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M.Sc. in Civil Engineering

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UNIVERSITY OF GAZİANTEP
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CONCRETE FILLED STEEL TUBE BEAMS WITH SQUARE SECTIONS: EFFECTS OF NOTCHES ON THE FLEXURAL PERFORMANCE

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M.Sc. Thesis
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Supervisor
Asst. Prof. Dr. Mehmet Tolga GÖĞÜŞ

by

Mohammed Muneam Mohammed
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This is to certify that we have read this thesis and that in our consensus opinion it is fully adequate, in scope and quality, as a thesis for of the degree of consensus or majority Master of Science.

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Mohammed Muneam Mohammed

ABSTRACT

CONCRETE FILLED STEEL TUBE BEAMS WITH SQUARE SECTIONS: EFFECTS OF NOTCHES ON THE FLEXURAL PERFORMANCE

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Bending tests of the Concrete-Filled Steel Tube (CFST) beams with artificial notches has been conducted in this thesis in order to study the effects of the material imperfections which is direct reduce the performances of steel tubes on beam. This thesis presents the results of an experimental study of bending behaviors of CFST beams with notches. The square shape of steel tube with two different thickness has been studied the design limitation; Thicknesses (2mm, 5mm) a total of 26-specimens which is consist of 24-with include notch CFST beams, whilst two tube control without notch. The flexural performance of Self-Compacting Concrete (SCC) filled steel tube beams are examined, the CFST with notches from the top to bottom and sidewall with two different thicknesses and two different lengths of the notch (long and short). The moment carrying capacities, ductility, failure modes, moment - mid-span deflection relations and bending moment capacities. Eventually, the results were illustrated that the notched CFST specimens had lower mechanical performances than the CFST beam specimens without notch. Furthermore, the investigating The performance of the beams decreases as the length of the notch increases for both the thicknesses (2mm,5mm).

Keywords: Self-compacted concrete (SCC), Concrete Filled Steel Tube (CFST) beams, notches, flexural strength, ductility.

ÖZET

KARE KESİTLİ İÇİ BETON DOLDURULMUŞ ÇELİK TÜP KİRİŞLER: ÇENTİKLERİN EĞİLME PERFORMANSINA ETKİSİ

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63 sayfa

Bu tezde,İçi Beton Doldurulmuş Çelik Boru (İBDÇB) kirişlerinin yapay çentikli eğilme testleri ile kiriş üzerindeki çentiklerin kiriş performanslarını doğrudan azaltan malzeme kusurlarının etkilerini incelemek amacıyla yapılmıştır. Bu tez, İBDÇB kirişlerinin çentik ile eğilme davranışları üzerine deneysel bir çalışmanın sonuçlarını sunmaktadır. İki farklı kalınlıkta kare çelik borunun tasarım sınırlaması üzerinde çalışılmıştır; Kalınlıklar (2mm, 5mm), 24 adet çentikli İBDÇB kirişleri, iki adet (kontrol) çentiksiz İBDÇB kirişleri olmak üzere 26 adet numuneden oluşan set iki farklı kalınlık (2mm, 4mm) olarak incelenmiştir. Kendiliğinden yerleşen beton (KYB) doldurulmuş çelik boruların eğilme performansını araştırmak için İBDÇB üstten, alttan ve yan yüzlerinden iki farklı kalınlıkta ve iki farklı çentik uzunluğunda (uzun ve kısa) sahip çentiklerle kirişler üretilmiştir. Moment taşıma kapasiteleri, süneklik, göçme modları, moment-orta açıklık deplasman ilişkileri, eğilme momenti kapasiteleri sunulmuştur. Sonuç olarak, sonuçlar çentikli İBDÇB kiriş numuneleri çentiksiz İBDÇB kiriş numunelerinden daha düşük mekanik performanslar olduğu gösterilmiştir. Ayrıca, kirişlerin araştıran performansı hem kalınlıklarının (2 mm, 5 mm) çentik uzunluğu arttıkça azalmıştır.

Anahtar Kelimeler: Kendiliğinden yerleşen beton, içi beton ile doldurulmuş çelik tüp kirişler, çentikler, eğilme dayanımı, süneklik.

To my parents….

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Special thanks to my family. Words cannot express how grateful I am to my father spirit that his words lead me to get certificates and my mother for all of the sacrifices that have made on my behalf, and also to my brothers and sisters.

TABEL OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS/ABBREVIATIONS

CFST	Concrete-Filled Steel Tube.
SCC	Self-Compacted Concrete.
F_u	Ultimate strength of steel.
F_y	Yield strength of steel.
Mu	Moment capacity.
Du	Mid-span deflection at ultimate moment.
DI	Ductility Index.
B/H	Width to depth ratio.
C100×100	Control beam 100mm×100mm.
S100-5-TLH	Beam square 100mm×100mm, 5mm thickness of tube, top, long notch horizontal.
S100-5-TSH	Beam square 100mm×100mm, 5mm thickness of tube, top, Short notch horizontal.
S100-5-TLV	Beam square 100mm×100mm, 5mm thickness of tube, top, long notch vertical.
S100-5-SLH	Beam square 100mm×100mm, 5mm thickness of tube, side, long notch horizontal.
S100-5-BLH	Beam square 100mm×100mm, 5mm thickness of tube, bottom, long notch horizontal.
S100-2-BLH	Beam square 100mm×100mm, 2mm thickness of tube, bottom, long notch horizontal.
S100-2-SSV	Beam square 100mm×100mm, 2mm thickness of tube, side, short notch vertical.
LVDT	Linear Variable Differential Transistor.

CHAPTER 1

INTRODUCTION

1.1 General

The purpose of using two or more materials in construction is to make full use of their properties. In order to get the best performance for the structure (Oehlers, D.J., and M. A,1999), Composite members consist of square, circular and rectangular steel tube filled with concrete are widely used for constructing modern buildings, bridges and electrical power transmission (Tao et al., 2006), (Wang et al., 2015) and (Tao et al., 2007), Therefore, it differs from material in terms of features. There are no materials meeting all construction requirements.

Composite Concrete Filled Steel Tube (CFST) is progressively being used as beam column, beam and column in braced and un-braced frame structures (Mohanraj et al., 2010), (Elchalakani, M., et al., 2004) and (Ghannam, S. 2016). The composite construction structure provides numerous structural advantages contain high strength and fire resistance, high energy absorption capacity and favorable ductility. There is also no need to use a shutter during concrete construction. Therefore, time and construction costs are reduced. These advantages are widely used and have resulted in widespread use of concrete in-filled tube structures in civil engineering structures (Han et al., 2014). Also used all over the world is used as a primary lateral resistance from a compression member of an open-floor plan of a low-rise structure utilizing cold-formed steel tubes in-filled with in-situ concrete large caliber cast in situ or precast (Ghannam, S. 2016). The difficulty of compacting concrete is that concrete bleeding can create weaknesses in the system that creates gaps between steel and concrete.

Also used all over the world is used as a primary lateral resistance from a compression member of an open floor plan of a low-rise structure utilizing cold-formed steel tubes in-filled with in-situ concrete large caliber cast in situ or precaconcrete Protect them from buckling. Concrete protects from corrosion, insulation at high temperature (Oehlers, D.J., and M. A,1999). Composite member is made of steel tuber filled with concrete, Steel distinguish.These composite CFST members are optimized for the use of both steel and concrete compared to steel structures and concrete structures. The steel tubes provide confinement in the concrete filling, but concrete filled delays local buckling of steel tubes (Tao et al., 2007) (Lai, Z., 2014) and (Alfawakhiri, F., 1997). The behavior of the CFST member combining axial load, flexure, and flexural loads may be more competent than reinforced concrete members or structural steels. In addition, the steel tube functions as a formwork for placing concrete, making construction easier and faster while reducing the cost of constructions (Zhichao Lai, 2014). The CFST beam is proper for all the towers in the high earthquake area. The use of CFST members is restricted because there is no information on inelastic behaviors and true strength of CFST members. They have many advantages, but the main advantages are as follows (Mohanraj et al., 2010) and (Ghannam, S. 2016).

a. Performance as a permanent and indispensable formwork.
b. Provides external reinforcement.
c. Supports many levels of the structure before pumping concrete.

by high tension and compressive strength. Conventional steel structures use applied fire protection systems to insulate the steel from the heat of the fire. Modern composite steel-concrete, structures (like structural steel sections encased in structural R.C.) concrete in which the concrete protects the steel because of the lower thermal conductivity of concrete. On the other hand, covering concrete by steel plates will provide external reinforcement and protection against external accidents besides being used as formworks.

As with other metal structural components, imperfection always exists external steel tube of CFST members may damage them it guarantees structural integrity and affects service life. There is a show imperfection Steel members are generally classified into two main types: Geometric imperfections and material imperfections. The former is actual geometric dimensions with deviation from perfect shape related to manufacturing tolerances, damage during processing or from the impact that the steel permanently deforms as it dents member (I. Tutuncu,.2001).

Incompleteness of materials is mainly due to defects or thickness variation due to welding, corrosion or other chemicals attack.

In this research, self-filling concrete SCC is used as a filling material for steel tube beams, but it is well known that SCC is widely used in the field of concrete technology. With the introduction of SCC technology, the way to execute a specific operation has changed significantly. In order to obtain more efficient results, by eliminating discontinuous mechanical. SCC one of the new types of high-performance concrete HPC is SCC that can be spread readily under its own weight and fill restricted sections beside the congested reinforcement in structures without a need for mechanical compaction and without any segregation of material constituents. The use of SCC can increase productivity in structural applications such as repair, and simplify the filling of restricted sections. Such concrete has been widely used to facilitate construction procedure, especially in sections presenting special difficulties to casting and vibration such as bottom sides of beams, girders, and slabs (S. Hwang, Khayat, K.H et al,2006).

1.2 Research objectives and significance

In view of the above literature, it can be conducted that there is limited research in the field of CFST beam with the notch. Furthermore, up to the best knowledge of the authors, no any previous study was carried out on the CFST beam with the notch. So that the research in this field is still in the primary stage and more efforts are required to fill this gap in the literature. Thus, the aim of this investigation was thus fivefold: first, to debate the action of four measurement methods in the axial deformation of beam specimens; second, to investigate the effect of concrete strength on the load capacity of the beams; third, to study the effect of small notched on the perimeter (with different vertical and horizontal dimensions) of the steel tube on the maximum capacity, the load-deformation behavior, and the failure mode of the beams; fourth, to check into the behavior of these beams in confinement; and fifth, to compare the experimental results of the unnotched control specimens with the CFST beam notch specimens. To examine the effect of the notch and different parameter on moment and ductility, such as, different steel plate thickness and arrangement.

1.3 Layout of the thesis

Consists of five-chapter summary and abridgment of each chapter are given as below:

Chapter 1. Introduction:

A general and simplified explanation about CFST, SCCFST and imperfection (notch) in CSFT members was presented as well as review the importance of research.

Chapter 2. Literature review:

Previous research based on the scope of the study has been reviewed and maintained. A review of the literature and general background about CFST and effect of notch in steel tubes beams.

Chapter 3. Experimental investigation:

This chapter provided an explanation and experimental investigation about the effect of notch on the CFST beam square steel tubes.

Chapter 4. Results and discussion:

Results obtained from the effect of notch in the CFSTs. Discussion of the results of the study was defined in this chapter.

Chapter 5. Conclusions:

Provide conclusions and summary based on the results of comparative investigations:

CHAPTER 2

LITERATURE REVIEW

2.1 Concrete Filled Steel Tube Beams

CFST members take advantage of both concrete and steel. These consist of a circular or rectangular or square steel hollow section filled with normal or reinforced concrete. They are using widely as a beam of low-rise industrial buildings that require a rugged and efficient structural system and as high-rise buildings such as columns, beam-columns. This type of composite beam is simply a structural steel section it is interconnected to a concrete slab attached to the top flange by the shearing connector. This is commonly used on bridges. Many theories have been suggested to simulate the actual behavior of such beams. One of the earliest studies presented to investigate the behavior and strength of such composite beams is attributed to (Newmark, N.M., Seiss.al,1951).

Chapman, (1964) studied the behavior of seventeen simply supported composite steel-concrete beams under the concentration of distributed loads. The amount of shear connection was different within the range and the effect of interface slip-on elastic and ultimate load behavior was observed. It was observed that the use of ultimate load design for a composite steel-concrete section might lead to working stresses closed from the yield stress because of the large shape factor. This paper suggested that the shear connection should be designed to carry the horizontal shear force existing in the beam at ultimate load. For this objective, experimentally, 80% the inherent maximum capacity of the shear connector should be used. In the case of the uniformly loaded beam, uniform spacing of shear connectors proved to be satisfactory.

Adekola (1968) formulated an interaction theory which picks account of slip, uplift and friction effects without assuming equal bends of interacting elements (mainly steel and concrete). The analysis of a differential element of the beam has led to two coupled simultaneous differential equations connecting the uplift tension arising from differential deflections of the two elements with the axial force within each of the elements. These equations were then solved by the finite difference approach.

A numerical analysis taking into calculation the inelasticity of steel, concrete and shear stud connection was presented by Yam and Chapman (1968). The model was based on several assumptions, one of which is that the concrete slab and the steel beam deflect equally at all points along the beam (i.e. equal curvature is assumed for both components), therefore; uplift or separation is not allowed. A nonlinear behavior is assumed for the three components of the composite section (i.e. Concrete slab, steel beam, and shear connectors). For the concrete material, an elastic-perfectly plastic behavior has been assumed, while for steel a bilinear relation for the stress-strain curve is employed with an elastic strain hardening behavior in tension and compression. Yam and Chapman (1972) studied the made elastic plastic behavior of composite beam of two spans experimental result. Concrete has no tensile strength; horizontal forces are transmitted through the reinforcement moment area. The computer program was written to solve the nonlinearity compute the differential equations using the predictor-corrector method. They are, the second hinge is formed in front of the first hinge, and therefore, a simple method of plastic design can be used. When a sufficient number of shearing connectors are separated uniformly between the points of maximum moment and minimum moment, Connectors will not occur. For continuous composite beams, Slip may have a more significant effect with limited strain compatibility.

An approximate method applicable only to simply supported composites It was announced by Johnson and Johnson in (1975). The range of this method is that all kinds of loads it is a concentrated load off center and usually produces a twisting effect. This method, a simple rule was derived to estimate the ultimate bending strength and the deflection in the use of a composite beam with partial shear coupling. This method has the ultimate flexural strength and the deflection is determined by the linear function by the degree of connection using information from full interaction theory. It was shown in tests that; this method was limited to a degree of interaction or shear connection of not less than 50 percent of full interaction. A smaller number of shear connectors would be ineffective.

Roberts (1985) studied the problem of analyzing composite beams with partial interaction in more detail. The basic equilibrium and compatibility equations were formulated in the expression of four independent variables, which were the horizontal and vertical displacements for both steel and concrete. The linear elastic material and linear shear connector behavior were assumed. Also, the reinforced concrete deck slab was assumed to stay uncracked under service condition. Both slip and separation at the interface were considered. Since then, Stress over the composite beam is divided into four Independent displacement variable; the analysis result is four simultaneous differential equations. Including third order differentiation in the horizontal direction displacement in the vertical displacement of the concrete and fourth order and Steel components. The numerical solution of the basic equation is express the displacement derivative in the finite difference and the whole used four equations at each node to determine the unknowns. Examining the application of the theory simply supports supported composite beam with server load. Normal rigidity assumed that the length per unit length of the shear connector is infinite, i.e. there is no separation equivalent curvatures of components were assumed. Shear the rigidity of the connecting portion per unit length is uniform, the triangular and discontinuous distribution of the shear connector is considered.

Jasim and N.A., (1997) studied to calculate the deflection of continuous composite beams on the same approach (Newmark's model). The accurate solution of the governing differential equations is acquired and the results are so arranged that the deflection of composite beams be to be related to those of corresponding composite beams. Charts are constructed to allow the computation of the mid spam deflections of beams.

Also Burnet and Oehlers (2001), develop procedure for design composite beam with arbitrary shear bond strength This study is a composite beam damage to the shear connector due to excessive slip, The degree of shear connection, the slip capability of the shear connector, and that Not only the span of the beam but also the cross-sectional properties of elastic and plastic as with the beam ductility requirements, in this paper, approach was developed to show the basic parameters that influence damage of shear joints, design rules were formulated. It is actually used.

Zona, A. and Ranzi, G presented in (2011). Done conducted a theoretical study and compared it three different nonlinear beam models and finite elements Analysis of composite members with partial interaction. These models are two Euler-Bernoulli obtained by coupling with deformable shear junction Beams (bending deformability of each beam and bending fracture mode only component), Euler · Bernoulli beam from Timoshenko beam (shear Deformability and shear fracture mode), two Timoshenko beam (addition of shear deformability and shear fracture mode both components). Simply supported and continuous steel-concrete composite the beams whose experimental results are described in the literature. side of the structural behavior believe include: (i) effects of the shear deformability of the steel and slab components at various load levels; (ii) differences in calculate collapse loads; (iii) differences in the internal actions, i.e. axial forces, bending moments, vertical shears and interface shear forces at several levels of loading. A study on the convergence average of the finite element solution and considerations on locking-free finite elements are given. Results show that the three models present small differences when composite beams dominated by the bending behavior are considered.

CFST members take advantage of both concrete and steel. These consist of a circular, rectangular, or square steel hollow section filled with normal or reinforced concrete. There used widely as a beam of low-rise industrial buildings that require a rugged and efficient structural system and as high-rise buildings such as columns, beam-columns.

Lai (2014) presented the evaluation and development of these design provisions. The CFST members have been reviewed available experimental databases and a new experimental database of tests performed on non-compact and slender CFST members has been compiled. Detailed 3D Finite Element Method (FEM) models have been developed for non-compact and slender CFST members and benchmarked using experimental results. Design provisions of the American Institute of Steel Construction (AISC) 360-10 for non-compact and slender CFST members have been evaluated by experimental test results and additional FEM analysis dealing with experimental database gaps. The current AISC 360-10 axial load bending moment (P-M) interaction equation has been updated using the results of a comprehensive parametric study (directed using a benchmarked FEM model). Effective stress-strain curves of steel tubes and concrete filled have also been advanced. Validation of these effective stress-strain curves has been confirmed by implementing them in a macro model based on benchmarked nonlinear fiber analysis.

Alfawakhiri (1997) presented additional experimental data on the behavior of these structural elements and assess recent design code provisions with regard to circular CFST columns. Three long beam-columns (L/D = 13.5) have been tested under continuous axial compression and incrementally enhanced lateral displacement reversals.In addition, one short column was tested in concentric compression. All specimens had the same cross-sectional geometry to allow for direct comparison. Both strength and deformability issues were addressed. Various aspects of concrete confinement in circular CFSTs were analyzed. A comparative study of North American and European design code provisions was carried out. The result of the test indicates that by using of high strength concrete for circular CFST members does not cause any dramatic changes in the behavior. North American codes were shown to be more conservative than Eurocode 4, but all codes gave conservative predictions of strength compared to the experimental results. A less conservative design approach was recommended for Canadian Standards Association (CAN/CSA-S16.1-94).

Elchalakani et al. (2001) showed investigational studies on the bending behavior of circular CFST subjected to large deformation pure bending of d/t 12 to 110 have been conducted. Their paper compares the behavior of void-filled and empty, circular hollow cross-sections cold-formed under purely plastic bending. The concrete filling has been found to completely prevent local buckling and elliptification of cold-formed steel tube with $13 < d/t < 40$, but in the inelastic range of CFST of $74 < d/t < 110$, A plastic ripple has been detected. Energy absorption, strengthening, and ductility is thinner than in the case of, particularly thick CHS. The plasticity limit of d/t 112 has been obtained for a CFST constructed under a pure bend from a cold-formed hollow section.

Chitawadagi et al. (2009) prepared a paper on the strength deformation behavior of circular steel tubes in-filled with concrete of different grade under bending. The influence of steel tube thickness, concrete filling strength, concrete cross sectional area and concrete confinement on moment capacity and curvature (CFST)s were investigated. The measured flexural strength is compared with the value predicted by EC 4-1994 and Load and Resistance Coefficient Design (LRFD) - AISC - 1999 code specification. A total of 99 specimens with all lengths of 1 meter was tested with concrete filling with characteristic strengths of 20, 30 and 40 Nmm^2 and a (d/t) ratio of 22.3 to 50.8.

Based on the investigational results, an interaction model has been developed to foresee the moment and curvature of the CFST specimens. The results indicate that in the experimental investigation are used to all the hollow section resistance moments and the corresponding increase in curvature showed higher ductility than the hollow. The ductility and moment of resistance of both the hollow and the CFST sample are increased by increasing the wall thickness of the steel tube. For a given wall thickness of the CFST specimen, increasing the strength of the filled concrete will not help to significantly increase its capacity. The Strength Increase Factor (SIF) proposed here helps to estimate the concrete contribution to the moment capacitance of the CFST beams. Detention of infill contributes of concrete to higher performance while higher strength infill concrete has been utilized for higher wall thickness steel tube. Furthermore, EC 4-1994 foresees the moment capacity of the test specimens moderately in the range of 30% to 35%. Since the influence of concrete filling is not taken into consideration and AISC / LRFD is also the same, it is very conservative to predict the moment capacity of the CFST section.

Han et al. (2014) studied the development of the family of (CFST) structure until now, and draw out the research framework of CFST members. Figure 2.1 shows a cross-section of a typical CFST. Recent CFST structural members, especially research and development in China, have been summarized and discussed. Several projects in China utilizing CFST members have also been introduced. Finally, some conclusions are given for CFST members.

The results observed that the rapid development of research and application of CFST structures in China and all over the world in the previous decades, the scope of "CFST" is extended greatly by researchers and engineers. The characteristic of these CFST members is that the structural properties can be improved due to the "composite action" between filled concrete and steel tube. Basic design approaches from various countries and some typical applications of CFST members in buildings, bridges, and other structures are presented and compared. The CFST structure can be treated as a substitute system to the steel or the concrete reinforcement.

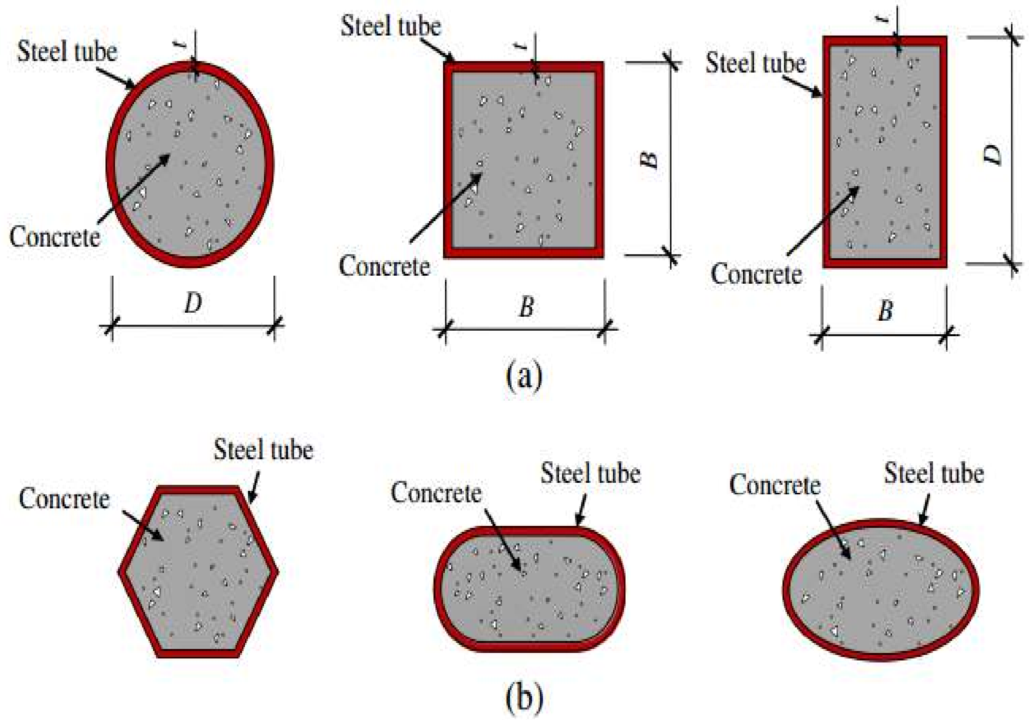


Figure 2.1 Typical concrete-filled steel tubular cross sections.

Ghannam (2016) studied an experimental investigation of the bending behavior of a square (CFST) subjected to pure bending, as shown in Figure 2.2 where the depth to thickness ratio (D/t) is 23.8 and 27.8. In this paper compares the behavior of rectangular hollow hot rolled sections concrete infill and empty under pure bending. The behavior of CFST beams partially replaced with coarse aggregate using granite under bending load has been shown. The effect of steel tube, concrete compressive strength and concrete confinement were investigated. Six specimens were tested at the concrete strength of 20 MPa. The beam has been 88.9 × 88.9mm, 114.3 × 114.3mm, and the length has been 1200mm.

From the result of tests, it has been found that the ultimate load carrying capacity of the steel tube beam in which the coarse aggregate is partially replaced with granite is nearly the same as the concrete conservative. Generally, filled steel tubes increase ductility, strength and energy absorption, especially for the thinner section. The steel shell results in restraint of the concrete core, resulting in improved ductility and strength of the core.

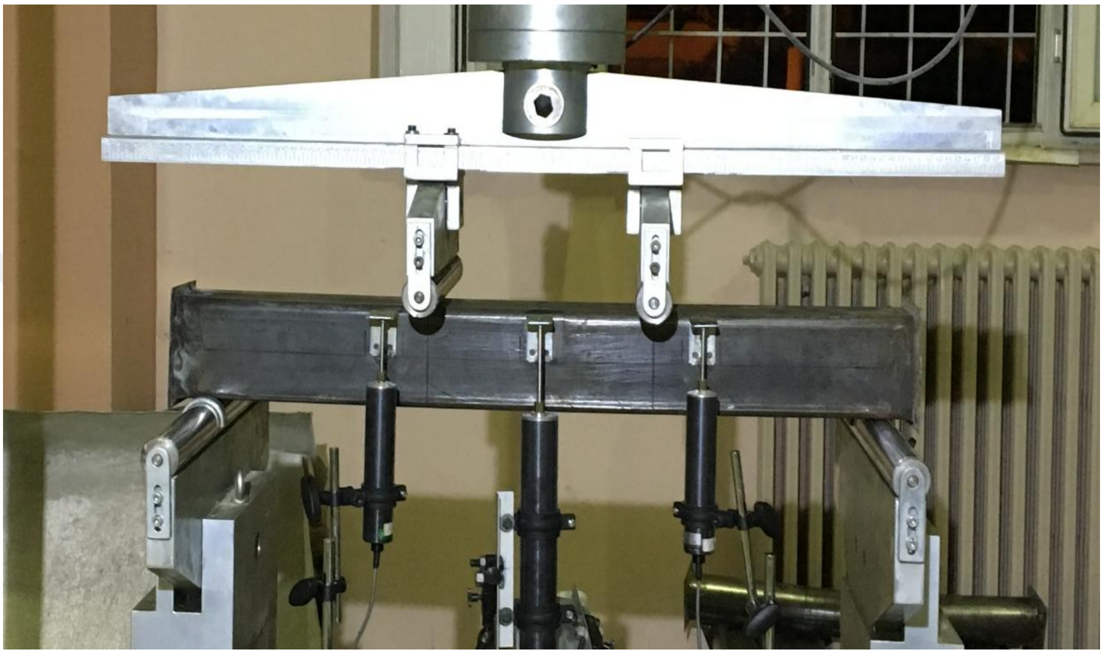


Figure 2.2 Investigational Test set up (Ghannam, 2016).

Tao, Z., & Han, L. H. (2006) described a chain's of tests performed on concrete infilled double-skin steel tube (CFDST) stub columns, beams, and beam columns. Rectangular Hollow Section (RHS) cold-formed both outside of the tube and inside of the tube. The fracture mode and load deformation behavior of CFDST specimens have been compared with those of conservative CFST members and empty double-skin tube members.

The results observed that the CFDST members showed behaviors similar to the conservative CFST specimens in the current test. Improvement in strength and ductility were observed for CFDST (RHS inner and outer) stub columns, beams and beam columns due to the composite action between the concrete core and the steel tube. A theoretical model has been advanced to foresee the load versus deformation relationship of CFDST stub column, beam and beam column. A good agreement has been attained between the prediction curve and the experimental curve. Simplified models were advanced to assess the strength of CFDST stub beam-columns, beams and columns. With reasonable agreement achieved, all predictions were compared with the result of tests.

Li et al. (2015) presented both experimental and numerical investigations on CFST tension members subjected to eccentric loads. Experimental parameters include load eccentricity and cross section steel ratio. Failure modes showed that in-filled concrete workings efficiently on steel tube and outside of tube improves concrete crack pattern and also modeling was used to predict the same result. The failure mode of CFST tension member subjected to eccentric load is a combination of elongation and all-inclusive bending. Composite members subjected to all eccentric loads showed large deformability when the rotation of the final end exceeded 0.1 rad. The compressive area may happen in the composite section while the member receives an eccentric load. Due to the composite effect of steel and concrete, the strength of composite specimen was much greater than hollow steel tube. The linear tension-to-moment interaction relationship shows acceptable results for CFST members. The recent design equation gives a safe and conventional estimate of the tension and flexural strength of CFST tension members subjected to eccentric loads.

Zhou et al. (2017) studied the mechanical behavior of the circular (CFST)s under axial tension. A group of three circular CFSTs with different steel ratios and in-filled concrete diameters are tested and compared with Hollow Steel Tubes (HTs). Based on the experimental results in the present and previous studies, three key issues of circular CFSTs under tension, i.e. the confining-strengthening effect, the confining stiffening effect, and the tension-stiffening effect, are studied at both the stress-resultant level (i.e., force displacement relationship) and stress levels (i.e., stress-strain relationship). At the stress-resultant level, the test results demonstrate that the strength and circular stiffness CFSTs are average 10% and 29% larger than those of HTs, respectively. The strength increase is contributed by the confining-strengthening effect, and the stiffness improvement is a combined result of the confining stiffening and tension-stiffening effects. At the stress-strain level, the measured strains show that the steel tube of the circular CFST is in the state of the bi-axial tensile stress state because of the difference of the Poisson’s ratio between steel tube and infilled concrete, which contributes to the confining strengthening and confining-stiffening effects of the circular CFSTs.

Roeder et al. (2010) studied the combination of axial force and bending load and decided the best model to predict stiffness and circular resistance CFST. 122 specimen databases have been gathered and assessed.

The result shows that the plastic stress way is a simple but effective method to predict the resistance of a circular CFST component under combined load. These data indicate that the current specification provides an inexact prediction of flexural stiffness and a representation of new stiffness is suggested. The suggested model allows simple but exact prediction of resistance and stiffness, allowing engineers to routinely use CFST constituents in structural design.

Varma et al. (2002) investigated the experimental behavior of square CFST columns witches made from high strength material. The influence of steel tube width to thickness ratio, yield stress and axial load level on stiffness, strength, and ductility of high strength CFST beam column have been studied. 16 3/4 scale CFST specimens containing 8 monotonic beam-column specimens and 8 cyclic beam-column specimens have been tested. Experimental results indicate that cyclic loading has no important influence on the strength or stiffness of the CFST beam column. However, it causes a more rapid diminution in the peak of post moment resistance.

Ammari et al. (2013) investigated the experimental behavior using the new system of the U-link of the CFST beam joining the concrete core on the compression side of the beam to the steel tube, resulting in a large confinement of the concrete core. Detention is attained by using the expected split-up among the two materials. As shown in Figure 2.3, two built-up CFSTs were experimentally tested to study the effect as reinforcement when U-link was used for a beam subjected to bending load. The state of the concrete in the tube did not show crash of the concrete when those beams have been cut at the position of the plastic hinge. Measurement of strain shown that the compressive strain of concrete is 5 to 6 times the crash strain of concrete. Investigational data exhibited that the use of a U-link as an added reinforcement in concrete filling tube improves beam bending capacity, stiffness, and ductility.

Figure 2.3 Flexural testing of specimens (Ammari et al., 2013).

Vijay and Manoj (2014) presented a study on the flexural behavior of CFST based on the previous effort carried out by Manojkumar as shown in Figure 2.4. An ANSYS model has been developed that can predict the behavior of CFST to determine the moment carrying capacity at ultimate point. The CFST beam is verified and verified against experimental data by the finite element method program ANSYS. The main parameters affecting the behavior and strength of concrete filled beams, geometric parameters, material nonlinearity, load, boundary conditions, and the degree of concrete containment. To account for all these characteristics, the ANSYS model has been developed. The main parameter that changes in analytical research is the D/t ratio, which is the characteristic strength of filled concrete. The suggested model foresees the ultimate moment capacity of the CFST beam. For numerical analysis, circular and rectangular CFST cross sections are taken into account using different grades of concrete. Compare the predicted value with the experiment result. Numerical analysis shows that a good confinement effect can be obtained with a rectangular CFST.

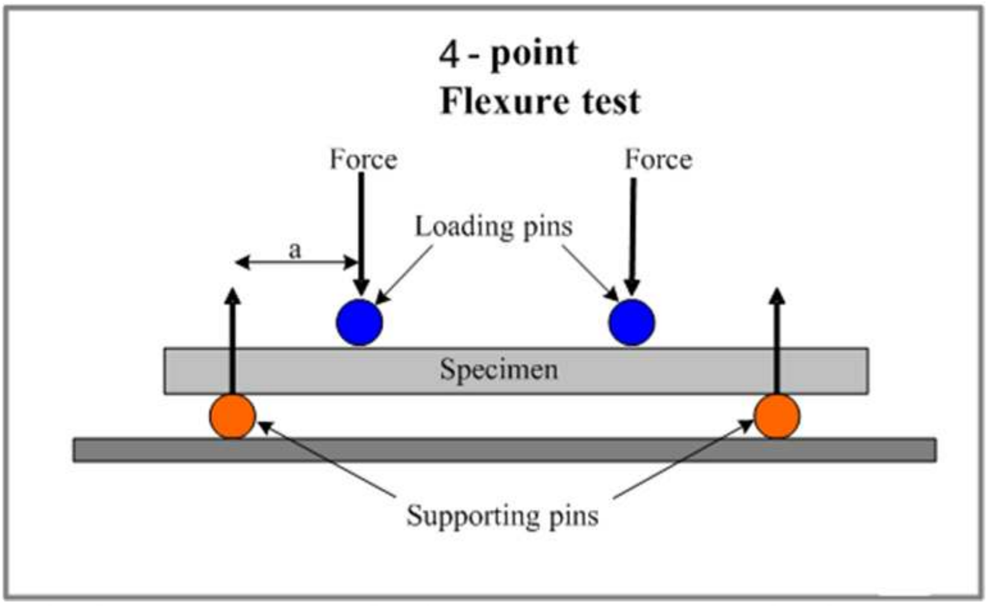


Figure 2.4 Flexure test on CFT beam (Vijay and Manoj, 2014).

Han (2004) developed a mechanics model that can expect the behavior of concrete infill Hollow Structural Section (HSS) beams. A series of rectangular and square CFST beam tests were performed. The main parameters varied in the tests were the depth to width (H/B) ratio and tube depth to wall thickness ratio. The relationship between load and lateral deflection was recognized for CFST beams both theoretically and experimentally. The results observed that the CFST beam behaved relatively ductile and the test continued in a smooth and controlled manner. The moment capacity of HSS beams could be conservatively predicted by using the recommendations of Architectural Institute of Japan (AIJ 1997), British Standard (BS5400 1979), EC4 (1994), LRFD-AISC (1999), and the suggested model in their paper. Generally, AIJ (1997) and LRFD-AISC (1999) showed moment capacity about 20% lower than the test. In addition, BS 5400 (1979) showed a moment capacity about 12% lower than the test. Also, the method suggested as EC 4 (1994) predicted a capacity about 10% lower than the result of the test.

Han et al. (2006) studied the flexural behavior of 36 composite beam specimens infill with Self-Concrete Concrete (SCC). The main parameters that change in the test are as follows. (1) The ratio of tube diameter-to-wall thickness. (2) Steel yield strength; (3) Section type. A comparison is made with the beam capacity predicted using the existing method. In general, the CFST beam filled with SCC was found to be very similar to the beam of composite beams filled with normal concrete. Simplified models for computing the moment versus deformation relationship of the composite beam have been advanced. Moment versus curvature relationships between mechanic models, test results, and simplified models are compared and generally, good agreement is achieved.

Lai (2015) presented the basis of the current AISC specification (AISC 360-10) on the design of non-compact and circular of slender CFST members under axial compression, bending, and combined-load. An investigational test database performed on compact, circular of slender CFST members have been collected and used to create maintainability of AISC specification design equations. Comparison with both the experimental results and finite element analysis showed that the circulating force of CFST members can be calculated conservatively using the (AISC 360-10) design equation.

Oyawa et al. (2004) sought alternative filling materials (polymer base) for very limited cement concrete used in CFST construction. Polymers are successfully used in other industries, are lighter, tensile strength is high, are durable and are known to withstand destructive environments. The results of this study on the elasto-plastic properties of homogeneously bending polymer CFST beams show an enormous increase in composite beam stiffness, strength, and ductility in empty steel tubes.

Results revealed that it became clear that epoxy polymer concrete creates composite steel beam with greatly improved strength and ductility. Epoxy polymer concrete has a wide range of interactions with steel sections because of low tensile and compressive and low inward motion of the cross section neutral axis due to high tensile adhesion even in the inelastic range.

Kvedaras et al. (2013) presented a method for designing a hollow and solid CFST beam based on test data. This is because the limit of increasing the strength of concrete due to detention contravenes the recommendation of 6.7.2 and full composite action of concrete and steel is assumed because conformity with the test result is better than EC 4 (CEN 1994). The proposed analytical method shows a good agreement with the natural and numerical test results with a hollow and solid CFST beam and makes it possible to recommend the method presented in such an efficient composite beam design practice.

Mohiuddin et al. (2016) investigated the flexural properties of CFST partially replaced with coarse aggregate. Their paper showed the use of CFST beam partially replaced with coarse aggregate by 10% and 30% under loading and an attempt has been made using recycled aggregate instead of convention aggregate Results concluded that the use of recycled aggregate up to 30% in CFST have similar strength while compared with that of normal plain reinforced beam. The ultimate load of steel tubular beam infill with normal concrete is about 42.30% increase than that of the hollow tubular reference beam and the percentage decrease of partially replaced coarse aggregate with infill normal concrete is about 2.73%, 4.37%.

Arivalagan et al. (2010) presented a study on the flexural and cyclic behavior of hollow beam sections of concrete infill steel. Test specimens in-filled with normal mixed concrete, fly ash concrete, quarry waste concrete and low strength concrete (brick - butt - lime concrete) and hollow steel sections have been tested. The beam capacity has been compared with the ultimate capacity achieved by international standards EC 4 - 1994, ACI - 2002 and AISC - LRFD - 1999. The investigational study revealed that the moment carrying capacity of ordinary mixed concrete 28%, fly ash concrete 27% and quarry waste concrete 25% was increased by the beam specimens compared to the hollow steel section. Concrete filled increases the residual-load capacity of thin SHS and RHS beams that withstand cyclic loads, especially while increasing lateral displacement and also increasing the energy absorption of hollow steel sections.

Soundararajan and Shanmugasundaram (2008) have experimentally studied the contribution of the normal mix, fly ash, quarry waste and low strength concrete (brick bat lime concrete) to the ultimate moment capacity of the hollow square steel. The same dimensions of 15 simple supported beam specimens of 1200mm long steel hollow sections in-filled with normal mixtures, quarry waste, fly ash and low strength concrete and identical dimensions of hollow sections have experimented. These experimental findings showed that normal mixes, fly ash, quarry waste and low strength concrete enhance the moment carrying capacity of steel hollow sections.

Mohanraj et al. (2010) showed experimentally investigation of the bending behavior of square CFST while d/t ratios were 23.6 and 25.4. Their papers compare the empty and void filling behavior of hot rolled square hollow sections under purely plastic bending. Partially replaced with fine aggregates by using fly ash and quarrying dust and by partially replaced with coarse aggregate and granite, construction and demolition debris, rubber, steel slag, tiles and marble behavior of square CFST beams have been presented. The steel tube effects, concrete compressive strength, and concrete detention have been investigated. In their study, 20 specimens were tested at the concrete strength of 20 MPa. The beams have been (91.5 x 91.5) mm, (113.5 x 113.5) mm cross section and 1200 mm long. From the test results, it has been detected that the ultimate loading capacity of steel tube beams infills with different waste materials, concrete is about the same as conservative concrete. Generally, void filling of steel tubes improves ductility, energy absorption, and strength, particularly of thin-walled sections.

2.2 Concrete Filled Steel Tube columns

For many years, Extensive studies were conducted to investigate the behavior of CFST columns. It was found that CFST columns had favorable behavior and load capacity due to merge the merits of concrete and steel L. Han et al (2014). The typical mechanical behavior of a column and strength of concrete and steel are also studied and can be found in references.

G. Giakoumelis and D. Lam (2004) carried out tests to or exploring the behavior of fifteen on circular CFFST columns. The parameters studied were the steel tube wall thickness, the bond between the steel tube and the concrete and the concrete confinement. All columns had a diameter equal to 114 mm and length equal to 300 mm. The compressive strength of concrete were 30, 60 and 100 N/mm2 and diameter to thickness (D/t) ratio were 22.9 to 30.5. The results show that the bond between the concrete and the steel tube is increased with the increase of CFST columns strength. Also, the small shortening appears for peak load and large displacement for the ultimate load was gained for normal concrete.

Amir Fam, et al (2004) conduct an analytical modeling and experimental testing for CFST columns under central axial loading and combination of axial and lateral cyclic loading. The bond, end loading conditions, and the level of axial compressive stress are the parameters considered in this investigation. A gross of ten samples, including 5 short CFST columns and 5 CFST beam-column specimens, were tested. The results show that the axial strengths of the unbonded short columns were slightly increased while the stiffness of the unbonded specimens was slightly reduced. On the other hand, the bond and end-loading conditions did not affect the flexural strength of beam-column members significantly.

A tested on 50 stub column filled with self-compact concrete subjected to axial load were carried out by L.H. Han et al (2005). Two hollow steel shape (circular and square), different steel tube yielding strength (varied from 282 to 404 Mpa) and different diameter (width) to tube wall thickness (*D(B)/t*) ratio (varied from 30 to 134) are the main parameters varied in the tests. They were developed a simplified model for calculating capacity and the axial load versus axial deformation relationships for the CFST stub columns. This method was in good agreement with the test results.

J. Zeghiche, K. Chau (2005) determine the performance of CFST columns. Twenty-seven circular CFST with 160 mm of an outer diameter and 5 mm wall thickness were tested. The columns length varied from 2.0 m to 4.0 m in increments of 500 mm. The strength of infill concrete was 40, 70, and 100 N/mm2. Columns were tested under different loading conditions (axial- eccentric-bending). The CFST length and the strength of concrete were the main parameters studied. The results show that the length of the columns has a worthy impact by increasing the load carrying capacity of CFST columns with decreasing it. Also with using of high concrete strength result in enhancing the load carrying capacity of the tested CFST.

An experimental investigation on circular CFST columns conducted by Ellobody, Young, and Lam (2006) to develop a numerical modelling to demonstrate the performance of circular CFST stub columns. Different concrete strength (ranging from 30 to 110 MPa of 40) and diameter to thickness ratio (ranges from 15 to 70) were the parametric studies. The numerical modeling predicted the failure mode, column capacity and load–axial formation curves and compared with the experimental test results which show a good agreement.

To inspect the CFST stub columns under various loading conditions that result in different mechanical performances, Yu, Ding, and Cai (2007) tested seventeen specimens filled with self- consolidated and normal concrete. The impact of concrete strength notched holes or slots, slenderness ratio, and different loading conditions were the main parameters. The results indicate that growth in concrete strength increase load capacity of columns. However, the notch has a negative effect on the columns by reducing the axial compressive stiffness of specimens.

A study was conducted axially loaded circular concrete-filled steel tube by de Oliveira, De Nardin, and El Debs to examine the restrained effect on the load capacity and behavior of circular CFST columns. Sixteen CFST columns with different values of concrete compressive strength (30, 60, 80, and 100 MPa) and the length/diameter ratio (3, 5, 7, and 10) was tested under axial loading. The test results show that the increments in slenderness led to the decreased load capacity of the composite columns due to decreases in confinement.

Dundu (2012) was undertaken an experimental study to explore the behavior of twenty-four CFST columns under concentrically compression load. The main parameters in the tests include the length (1, 1.5, 2, and 2.5 m), diameter (114.85, 127.3, 139.2, 152.4, 165.1, and 193.7 mm), the steel tubes strength (30 and 40 MPa). Overall flexural buckling happens with the large slenderness ratio. Although the crushing of the concrete and yielding of the steel tube obtained as failure mode for stockier columns. The higher ultimate load can be achieved with large diameter columns when compared to the smaller diameters of the same lengths.

Experimental investigation has been performed by Abed, AlHamaydeh, and Abdalla (2013) to study the effects of two main parameters, the strength of concrete (44 MPa and 60 Mpa) and the diameter-to-thickness (D/t) (54, 32, and 20) ratio on the CFST columns behaviour and failure modes. Sixteen CFSTs specimens were tested in this study. The results present that there was an increase in axial load capacity and stiffness of CFST with increasing concrete strength and D/t ratio due to the increase in the confinement.

Ekmekyapar and AL-Eliwi (2016) conducted a test on eighteen circulars (CFST) columns to determine the experimental behavior of columns and to check the effectiveness of AISC 360-10 and EC4 design code with different limits. CFST specimens, with 114.3 mm diameter and (2.74, 5.9 mm) thickness, were tested for failure under compressive loading. Three L/D ratios (2.62, 5.25, 7.87), two D/t ratios, three compressive strength of concrete (56 MPa, 66 MPa, and 107 Mpa) and two yield strength of steel (235, 355 MPa) are the main parameters employed in this study. The results achieved from tests and collected data from the literature show that the higher confinement behavior can be achieved with lower concrete strength. The deformation capacity and ductility of CFST columns with lower concrete strength have greater. Also, the thicker tube shows better confinement than thinner with same concrete strength. Contrary to D/t ratio and confinement factor, slenderness ratio has a direct impact on CFST column capacity and behavior. In general, the test results indicated by EC4 much better satisfied than AISC. Also, EC4 can be applied to CFST columns which have properties beyond the applicable limits.

2.3 Effect notch in steel tubes

In recent times, several scientists have addressed the study of the effect of the defect found in the substances that enter into vital structures. The study summarized the effect of defects in the height of the tube used in the columns used in bridges and has been used in the very high structural task. This was defined as a function of the industrialization process. Edmundo and Suhas (1996) reported square tubular net bending response, buckling and destruction of long, thin-walled welded steel. The doing a complex experimental and analytic approach, Liu and Young (2003) in this area, studied the imbalance in the columns and how they affect them. Jensen et al. (2004) then he completed the tests in this field how to walk and affect the structures and buildings and used in his experience column square. Such as steel structures, may permanently contain defects as in the metal tubes that are outside of the CFST members, which may result in structural weaknesses and are ineffective, thereby reducing their lifetime in the buildings. Where classify the faults and weaknesses of the solid organs in general to two defects: the engineering defect by specifying the desired and a small amount of material. The first disadvantage is the geometric dimensions corresponding to the reality, which differ from the ideal shape required, which is related to the different materials manufactured, that is a defect of origin, or it is wasted and lost during the conduction processor through the factors that are targeted permanently and affect the steel members I. Tutuncu,(2001) the defect in the materials relates primarily to industrial defects or changes in dimensions caused by damage due to external factors such as moisture that may cause corrosion and may be due to the welding process to connect the metal parts or the impact of materials

Experiments and tests were also completed to ascertain the extent of the engineering errors and defects in the performance of the square tubes. Stull et al. (2011) also studied the extent to which engineering defect or engineering defect in the structure of the crust. Alavi Nia et al. (2012) also investigated the impact of falling collapse on the properties of the tube response to the energy absorbed by the square-shaped tubes if under the influence of the semi-static force in a tilted manner.

The geometric imperfections of square steel tube columns have been intensively studied over the past few decades, its shape and amplitude, the effect on buckling strength, and design tolerance and design strength. Donnell and Wan defined geometric imperfections with irregularity factor (L.H. Donnell,1950).

Enhancement of CFST beam in structural properties Due to the combined action between the components element. Steel tube works both vertically and horizontally reinforcement and provides a confining pressure to the concrete, Concrete is placed in a triaxle stress state. On the other hand, the concrete core reinforces the steel tube. The maximum strength of CFST column it is influenced by their constituent material properties, such as the bending strength of concrete, the yield strength of steel, and the steel ratio.

Sadovsky and Krivacek (2012) describe the computational modeling of engineering defects and the hardness of cold-formed steels. Zhang studied the energy absorption of the square powder tube axially with the twisting starter (2009). For thin-walled steel tube panels with physical defects, their torsion and strength patterns have also been verified in recent years. Alavi Nia et al. (2011). The effect of cracks on the properties of energy absorption of square tubes and tube with a cylindrical shape where they made these cracks in different positions as well as the nook of those cracks were different. Han et al (2012) studied parts of rectangular steel tubes by lifting a load to measure the erosion and impact of loading in that part. Estekanchi and Vafai also studied by experimenting with the sureness of the twisting process that gets in the plastic cylinders containing the reinforcement (1999 & 2006).

Julian and Lehman (2005 & 2009) also conducted an investigation into the effect of sizes, shapes, cracks, and location on the compressive load strength of these cylinders. Contributions from missing locations, related to border conditions, properties of materials, and crust with thickness, were also important for consideration as an attempt to investigate the effect of local structural defects on the steel tube on the mechanical performance of CFST packages, Tao et al. (2006). A function related to local imbalance was used as a numerical example. Between the test result and the FE result, the behavior of the torsion, in the conversion response of the load and the final force is equally large. Zhang et al. (2013). Studied the design defects of steel tubing related to the mechanical behavior of CFST packages. Also conducted tests and parametric studies related to the length of the incision, the incisions of the incision, and the location of the incision, the bearing of the concrete and the extent of the effect of sclerosis.

Accordingly, CFST models with cracks and defects in which failure patterns are quite different from sound CFST models are found. Accordingly, it was found that the defective CFST model showed less mechanical performance than the intact CFST sample, as the defect in the steel tube contributed to the failure to give the real effect of concrete pressure. Also proposed an experimental formula for predicting the beam density of the CFST package. Their experience also showed that the percentage of local structural faults and defects of steel tubes was relatively small, as their concrete essence contributed most of the final strength of the thin composite samples.

This means that the mechanical behavior of the steel tube package that is tapered is more sensitive to the defects and engineering errors of CFST. In recent years, CFST members been used in deep underground spending and road support systems. In such a case, the outer steel tube must be cut into the concrete Zhang et al. (2013). In this case, corrosion or corrosion can occur as well as material defects in steel tubes that are outside the CFST member when applied in an unsuitable or anodic place. As mentioned above, the effect of the physical defect on a circular steel tube package was studied experimentally and the results were taken to compare and evaluate with complete CFST members.

CHAPTER 3

EXPERIMENTAL INVESTIGATION

3.1 Material Properties

3.1.1 Self-compacted concrete

In this study, the mix ratio of the SCC consisted of Water (W), Super Plasticizer (SP), Cement (C), Fly Ash (FA), Natural Fine Aggregate (NFA) with 4mm maximum particle sizes, Crushed Sand (CS) with range of particle sizes are 2-0.3mm, and <0.3mm as finer, and Natural Course Aggregate (NCA) with 10mm maximum particle sizes are used. The mix design is done for M30 grade, the water-cement ratio is used as 0.33, and the mix proportion of self-compacted concrete is given in Table 3.1 below.

Table 3.1 mix ratio of SCC

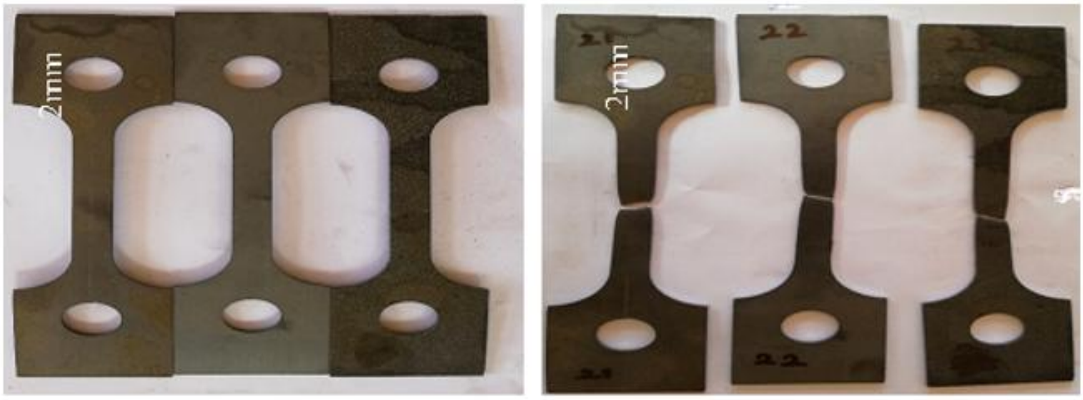
Mix ratio	W/B	C	FA	W	SP	CA	NS	CS2-0.3mm	CS<0.3mm
Kg/m³	2338.27	14.25	14.25	9.41	.14	35.49	32.53	7.23	3.61

The mean compressive strength of the mix proportion is determined in the laboratory at testing day after casting of three cylinders 100mm by 200mm as shown in Figure 3.1. The average concrete compressive strength is 50.31 Map.

Figure 3.1 (a) Concrete cylinders before testing, (b) Concrete cylinders after test.

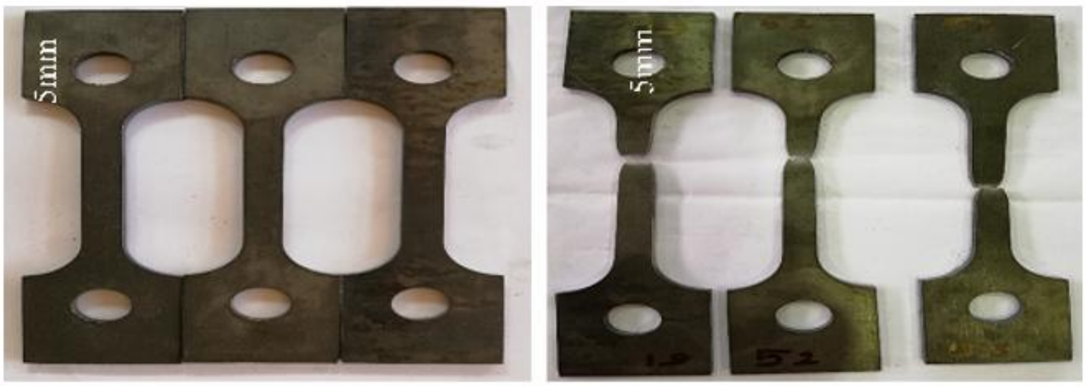
3.1.2 Steel tubes

Mild steel tube, cold-formed is square with two different in thicknesses are used, the thicknesses of the tube (5mm,2mm) with 6-meter length. The yield and ultimate strength are listed in Table 3. 2 and Figure 3.2. (a - d).



(a) (b)

Figure 3.2 (a) Steel tube before testing (2mm), (b) steel tube after testing (2mm).



(c) (d)

Figure 3.3 (c) Steel tube before testing (5mm), (d) steel tube after testing (5mm).

Table 3.2 Properties of steel tubes.

NO	Tube cross section (mm)	Yield strength (Fy) (Map)	Ultimate strength (Fu) (Map)
1	2	273.67	351.7
2	5	314.67	389

3.2 Test specimens

In the tests reported in this paper, machine notches simplify material defects. According to the direction and position of the notch, the twelve notches and the two sections are classified as follows: The horizontal notch in the two-side wall, two top and two bottoms, the vertical notch located in the two sidewalls, two top and two bottoms, respectively. But with the dimensions of the notch, there is a length of the notch 5 cm and there is 2.5 cm, Either the thickness of the notch shall be according to the thickness of the specimens (5mm, 2mm). In order to reduce a large number of variables, the center of the notch coincides with the center of the top, bottom and sidewalls. Therefore, the notch can be described strictly in accordance with the notch position, the notch width (b), and the notch length (l). And the depth of the cut (t), as shown in Figure 3.4 (a - e). A total of 26 specimens were constructed and tested under bending load: 24 artificial notches of various features of specimens of length and size, such as position, orientation and two control specimens without any notches. Cutting the end of the steel tube section and processing it to the required length all specimens 820mm and two pieces of plates shall be added to the ends of the beam with the weld until a concert is made into the confining during the test, used two types of thickness in the testing. The dimension of each specimen being 100 (B) mm×5 (t) mm×820 (H) mm and 100 (B) mm×2 (t) mm×820 (H) mm, where B The width of the square steel tube section, (t) is the wall thickness of the steel tube, and (H) is the height of the sample. All details regarding the sample are listed in Table 3.3. The steel tube is cleaned of dust and oil from the inside by a dry cloth. The silicon is also placed inside the notch and then the adhesive tape to prevent the exit of the concrete out of the tubes of the notch during the casting and after the dry concrete is removing the silicon. Then when it is put one ends were sealed by epoxy to block moisture loss during curing. For a better observation of deformation of the test specimens, line in the middle of the specimen along its length as well as a vertical line on its center of the specimen.

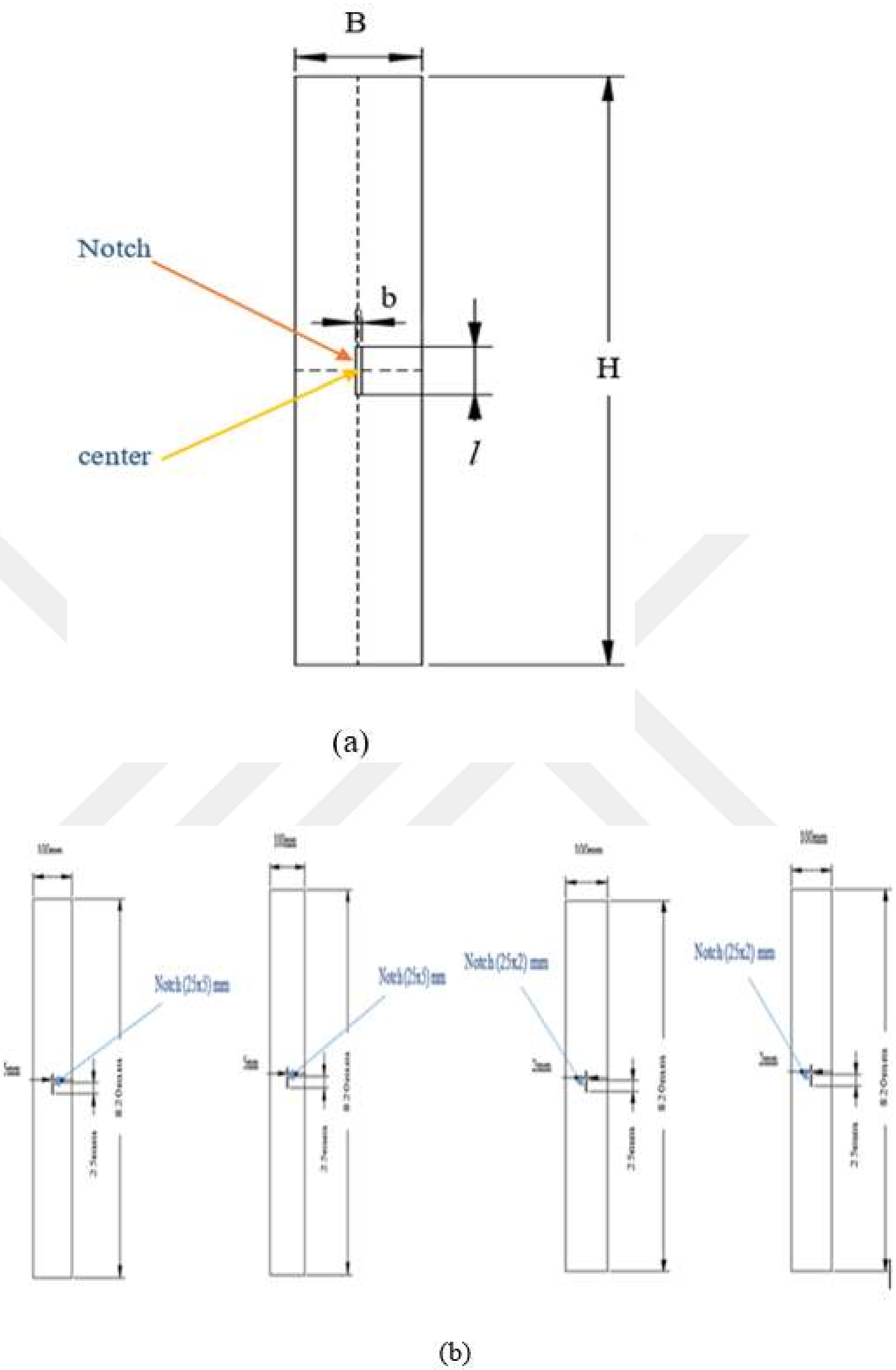


Figure 3.4 Sketch for a notch and (a) explain the length of the notch (b).

Table 3.3 Specimen details, test, and calculated results (5mm).

Group name	Specimen	Orientation	Location	$l \times b$ (mm)	$D \times t \times H$* (mm)	D/t**
	Control-5mm	Control			100×5×820	20
Group one	S100-5-BLH	Horizontal	Bottom	5o×5	=	=
	S100-5-BSH	Horizontal	Bottom	25×5	=	=
	S100-5-BLV	Vertical	Bottom	5o×5	=	=
	S100-5-BSV	Vertical	Bottom	25×5	=	=
Group two	S100-5-TLH	Horizontal	Top	5o×5	=	=
	S100-5-TSH	Horizontal	Top	25×5	=	=
	S100-5-TLV	Vertical	Top	5o×5	=	=
	S100-5-TSV	Vertical	Top	25×5	=	=
Group three	S100-5-SLH	Horizontal	Sidewall	5o×5	=	=
	S100-5-SSH	Horizontal	Sidewall	25×5	=	=
	S100-5-SLV	Vertical	Sidewall	5o×5	=	=
	S100-5-SSV	Vertical	Sidewall	25×5	=	=

* =20 for all specimen in the table above, **=100×5×820 for all specimen in the table above.

Table 3.4 Specimen details, test, and calculated results (2mm).

	Specimen	Orientation	Location	$l \times b$ (mm)	$D \times t \times H$* (mm)	D/t**
	Control-2mm	Control			100×2×820	50
Group four	S100-2-BLH	Horizontal	Bottom	5o×2	——	—
	S100-2-BSH	Horizontal	Bottom	25×2	——	—
	S100-2-BLV	Vertical	Bottom	5o×2	——	—
	S100-2-BSV	Vertical	Bottom	25×2	——	—
Group five	S100-2-TLH	Horizontal	Top	5o×2	——	—
	S100-2-TSH	Horizontal	Top	25×2	——	—
	S100-2-TLV	Vertical	Top	5o×2	——	—
	S100-2-TSV	Vertical	Top	25×2	——	—
Group six	S100-2-SLH	Horizontal	Sidewall	5o×2	——	—
	S100-2-SSH	Horizontal	Sidewall	25×2	——	—
	S100-2-SLV	Vertical	Sidewall	5o×2	——	—
	S100-2-SSV	Vertical	Sidewall	25×2	——	—

* =50 for all specimen in the table above, **=100×2×820 for all specimen in the table above.

The following system of remarking each tube was designated with the names such as, (S100-5-BLH, its mean S Square, B bottom, L long, H Horizontal), (S100-5-BSH, B bottom, S short, H Horizontal), (S100-5-BLV, bottom long V vertical), (S100-5-TLH, T Top long horizontal), (S100-5-SLH, sidewall long horizontal) etc. for all Specimen notch where used two thicknesses (5mm,2mm) As in the following Figure 3.5.

Figure 3.5 Thirteen specimens from each cross section with thickness differences.

Steel tubes place it upright in a bracket specially prepared for pouring concrete. The bottom end of the steel tube is tightly covered with steel plate 3mm thick by welding. Then the concrete is casting into the beams tube infilled SCC without strengthening as shown in Figure 3.6 a, and b.

Figure 3.6 Hollow steel tubes(a), beams infilled SCC (b).

Then SCC is casting in the steel tubes from the top without any compaction. After filling, the concrete is trimmed off using a trowel as shown in Figure 3.7. After the curing period, if any shrinkage occurs, the gap filled with epoxy and then covered with another steel plates by welding, finally the specimens prepared for testing.

Figure 3.7 The gap filled with epoxy.

3.3 Test procedure

The testing of beams is done by using of flexure machine test with 500 KN capacity. All beams are tested under simply supported end conditions and under two-point loading with displacement control loading method with a rate of 1mm/min. Supports are fixed at a distance of 820 mm from both the ends of beams so that the clear span is 750mm. Two-point loading is adopted for testing and provide a pure bending zone of 250mm as shown in Figure 3.8 and 3.9.

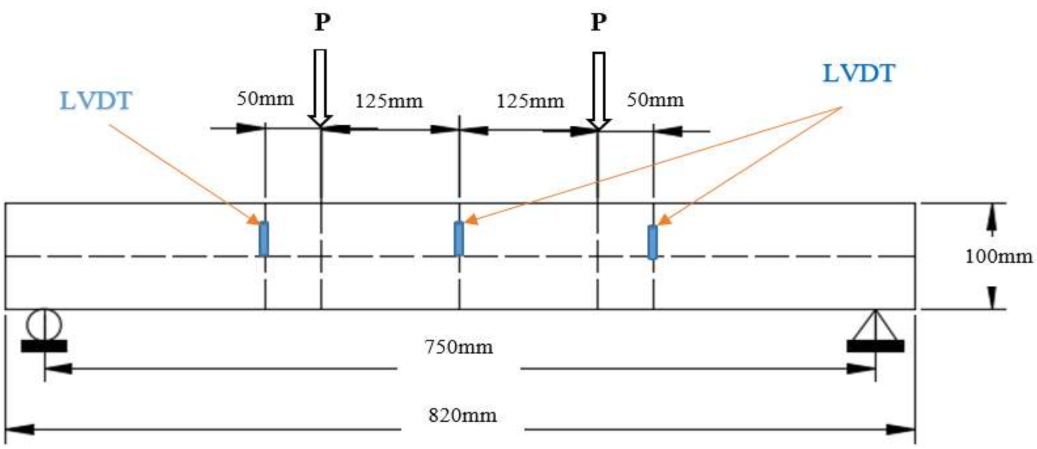


Figure 3.8 Drawing the user's device in the test

The axis of the specimens is carefully aligned with the axis of the loading device. Three Linear Variable Differential Transistor (LVDTs) are fixed along the length of the beams to measure deflection. Among the three LVDTs, two of them are placed at a distance of 175mm at each side from the mid-span and the third one is fixed exactly at the mid-span. All beams are loaded up to failure.

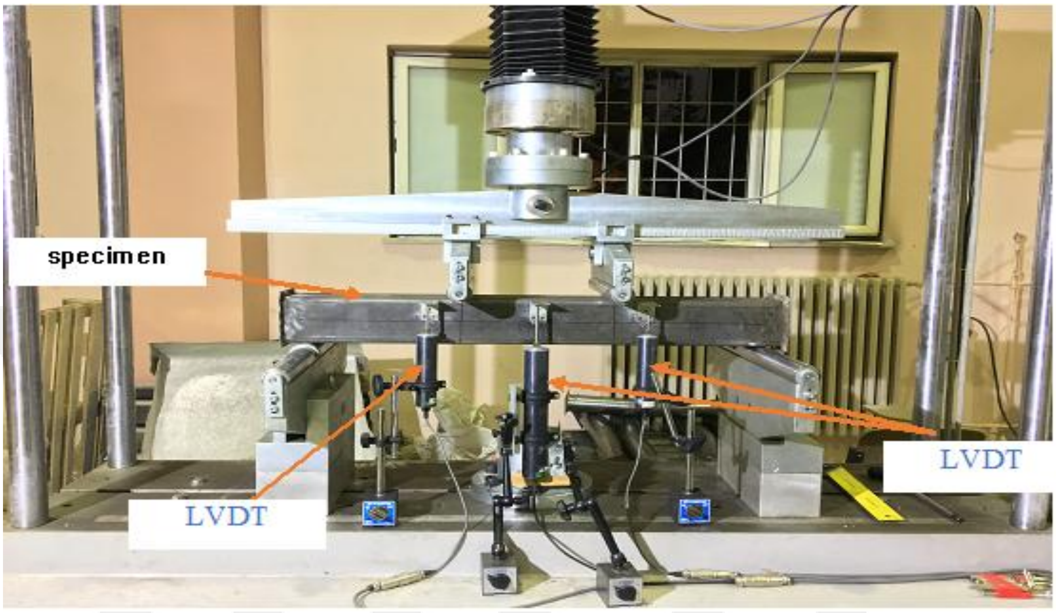


Figure 3.9 The device used in the test.

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 Failure Modes

All beams have large deflection capacity and CFST with notch beams failed by the notch (imperfections of steel) and rupture of tensile large at the end of test. According to the test results, the sample can be divided into many typical failure modes. For the first group, the sample with the top notch (5 mm) has a vertical notch that is always closed under axial compression. The specimen S100-5-TLV happened inward local buckling at the end of testing are occurred with rupture. Also, the specimen S100-5-TSV without inward local buckling at the end of testing occur with rupture. In addition, for samples with horizontal notches, the specimen S100-5-TLH, S100-5-TSH happened bending outward in the middle of the notch at the end of testing are occurred with rupture excepting of the specimen S100-5-TSH without rupture, as indicated in Figure. 4.1(a)-(c). For group two, the specimen with a bottom notch (5mm) along the welding line, for all the specimens with a vertical and horizontal notch without local buckling. Finally, rupture is occurred at the bottom for all specimens CFST beams with the notch. as indicated in Figure. 4.2 (a) and (b).

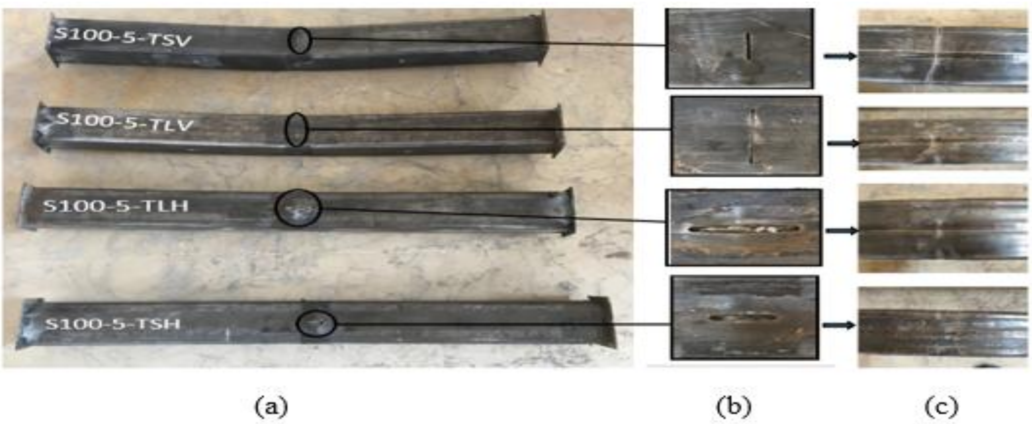


Figure 4.1 Failure modes (a), deformation of the notch (b) and rupture the specimens or GR1(c)..

(a) (b)

Figure 4.2 Failure modes (a) deformation of the notch and (b) rupture the specimens for GR2.

For group three, the specimen with a side notch (5mm), for all the specimens with a vertical and horizontal notch without local buckling. For the specimens S100-5-SLV and S100-5-SSV at the end of testing are occurred with rupture excepting of the specimens S100-5-SSH and S100-5-SLH without rupture, as indicated in Figure. 4.3 (a), (b) and (c).

(a) (b) (c)

Figure 4.3 Failure modes (a), deformation of the notch (b) and rupture the specimens for GR3(c).

For group four, the specimens with a top-notch (2mm), at the vertical notch, the notch is always closed under axial compression and happened inward local buckling at the end of testing but with a horizontal notch bending outward in the middle of the notch. In addition, all the specimens rupture is occurred at the bottom for all specimens CFST beams with a notch from the top. As indicated in Figure. 4.4 (a), (b) and (c).

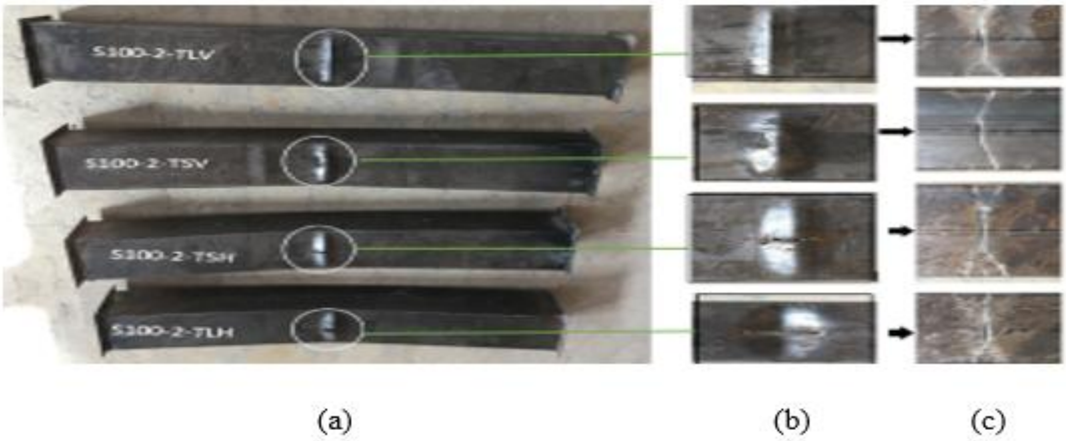


Figure 4.4 Failure modes (a), deformation of the notch (b) and rupture the specimens for GR4(c).

For group five, the specimens with a bottom notch (2mm), the all of the specimens CFST beams with a notch from bottom occur outward local buckling occurred in the middle of the notch and side buckling especially in the area of the corner. In addition, at the end of testing are occurred with rupture all the specimens from tow side of the notch. as indicated in Figure. 4.5 (a), (b) and (c).

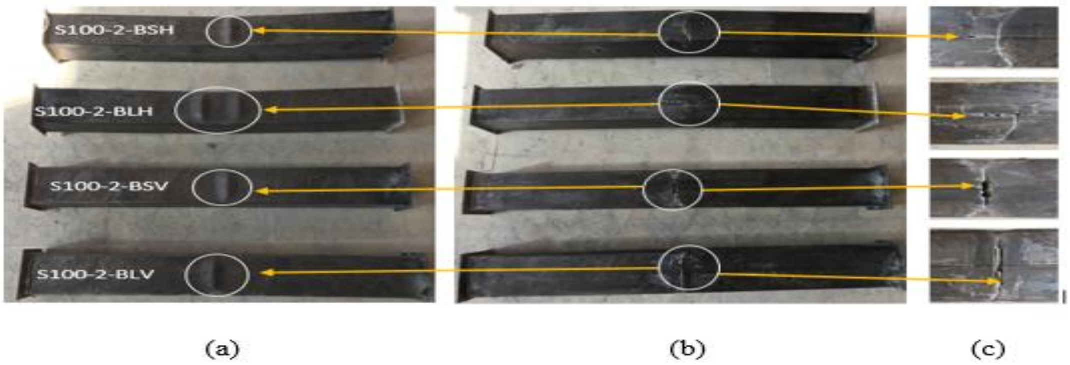


Figure 4.5 Failure modes (a), deformation of the notch (b) and rupture the specimens for GR5(c).

For group six, the specimens with a side notch (2mm), the all of the specimens CFST beams with a notch from side occur outward local buckling in top the specimens and side buckling especially in the area of the corner. It also observes the deformity of the horizontal notch, but there is no rupture within the notch. However, the vertical notch occurs rupture from the bottom of the notch. At the end of testing are occurred with rupture all the specimens from the bottom. As indicated in Figure. 4.6 (a), (b) and (c).

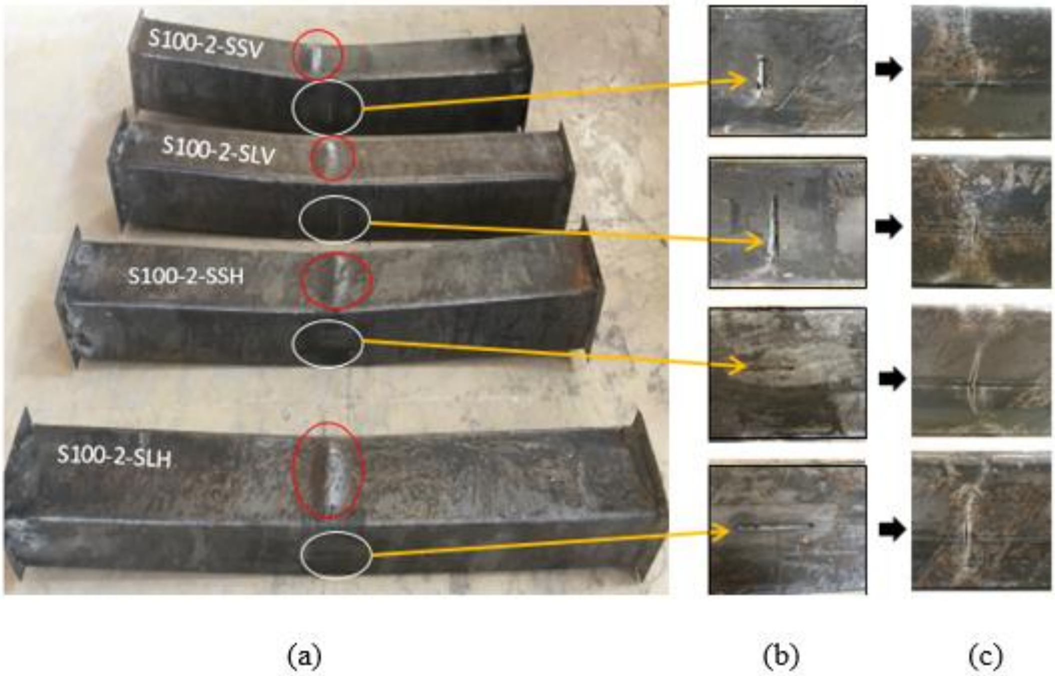


Figure 4.6 Failure modes (a), deformation of the notch (b) and rupture the specimens for GR6(c).

4.2 Moment - Mid-Span Deflection Relations

The moment – mid-span deflection is shown in Figure 4.7- 4.12 for a beam with notch group one, two, three, four, five and six, respectively. All the beams showed a smooth moment deflection curve with large deflection up to failure the control CFST beams without notch have been compared with all specimens steel tubes with a notch for all groups, it found that the concrete inside the tubes have significantly increased the moment carrying capacity, stiffness and reduce the deflection. Used two thickness (2mm,5mm) the control CFST beams without notch (2mm,5mm) showed elastic behavior at the first stage of loading both of them. Then, the curves follow an elastic path with decreasing stiffness until reaching the maximum bending capacity. However, the control specimens (5mm) show greater bending moment capacity and stiffness than the control specimens (2mm). For the specimens with a notch, in this case, the bending moment-curvature relationship at the mid-span of the specimen can be represented into 3 stages: elastic stage, yield stage, and hardening stage.

elastic stage, in this case, the stiffness of the specimen remains constant and the moment is initially applied on the beam, during this stage, relation shows generally linear response. Even if the bottom concrete damage inside tube. In addition, occur the deformation of compressive concrete and steel tube keep elastic. Yield stage, the moment will be stay kept as constant whilst effect of the notch and takes place, leading to a stress redistribution between steel and concrete also observed in this stage the mid-span deflection is increased. Hardening stage, the load is increased to the ultimate capacity during this stage the maximum stress of the steel tube in compressive zone exceeded. The CFST beam with notch shows elasto-plastic behavior response. The ultimate flexural strength and stiffness are both decreased due to the notch. After the peak, point of the curve increase effect of the notch on the bending moment maintains whilst deflection keeps increasing. The specimens' beams (5mm) with notch failed by rupture at the end of the test accept (S100-5-TSH, S100-5-SSH, and S100-5-SLH) with almost the difference deflection capacity for group one, two and three. While the specimens' beams (2mm) with notch all specimens failed by rupture had more deflection capacity at the end of the test.

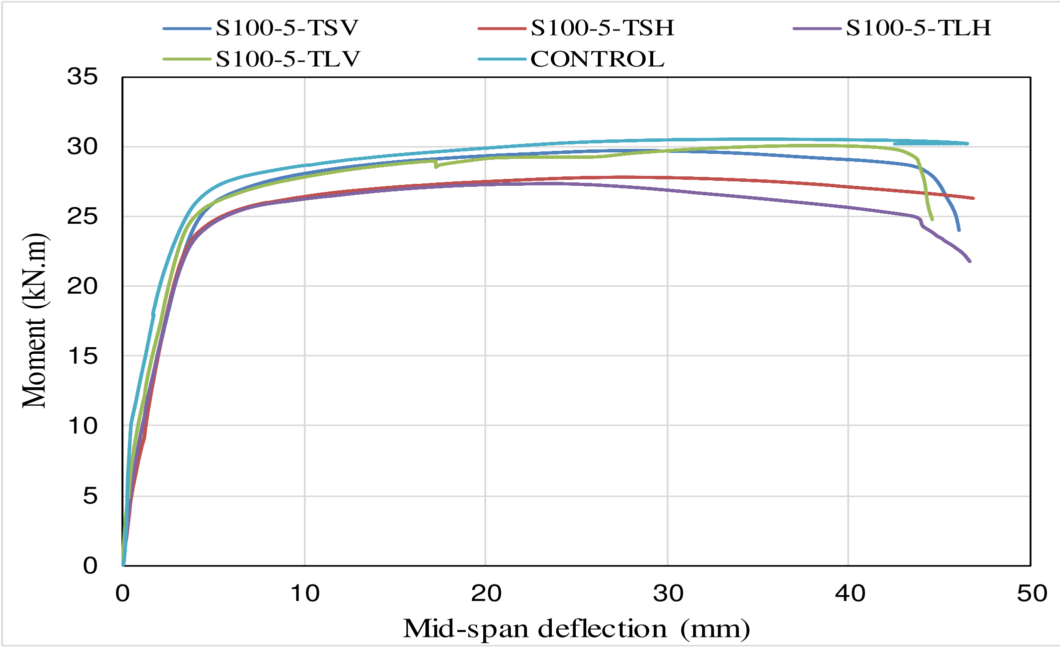


Figure 4.7 Moment capacity and mid span deflection for group one.

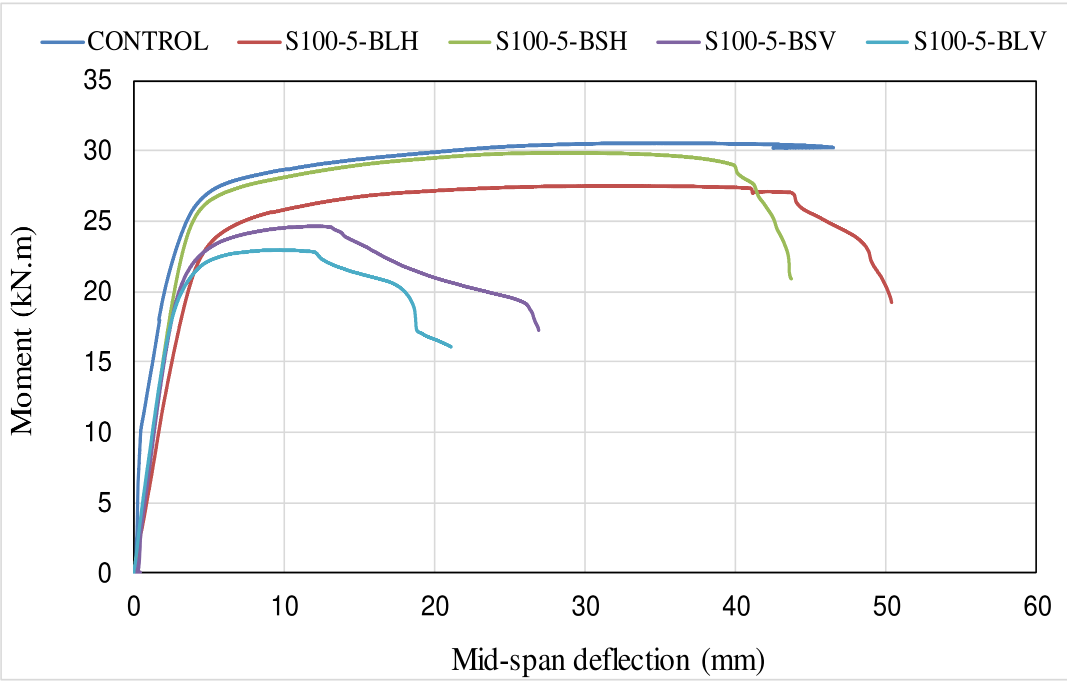


Figure 4.8 Moment capacity and mid span deflection for group two.

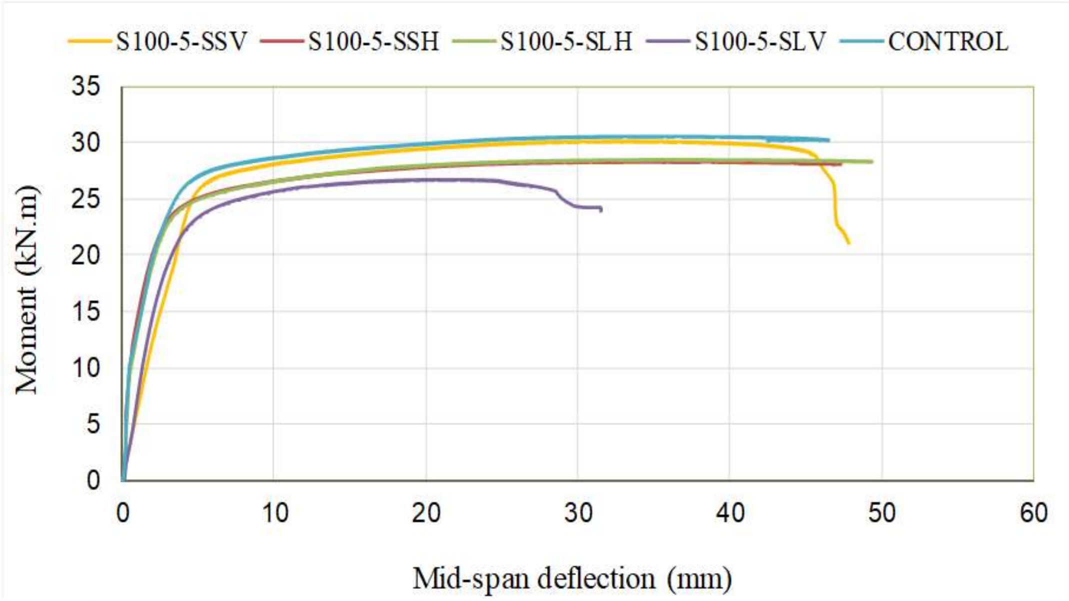


Figure 4.9 Moment capacity and mid span deflection for group three.

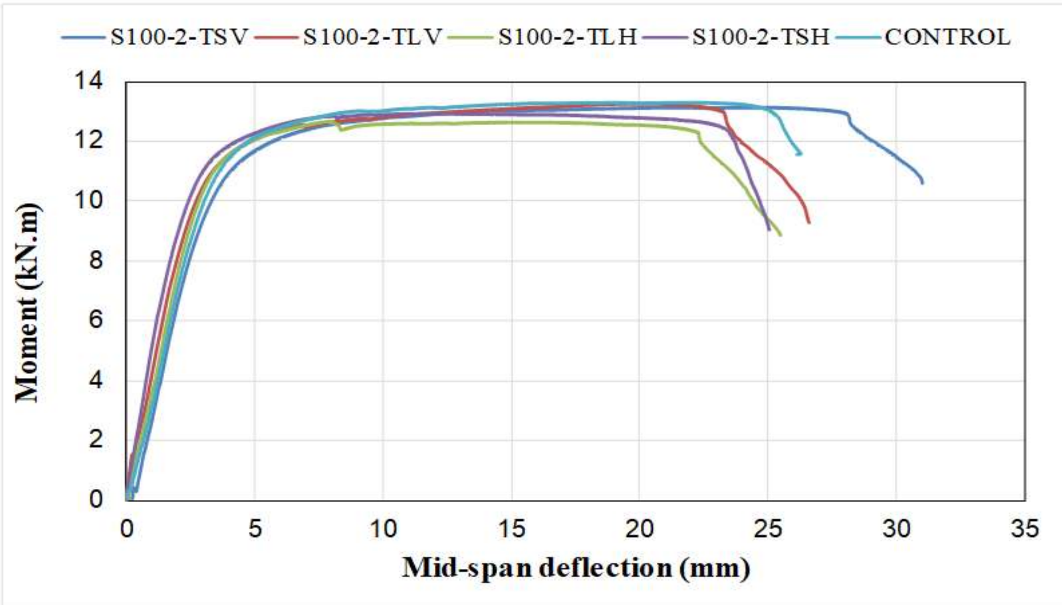


Figure 4.10 Moment capacity and mid span deflection for group four.

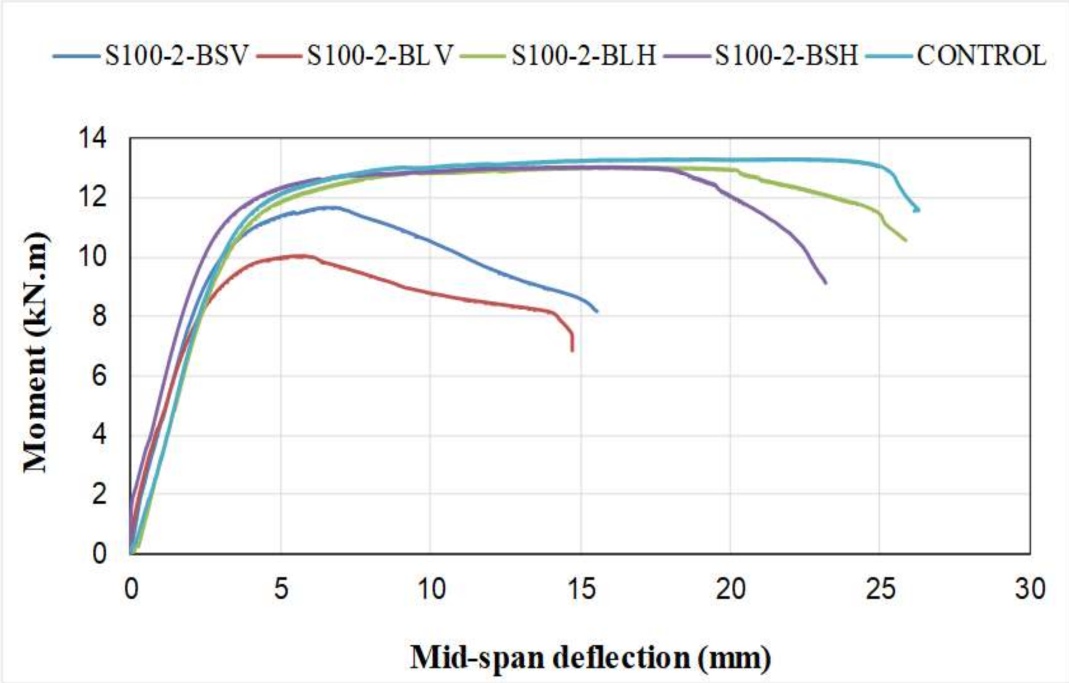


Figure 4.11 Moment capacity and mid span deflection for group five.

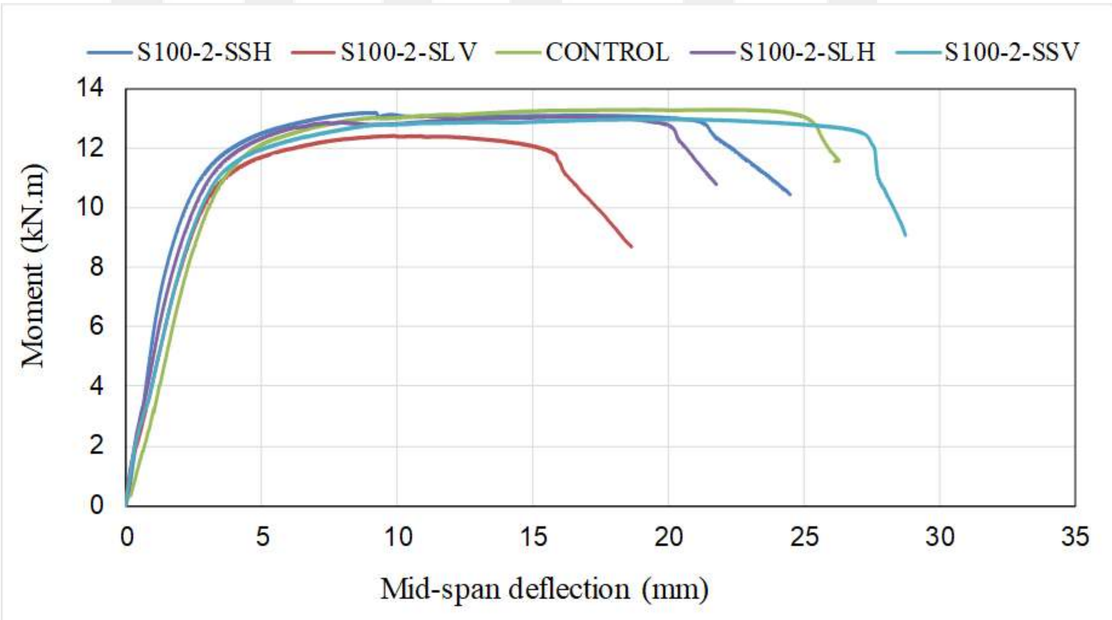


Figure 4.12 Moment capacity and mid span deflection for group six.

4.3 Bending Moment capacities

The maximum moments and percentage of decrease in moment capacity for all groups specimens with notch were compared them with the control beams without notch were summarized and tabulated.

Table 4.1 Moment and percentage of decrease in moment capacity group one (5mm).

Group name	Designation of beams	Moment (KN.M)	% decrease in Moment. (kN.m)
	Control-5mm	30.56	-----
Group one	S100-5-TLH	27.37	10.42
	S100-5-TSH	27.83	8.92
	S100-5-TLV	30.10	1.50
	S100-5-TSV	29.74	2.67

For group one, the percentage decrease in the ultimate moment due to notch CFST beams are ranged from 10.42% for specimen S100-5-TLH to 1.50% for specimen S100-5-TLV. Showed the effects of notch length on the load- displacement response, where these two specimens had the same notch orientation, therefore showed the influence long length in the top of the beam the orientation horizontal 10.42% for the specimen S100-5-TLH and 8.92% for the specimen S100-5-TSH. The moment capacity decreased by increasing the length of the notch. While the orientation vertical showed obverse, where1.50% for the specimen S100-5-TLV and 2.67% for the specimen S100-5-TSV. From the result observed the orientation vertical had the maximum moment more than the orientation horizontal show as in Figure 4.7. That is because when the applied load on the beam with notch orientation vertical occur closed the notch. It is the confinement of steel on concrete that contributes to the increase of moment carrying capacity of concrete. Compared to the horizontal when applied load the notch it will be opening outward will significantly weaken the restraining effect of steel tube on core concrete and occur damage for concrete Thus being unable to resist. The show is in Figure. 4.1.and Table 4.1.

Table 4.2 Moment and percentage of decrease in moment capacity group two (5mm).

Group name	Designation of beams	Moment (KN.M)	% decrease in Moment. (kN.m)
Group two	S100-5-BLH	27.52	9.94
	S100-5-BSH	29.87	2.23
	S100-5-BLV	22.96	24.86
	S100-5-BSV	24.64	19.37

For group two, the percentage decrease in the ultimate moment due to notch CFST beams are ranged from 24.86% for specimen S100-5-BLV to 2.23% for specimen S100-5-BSH. showed the effects of notch length on the load-strain response, where these two specimens had the same notch orientation, therefore showed the influence long length notch in the bottom of the beam the orientation horizontal 9.94% for the specimen S100-5-BLH and 2.23% for the specimen S100-5-BSH. From this result observed very clear the effective length of the notch. While the orientation vertical same effect for horizontal by the length of notch the peak load (moment capacity) and the corresponding displacement also decreased by increasing the notch length show as in Figure 4.8. Where the comparison between the orientation horizontal and vertical found it. The notch in a vertical orientation more than effect horizontal. Due to occur rupture outward from center and end of the notch in addition to small-scale section (vertical on the short direction) significantly weaken the confining effect of the steel tube on the core concrete of beam from the bottom when compared with a horizontal notch (horizontal on the long direction) of the beam shown as in Figure 4.2.and Table 4.2.

Table 4.3 Moment and percentage of decrease in moment capacity group three (5mm).

Group name	Designation of beams	Moment (KN.M)	% decrease in Moment. (kN.m)
Group three	S100-5-SLH	28.48	6.78
	S100-5-SSH	28.31	7.34
	S100-5-SLV	26.72	12.55
	S100-5-SSV	30.12	1.44

For group three, the percentage decrease in the ultimate moment due to notch CFST beams are ranged from 12.55% for specimen S100-5-SLV to 1.44% for specimen S100-5-SSV. Same procedure for group one and two take two specimens had orientation horizontal notch on the sidewall and check to affect the length of the notch. The orientation horizontal 6.78% for the specimen S100-5-SLH and 7.34% for the specimen S100-5-SSH. From the result observed the orientation horizontal long of notch had the maximum moment more than the horizontal short notch shown in Figure 4.9, this was because that the concrete of confining effect was better than in the horizontal long notch of the sidewall for the CFST beam, from horizontal short notch. While the orientation vertical 12.55% for the specimen S100-5-SLV and 1.44% for the specimen S100-5-SSV. Where showed the effect notch on the long vertical more than the short vertical due to the moment capacity decrease by increase length of the notch. While the comparison between the orientation horizontal and vertical found it. For the specimen sidewall long notch horizontal (SLH) observed the moment capacity more than the specimen sidewall long vertical (SLV) due to the small-scale section for the beam (SLV) significantly weaken the confining effect of the steel tube on the core concrete of beam in addition to the notch orientation parallel applied load. While the length short notch its inverse long notch same conclusion for group one and two. Shown as in Figure. 4.3.

Table 4.4 Moment and percentage of decrease in moment capacity group four (2mm).

`Group name	Designation of beams	Moment (KN.M)	% decrease in Moment. (kN.m)
	Control-2mm	13.30	---
Group four	S100-2-TLH	12.67	4.75
	S100-2-TSH	12.93	2.78
	S100-2-TLV	13.26	0.27
	S100-2-TSV	13.13	1.22

For group four, the percentage decrease in the ultimate moment due to notch CFST beams are ranged from 4.75% for specimen S100-2-TLH to 0.27% for specimen S100-2-TLV showed the effects of notch length on the load- displacement response, where these two specimens had the same notch orientation, therefore showed the influence long length in the top of the beam the orientation horizontal and vertical. The same as group one it seemed that the discussion and conclusion. Shown as in Figure 4.4 and 4.10.

Table 4.5 Moment and percentage of decrease in moment capacity group five (2mm).

Group name	Designation of beams	Moment (KN.M)	% decrease in Moment. (kN.m)
Group five	S100-2-BLH	13.02	2.05
	S100-2-BSH	13.03	2.00
	S100-2-BLV	10.03	24.56
	S100-2-BSV	11.66	12.28

For group five, the percentage decrease in the ultimate moment due to notch CFST beams are ranged from 24.56% for specimen S100-2-BLV to 2.00% for specimen S100-2-BSH. Showed the effects of notch length on the load- displacement response, where these two specimens had the same notch orientation, therefore showed the influence long length in the bottom of the beam the orientation horizontal and vertical. The same as group two its seemed that the discussion and conclusion. Shown as in Figure 4.5 and 4.11.

Table 4.6 Moment and percentage of decrease in moment capacity group six (2mm).

Group name	Designation of beams	Moment (KN.M)	% decrease in Moment. (kN.m)
Group six	S100-2-SLH	13.06	1.81
	S100-2-SSH	13.19	0.79
	S100-2-SLV	12.42	6.59
	S100-2-SSV	12.97	2.44

For group six, the percentage decrease in ultimate moment due to notch CFST beams are ranged from 6.59% for specimen S100-2-SLV to 0.79% for specimen S100-2-SSH showed the effects of notch length on load- displacement response, where these two specimens had the same notch orientation, therefore showed the influence long length in the sidewall of the beam the orientation horizontal and vertical. The same as group three it has seemed that the discussion and conclusion. Shown as in Figure 4.6 and 4.12.

4.4 Ductility

Is the ability of a material to undergo permanent deformation through elongation, to study the influence of various parameters on the ductility of notched concrete-filled steel tubular CSFT beams deformation and bending without rupture, compared between control CFST beams without notch with all specimens with the notch.

Table 4.7 Deflection control and ductility index group one (5mm).

	Specimen	Mid span deflection Du. at Mu. (mm)	Mid span deflection Du. at (0.6Mu) (mm)	Ductility index Du / Du(0.6Mu)
	S100-5- Control	35.22	1.74	20.28
Group one	S100-5-TLH	23.56	2.17	10.87
	S100-5-TSH	27.23	2.23	12.22
	S100-5-TLV	38.49	2.21	17.43
	S100-5-TSV	27.70	2.46	11.28

The ductility index (DI=Du/Dy) which represents the ratio of mid-span deflection (Du) at the ultimate moment (Mu) to the mid-span deflection (Dy) at M = 0.6Mu, was used to predict and compare the ductility of the notch CFST beams compared to the control specimens. According to the obtained results for group one, the ductility Index (DI) ranged from 10.87 for specimen S100-5-TLH to 17.43 for specimen S100-5-TLV. It is clear that the maximum DI obtained for the notch CFST beams is 17.43 for specimen S100-5-TLV. However, the corresponding control beam had DI equal to 20.28, So that ductility of the control beam is better than that of the notch CFST beams in this group. It could be seen that the notched specimens could lead to a weaker ductility.

Table 4.8 Deflection control and ductility index group two (5mm).

	Specimen	Mid span deflection Du. at Mu. (mm)	Mid span deflection Du. at (0.6Mu) (mm)	Ductility index Du / Du(0.6Mu)
Group two	S100-5-BLH	32.43	2.84	11.41
	S100-5-BSH	28.31	2.37	11.97
	S100-5-BLV	9.56	1.78	5.37
	S100-5-BSV	11.91	2.00	5.95

For group two, the DI ranged from 5.37 for specimen S100-5-BLV to 11.97 for specimen S100-5-BSH with an average value of DI equal to 8.67.all this group less than the control spacemen, the same as groups one it seemed that the discussion and conclusion.

Table 4.9 Deflection control and ductility index group three (5mm).

	Specimen	Mid span deflection Du. at Mu. (mm)	Mid span deflection Du. at (0.6Mu) (mm)	Ductility index Du / Du(0.6Mu)
Group three	S100-5-SLH	35.97	1.62	22.23
	S100-5-SSH	32.83	1.39	23.58
	S100-5-SLV	20.49	2.26	9.05
	S100-5-SSV	32.54	3.18	10.25

For group three, the DI ranged from 9.05 for specimen S100-5-SLV to 23.58 for specimen S100-5-SSH. in this group that the maximum DI obtained for the notch CFST beams is 23.58 for specimen S100-5-SSH and 22.23 for specimen S100-5-SLH. However, the corresponding control beam had DI equal to 20.28, from this result showed clear the two specimens had large ductility due to the concrete of confining inside the tube. in addition, in the same group also two specimens 9.05 for specimen S100-5-SLV and 10.25 for specimen S100-5-SSV it's less than from control beam because it's a clear influence of the notch on the specimen.

Table 4.10 Deflection control and ductility index group four (2mm).

	Specimen	Mid span deflection Du. at Mu. (mm)	Mid span deflection Du. at (0.6Mu) (mm)	Ductility index Du / Du(0.6Mu)
	S100-2- Control	19.51	2.27	8.61
Group four	S100-2-TLH	7.89	2.01	3.92
	S100-2-TSH	11.89	1.64	7.24
	S100-2-TLV	20.51	1.95	10.52
	S100-2-TSV	23.77	3.06	7.77

For group four, the DI ranged from 3.92 for specimen S100-2-TLH to 10.52 for specimen S100-2-TLV. It is clear that the maximum DI obtained for the notch CFST beams is 10.52 for specimen S100-2-TLV. However, the corresponding control beam had DI equal to 8.61, So that ductility of the control beam is less than that of the specimen S100-2-TLV in this group. However, the control beam is better than that of the notch CFST beams in this group when compared with S100-2-TLH, S100-2-TSH and S100-2-TSV. It could be seen that the notched specimens could lead to a weaker ductility.

Table 4.11 Deflection control and ductility index group five (2mm).

	Specimen	Mid span deflection Du. at Mu. (mm)	Mid span deflection Du. at (0.6Mu) (mm)	Ductility index Du / Du(0.6Mu)
Group five	S100-2-BLH	15.93	2.29	6.95
	S100-2-BSH	15.52	1.66	9.36
	S100-2-BLV	5.62	1.49	3.78
	S100-2-BSV	6.74	1.73	3.89

For group five, the DI ranged from 3.78 for specimen S100-2-BLV to 9.36 for specimen S100-2-BSH. One specimen in this group large than the control specimen, but three less than the control specimen the same as groups four its seemed that the discussion and conclusion shown in table 4.11.

Table 4.12 Deflection control and ductility index group six (2mm).

	Specimen	Mid span deflection Du. at Mu. (mm)	Mid span deflection Du. at (0.6Mu) (mm)	Ductility index Du / Du(0.6Mu)
Group six	S100-2-SLH	16.85	1.70	9.89
	S100-2-SSH	9.40	1.54	6.12
	S100-2-SLV	9.90	1.85	5.36
	S100-2-SSV	19.07	1.93	9.87

For group six, the DI ranged from 5.36 for specimen S100-2-SLV to 9.89 for specimen S100-2-SLH. in this group that the maximum DI obtained for the notch

CFST beams is 9.89 for specimen S100-5-SSH and 9.87 for specimen S100-2-SSV. However, the corresponding control beam had DI equal to 8.61, from this result showed clear the two specimens had large ductility due to the concrete of confining inside the tube. in addition, in the same group also two specimens 5.36 for specimen S100-2-SLV and 6.12 for specimen S100-2-SSH it's less than from control beam because it's a clear influence of the notch on the specimen shown in Table 4.12.

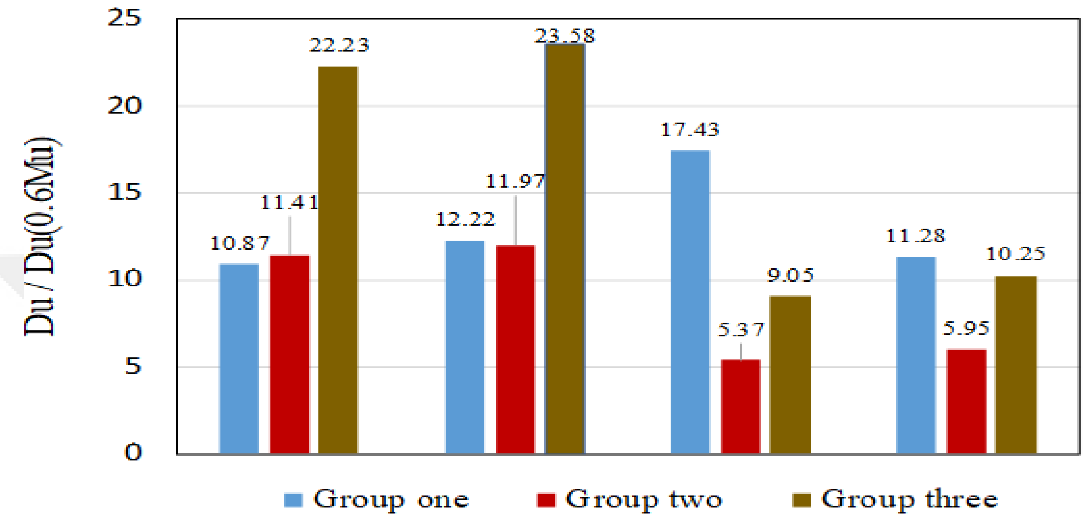


Figure 4.13 Ductility index for specimen (5mm).

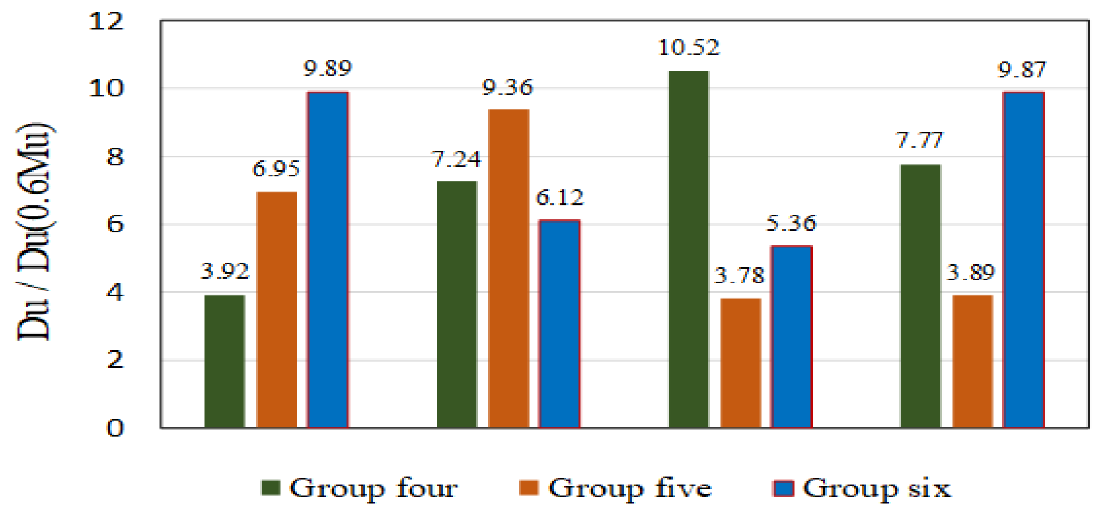


Figure 4.14 Ductility index for specimen (2mm).

CHAPTER 5

CONCLUSION AND RECOMNDSTION

5.1 Conclusions

This thesis presents the results of an experimental study the bending behaviors of CFST stud beams with notches in the steel tube were investigated. In the present work. Investigating the flexural performance of self-compacting concrete-filled steel tube beams with width to depth ratio (B/H) equal 1, the CFST with notches from the top, bottom, and sidewall with two different thicknesses and two different lengths of the notch (long and short). The moment carrying capacities, ductility, failure modes, Moment - mid-span deflection relations, bending moment capacities were listed and discussed.

The typical failure modes of the notch CFST beams were characterized by outward local buckling of the top specimen at the mid-span location between the two loads. This local buckling characterized by single or multiple local buckling according to the thickness of the specimen where the specimen (5mm) did not occur local buckling but in the thickness (2mm) observed all the specimens occur local buckling.

The failure mode of the notched CFST beam is completely different from that of the complete steel tube concrete under bending. Typical failure modes are in the first group notch on the top when the applied load on the specimen the notch orientation vertical it closing of notch but the notch orientation horizontal it opening outward. For group two the notch in the bottom all specimens that orientation vertical and horizontal opening outward. For group three the notch in the sidewall of the beam, the notch orientation horizontal deformed but the notch orientation vertical opening outward. For group four the same as groups one it seemed that the discussion and conclusion. For group five the same as groups two its seemed that the discussion and conclusion. For group six the same as groups three it has seemed that the discussion and conclusion.

All the specimens failed by bulging of the notch. The failure of the concrete core was also determined by the notch orientation and location. For specimens (5mm) group one, two and three also failed by rupture of the bottom sides occurred at the end of the test. These ruptures occurred only at the bottom tension flanges of the beam specimens. Excepting of the specimens S100-5-SSH and S100-5-SLH without rupture. For specimens (2mm) group four, five and six all specimens of beams failed by rupture of the bottom and sides at the end of the test.

All the beams showed a smooth moment deflection curve with large deflection up to failure. The control CFST beams without notch have been compared with all specimens steel tubes with a notch for all groups; it found that the control had a moment more than of other specimens due to there is no notch. The concrete inside the tubes has significantly increased the moment carrying capacity, stiffness and reduce the deflection.

The maximum moments and percentage of decrease in moment capacity for all groups specimens with notch were compared them with the control beams without notch were summarized and tabulated. Showed the effects of notch length on the load-displacement response, where these two specimens had the same notch orientation, therefore showed the influence long length in the top of the beam the orientation horizontal 10.42% for the specimen S100-5-TLH and 8.92% for the specimen S100-5-TSH.

The moment capacity decreased by increasing the length of the notch. While the orientation vertical showed obverse where1.50% for the specimen S100-5-TLV and 2.67% for the specimen S100-5-TSV. From the result observed the orientation vertical had the maximum moment more than the orientation horizontal.

For group two, showed the influence long length notch in the bottom of the beam the orientation horizontal, from this result observed very clear the effective length of the notch. Where the orientation vertical same effect for horizontal by the length of notch the maximum load, where the comparison between the orientation horizontal and vertical found it. The notch in a vertical orientation more than effect horizontal.

For group three, showed the influence long length notch in the sidewall of the beam the orientation horizontal, where S100-5-SLH equal 6.78% and S100-5-SSH equal 7.34% from the result observed the orientation horizontal long of notch had the maximum moment more than the horizontal short notch, for the orientation vertical where S100-5-SLV equal 12.55% and S100-5-SSV equal 1.44%. Showed the effect notch on the long vertical more than the short vertical due to the moment capacity decrease by increase length of the notch. While the comparison between the orientation horizontal and vertical found it. For the specimen sidewall, long notch horizontal observed the moment capacity more than the specimen sidewall long vertical also for short notch same effect.

For group four, showed the influence long length notch in the top of the beam the orientation horizontal, where S100-2-TLH equal 4.75% and S100-2-TSH equal 2.78% from the result observed the, therefore, showed the influence long length in the top of the beam the orientation horizontal and vertical. The same as group one it seemed that the discussion and conclusion.

For group five, showed the influence long length notch in the bottom of the beam the orientation horizontal, where S100-2-BLH equal 2.05% and S100-2-BSH equal 2.00% from the result observed the, therefore, showed the influence long length in the top of the beam the orientation horizontal and vertical. The same as group two it has seemed that the discussion and conclusion.

For group six, showed the influence long length notch in the sidewall of the beam the orientation horizontal, where S100-2-SLH equal 1.81% and S100-2-SSH equal 0.79% from the result observed the, therefore, showed the influence long length in the top of the beam the orientation horizontal and vertical. The same as group three it has seemed that the discussion and conclusion.

From the above point made comparison between the length of notch from group one, two and three to the notch orientation: for specimens with a horizontal long notch 10.42% for S100-5-TLH 9.94% for S100-5-BLH and 6.78% for S100-5-SLH showed from this result the effect notch on the beam top more than of the other. For specimens with a horizontal short notch 8.92% for S100-5-TSH 2.23% for S100-5-BSH and 7.34% for S100-5-SSH same effect long notch.

Comparison between the length of notch from group one, two and three to the notch orientation: for specimens with a vertical long notch 1.50% for S100-5-TLV 24.86% for S100-5-BLV and 12.55% for S100-5-SLV showed from this result the effect notch on the beam bottom more than of the other, for specimens with a horizontal short notch 2.67% for S100-5-TSV 19.37% for S100-5-BSV and 1.44% for S100-5-SSV same effect long notch.

When compared between group four, five and six to the notch orientation horizontal and vertical also compare between in length of the notch and short. Its same behavior and effect notch on the specimens.

Whilst compared effect thickness between (5mm) for group one, two and three with (2mm) for group four, five and six. Found the effect of the notch on the thickness (2mm) more than (5mm) and the moment capacity for (5mm) more and better than (2mm).

According to the result, for thickness (5mm) the ductility index from group three is greater than the group one and two, it means that the ductility index from group three is better than the group one and two.

According to the result, for thickness (2mm) the ductility index from group six is greater than the group four and five, it means that the ductility index from group six is better than the group four and five.

From the result, ductility for (5mm and 2mm) found the thickness (5mm) had more ductility.

Finally, for all the point above the notch in the bottom more effect and danger to the structures when compare with the notch top and sidewall for (2 mm,5 mm) in addition to the notch in the bottom had less than ductility from top and sidewall for (2mm,5mm).

5.2 Recommendations for future research

This research focused on the performance effect of notch on the beams square shape with mix Self-Compacting Concrete (SCC). Specimens ranged in different thickness (2mm,5mm) with two different lengths of notch and three position from the top to bottom and sidewall. The following recommendations would be taken to expand this experiment:

- There is still a wide range of CFST beams with notch and different thickness to use in future research. This will the risk ratio is determined and reduced.
- Besides square tube, situations may arise in practical applications to use another shape tube. Thus, additional research is necessary to understand the behavior of notch on the CFST beam with different shape of tube section.
- Through this study we conclude the effect of the notch. For all the point above the notch in the bottom more effect and danger to the structures when compare with the notch top and sidewall for (2 mm,5 mm) in addition to the notch in the bottom had less than ductility from top and sidewall for (2mm,5mm).

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