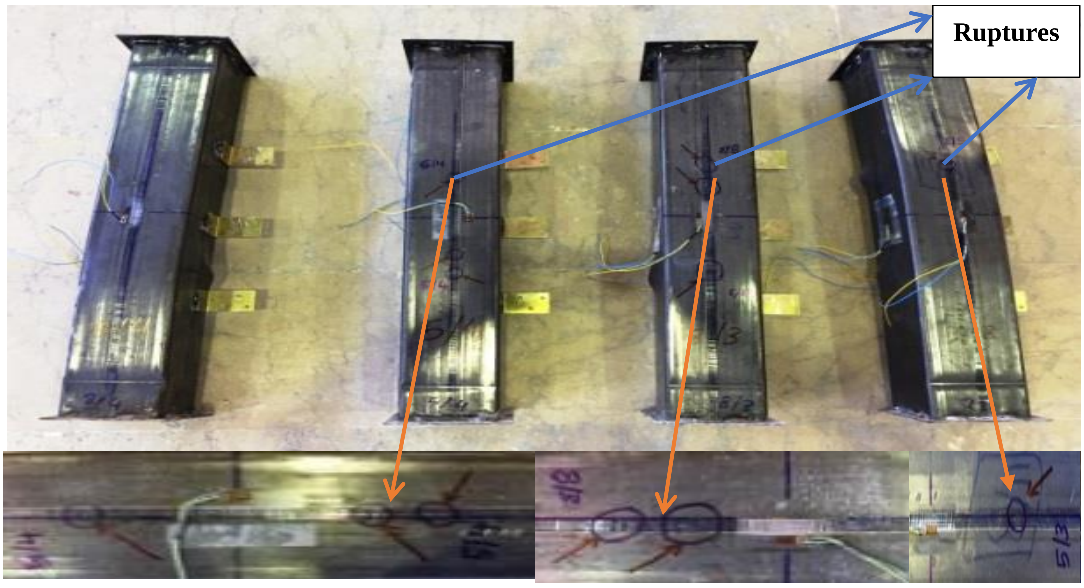


Figure 4.21 Ruptures for RCFST (ρ= 7%) beams

Generally, during the test procedure for LWCFST beams and reinforced LWCFST beams, sounds of crackling were heard due to crushing concrete core. The rupture of the steel tube occurred beyond the end of the plastic stage with a drop in the flexural load. The observed failure mode in case of the reinforced LWCFST beams was slightly different from that in the unreinforced LWCFST beams. The reinforced LWCFST beams failed progressively in a successive manner.

4.2.2 Ultimate moment capacity

The ultimate moment capacity obtained for the square hollow section beams, control beam (LWCFST) and RLWCFST beams are shown in Table 4.4 and Figure 4.22. The percentage of increase in ultimate moment capacity is found to be about 203% for LWCFST beams without reinforcement and 279–396% for reinforced LWCFST beams with respect to hollow section beams respectively. In general, the percentage of increase for the RLWCFST is about 21-64% compared to control beam (LWCFST). The percentages of increase were 21-26%, 25-38% and 38-64% for reinforced LWCFST beams with ρ= 3%, ρ= 5% and ρ= 7% respectively with respect to control beam. The above results refer that void filling increases the ultimate moment capacity of hollow section beams. On the other hand, there is a significant increase in the ultimate bending strength of LWCFST over hollow section tubes beams due to filling the hollow section beams with LWC. Also, there is a considerable increase in the ultimate bending strength of reinforced LWCFST that mentioned above over the unreinforced LWCFST beams where the percentage of increase is 21-26%, 25-38% and 38-64% for RLWCFST with ρ= 3%, ρ= 5% and ρ= 7% respectively as shown in Table 4.3. The RLWCFST beam B15 has the higher ultimate bending strength between the other RLWCFST beams where it has a percentage of increase about 64% over the unreinforced LWCFST beams and about 26% over the RLWCFST beams that have the same ρ= 7% (B13, B14 and B15). Also, B15 has a percentage of increase about 9% over the B16 that has the same ρ and the same arrangement of longitudinal reinforcement (two layers) but different in spacing for transverse reinforcement which in turn increase the confinement for concrete core and therefore increase bending strength. In general, the spacing of stirrups was effective with two layers’ arrangement of longitudinal reinforcement and without obvious effect with three layers of arrangement. On the other hand, it can be noticed that the effect of arrangement of longitudinal reinforcement in increasing the ultimate bending strength was obvious in beams with ρ= 5% and ρ= 7% while without effect in beams with ρ= 3%. The results for B15 show the best arrangement of longitudinal reinforcement (two layers) with a spacing for transverse reinforcement equal to 6cm. In general, it can be concluded from the results above that the ultimate bending strength is obviously increased along with the increase of ρ and this conclusion disagreed with the opinion of the other researchers Xiamuxi et al. (2014). Xiamuxi et al. (2014) concluded that the increasing in bending strength is stopped at ρ = 3.0% and the optimal range of ρ in their study was 1.5% ≤ ρ ≤ 3.0%. The results of this study refer to that the ratio of longitudinal reinforcement ρ= 3%, ρ= 5% and ρ= 7% has obvious increasing in bending strength and has the effect on bending performance of RLWCFST beams, and would help the steel tube and concrete to improve their performance.

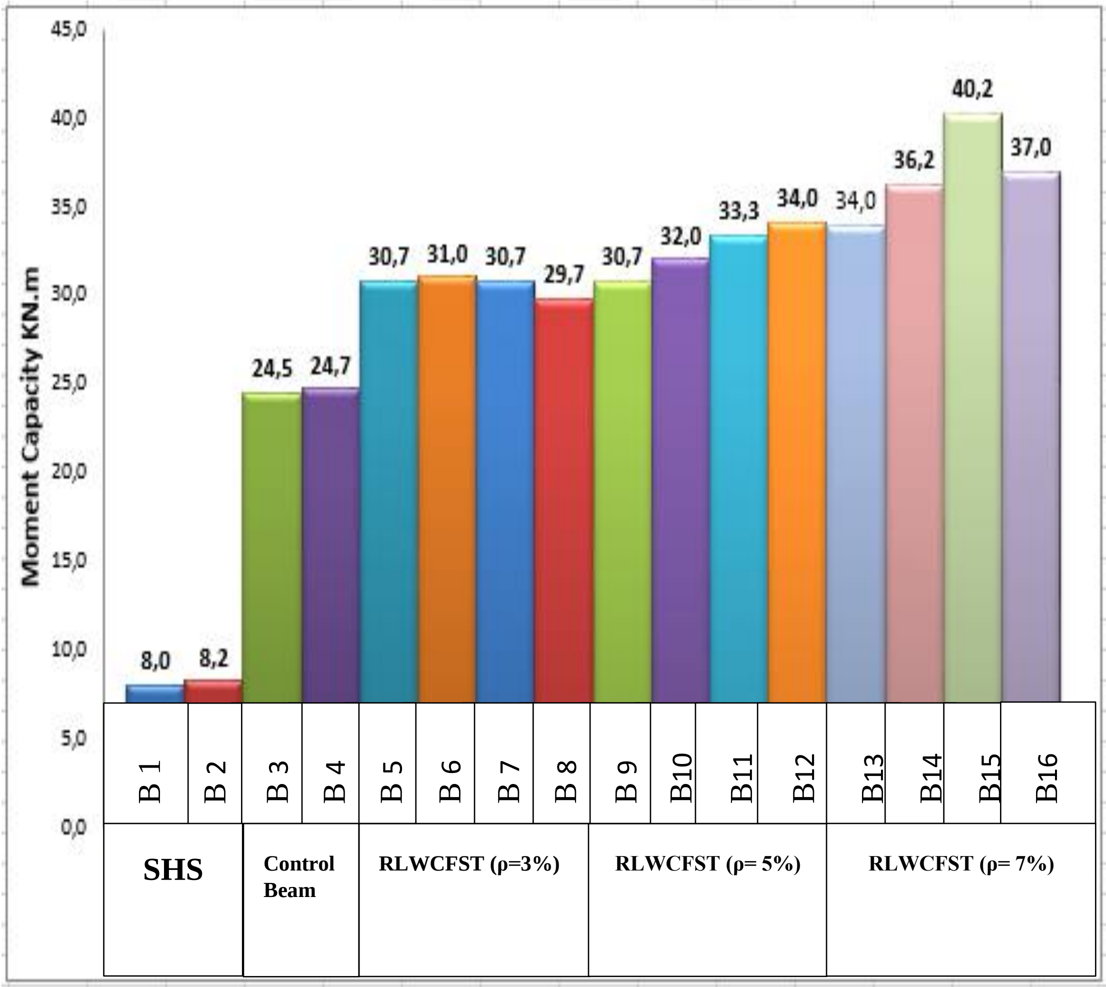


Figure 4.22 Ultimate Moment Capacity

Table 4.4. Test results

Beam. No.	Lab symbol	Beam Type	Ultimate Moment KN. m	Average Ultimate Moment KN. m	Percentage of Moment Increase		Mid-Span Deflection at Mu (mm)	Mid-Span Deflection at My (mm)	Ductility Index
					Mu/ MH	Mu RLWCFST / Mu LWCFST			
B1	1-3	Hollow Beam	8.01	8.12	1.00	-----			
B2	1-4		8.22						
B3	2-3	Control Beam	24.52	24.6	203.18	1.00	3.64	1.33	2.74
B4	2-4		24.68						
B5	3-3	RCFST (ρ= 3%)	30.74	30.74	278.81	24.95	7.00	1.67	4.19
B6	3-4		30.99	30.99	281.88	25.96	5.57	1.33	4.17
B7	6-3		30.73	30.73	278.74	24.92	6.95	1.66	4.18
B8	6-4		29.75	29.75	266.61	20.92	8.04	1.71	4.69
B9	4-3	RCFST (ρ= 5%)	30.74	30.74	278.88	24.96	5.29	1.59	3.32
B10	4-4		31.99	31.99	294.25	30.04	5.00	1.65	3.03
B11	7-3		33.34	33.34	310.85	35.51	4.04	1.10	3.65
B12	7-4		34.03	34.03	319.34	38.31	4.11	1.21	3.39
B13	5-3	RCFST (ρ= 7%)	33.96	33.96	318.52	38.04	8.57	1.67	5.15
B14	5-4		36.24	36.24	346.59	47.30	6.37	1.81	3.52
B15	8-3		40.25	40.25	396.01	63.60	9.46	1.28	7.41
B16	8-4		36.96	36.96	355.46	50.23	8.58	1.29	6.64
Discretions		Mu: Specimen ultimate moment MH: Ultimate moment for hollow			Mu LWCFST: Ultimate moment for LWCFST Mu RLWCFST: Ultimate moment for RLWCFST				

4.2.3 Moment – deflection relationship

Figures 4.23-25 displays a measured bending moment versus deflection curves of the test beams in this study. Figure 4.12 a represents the hollow section beams and control beams. Figure 4.12 b, represent the hollow section beams and control beams with RCFST beams with ρ= 3%.

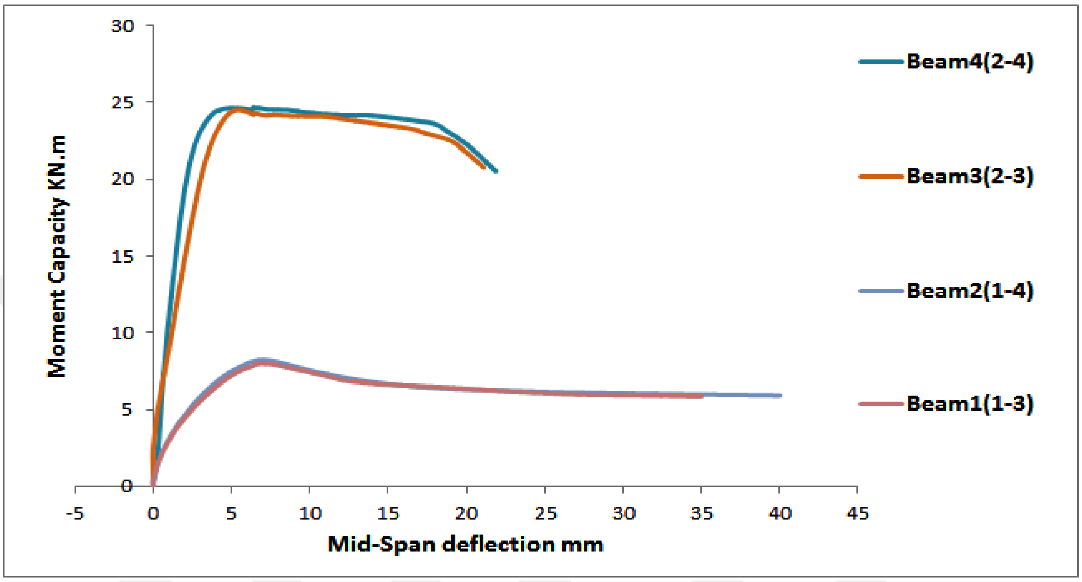


Figure 4.23 Moment versus mid-span deflection Curves for hollow and control beams

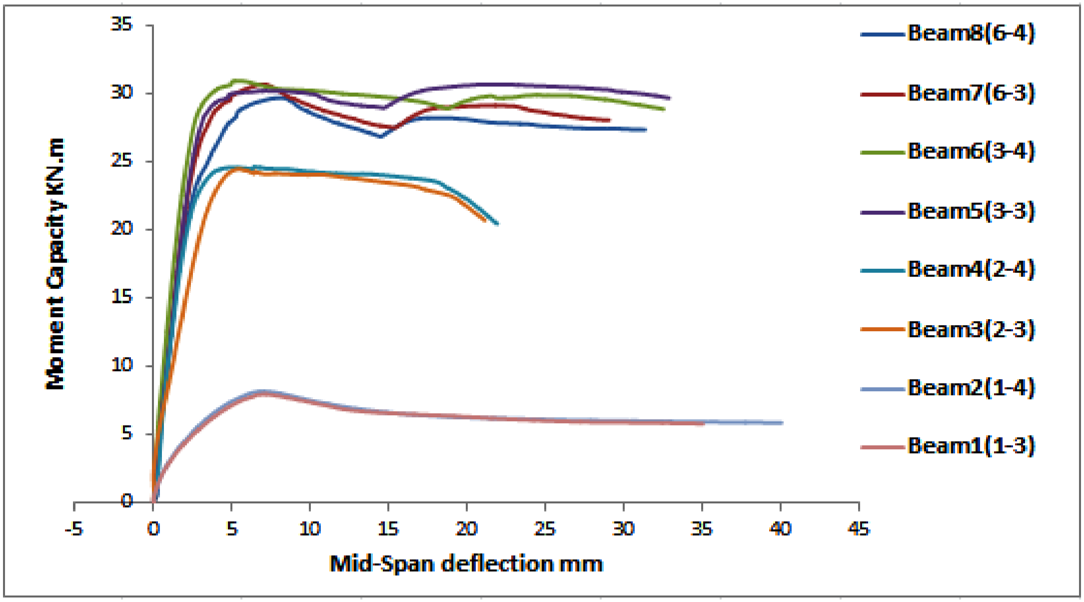


Figure 4.24 Moment versus mid-span deflection Curves for hollow, control and RCFST beams with ρ= 3% beams

Figure 4.25 represent the hollow section beams and control beams with RCFST beams with ρ= 3%, RLWCFST beams with ρ= 5% and RLWCFST beams with ρ= 7% respectively.

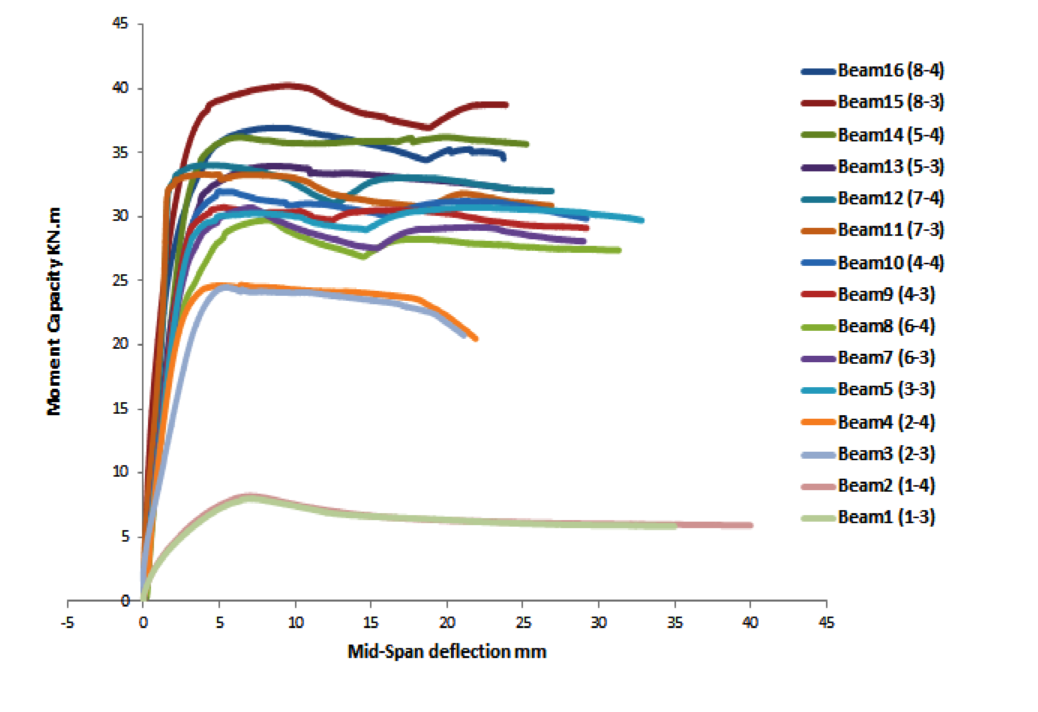


Figure 4.25 Moment versus mid-span deflection Curves for hollow, control and RCFST beams with ρ= 3%, ρ= 5%, ρ= 7%, beams

In general, from these curves it can be observed that the defection increased tardily as the moment increased before the specimens reach to the yielding where, it is almost a straight line. The square hollow steel tubular beams filled with LAC under bending exhibited high stiffness and great increase in the moment capacity compared with the hollow tubular beams due to concrete filling of the hollow sections. RLWCFST beams exhibited higher stiffness and moment carrying capacity than hollow section beams and control beams. The good performance in terms of stiffness and moment carrying capacity of RLWCFST beams refer to the effect of longitudinal reinforcement to develop the features of lightweight concrete filled-steel tubular beams. Finally, it can be seen that RLWCFST beams developed larger deformation capacity at the end of test (rupture at tension flange compared to LWCFST beams especially for the specimen with ρ= 3% and ρ= 7%). Figure 4.26 shows deflection curves along the length of effective span for the beams (B1, B3 and B15) under bending moment. From this figure, symmetrical curves were observed and the deflection curves corresponded with curves of half sine wave.

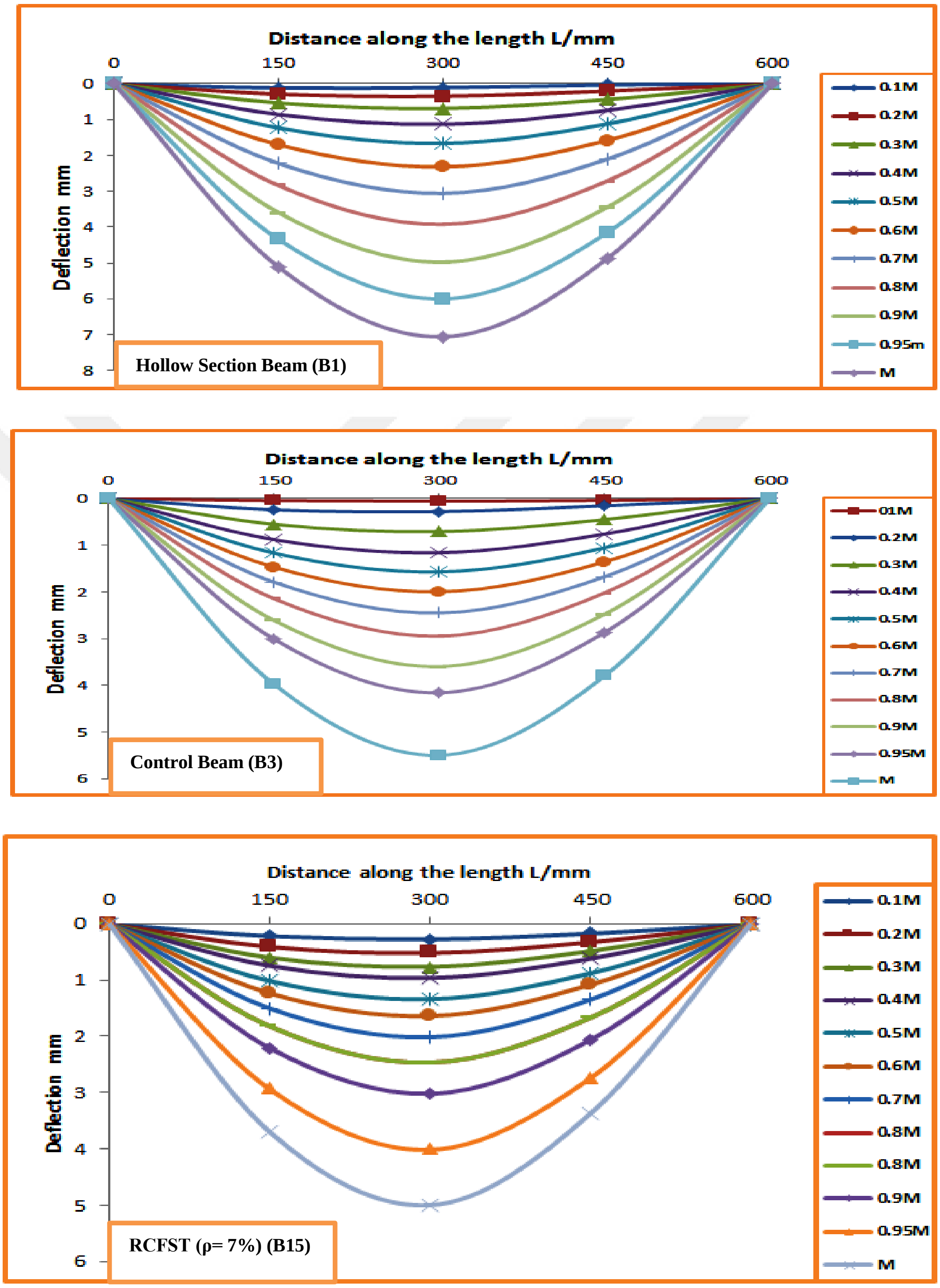


Figure 4.26 Mid-span deflection distributions along the Length for B1, B3 and B15

4.2.4 Ductility

The results for the ductility index obtained for the control beam (LWCFST) and RLWCFST beams are shown in Table 4.4 and Figure 4.27. The ductility index is found to be about 2.74 for LWCFST beams without reinforcement, 4.31 for reinforced LWCFST beams with ρ= 3%, 3.35 for reinforced LWCFST beams with ρ= 5%, and 5.68 for reinforced LWCFST beams with ρ= 7% respectively. In general, we observe that the ductility for reinforced LWCFST beams higher than that for LWCFST beams. Also, the ductility for reinforced LWCFST beams with ρ= 7% more than other reinforced LWCFST beams. Also, the B15 is the higher ductility among the reinforced LWCFST beams with ρ= 7%. B15 shows the best ductility in this study and the best ultimate moment capacity as mentioned in 4.2.2 section. Finally, it can be observed that longitudinal steel reinforcement developed the ductility of LWCFST with increasing the ρ.

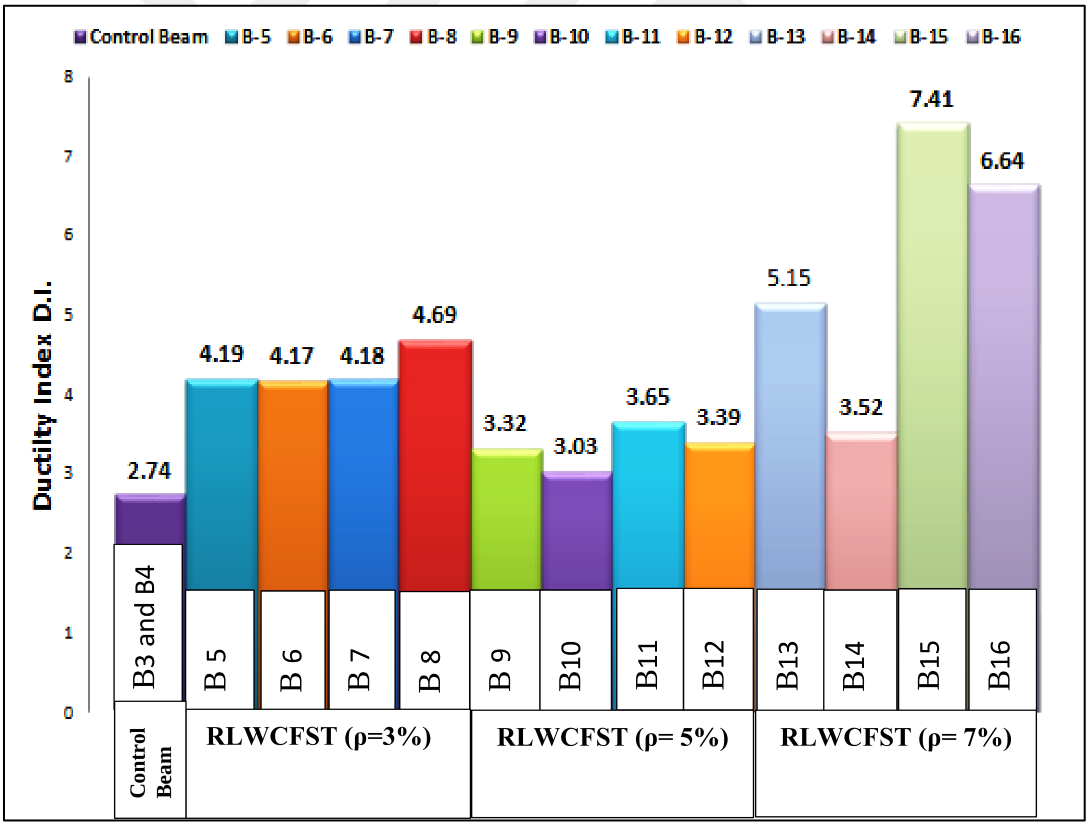


Figure 4.27 Ductility Index

4.2.5 Moment versus strain relationships

The typical moment versus maximum compressive and tensile strains, in addition to the section mid height strain relationships are shown in Figures 4.28-31. These curves represent the recorded strains on the top flange, bottom flange, and mid-height section, for the beam specimens B4, B5, 12, B13, respectively.

It is clear that the strains on the top flange were under compression since the steel section and concrete infill are both in compression, and the strains at the mid-height section and the bottom flange were under tension because the steel section and concrete infill are both in tension. It was due to the fact that; the neutral axis location was moved up when the concrete was cracked at the initial loading stages.

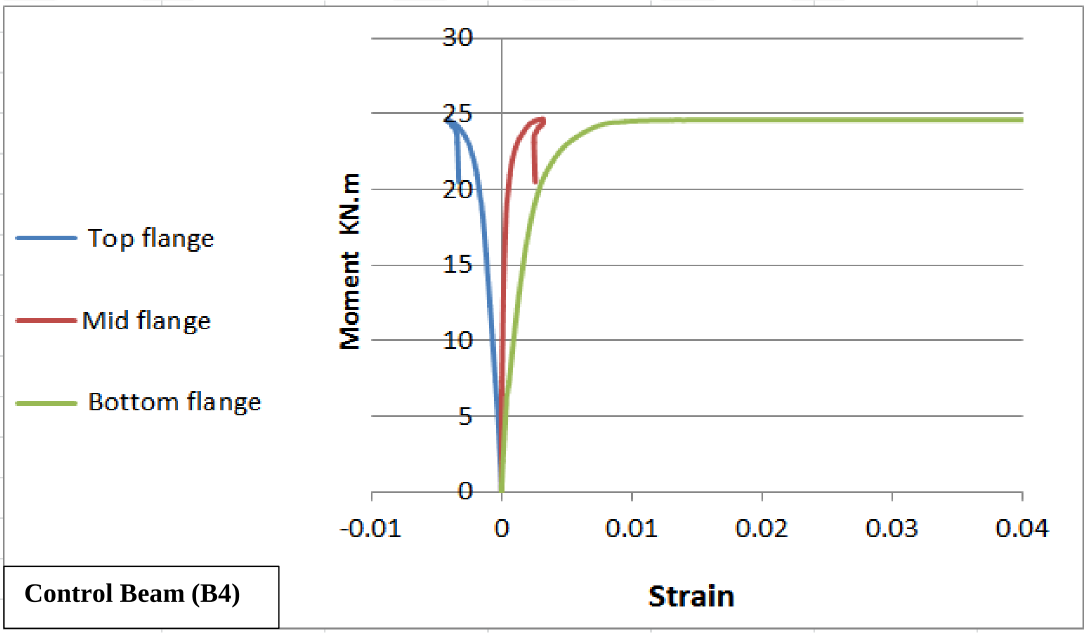


Figure 4.28 Typical moment vs. maximum compressive and tensile strains for B4

The test results indicated that, at the test beginning, the axis of symmetry of the RLWCFST beams is considered to coincide with the neutral axis of the section. Therefore, the strain readings on the mid height, show approximately nil values before the tension flange strain reach to yield strain. After that, all strains increased with a slow rate as the moment increased. After the yielding of the tension flange had been reached, all strains grew quickly. The neutral axis of the composite section moved up, and thus, the mid-height web strain increased as a tensile strain.

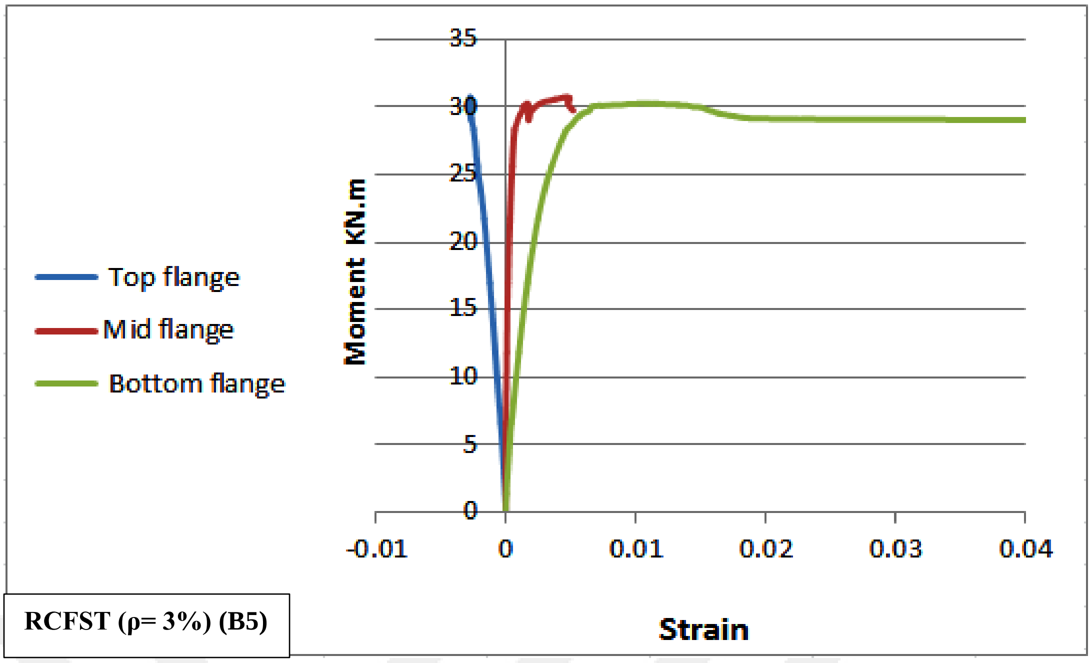


Figure 4.29 Typical moment vs. maximum compressive and tensile strains for B5

The figures show that both the compressive and tensile strains of the LWCFST and RLWCFST beam specimens are exceeded the yielding stage before reaching the yield plateau, which indicates that the top and bottom steel flanges of the beams specimens yielded before reaching their bending moment capacity.

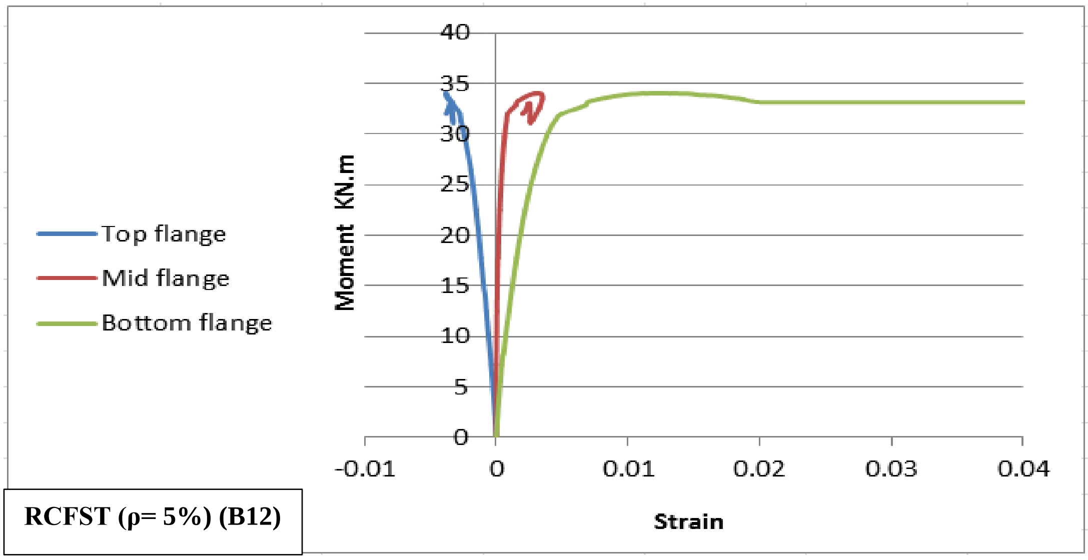


Figure 4.30 Typical moment vs. maximum compressive and tensile strains for B12

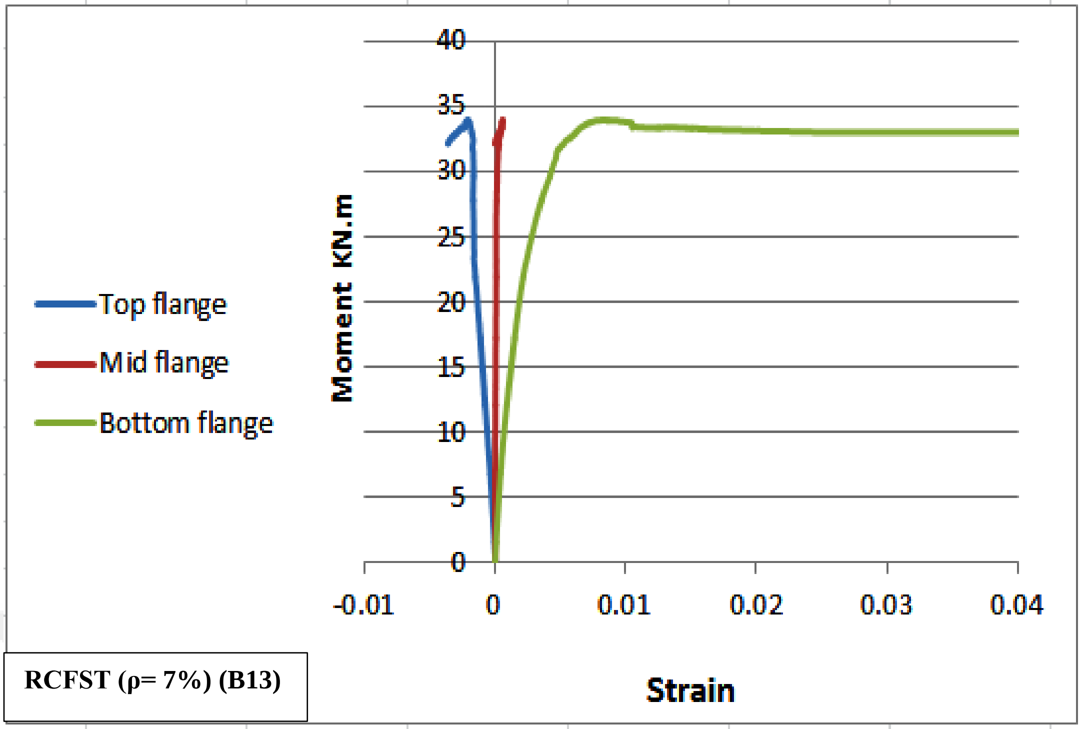


Figure 4.31 Typical moment vs. maximum compressive and tensile strains for B12

Finally, the results revealed that the moment versus strain relationships of the RLWCFST beam exhibited an approximately similar trend to that of the control LWCFST beam specimen without reinforcement. However, the main differences are: First, RLWCFST beams gained higher levels of moment capacity and stiffness, due to the existence of the additional amount of steel which provides more efficient confinement for the core concrete. And this performance increase with the increase in reinforcing bar ratio. Second, the moment versus the tension side strain curves of the control specimen showed smooth and almost constant bending moment after reaching the maximum moment. On the other side, the moment versus the tension side strain curves of the RLWCFST beams showed some different behavior represented by a reduction in flexural strength after reaching the peak value.

4.2.6 Comparison of experimental moment capacities with EC4-2004 and AISC -360 -2010

4.2.6.1 General

The moment capacities of the specimens were computed by the specifications in the codes EC4-2004 and AISC-360-2010.

4.2.6.2 Comparison of results between the codes and the tests

Table 4.5 shows the calculations for the two codes. The codes, EC4-2004 and AISC-360-10, underestimated the moment resistance of LWCFST beams. In Table 4.4, M_{ue} and M_{uc} shows the experimental and the calculated moment capacity from the codes, respectively.

In general, EC4 and AISC provided a good assessment of the moment capacities with the same average calculated-to-experimental moment capacity (M_{uc}/M_{ue}) ratio of 0.96. The two codes had the same minimum (M_{uc}/M_{ue}) ratio were 0.85.

The minimum (M_{uc}/M_{ue}) ratio (0.85) that predicted by two codes was with LWCFST beams. the maximum (M_{uc}/M_{ue}) ratio was 1.07 with B8 and 1.05 with B13 by EC4 and AISC respectively. On the other hand, the average (M_{uc}/M_{ue}) ratios were 0.87, 0.97,0.97 and 1.00 that predicted by AISC code for LWCFST beams and RLWCFST beams with ρ= 3%, ρ= 5% and ρ= 7% respectively while 0,87,0.98,0.97 and 0.97 with EC4 code as shown in Table 4.4.

From the comparisons mentioned above, it can be seen that both of these design codes are successful to predict the ultimate moment capacities and safe for RLWCFST beams, especially EC4 that limited the maximum ρ with 6% while this study exceeded this limit up to 7%.

Table 4.5 Design codes calculations

Beam. No.	Lab symbol	Beam Type	Ultimate Moment KN. m	Average Ultimate Moment KN. m	Mu calculate d by AISC KN. m	Mu calculate d by EC4 KN. m	M_{uc} AISC / M_{ue}	M_{uc} EC4 / M_{ue}
B1	1-3	Hollow Beam	8.01	8.12				
B2	1-4		8.22					
B3	2-3	Control Beam	24.52	24.6	21.75	20.94	0.89	0.85
B4	2-4		24.68		20.99	21.94	0.85	0.89
Average					21.37	21.44	0.87	0.87
B5	3-3	RCFST (ρ= 3%)	30.74	30.74	29.00	29.26	0.94	0.95
B6	3-4		30.99	30.99	29.68	29.13	0.96	0.94
B7	6-3		30.73	30.73	29.55	30.04	0.96	0.98
B8	6-4		29.75	29.75	30.27	31.69	1.02	1.07
Average			30.55		29.63	30.03	0.97	0.98
B9	4-3	RCFST (ρ= 5%)	30.74	30.74	30.56	30.34	0.99	0.99
B10	4-4		31.99	31.99	31.27	29.95	0.98	0.94
B11	7-3		33.34	33.34	32.02	32.86	0.96	0.99
B12	7-4		34.03	34.03	32.05	33.18	0.94	0.98
Average			32.52		31.48	31.58	0.97	0.97
B13	5-3	RCFST (ρ= 7%)	33.96	33.96	35.77	33.51	1.05	0.99
B14	5-4		36.24	36.24	36.47	35.57	1.01	0.98
B15	8-3		40.25	40.25	37.36	37.64	0.93	0.94
B16	8-4		36.96	36.96	37.24	36.77	1.01	1.00
Average			36.85		36.71	35.88	1.00	0.97
Avg. (all specimens)							0.96	0.96
Discretions:	M_{uc}: Theoretical ultimate moment calculated by code M_{ue}: Experimental ultimate moment							

CHAPTER 5

5.1 Conclusions

This study presented an experimental research and evaluated the flexural performance of Reinforced Lightweight Concrete Filled Steel Tubular beam. 16 beam specimens were tested, 12 specimens as Reinforced Lightweight Concrete Filled Steel Tube (RLWCFST) beams, two specimens as Lightweight Concrete Filled Steel Tube (LWCFST) beams and two specimens as Square Hollow Steel Tube (SHS) beams. The main experimental parameters were: (1) ratio of longitudinal steel reinforcement (ρ) from 3% to 7%; (2) arrangement of longitudinal steel reinforcement (in two or three layers), and (3) the spacing of stirrups (6cm or 12 cm). The following conclusions are based on the results of this study:

- The ratio of longitudinal reinforcement (ρ) has significant effect on bending performance of RLWCFST beams by increasing the bending strength from 21% (ρ=3%) to 64% (ρ=7%) with respect to LWCFST beams, and therefore would help the steel tube and concrete to improve their performance.
- The effect of arrangement of longitudinal reinforcement in increasing the ultimate bending strength was obvious in beams with ρ= 5% and ρ= 7% while it was with slight effect in beams with ρ= 3%.
- The RLWCFST beam (B15) had the higher ultimate bending strength between the other RLWCFST beams where it had a percentage of increase about 64% over the unreinforced LWCFST beams and about 26% over the RLWCFST beams that have the same ρ= 7% (B13, B14 and B15). Also, B15 had a percentage of increase about 9% over the B16 that had the same ρ and the same arrangement of longitudinal reinforcement (two layers) but different in spacing for transverse reinforcement which in turn increase the confinement for concrete core and therefore increase bending strength.

- RLWCFST beams developed larger deformation capacity at the end of test (rupture at tension flange compared to LWCFST beams especially for the specimen with ρ= 3% and ρ= 7%).
- The ductility for reinforced LWCFST beams higher than that for LWCFST beams. The longitudinal reinforcement had significant effect on the ductility of LWCFST beams and developed it to better performance.
- EC4 and AISC design codes predicted good estimate and safe design moment capacities for RLWCFST beams especially EC4 that limited the maximum ρ with 6% while this study exceeded this limit up to 7%. However, test results showed that exceeding this ratio (6%) up to 7% is possible and EC4 design method is applicable, even if the limitation of ρ was increased up to 7%.

5.2 Recommendations for future research

- The current test series on the performance of RLWCFST beams focused on a single b/t ratio (120/3). However, to evaluate the impact of the b/t ratio on the response of RLWCFST beams, additional tests, using different b/t ratios, are required.
- The current test series of RLWCFST beams only focused on the square section for steel tube (120 mm x 120 mm). However, additional tests, using other sections as rectangular section to study the influence of depth to width on the performance of RLWCFST beams.
- This study takes the range of longitudinal reinforcement (ρ) from 3% to 7%. Additional tests out of the range of this study, especially more than 7% can be conducted on RLWCFST beams.
- This study takes a symmetrical distribution for the layers of longitudinal reinforcement. However, to evaluate more obvious on the effect of longitudinal reinforcement on the neutral axis of RLWCFST beams, additional tests, using unsymmetrical distribution for the layers of reinforcement.
- This research depends on steel bars as reinforcement therefore, other research by using a different type of reinforcing bars such as FRP bars with lightweight concrete-filled steel tubular beams can be carried out.

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