SEPTEMBER 2017 M.Sc. in Civil Engineering BASHAR AHMED SHEHAB

UNIVERSITY OF GAZİANTEP
GRADUATE SCHOOL OF
NATURAL & APPLIED SCIENCES

COMPRESSION BEHAVIOR OF STEEL BAR REINFORCED CONCRETE FILLED STEEL TUBE COLUMNS

M.Sc. THESIS
IN
CIVIL ENGINEERING

BY
BASHAR AHMED SHEHAB
SEPTEMBER 2017

Compression Behaviour of Steel Bar Reinforced Concrete Filled Steel Tube Columns

M.Sc. Thesis

in

Civil Engineering

University of Gaziantep

Supervisor

Assist. Prof. Dr. Talha EKMEKYAPAR

by

Bashar Ahmed SHEHAB

September 2017

REPUBLIC OF TURKEY
UNIVERSITY OF GAZİANTEP
GRADUATE SCHOOL OF NATURAL & APPLIED SCIENCES
CIVIL ENGINEERING DEPARTMENT

Name of the thesis: Compression Behaviour of Steel Bar Reinforced Concrete Filled Steel Tube Columns.

Name of the student: Bashar Ahmed SHEHAB

Exam date: 06.09.2017

Approval of the Graduate School of Natural and Applied Sciences

Prof. Dr. Ahmet Necmeddin YAZICI

Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

Prof. Dr. Abdülkadir ÇEVİK

Head of Department

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 the degree of Master of Science.

Asst. Prof. Dr. Talha EKMEKYAPAR

Supervisor

Examining Committee Members:	Signature
Assoc. Prof. Dr. Ali Fırat ÇABALAR	....
Asst. Prof. Dr. Fatih ÇELİK	....
Asst. Prof. Dr. Talha EKMEKYAPAR	....

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Bashar Ahmed SHEHAB

ABSTRACT

COMPRESSION BEHAVIOR OF STEEL BAR REINFORCED CONCRETE FILLED STEEL TUBE COLUMNS

SHEHAB, Bashar Ahmed
M.Sc. in Civil Engineering
Supervisor: Assist. Prof. Dr. Talha EKMEKYAPAR
September 2017
90 pages

In-filled composite columns combine the benefits of steel of high ductility and tensile strength; and concrete of high stiffness and compressive strength, thus, it turns out to be increasingly well known in tall buildings, bridges and multi-story structures. The uses of reinforcing bars with in-filled composite columns have a few popularity, in spite of the benefits of reinforcing bars which could be maximized since the core concrete in CFST columns is also supported by the outer steel tube. So, this study was carried out to increase the knowledge of using reinforcing bars with concrete in-filled circular steel tubes (CFST) columns and its effect on compressive strength capacity, ductility, stiffness, and failure mode. Twenty-two column specimens were tested under axial compression load with different parameters, two different D/t ratio, the method of using reinforcing bars (welded or with stirrups), number of reinforcing bars, and the diameter of reinforcing bars as well. It was observed that the reinforcing bars increased the compressive strength capacity, ductility, and stiffness of the column specimens and that enhancement was increased with increasing the number and diameter of reinforcing bars. However, by applying a comparison between the welded reinforcing bars column specimens and the specimens with stirrups, it was observed that for both the D/t ratio (36.28, and 20.3), the strength index (SI) for column specimens with stirrups was slightly greater than the specimens of welded bars, which increased the ultimate load capacity. On the other hand, the initial stiffness, ductility, and failure mode were a slightly more enhanced for the specimens with welded bars. Since the steel tubes are closed section, the construction process of stirrups with in-filled composite column in real life is harder than welded bars which can be manufactured in origin, which make the welded reinforcing bars as a superior alternative to stirrups, which give approximately the same behavior of stirrups with easier construction process.

Keywords: In-filled steel tube, composite column, reinforcing bars, compressive behavior.

ÖZET

İÇİ BETON DOLDURULMUŞ ÇELİK DONATILI ÇELİK TÜP KOLONLARIN BASINÇ DAVRANIŞI

SHEHAB, Bashar Ahmed
Yüksek Lisans, İnşaat Mühendisliği
Danışman: Yrd. Doç. Dr.Talha EKMEKYAPAR
Eylül 2017
90 sayfa

İçi beton doldurulmuş çelik tüp kolonlar çeliğin yüksek sünekliği ve çekme mukavemetini ve betonun yüksek basınç mukavemeti ve rijitliğini bir araya getiririler. Böylece yüksek binalarda, köprülerde ve çok katlı yapılarda tercih edilirler. Faydalarının iyi belirlenmiş olmasına karşın, çelik donatıların içi beton doldurulmuş çelik tüp kolonlarda uygulanması ile ilgili araştırma sayısı oldukça azdır. Bunu dikkate alarak bu tez çalışması çelik donatıların içi beton doldurulmuş çelik tüp kolonlarda ortaya koyacağı etkilerin araştırmasını ve bu donatıların basınç mukavemetine, rijitliğe, sünekliğe ve çökme biçimlerine yapacağı katkıları araştırmaktadır. Bu çalışmada farklı parametreler kullanılarak 22 kolon numunesi basınç yüklemesi altında test edilmiştir. Parametre olarak D/t oranı, çelik donatı uygulama şekli, donatı sayısı ve donatı çapı çalışılmıştır. Deney sonuçları göstermiştir ki çelik donatılar kompozit kolonların kapasitesini, sünekliğini ve rijitliğini oldukça artırmaktadır. Artan donatı sayısı ve çapı kolon davranışına daha olumlu katkılar yapmaktadır. Ancak, etriyeli ve kaynaklı donatı düzenlerinin karşılaştırılması sonucunda 36.28 ve 20.3 D/t oranlarının her ikisi içinde etriyeli donatı düzenine sahip kolonların bir miktar daha fazla mukavemet indeksine sahip oldukları görülmüştür. Diğer taraftan, kaynaklı donatı düzeninin kolon rijitliğini, sünekliğini ve çökme şeklini etriyeli donatı düzenine göre bir miktar daha fazla iyileştirmiştir. Ayrıca çelik tüpler kapalı kesit olduklarından, etriye kullanılarak hazırlanan donatı uygulaması gerçek hayatta zordur. Bu durumda kaynaklı donatı düzeni üretimi kolay olduğundan ve etriyeli donatı düzenine sahip kolon ile yakın davranış gösterebildiğinden uygulama anlamında önemli bir alternatif olabilecektir.

Anahtar kelimeler: İçi beton doldurulmuş çelik tüp, kompozit kolon, çelik donatı, basınç davranışı.

To My

Father and mother, which they spend their life for me and give the support to reach this level from the knowledge

ACKNOWLEDGEMENTS

First of all, praise be to my God, the Cherisher and the Sustainer of the World. A thesis work is a product of collective efforts and able guidance. A work has a life and spirits of its own. I believe that no work can achieve its ultimate objectives without proper guidance and support from others. I wish to express my heartiest thanks to my respected guides **Assist. Prof. Dr. Talha EKMEKYAPAR**, Department of Civil Engineering, University of Gaziantep, Gaziantep - Turkey, who devoted their valuable time and provided enthusiastic guidance, advice and continuous encouragement, which were the constant source of inspiration for the completion of this thesis work.

I cannot forget to recall with my heartiest feelings, the never-ending heartfelt stream of caring and blessings of **my Father** and **my Mother**, to support me with everything from my early childhood till reach higher education, They were always denying themselves for supporting and pushing me to every success.

Finally, I wish to record my heartfelt gratitude and indebtedness to all of my friends, whose help me during the experimental work. I appreciate them for the kind of support that they has extended to me.

TABLE OF CONTENTS

Page

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 General

The utilizations of composite columns with concrete-filled hollow section in tall buildings, bridges and multi-story structures turn out to be increasingly well known. This kind of column combines the benefits of steel of high ductility and tensile strength; and concrete of high stiffness and compressive strength. However, they are a compression members that have the valuable qualities of both materials in an extraordinary way and shows outrageous resistance and high flexibility in the mix with small cross-sectional dimensions [1].

A steel-concrete composite column is including either a concrete encased steel section or a concrete-filled tubular section of steel and for the most part is utilized as a compression members in the composite structure with outstanding mechanical behavior such as: high strength, high stiffness, and high ductility. The large energy absorption capacity and the mentioned properties guarantee that the composite columns could be used in the seismic regions to resist the seismic loads. Furthermore, the steel-concrete composite columns have numerous advantages which allow such members to be employed in different type of engineering construction such as seismic-resistant structures, bridge piers, storage tanks columns, decks of railways, and as piles.

The composite columns have more advantages than the normal steel reinforced concrete (RC) columns, the main benefits are:

- Increased strength for the similar cross sectional dimension.
- Improved stiffness, leading to reduced slenderness and increased buckling resistance.

- Sufficient fire resistance for the concrete encased column types.
- Produce a good protection against corrosion for encased columns type.
- In case of in-filled columns the external steel works as a formwork, and so, reduce the cost of the construction.

Classical cross-section areas of composite columns for completely and incompletely concrete encased steel are outlined in Figure 1.1, and Figure 1.2 indicates three ordinary cross-sections of the concrete filled tubular section.

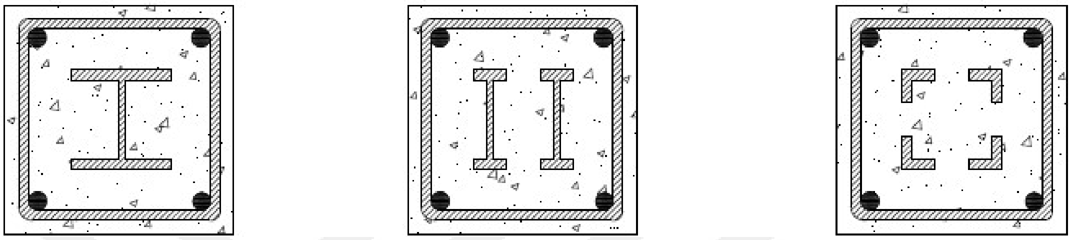


Figure 1.1 Encased composite columns.

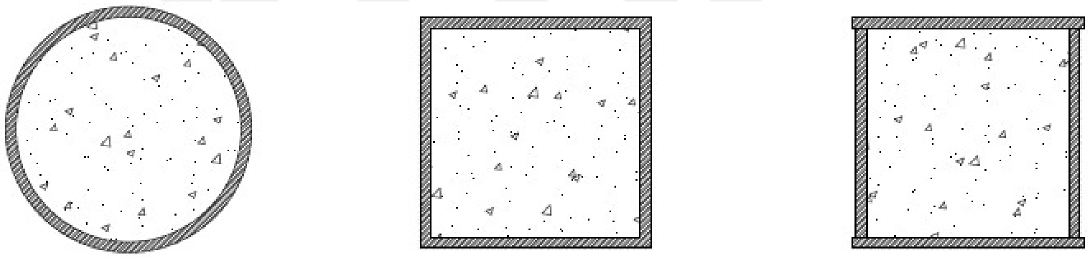


Figure 1.2 In-filled composite columns.

1.2 Encased Composite Columns

The encased composite column is shown in Figure 1.1, was the first type established in composite columns, it was an effort to enhance the steel column's fire resistance and protect the steel core from sudden overheating when fire takes place. Generally, in the encased composite column constructions, the steel core support the loads of the structure during construction process. Later, the steel section will be surrounded by casted concrete. Steel core and concrete are consolidated in such a mold that the promising properties of two materials are used efficiently in the composite column. The steel properties of higher strength and lighter weight allow the utilizations of lighter and smaller foundations. The concrete addition enables the building frame to easily limit the lateral deflections and sways [2].

As the concrete-encased steel sections are utilized in bridges and buildings in seismic regions, the research on the cyclic behavior of these composite columns was conducted [3]. In general, this type of columns displayed a favorable energy dissipation capacity. The energy and deformation capacity of the concrete-encased steel column decreased with the increase of axial load level [4].

1.3 In-Filled Composite Columns

As exposed in Figure 1.2, there are many types of in-filled composite columns, but the circular tubular columns have more benefits than the further sections when they used as compression members. Circular tube columns have a large uniform flexural stiffness in all directions compared to other sections with the same cross-sectional areas. When the steel tube filled with concrete core, the maximum strength of the construction member will increase without considerable increases in cost. The local buckling of the tube wall and the concrete itself will be delayed by the concrete and that is the main effect of concrete. In the restrained state, it is able to provide higher stresses and strains.

It is recognized that the circular cross-section sustains the most grounded repression to the concrete core, and the local buckling usually inclined to happen in square or rectangular cross-sections. Though, the concrete-filled steel tubes with a rectangular hollow section or a square hollow section are still widely used in construction, because of the easier design in beam-to-column connection, aesthetic reasons and for high cross-sectional bending stiffness [4].

It is essential to guarantee that in the outrageous occasion of the structural failure, it should be in a ductile manner. This implies that the structure will not fail in brittle fashion without notice of failure. The present seismic design philosophy relies on dissipation and load capacity by post-elastic deformation for endurance in the earthquake times. When a ductile structure is subjected to over-burdening it will have a tendency to behave inelastically and doing, will redistribute the overindulgence load to multilateral part of the structure. In the event that a structure is ductile, it can be relied upon to adjust to surprising over-burdens, impacts and auxiliary developments because of the establishment.

The use of concrete-filled steel tube columns provides economical solutions, because it takes less area compared to the normal reinforced columns and so, it provides a larger floor area, and there is also no need for the use of shuttering during concrete construction, hence, the construction time and cost are reduced. This is an advantage for skyscraper design in some cities where the cost of floor spaces is high, especially in the lower story of tall structures where the stubby columns typically exist [5].

1.4 Components of In-Filled Composite Columns

Apart from the concrete-filled steel tubes shown in Figures 1.1-2, there are a different type of "general" part assignment in the CFST family, i.e. concrete-filled double skin steel tubes (CFDST), as well as the special concrete filled steel tubes sections. Some of the CFST types will be sustained within the component of composite columns.

1.4.1 Structural Steel

Hollow sections could be manufactured welded or seamless. Seamless hollow sections are completed in two stages, first stage comprises of an ingot piercing process, and the second one is elongate this hollow section into a desired shape. In order to employ the required diameter, the tube goes through a sizing machine. For the time being, a longitudinal weld for hollow sections is generally made with an induction welding process or with electrical resistance welding processes, as shown in Figure 1.3. The plate is welded longitudinally after being formed by rollers into a cylindrical shape. By using the electrical resistance , plate edges were heated while being pushed together by rollers, resulting in a pressure weld. After the weld process finished, all external remaining parts were trimmed. By deforming the circular hollow sections through forming rollers, the rectangular hollow sections are made, as exposed in Figure 1.4. The previous process could be made in the cold or hot procedure, and longitudinally welded or seamless can be used. Seamless sections would be reasonable for thicker sections and special applications, otherwise, it is ordinary perform to use longitudinally welded hollow sections which are cost effective [6].

For the larger circular hollow sections, also they could be prepared by rolling a strip through a U-O press process which outlined in Figure 1.5. When the plate been manufactured by the one of the previous methods to the required shape, and by using a submerged arc welding process, the longitudinal weld is made. Another operation for large tubular sections which is required to use wide and continuous strip to be deployed into a shaping machine at specific angle to form the spiral circular section and a welding process are applied to weld the edges together, as Figure 1.6 shows [6].

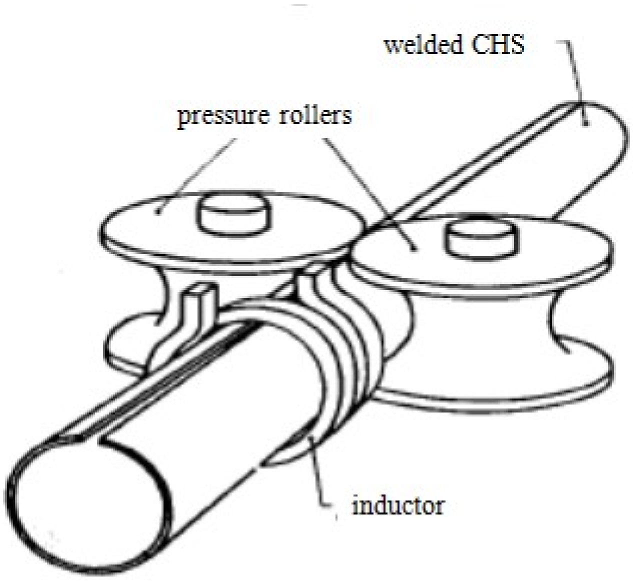


Figure 1.3 Induction welding process [6]

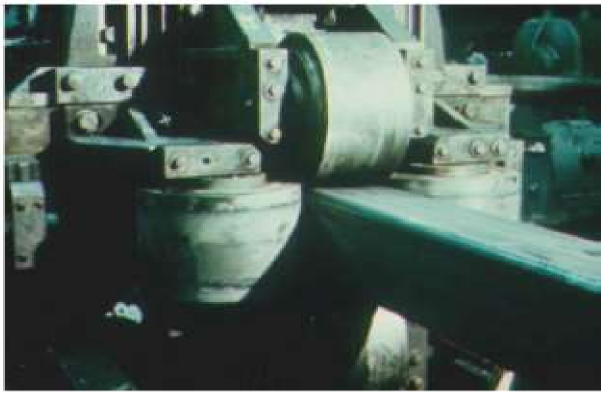


Figure 1.4 Manufacturing of rectangular hollow section [6]

Figure 1.5 Forming of large CHS [6]

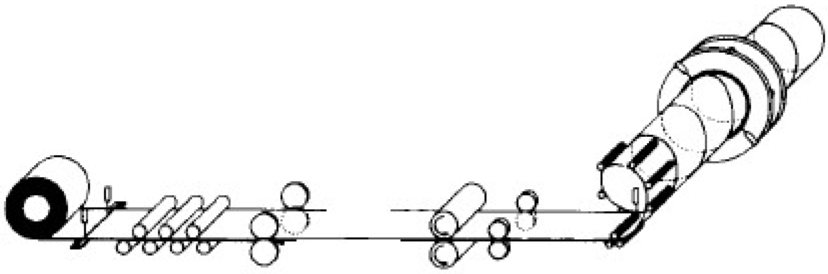


Figure 1.6 Spirally welded CHS [6]

1.4.2 Concrete

When the existing wall thickness of steel tube was not adequate to sustain the needed load capacity resistance, the concrete might be used to fill the hollow section. For instance, when it favored in the building to have the same outside size of the hollow column sections in each floor, the smallest wall thickness could be used in the top floor and the column dimensions increased as the load increased for lower floors. When the largest available wall thickness of the hollow sections are not adequate for the lower floor, a concrete mix can be used to fill the hollow section and the load capacity will be increased to sustain a sufficient resistance [6].

An essential cause of using in-filled composite sections is that the sections can be generally slender. A promising design formulas are given in Eurocode 4 and other specifications. The in-filled steel tub column contributes not exclusively to an expansion in load caring capacity, also enhances the fire resistance time. Wide test projects completed by ECSC and CIDECT had demonstrated that hollow steel tube filled with reinforced concrete with no fire protection at the external surface like vermiculite panels, or plaster can achieve a fire resistance of 2 hours relying upon the reinforcement percentage of the concrete, the cross-section ratio of the concrete and steel tube, and the subjected load [7-8].

1.4.3 Reinforcing Steel

Steel bar reinforcement has a wide use in civil engineering structures. The strength and unity provided in reinforced concrete members are the results of using steel bar reinforcement. Steel bar reinforcement has the further benefits of eliminating the adverse effects of time-dependent behavior of the concrete. The shrinkage and creep behavior of the concrete could minimized by employ a steel bar reinforcements. The application of steel bar reinforcement in civil engineering structures is conventional and a basic routine. The cost of the steel bar reinforcement is very low compared to additional benefits of such reinforcement. The benefits of longitudinal steel reinforcement could be maximized since the core concrete of CFST columns is also supported by the outer steel tube. Steel reinforcements are typically used to enhance the resistances of concrete-filled steel tubes. The structural steel sections contribute a considerable measure to column capacities without changes of column section. That contribution to the column capacities can be counted as the combined capacities of the structural steel and the in-filled steel tubular parts. For the reinforcing bars, since they are all around embedded in the concrete, they might be considered for the resistance of the column [2].

1.5 Purpose of the Thesis

The major aim of this thesis is to generate detailed knowledge about the performance of the steel bar reinforced concrete-filled steel tube columns (CFST), the influence of steel reinforcement bars to the ductility, compression capacity, and the toughness will be investigated.

1.6 Outlines of the thesis

Chapter two presents literature review of the past research that deal with composite columns and its behavior under different load condition and various parameters like sectional shape and slenderness ratio and other. Chapter tow is divided into two parts, the first part presents the composite columns without reinforcement, while the second part presents the composite column with any type of reinforcement.

Chapter three extends two of the most promising international specifications for composite columns. This chapter presents Eurocode 4, and ANSI/AISC 360-10 design specification with their formulations and limitation.

Chapter four explains the experimental work of this research with the detail of composite column specimens. The preparation and manufacturing of these specimens can be found in this chapter, as well, the test method with instrumentation that used to record data

Chapter five presents the results of the compression test. Results are discussed in this chapter and the columns behavior are observed by analyzing the results. The contributions of the composite column parameters to compression capacity and the ductility are also detailed in this chapter in a comparative form. Chapter six presents the conclusion of this study.

CHAPTER 2

LITERATURE REVIEW

2.1 General

Composite columns, in general, are made of structural steel, concrete and reinforcing steel. There are three main types of composite columns can be distinguished, totally encased composite column, partially encased composite column, and filled composite columns. Spite of that, the composite column was endorsed from the end of World War II till the first of the 1970's (Viest et al. 1997, 1.13); the research had launched a long time before that, toward the start of the twentieth century. The joining of these materials had various inspirations, steel sections were frequently encased in concrete to shield them from fire, while concrete columns were combined with structural steel as reinforcement [9].

This chapter will present a summary of the literature review pertained to using of reinforcement in composite columns and its effects by the present (i) composite columns without reinforcing bars and (ii) reinforced composite columns.

2.2 Composite Columns without Reinforcement

Dutta and Bhattacharyya [10] made an experimental study on the in-filled steel tubular columns behavior. They concluded that the load carrying capacity of CFST's section was larger compared to hollow steel tubular columns. They reported that this excess in strength is not just due to the addition of individual capacity, but also because of the confinement of core concrete. The strength increases by a certain factor λ. The strength increases factor λ obtained in that study was 2.206.

O'Shea and Bridge [11] made studies on short CFST columns. They examined the loading condition within axial loading of the concrete only, axial loading of the steel only, and an axial compression load on the column section within concrete and steel tube. The studied specimens have an L/t ratio equal to 3.5 which considered as short columns and a diameter-to-thickness ratio 60 up to 220. The unconfined compressive strength of the concrete core found to be 50, 80 and 120 MPa. It was observed that the hollow circular steel tubes strength without filling with concrete was extensively influenced by local buckling, even with improvement by adding internal lateral restraint, at the opposite with square tube. As an alternative, the usual outward buckling was uninfluenced by the filled core. But for the thin-walled steel tubes under axial load and with adequate bond strength of concrete core and inner surface of steel tube, the local buckling of the steel tube wasn't occurred. They developed an equation depends on an effective area approach of column section to predict the ultimate strength of the thin-walled steel tube.

Schneider [12] studied the behavior of 14 in-filled steel tube specimens subjected to concentric load, five were square, three were circular and six were rectangular in cross section. In all these specimens, a yield strength of 317 MPa of cold-formed carbon steel was adopted. The concrete used for the composite columns acquired a 20 MPa compressive strength at 28-day. In this study, the shape of steel tube was investigated to identify their effect on the concrete filled steel tube column specimens ultimate strength and the sequestration made by steel tube. From the measured perimeter-to-longitudinal strains of the steel tube, it was observed that significant confinement for specimens with circular cross sections was not developed until the axial compression load reached approximately 92% of the yield strength of the specimen. While the specimens with rectangular and square steel tubes didn't provide sufficient confinement as a circular cross section, even when the load exceed the yield load of the composite column. The circular tubes offer better axial ductility than that of square and rectangular section tubes. Therefore, circular steel tubes are preferable to square and rectangular ones if concrete-filled steel tube columns are sustained in a regions of seismic risk.

O'Shea and Bridge [13] evaluated a number of design methods that predicted the strength of circular thin-walled composite columns under various conditions of loading. The examined loading conditions was included an axial load applied only on concrete core, on steel tube only, on the concrete and steel simultaneously, and to the total cross section at small eccentricities. In spite of the verification of the design codes and proposed design methods, it was known that the bond behavior between the inner surface of steel tube and the concrete core was fundamental in evaluating the formulation of local buckling. Though there was no design method presented for the local buckled specimens due to inadequate tests, it was recommended that the thin-walled CFST columns should be conservatively designed, as the previous design methods assumed full bond on the interface of concrete and steel.

Uy [14] conducted a set of experiments to study the strength of short CFST column specimens by using high strength concrete core and box section for steel tube. Numerical models were presented in these studies. Nevertheless, a comparisons with Eurocode 4 for composite columns were carried out and it was obtained to be un-conservative in prognostication of the axial and combined load strength. Because of that, a mixed analysis system was proposed, which treat steel as linear elastic and concrete as rigid plastic. And this presented model was compared with the presented numerical model, it was observed that both these models were conservative in estimating the results.

Johansson and Gylltoft [15] conducted a study on 9 circular CFST column specimens subjected to three kinds of axial loading condition to investigate the mechanical behavior of the CFST. Numerical analysis was then established based on finite element (FE) models to compare with the experimental results and to further explore the influence of bond strength on the behavior of CFST column. All column specimens were 159 mm in diameter and 650 mm in length, and have a wall thickness of 4.8 mm. The three condition of the loading that was attributed to this study: (1) load subjected to the steel tube only, (2) load subjected to the core concrete only, and (3) load subjected to concrete and steel simultaneously. 4 hollow steel tube were tested under axial load. For that CFST column with axial load subjected to the whole cross-sectional area, and that to the core concrete. The concrete section obtained better compressive strength than estimated, which was suggested that the steel tube could produce sufficient confinement. When the axial compression load was subjected to the filled concrete only, the confinement effect to concrete core was advanced due to steel tube when the concrete expanded laterally. Under circumstance like that, the bond strength of core concrete and steel tube was beneficial to the confinement influence of the core concrete. On other hand, when the axial load was subjected to the whole cross-section, the bond strength had no effect on the structural performance as there was no relative displacement between steel tube and core concrete. Furthermore, it was worthless that the stiffness was improved by the increase in bond strength of the steel tube and core concrete. Though the eligibility confinement presented by steel tube was better when the axial load was subjected to core concrete only, it was not reliable to depend on the natural bonding to obtain the complete concrete-steel composite action. Also, it was not practical to guarantee that the load is resisted by the concrete core only in building construction.

Giakoumelis and Lam [16] carried out a study on 15 in-filled circular steel tube column specimens under concentric load. The concrete of 30, 60, and 100 MPa concrete strengths were used in this research. The specimen's slenderness ratio was from 22.9 up to 30.5. Column diameter was 114 mm, and 300 mm in length for all specimens. The structural steel tubes used in this study were hot-finished with thickness of 3.6 and 5 mm. The aim of this study was to examine the core concrete bond strength with steel tube by comparing with 5 specimens who greased on the interface of the concrete in fill and steel tube. It was concluded from compression load-deformation curves, that for in-filled steel tube columns with normal concrete strength (30 and 50 MPa), which suggested that the reduce in axial compression capacity due to bond strength is negligible, the differentiation of axial capacity for greased and non-greased specimens were very small. While the specimens with high-strength concrete (100 MPa) and non-greased have a load capacity 14% higher than that of greased specimen. The theoretical results calculated based on Eurocode4 [1994], ACI 318-95 [1995], and Australian standards AS 3600 [1994] & AS 4100 [1998], were used to make a comparison between the experimental results of non-greased specimens and specifications. A coefficient was proposed to modify the design formula of CI and A_S, because they were conservative in predicting the axial strength. While the Eurocodes4 predicted the axial strength reasonably well. But for greased columns, all experimental results are approximately 30% lower than that of Eurocode4, leading to that the reduction is caused by the lack of bond strength between core concrete and steel tube.

Sakino et al. [17] managed a five-year research for 114 CFST specimens with square and circular cross-sections applied under axial loading, to find out the effect of the steel-concrete composite action and to deduce the design formulas for the maximum load of the CFST column, the following primary parameters with extensive ranges were studied: (1) tube shapes, comprise square and circular cross-section; (2) the structural steel tensile strength, comprises 400, 600 and 800 MPa; (3) tube diameter-to-thickness, and tube width-to-thickness ratio; and (4) concrete strength, comprise 20, 40 and 80 MPa. For the circular concrete filled steel tube column specimens, it was concluded that the ultimate axial compression load was greater than the sum of core concrete and steel tube strength, which denotes that the circular shape steel tubes could present effective confinement pressure to the core concrete. A linear formula of the tube yield strength had been presented that difference between the ultimate load and the summation of steel and concrete strength. As well, it seemed that the current design formula presented from the Architectural Institute of Japan (AIJ) was a little conservative and design formulas based on the US-Japan data were suggested. Furthermore, it had been concluded that the confinement affect for square columns with large *B/t* ratios was regarded as negligible because the maximum axial load is less than the sum of core concrete and steel tube strength. That because of the local buckling of steel tube wall decrease the axial load capacity of CFST columns with large *B/t* ratios. Further, the axial bearing load of square CFST columns derived from the US-Japan experiments were slightly greater than theoretical results. The main cause for this increase was attributed to the strain hardening influence more than confining effect.

Zeghichea and Chaouib [18] conducted tests on twenty-seven in-filled steel tubular columns. Key parameters were, column slenderness, t compressive strength of concrete core, axially load eccentricity, and eccentrically loaded columns with double or single curvature bending. They studies the effect of these parameters on strength capacity and performance of mentioned column specimens. However, a comparison with Eurocode 4 Part 1.1 for the experimental failure was carried out. It was concluded that the load carrying capacity of composite columns have an inverse relationship with slenderness ratio. Furthermore, a high concrete strength uses improved the load bearing capacity of column specimens that have been tested, but with a higher rate decreasing of load–slenderness relationship compared to that for normal strength concrete columns. The prediction of Europe Cod 4 were on the safe side for the single curvature of bending in axially and eccentrically loaded columns. But, for column specimens in the double curvature of bending, the result of the test and numerical analysis show that the predictions of Europe Code 4 are on the unsafe side. The failure mode of the tested columns was by overall buckling with no sign of local buckling.

Liu [19] carried out a study for twenty-two specimens in 4 series that were manufactured and tested in the research program, to determine and analyze the ultimate capacity and the load-shortening curve of the specimens. Tensile and crushing tests were sustained to determine the properties of the steel tube and concrete core respectively. Four flat plates were welded together to form the rectangular steel hollow section. At age of 32 days after casting, the specimens were tested by applying an axial load of 5000 kN. The overall deformation was obtained by placing 4 displacement transducers between the two platens of the test machine. As well for longitudinal strain, 4 strain gauges were affixed at mid-length of the external face of steel tube. It was proposed that the steel hollow section were to be less efficient than the circular section, that because in rectangular section, the concrete core confinement is by plate bending, while in circular steel tube the core concrete be confined by hoop stress. Test result obtained the favorable ductility behavior of the high-strength concrete composite columns. It was concluded that because of the confinement of steel section to core concrete, the concrete strength was increased, and these increases were negatively influenced by the cross-sectional ratio. By comparing the load at failure for tested specimen with design codes, showed that the ultimate capacity is conservatively estimated in EC4, ACI, and AISC by 1, 9, and 11% respectively.

Han et al. [20] tested thirty-two specimens to explore the behavior of the CFST stub columns subjected to axial compression load. The key parameters in this study were, structural steel section: square and circular section shape; the local compression area ratio: 1.44 and 16; and endplate thicknesses: 2 to 12 mm. However, a finite element modeling used to analyze the in-filled steel tube stub column specimens in a comparison with results of tested specimen. In general, concrete-filled steel tube columns are in a ductile manner when it is under local compression, and so it was concluded that the carrying capacities of the composite columns reduce by the local compression influence. The behavior of composite columns could be enhanced by the top endplate under axial local compression since the pervasion local compression force can be made by constraining the deformation at the end sections. The increasing of the local compression area ratio (β) could be decrease the strength index (SI). Anyhow, the influence of β on the ductility index (DI) is restrained.

Yu et al. [21] explored the influence of different load conditions on the maximum compression load capacity for 17 specimens, and load–deformation curves; concrete strength, and notched holes of the column specimens. It was observed that the increase of compressive strength for both normal concrete and self-consolidation concrete lead to increasing the compressive capacity of the column. Because of the notches that was carried out in the mid-length region with full perimeter slot, no stress flow could exceed through the notched holes anymore; the behavior of the concrete in three-dimensional compressive stress was changed; and the axial compressive modulus of elasticity and the capacity also reduced as a total. With the dimension decreasing of the full slot, the confinement influence was improved. Adverse, with dimension increasing of the slot, the confinement effect decrease causing a hardly strengthened for the concrete core, and so, the ultimate capacity of the specimen was decreased. By applying the load to the concrete core only or initially, the influence of confinement observed later but enhanced. Anyhow, the residual capacity of stub columns subjected to the mentioned four loading condition was hardly effected.

Tao et al. [22] made a study on 36 square specimens to enhance the in-filled thin-walled steel tube stub column behaviors by adding longitudinal inner-weld on the steel structure, including thirty stiffened stub columns and six unstiffened specimen. The purpose of study was to observe the enhancement in ductility of mentioned column specimens with different methods: increasing the stiffener number on each side, increase stiffener height, using saw-shaped stiffener, anchor bars or welding binding on stiffener, and using steel fiber with concrete. It has been found that only the effect of specimen elastic modulus has been moderated. Increasing in stiffener number has a postponed effect with local buckling of steel tube. Using saw-shaped stiffener and increasing stiffener height have only improved effect on the maximum strength. The capacity of the section can be increased by welding binding or anchor bars on stiffener; or increase the stiffener number on the steel tube face; and by adding steel fiber to concrete. Using fibers with concrete core is more reliable and effective measurement in increase the ductile behavior of the composite columns.

de Oliveira et al. [23] experimentally analyzed sixteen circular CFST columns under axial compression load concerning, two main key parameters were selected: concrete compressive strength, and columns slenderness ratio, to investigate the confinement effect of steel-concrete composite columns. The filled concrete compressive strength was 30, 60, 80, and 100 MPa, the steel tube *L*/*D* ratio was 3, 5, 7, and 10. It have been observed that for high-strength filled concrete, the ultimate load was obtained with lower value of strain compared to that for normal strength concrete. However, by taking the same concrete compressive strength specimens, the specimens with L/D = 3 have a higher increase of load capacity and that was attitude to the confinement effect, till a point when the concrete crushes and local buckling of steel tube take a place. For these with L/D = 10 showed less strain value because of the overall buckling happened before the full development of concrete core capacity and that cause a reducing in the radial strain of the concrete core which couldn't reach the point of employing the confinement effect of steel tube. The comparison of test results with that based on the specification codes present acceptable results, especially for the higher values of L/D ratio. Both the Brazilian code NBR 8800:2008, and AINSI/AISC 360:2005 gave results 10.7% and 10.4% respectively lower than the obtained result. While for all 16 specimens, the ultimate capacity was higher than estimated in ANSI/AISC Code, and the same for the Eurocode4 and CAN/CSA, they were not conservative, they have different in predicted results compared to the measured maximum load, lower by 2.4%, and 2.3% on average respectively. But in general, for higher L/D values, all the specification codes showed good agreement results.

Uy et al. [24] made a series of test on slender short in-filled stainless steel tube column specimens, to explore their performance under compression axial force, or under combined effect of bending moment and axial force. A comparison of the test result with many presented design methods of conventional in-filled carbon steel tube column. Three square and six circular, stub columns have been tested to observe the affect of different loading conditions. By comparison of the results with empty tubes that have been also tested, it was concluded that, for the in-filled columns, a strength increase was recognized even if the load was subjected only to the steel tube. Under a pure compression and by comparing with conventional concrete-filled carbon steel tube column, the ductile behavior of stainless steel CFST columns was more obvious and have a higher residual strength. When an axial force and bending moment were subjected as a combined action, the short composite columns showed very ductile behavior. The overall strength and stability of the in-filled steel tube columns were improved by the concrete core affect. In case of the slender columns, no significant influence between concrete-filled stainless steel columns and conventional carbon steel CFST columns by observation of test result and failure mode.

Hu et al. [25] conducted a test of three groups of in-filled circular steel tube columns, to evaluate the effectiveness of an external confinement stiffener which made by fiber reinforced polymer (FRP) under axial compressive load. Depending on a thickness of 0.17 mm per ply, the fiber reinforced polymer (FRP) was with an average ultimate strain of 2.28%, and an average elastic modulus of 80.1 GPa. For all groups of specimen was contained three FRP-confined columns and one unconfined column with different number of FRP layers. It was observed that the FRP-confined columns failed in the mid-length of the column specimens at the explosive rupture of FRP wrap due to the lateral expansion, and after that an immediate drop of the axial capacity have been happened. The increase of outward local buckling of the columns cause that rupture of FRP wrap with no any significant degradation of the load bearing capacity of tested column. It was observed from the load-shortening curves, that there was a 60% increase in the maximum load for the FRP-confined column specimens, in comparison with that of bare CFST columns. The axial shortening capacity has been effectively increased up to 153% by using the FRP. However, when the FRP layers was increased, more improvement in column performance was achieved. If the thickness of steel tube wall was smaller for the same FRP wraps thickness, the contribution of the FRP wrap is more clearly to observe. The use of the FRP wraps were effectively delayed the local buckling or completely suppressed it.

Dundu [26] made a study to examine the behavior and load capacity of a 24 concrete-filled circular steel tube column specimens, with different diameter and length; and by using 30 MPa, and 40 MPa concrete strength. In the series 1 tests, the failure mode was mainly flexural buckling without a sign of local buckling existence, and that failure mode happened because of the large L/D ratio of the steel tube. While in series 2, the failure mode caused by the failure of material for the specimens with 1 m length (L/D = 5.16 to 6.56), and 1.5 m length (L/d = 7.74 to

9.84). And it was observed in these columns that bulging of steel tube also happened. For the columns with the same length, the 193.70 mm diameter CFST columns achieved higher ultimate load capacity. These variations in maximum load capacities were attributed to circumferential stress. The load capacities of the CFST columns significantly increase with higher hoop or circumferential stress. For series 1, the average load predicted by Eurocode4 and the South African code (SANS 10162-1) were conservative by 13.6% and 8.4% respectively, and for series 2 were conservative by 20.2% for Eurocode4 and 10.2 for SANS 10162-1 in the prediction of the failure load. In general, all columns were in fully ductile manner but it was observed to be larger in short columns than slender columns.

Abed et al. [27] made a study to explore the behavior of the CFST columns subjected to pure compression load at rate of 0.6 kN/s. Three different D/t ratio 54, 32, and 20 have been carried out in this experiment and filled by two different concrete compressive strength of 60 MPa and 44 MPa. The compressive axial capacities of tested specimen were compared to the theoretical values calculated by the following different specification and codes: the American Institute of Steel Construction (AISC), the American Concrete Institute (ACI 318), the Australian Standard (AS), and Eurocode 4. The conclusion are summarized as: the overall compressive capacity of CFST column specimens was increased by increasing the compressive strength of concrete core. But the ductility of the column specimens decrease with increasing concrete core compressive strength for the higher D/t ratio, and for low D/t the opposite is true. The increase in D/t ratio will decrease the stiffness of column specimens, and the strength in same time because the confinement will be decreases.

Ferhoune [28] carried out a study on 20 specimens of thin wall thickness in-filled steel stubs with a rectangular cross-section that have been manufactured by welding two U-shape steel plates with electric arc to form the required rectangular section of dimension: 100 x 70 x 2 mm^3. The gravel used in the concrete mix was crushed crystallized slag stone type which sieved by 10 mm sieve. The key parameters of this study were the column specimens height (200, 300, 400, and 500 mm), the continuity of welding, and load eccentricity. The test was carried after 28 days of casting date. 16 stubs specimens were tested under an eccentric compression load subjected along the two axis of rigidity, and the other 4 specimens were tested under an axial compression load. The objective of the study was to explore some facts of using of crushed slag in concrete mix might integrated in the manufacturing of non-conventional concrete. The Eurocode4 and the proposed design method of Z. Vrcelj and B. Uy were used to predicate the failure loads. It was observed that the using of crushed crystallized slag have a good influence to the behavior of eccentrically loaded of concrete core compared to that made of normal concrete. The prediction results of the Eurocode4 showed a good agreement, but the proposed design method of Z. Vrcelj and B. Uy was on unsafe side in the case of columns with a length of 400 mm and 500 mm under eccentric load.

Huang [29] conducted a study to investigate the CFST columns behavior in preload case. Total of 12 CFST column specimens were carried in test with preloading ratio of about 0.0, 0.25 and 0.5 and compared the test result with that of the finite element numerical analysis based on ANSYS. The specimens were consisted of short (324 mm), intermediate (1296 mm), and long (1944 mm) columns, with diameter of 108 mm, and 4 mm steel tube thickness, and a slenderness ratio of 12, 48, and 72 respectively. It was concluded that with increasing slenderness ratio the preloading effect will decrease, and the light preloading haven't that considerable effect on the ultimate load capacity but increased the relative deformation. By comparing the test result with that of the FEM based on NSYS, it was showed that FEM modeling agree well with test results. Under concentrically and eccentrically load the structural responses are similar. The maximum load and stiffness of in-filled composite column specimens does not effected at small preloading ratio, Nevertheless, high preloading ratio decreased the ultimate load, and significantly increased the deformation. The higher slenderness and eccentricity of load reduced the ultimate load up to 20%, and further intensify the effect. The preloading influence was more clearly for a higher grade of steel, and this influence increased with preloading ratio.

Ding et al. [30] conducted a track-shaped concrete filled steel tubular stub column specimens stiffened by rebars and subjected to a compressive load, to investigate the mechanical behaviors of SCRT by a experimental, theoretical, and numerical study, with key parameter of flakiness ratio, concrete strength, and stiffener. 12 specimens of 18 in total were stiffened by rebars, and 6 specimens were a control without stiffeners. Finite element (FE) with a software ABAQUS was used to present the numerical simulation. Based on the experimental result, the failure mode, carrying capacity, and ductile behavior were analyzed and discussed. It was concluded that with increasing of flakiness ratio, local outward became bigger and the failure of shear force became more obvious. The use of stiffener gave the columns an advanced mechanical performance in terms of ductility, carrying capacity and confinement effect. So the deformation was smaller than that without stiffener. With increasing the concrete compressive strength, the ultimate load carrying capacity increase as well and a decrease of the ductility were happened. The ductility of the column specimens could be reduced by a higher flakiness ratio. The predicted result from the numerical modeling based on FE software ABAQUS agree well with the experimental result and the formula proposed can be used for the design of track-shaped in-filled steel tubular (SCFRT) stub columns under axial load.

Ekmekyapar [31] carried out a study on 18 specimens to investigate the influence of the laterally and longitudinally welding on the behavior of concrete-filled steel tube (CFST) columns. The key parameters used in this study were, the length of columns specimens (short, medium, and long columns), variation of D/t ratio, and the location of welding. The steel tubes were a circular cross section, cold manufactured, having a diameter of 101 mm, with three different L/D ratio, and two D/t ratio with an average concrete strength was 75.3 MPa. Three different configurations were considered for each group of specimens to investigate the lateral weld behavior, (1) un-welded specimens (seam welded but not laterally welded), (2) laterally welded specimens at mid-high location of column specimen, (3) lateral weld with location at one-third (L/3) of the length of the columns. All test results were compared with the AISC 360-10 specification and the Eurocode4 (EC4). It was concluded that for short column specimens, two specimens was medium length columns, and one long column specimens, the failure mode was a crushing of concrete core and yielding of steel tube. Furthermore, for other specimens, the failure mode was more complicated and involved both of local material and global column failure. The uses of high strength concrete increased the compression load bearing capacity of column specimens. The failure of columns with seam welded was by local material, can be presented in combination of possible weld imperfection and a high tensile circumference stresses. For different D/t ratio it was observed that the lateral weld joints might used for thicker and thinner in-filled steel tube. The different in lateral weld location of the specimen have no significant influence on load carrying capacity and the mode of failure of the column specimens.

2.3 Composite Columns with Reinforcement

Ge and Usami [32] conducted an experimental study on the strength of square CFST columns, and deformation as well under concentric compression load to study the influence of the internal stiffeners on the failure mode, and behavior of composite columns. The stiffeners were fabricated by mild steel of grade SS400 which have a yield strength of 235 MPa and were welded at the longitudinal direction in the mid-length of the steel tube as shown in Figure 2.1. A total of ten specimens were tested, the properties of the two CFST columns with internal steel stiffeners were, the width of the stiffener b_s was 38.25 mm and the counterpart thickness t_s were 4.36 and 4.34 mm. It was observed that the CFST columns with internal stiffeners have been achieved more ultimate strength compared to those of the unconfined CFST column specimens. The failure mode of the tested columns shown in Figure 2.2. For unstiffened columns, the buckling of the two opposite faces were outward and the two perpendicular faces buckles inward. In the case of stiffened steel columns, the buckling direction were almost the same in the unstiffened columns, except for the stiffened nodes. The conclusion was, the stiffeners were effective at improving the axial load capacity and sustaining the overall shape at failure.

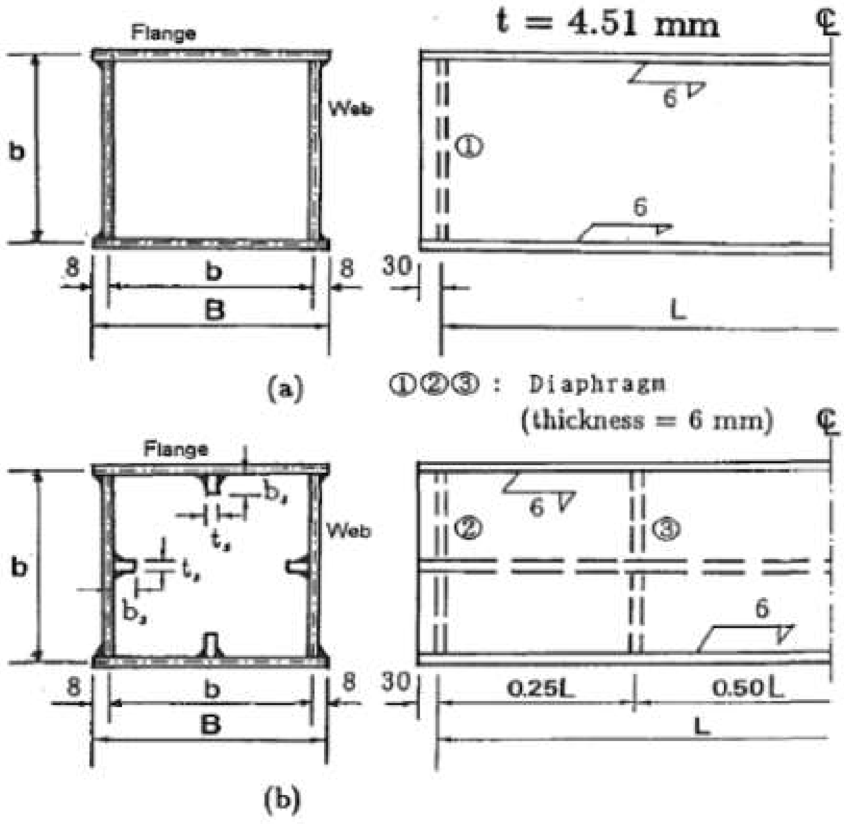


Figure 2.1 Square CFST specimens with and without stiffeners: (a) Unstiffened section; (b) Stiffened section [32]

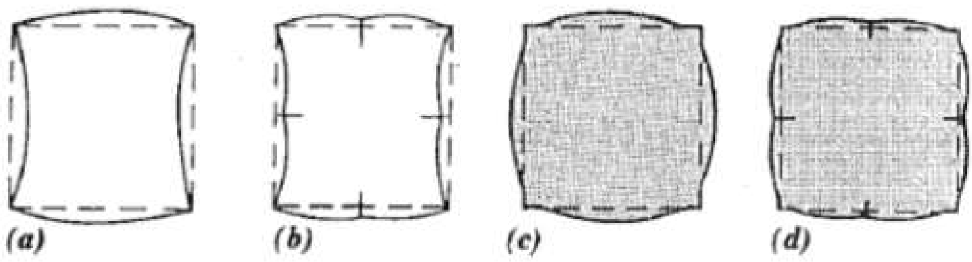


Figure 2.2 Failure modes of test specimens: (a) Steel column; (b) Stiffened steel column; (c) CFST column; (d) CFST columns with internal stiffener [32]

Claeson and Gylltoft [33] carried out tests on series to investigate the behavior of six slender columns filled with reinforced concrete under short-term loading. A concrete of 35 Mpa and 92 Mpa concrete strength was used, and the load eccentricity was 20 mm. The major parameters were the concrete and steel strains, compressive strength of concrete, cracking, mid height deformation, and loading rate. It was observed that the columns of high strength care concrete (HSC) under short-term load condition product less ductile behavior and had a chance for sudden failure more than the columns with a concrete core of normal compression strength. Additionally, the tests shows that, the structural behavior of HSC was preferable under providing load condition, the HSC column presented fewer tendencies to creep and absorbed the compression load with less deformation for longer period of time.

Chen et al. [34] conducted a series on steel reinforced concrete filled glass fiber reinforced polymer (GFRP) tube, and steel bar reinforced concrete filled glass fiber reinforced polymer columns under action of eccentric load. The outer diameter of GFRP tube was 200 mm with height of 700 mm, and the trial value of concrete compressive strength was 24.6 Mpa. The reinforcement was ϕ12 mm longitudinal bars, and the stirrup was ϕ6 @150 mm. The summary of the research was 4 reinforced concrete filled GFRP tubes under eccentric compression load, 1 steel reinforced concrete filled GFRP tube, and 1 GFRP tube filled with concrete only, all as composite columns. It was concluded that under eccentric load, the Glass fiber reinforced polymer can sustain different failure mode depending on the load eccentricity. The failure sign of composite column with a small eccentricity, was in GFRP tube at the compression area was crushed, in other hand, for the composite column with large eccentricity, the failure was in fiber of GFRP tube at the tensile area was ruptured. However, the confinement of concrete given by GFRP tube existed in the compressive zone, and for tensile zone only. The constraint to the concrete was not obvious. In tensile zone the loads carried by GFRP tube and steels only. The effect of the shaped steel that embedded inside the GFRP was a positive on the bearing capacity, and bending rigidity of the column specimen. Using the limit equilibrium theory, a bearing capacity calculation formula of GSRC under an eccentric load was created. The calculated results were agreed well with the experimental results.

Lai and Ho [35] carried out a study on the behavior of tie bars confinement in-filled steel tube columns (CFST) subjected to uni-axial compression load. A total of twenty-four specimens was divided into four groups, depending on the variation of the sectional properties, and concrete grade were tested and examined. The specimen's outer diameter was 168.3 mm and the height was 330 mm for the all 24 specimens. The tie bar properties were 8 mm in diameter with a yield strength of 250 Mpa, and the spacing between ties were 5t, 10t, 12.5t, 15t, and 20t. The tie bars were manufactured to be a little bigger than the external diameter of the steel tube and perpendicular to each other so the nuts could be installed at the both ends of ties tighten them against the outer surface of the steel tubes without applying any initial pre-stressing to steel tube of the columns. The concrete cylinder compressive strength was 30 Mpa on average at testing date. The test result showed that the using of tie bars was good to decrease the lateral deformation of concrete core, and steel tube at a tie caps location, and the axial load-carrying capacity of the columns was increased (5% in average and 16% maximum) and the strength degradation rate of CFST columns was decreased. The tie bars were not much effective in enhancing the elastic stiffness of the CFST columns specimens.

Xu et al. [36] conducted an experimental study on thin-walled concrete-filled steel tube columns with reinforced lattice angles, under a compression load. The lattice angles were designed to reinforce the CFST column by rising the steel cross-sectional area percentage amount. The tested tubes were having the length of 500 mm, 1500 mm, 2500 mm, and 3500 mm, and the same columns size was a control specimens without lattice angle reinforcement. A comparison was carried out between test result and design strength calculated by the Eurocode, and AISC Specification formulas for composite columns. From the load-displacement curves and load-strain curves, it was observed that the strength of in-filled steel tube could be efficiently enhanced by reinforcing lattice angle. In addition, the concrete core resistance was enhanced by using the reinforced lattice angle against the diagonal crack. For the ductility and axial stiffness of column specimens, the effect of the reinforced lattice angle was small and not obvious. The results predicted by the AISC design specification and Eurocode by comparing with that of the tested specimen it showed that: for AISC standard it was slightly unconservative without the consideration of lattice angles; for the Eurocode, it was totally unconservative. A design method was proposed to estimate with reasonable accuracy the strength of concrete-filled steel tubes reinforced with lattice angle.

Brown et al. [37] made a test on seven specimens with reinforced concrete-filled steel tube (RCFST) composite structure as a piles, to investigate the seismic behavior of the piles under reversed cyclic four-point bending with constant moment region centered in the pile, with focusing on variation of diameter-to-thickness ratio. The geometric characteristic of pip-piles were 20 to 24 inches in diameter, with D/t ratios between 33, and 192. It was identified by equations based on the test result the effect of diameter-to-thickness ratio on local buckling of pile specimen wall and equivalent viscous damping. In additionally, it was concluded that diameter-to-thickness ratio had a deep effect on local buckling of the pipe, but had no effect on the maximum limit state as distinct by pipe fracture.

Cai et al. [38] carried out a study to examine the mechanical behavior and the failure mechanism of the steel-reinforced concrete filled steel tubular (SRCFST) columns under uniaxial compressive loading. Previous experimental results were used to compare the numerical results by using the software ABAQUS/ Standard solver based on the finite element modeling. 22 specimens on total were studied to explore the influence of section steel ratio, structural steel tube ratio, compressive strength of concrete, structural steel tube yield strength, and section steel yield strength, on the mechanical behaviors and ultimate resistance of the SRCFST. It was concluded that attributable to the existence of inner section steel, the carrying capacity of steel reinforced CFST columns was much bigger than that of CFST columns which have the same cross-section area. “Strength reserve” for whole column could be also provided by the inner section steel, which has a large influence on the fire resistance of the SRCFST columns. Due to the existence of out CFT columns, the inner section steel local buckling could be eliminated. The result of FE modeling for the 22 specimens and by considering all parameters mentioned above showed that the peak strength, and initial stiffness of concrete-filled steel tube columns was increased with increasing of all parameters. The capacities of the SRCFT were significantly underestimated by the Eurocode4 because of disregarding the strength increase of concrete core and structural steel section due to confinement effect. A new model was proposed to predict the carrying capacity of the SRCFST columns and verified with the experimental, and simulation results.

Lu et al. [39] conducted an investigation on the steel fiber reinforcement effect on short in-filled steel tube columns behavior. The experimental test was on plain concrete-filled steel tube, and steel fiber reinforced concrete-filled steel tube subjected to axial compression load. A total of thirty-six column specimens were examined with a different parameter, volume percentage of steel fiber 0%, 0.6%, 0.9%, and 12%; steel tube thickness 3mm, 4mm, and 5mm; and the concrete strength was various from 50 Mpa up to 70 Mpa. Ultimate load, failure mode, and load-shortening relationship were presented. Test results explored that the steel fiber reinforced concrete columns showed a faintly higher maximum load by comparison compared to plain concrete-filled steel tube columns. The composite action between steel tube and core concrete was improved by the addition of steel fiber, and a higher concrete strength enhancement have been gained. That enhancement increased with increasing thickness of steel tube and decreasing the compressive strength of concrete core. The local buckling was delayed by adding the steel fiber into concrete core, because the confinement of steel fiber provides shear-frictional resistance against sliding and allows to shear force to transfer through the crack pattern, and it had a little influence on the failure mode. Furthermore, adding steel fiber is more efficient and economic than increasing the thickness of steel tube to improve the ductility, and energy dissipation capacity of CFST columns with considering that to establish a ductile behavior, the needed steel fiber volume percentage is 0.9% at least. Proposed formulas was provided to predicate the carrying capacity of CFST columns, and the predictions agree well with experimental results of this study, and so a design expression was proposed to predicate the ductility with reasonable prediction.

Zhou et al. [40] investigated the behavior and design of the reinforced concrete-filled slender circular steel tube columns under an eccentric compression load, the external thin-walled steel tube was not continuous at the beam-column joint and so, the carrying load was transferred to the reinforced concrete only. A total of 60 specimens were tested considering the key parameter: tow different slenderness ratio 24 and 40; two D/t ratio of steel tube 133 and 160; two various load eccentricities 25 mm, and 50 mm; and two continuity condition of steel tube, continuous, and not continuous at mid-length of the columns. 8 specimens of 60 in total which the steel tube was continuous at mid-length; and 8 specimens which the steel tube was not connected with a girth gap at mid-length of column specimens. The ultimate load, failure mode, and deformation were examined and discussed. Finite element (FE) modeling was used to present the behavior of the circular tubed-reinforced-concrete columns under eccentric load to compare the results with that of experimental tests. It was observed that the tested reinforced concrete-filled slender circular steel tube columns presented a moderate ductility under eccentric load. The axial compression bearing capacity and initial stiffness decreased with increasing the slenderness ratio and eccentricity. The failure mode of the slender specimens was a global bending failure with a critical area at mid-length. The discontinuity of the steel tube caused a small effect (≈5% on average) on the carrying capacity of the columns at mid-length. Using the nonlinear finite element analysis, the results were in a good agreement with that of the experimental test. Based on the moment magnification method a simplified design expressions was proposed.

Ding et al. [41] conducted a 3D finite element (FE) modeling by ABAQUS software and a comparative study on the composite columns with square stirrups confined concrete. A total of 23 specimens was examined, 12 of the specimens in a scenario and the other 11 specimens in a different scenario in comparison of square stud-confined concrete-filled stub columns with traditional square concrete-filled stub columns. All modeled specimens were compared with that corresponding specimen of the experimental results. Apart of internal confinement, the steel tube of the square CFT stub columns showed that the internal studs have limited enhancement to maximum load carrying capacity, but it was effectively enhanced the ductility of the concrete-filled tubular columns. The local buckling of square steel tube was effectively reduced due to the confinements of square stirrup with cross ties, rhombus, and spiral stirrups. By considering a same steel ratio, the steel tube confinement to the core concrete was better than the confinement of increasing the steel tube wall thickness. Both the modeling and experimental results showed that, the ductility and maximum carrying capacity of composite columns with square stirrups confined concrete were much higher compared to square concrete filled stub columns. The finite element modeling by ABAQUS software results for the axially loaded square concrete filled tube stub columns presented a good agreement with that of experimental tests.

It is obvious from literature, that there are a few researches about the in-filled composite column with steel reinforced bars compared to the normal in-filled composite column with different parameters. Thus, there are a need to extend the researches about the uses and benefits of reinforcing bars with concrete filled composite column. Therefore, this study was carried out to increase the knowledge about using reinforcing bars and its effect on the concrete filled composite column behavior.

CHAPTER 3

DESIGN SPECIFICATIONS

3.1 General

The composite columns are the most effective compression members due to the higher load bearing capacity, good ductility, fire resistance (in the case of encased composite columns), framework cost saving (in the case of in-filled columns), and many other advantages in comparison with other compression members. For that it became important to determine the all composite characteristics under the official specification that providing a details and prediction results of composite columns with a good agreement, and giving a geometric limit and a reduction factor of the different circumferences situation of a composite columns for designing to avoid the engineering mistakes and constructions failure under different overall loads.

In early of 1970's the technology of concrete-filled steel tubular (CFST) columns was evaluated based on enough researches have been carried out to understand the complete behavior of the CFST columns. The composite columns resist the applied loads through the composite action of steel and concrete together acting as a composite structural member. In the past forty years, much wide research was carried in the field of concrete-filled steel tubular columns which are used as compression members in high rise buildings, piles, bridges, and offshore structure. Based on several experimental works with different geometric parameters, the different material used in presenting the composite columns (steel bar reinforcement, fiber reinforced steel tube, height compressive strength concrete, etc.), and different load conditions many different specifications have been obtained to present the behavior of composite columns. Some specifications are conservative for a specific condition and un-conservative in other condition and some other conversely. In this chapter, some of the most important specification will be presented with the most effective parameters of the concrete-filled steel tube columns.

3.2 Eurocode 4

In composite structures, Eurocode 4 (EC4) [42] is the most promising international specifications. EC4 covered the concrete-filled columns section with reinforcement or without reinforcement and concrete-encased and partially encased steel sections. EC4 applies to the design to comply with the principles and requirements for the safety and serviceability of the structure with composite members. Thermal or sound insulation is not considered in EC4. Only resistance, durability, serviceability, and fire resistance of composite structure are considered.

3.2.1 Limitations

The limitation of the reinforced if included in design calculation is to not exceed 4% of cross-sectional area of concrete core, without minimum limitation. In case of reinforcement was included in the load carrying capacity, the minimum reinforcement required will be 3% of the concrete area.

The steel contribution ratio δ can be calculated as:

$$\delta = \frac{A_a\, f_{yd}}{N_{pl,Rd}} \tag{3.1}$$

By calculating the value of δ from the equation above, it has to be: $0.2 \leq \delta \leq 0.9$, to consider the composite column as "composite". If the contribution ratio δ was lower than 0.2, the column will be considered as a concrete column and the design will be according to Eurocode 2. And if the value of δ was more than 0.9, the column will be considered as a steel column and the design will be according to Eurocode 3. For the geometric limitation, Figure 3.1 shows the notation of the concrete filled circular section.

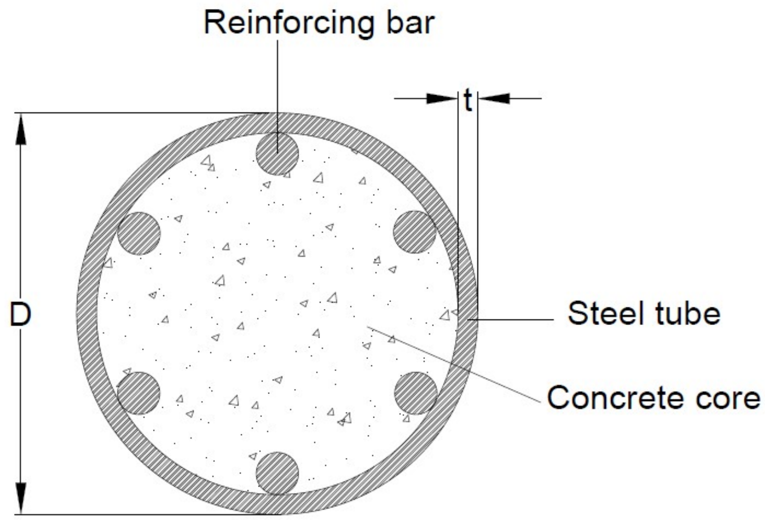


Figure 3.1 CFST circular column with notation

The limitation is: d/t $\leq$ 90 ε^2

The factor ε depends on the structural steel yield strength as follow:

$$\varepsilon = \sqrt{\frac{235}{f_y}} \tag{3.2}$$

With f_y in N/mm^2

3.2.2 Simplified Method of Design

This method presented for members of doubly symmetrical and uniform cross-section with welded or cold-formed, and rolled sections. When the structural steel was made by two or more unconnected sections, the simplified method is not applicable.

$$\frac{N_{Ed}}{\chi\, N_{Pl,Rd}} \leq 1.0 \tag{3.3}$$

Where:

$N_{Pl,Rd}$ is the resistance of the composite section against normal force according to Equation 3.2.

N_{Ed} is the design normal force value.

χ is the reduction factor for the relevant buckling mode given in EN 1993-1-1, 6.3.1.2 in terms of the relative slenderness $\bar{\lambda}$.

$$N_{\mathrm{pl,Rd}} = A_{\mathrm{s}} f_{\mathrm{yd}} + A_{\mathrm{c}} f_{\mathrm{cd}} + A_{\mathrm{s}} f_{\mathrm{sd}} \tag{3.4}$$

Where:

A_a, A_c, and A_s are the cross-sectional areas of the structural steel, concrete, and reinforcement.

f_{yd}, f_{cd}, and f_{sd} are the design strength of structural steel, concrete, and reinforcement, which can be calculated from the equations below:

$$f_{\mathrm{yd}} = \frac{f_{\mathrm{y}}}{\gamma_{\mathrm{a}}} \tag{3.5}$$

$$f_{\mathrm{cd}} = \frac{f_{\mathrm{ck}}}{\gamma_{\mathrm{c}}} \tag{3.6}$$

$$f_{\mathrm{sd}} = \frac{f_{\mathrm{sk}}}{\gamma_{\mathrm{s}}} \tag{3.7}$$

For concrete-filled steel tube columns, an increase in strength of concrete caused by the confinement of the steel tube may be taken into account with $\bar{\lambda} < 0.5$ and $e/d < 0.1$, where e is the load eccentricity, and d is the outer diameter of the column. However, the plastic resistance can be evaluated from the following formula:

$$N_{Pl,Rd} = \eta_a A_a f_{yd} + A_c f_{cd} \left(1 + \eta_c \frac{t}{d} \frac{f_y}{f_c}\right) + A_a f_{sd} \tag{3.8}$$

Where:

t is the steel tube wall thickness.

For columns with $e = 0$, the value of η_a, and η_c are given below:

$$\eta_a = \eta_{ao} = 0.25\,(3 + 2\,\bar{\lambda}) \qquad \text{(but } \leq 1.0\text{)} \tag{3.9}$$

$$\eta_c = \eta_{co} = 4.9 - 18.5\,\bar{\lambda} + 17\,\bar{\lambda}^2 \qquad \text{(but } \geq 0\text{)} \tag{3.10}$$

For columns in combined action of compression and bending effect with ($0 < e/d \leq 0.1$), the value of η_a, and η_c have to be calculated from Equations 3.11-12, where η_{ao} and η_{co} are given by Equations 3.7-8.

$$\eta_a = \eta_{ao} + (1- \eta_{ao})\,(10\, e/d) \tag{3.11}$$

$$\eta_c = \eta_{co}\,(1- 10\, e/d) \tag{3.12}$$

For $e/d > 0.1$, $\eta_a = 1.0$ and $\eta_c = 0$.

The reduction factor (χ) is determined for the relative slenderness $\bar{\lambda}$:

$$\bar{\lambda} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}} \tag{3.13}$$

Where:

$N_{pl.Rk}$ is the section resistance to the axial loads $N_{Pl,Rd}$ according to (3.6). However, with $\gamma_a = \gamma_\circ = \gamma_s = 1.0$.

N_{cr} is the load of elastic buckling (Euler critical load).

$$N_{Cr} = \frac{(EI).\pi^2}{\ell^2} \tag{3.14}$$

Where:

ℓ is the buckling length of the column.

EI is the section's effective stiffness.

Column length could be considered as the buckling length of the columns in the non-sway system, as a safe approximation. The effective flexural stiffness $(EI)_{eff}$ of a cross section of the composite columns should be calculated from Eq. (3.15), to determine the elastic critical force N_{cr} and relative slenderness ratio $\bar{\lambda}$.

$$(EI)_{eff} = E_a I_a + E_s I_s + \ 0.6\, E_{cm} I_{cm} \tag{3.15}$$

Where: I_a, I_c, and I_s are the second moment of area of the structural steel section, the un-cracked concrete section, and the reinforcement.

By taking into account the effect of long-term on effective elastic flexural stiffness, The concrete modulus of elasticity (E_{cm}) must be minimized to the value of $E_{c,eff}$ by the following expression:

$$E_{c,eff} = E_{cm} \frac{1}{1 + \left(N_{G,Ed}/N_{Ed}\right)\varphi_t} \tag{3.16}$$

Where:

φ_t is the creep coefficient.

N_{Ed} is the total design normal force.

$N_{G,Ed}$ is the part of the normal force that is permanent

3.3 ANSI/AISC 360-10

This is the American national standard for structural steel building from the American institute of steel construction [43], it is based on the past successful research and usage. Advances in the state of knowledge, and design practice changes, it provides two treatment of the design, the allowable stress design (ASD), and the load and resistance factor design (LRFD). And provide two methods to predicate the nominal strength of composite section which they are, plastic stress distribution method, and strain compatibility method.

3.3.1 Plastic Stress Distribution Method

In determining the nominal strength of the rounded hollow section filled with concrete by this method, it should be assumed that, the steel component has reached the yield stress (f_y) in tension or compression under axial force and/or flexure. And for the concrete in-filled, assuming that the stress reached a value of $0.95f_c$ under the same condition to account for the effect of concrete confinement.

3.3.2 Strain Compatibility Method

Across the column section, a linear strain distribution shall be assumed in this method. And the maximum compressive strain of concrete shall assume to be equal to 0.003 mm/mm. The stress-strain curve for concrete and steel have to be determined from tests for the similar materials.

3.3.3 Material Limitations

In American national standard, the composite component, structural steel, concrete, and reinforcing bars shall be met with the following limitation, just if justified by testing or analysis:

- In evaluating the available strength, the minimum compressive strength of concrete f_c of normal weight concrete have to be 21 MPa and 70 MPa in maximum. And 21 MPa to 42 MPa for lightweight concrete.
- The yield strength of structural steel and reinforcing bars shall not exceed 525 MPa.
- The minimum cross sectional area of steel section has to be 1% of the total composite cross section.
- The width-to-thickness ratio for members under axial compression and flexure are shown in Tables 3.1.

Table 3.1 Limitation D/t ratio for the compression steel element [43]

Description of Element	λ_p Compact/ Noncompact	λ_r Noncompact Slender	Maximum Permitted
Round HSS subjected to axial compression	$\frac{0.15\,E}{f_y}$	$\frac{0.19\,E}{f_y}$	$\frac{0.19\,E}{f_y}$
Round HSS subjected to flexure	$\frac{0.09\,E}{f_y}$	$\frac{0.31\,E}{f_y}$	$\frac{0.31\,E}{f_y}$

3.3.4 Compressive Strength

For the axially loaded doubly symmetric concrete filled composite columns, the available compressive strength has to be evaluated for the limit state of flexural buckling.

ϕ_c = 0.75 (LRFD) Ω_c = 2.00 (ASD)

Where:

ϕ_c is the resistance factor for composite column under axial load.

Ω_c is the safety factor for axially loaded composite columns.

a) For compact section:

$$P_{no} = P_p$$

Where:

$$P_p = F_y A_s + C_2 \grave{f}_c \left(A_c + A_{sr} \frac{E_s}{E_c} \right) \tag{3.17}$$

Where:

C_2 = 0.95 for round section.

E_s, E_c are the modulus of elasticity of steel, and concrete respectively.

A_s, A_c, and A_{sr} are the cross sectional area of steel section, concrete, and continuous reinforcing bars

b) For Noncompact section:

$$P_{no} = P_P - \frac{P_P - P_y}{(\lambda_r - \lambda_P)^2} (\lambda - \lambda^2) \tag{3.18}$$

Where:

λ, λ_p, and λ_r are the slenderness ratios determined from Table 3.1.

P_p is determined from Equation 3.15.

$$P_y = f_y A_s + 0.7 \grave{f}_c \left(A_c + A_{sr} \frac{E_s}{E_c} \right) \tag{3.19}$$

c) For slender section:

$$P_{no} = F_{cr} A_s + 0.7 \grave{f}_c \left(A_c + A_{sr} \frac{E_s}{E_c} \right) \tag{3.20}$$

Where for rounded, filled section:

$$F_{cr} = \frac{0.72 F_y}{\left(\left(\frac{D}{t} \right) \frac{F_y}{E_s} \right)^{0.2}} \tag{3.21}$$

The effective stiffness are calculated from the expression below:

$$(EI)_{eff} = E_s I_s + E_{sr} I_{sr} + C_3 E_c I_c \quad (3.22)$$

Where:

I_s, I_{sr} and I_c are the moments of inertia of steel shape, reinforcing bars, and the concrete section respectively.

C_3 is the effective rigidity coefficient.

$$C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.9 \quad (3.23)$$

The tensile strength of in-filled composite members under axial load can be determined from the equation below:

$$P_n = A_s f_y + A_{sr} f_{ysr} \quad (3.24)$$

$\phi_t = 0.90$ (LRFD) $\Omega t = 1.67$ (ASD)

Where:

f_y, f_{ysr} is the minimum yield strength of the structural steel, and reinforcing bars respectively.

ϕ_t, is the resistance factor of tension.

Ωt is the safety factor of tension.

CHAPTER 4

EXPERIMENTAL WORK

4.1 Material Properties

The properties of materials that used with in-filled steel tube columns have a major influence on the structural behavior and failure mode of the composite column specimens. However, the properties of structural steel, reinforcing steel, and concrete with its component were carefully examined to be maintained within the limit of Eurocode 4 [42], American National Standard for Structural Steel Building (ANSI/AISC 360-10) [43], and American Society for Testing and Materials (ASTM) [44].

4.1.1 Structural Steel

Two different steel wall thicknesses were used for CFST column specimens in this study. The first one was with an average thickness of 5.63 mm and the yield strength was 410 MPa in average. The other one was with an average thickness of 3.15 mm and the yield strength was 460 MPa in average, and so, the behavior of columns will be examined with different properties of compressive capacity, ductility and toughness. Also, the concrete core behavior will be examined by calculating the strength index (SI), which Shown in Equation 4.1, as this equation has been used by several researchers to measure the composite action and to compare the behavior of the column specimens [45-47].

$$SI = \frac{N_u}{A_s f_y + 0.85 A_c f_c + A_{sr} f_{sr}} \tag{4.1}$$

Where:

A_s, A_c, and A_{sr} are the cross sectional area of the steel tube, core concrete, and reinforcing bars respectively.

f_y, f_c, and f_{sr} are the yield strength of steel tube and the compressive strength of concrete respectively.

4.1.2 Reinforcing Steel Bars

Two diameters of reinforcing steel were used, ϕ8 mm with a yield strength of 534.20 MPa, and ϕ 12 mm with a yield strength of 470.51 MPa, and a stirrups were made by 5.5 mm thickness reinforcing bars which have a 520 MPa yield strength.

4.1.3 Concrete

A concrete mix of grade (SC50M) was used in this research to sustain an approximate compressive strength of 50 ± 2 MPa in average at the specimens test day, four molds for each casting process have been taken and filled by the same concrete batch that used in casting the column specimens..

4.1.3.1 Cement

Turkish ordinary Portland cement (limak cement, Type II) manufactured in the city of Gaziantep according to SR EN 197-1:2002, the main constituents are Portland clinker (K) (80-94%) and other components (6-20%). This type of cement has an advantage of moderate sulfate resistance due to relatively low C_3A content (≤ 8%), in reason this type has wide uses in the construction process.

4.1.3.2 Coarse Aggregate

River coarse aggregate rounded shape was used in concrete mix. Taking into consideration the diameter of the structural steel 114.3 mm and the distance between the longitudinal reinforcing steel and the structural steel's wall 16 mm, the coarse aggregate was sieved by 10 mm sieve to obtain the perfectibility of material distribution inside the tube and prevent any segregation between the mix constituent.

4.1.3.3 Fine Aggregate (Sand)

Natural rounded-shape particles and smooth texture sand were used for the concrete mix in this study.

4.1.3.4 Crushed Aggregate

A crushed aggregate made by crushing a natural rock to two different grading were used in concrete mix. The first grade was 2 – 0.3 mm, while the other was ≤ 0.3 mm.

4.1.3.5 Fly Ash

Type F fly ash was used in concrete mix. The fly ash properties showed in Table 4.1.

Table 4.1 Chemical composition and physical properties of Fly Ash (FA)

Item	FA
CaO (%)	2.24
SiO_2 (%)	57.5
Al_2O_3 (%)	24.4
Fe_2O_3 (%)	7.1
MgO (%)	2.4
SO_3 (%)	0.29
K_2O (%)	3.37
Na_2O (%)	0.38
Loss on ignition (%)	1.52
Specific gravity	2.04
Blaine Fineness (m^2/Kg)	379
Surface-volume ratio (m^2/g)	-
Average primary particle size (nm)	-

4.1.3.6 Water-Reducing Admixture

Master Glenium 51 is a second generation superplasticizer concrete admixture and a high range water reducing and by that it gives a high early and final compressive and flexural strength and durability of the concrete construction. It is used in the production of self-compacting and self-consolidation concrete, and in improving the wear resistance of concrete by reducing the segregation and bleeding. Table 4.2 shows some of the Glenium properties.

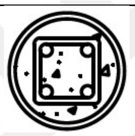
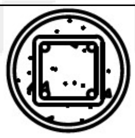
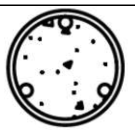
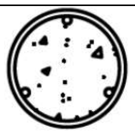
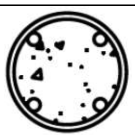
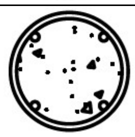
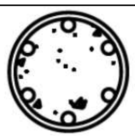
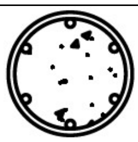
Table 4.2 Water-Reducer admixture properties

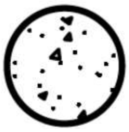
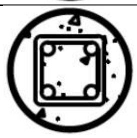
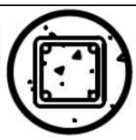
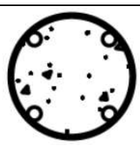
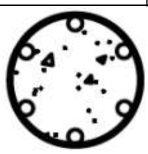
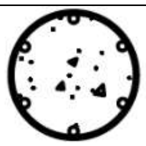
Structure of a material	Polycarboxylic ether based
Color	Amber
Density	1.082 – 1.142 kg/liter
Chlorine Content% (EN 480-10)	< 0.1
Alkaline content% (EN 480-12)	< 3

4.2 Specimens Manufacturing

All columns specimens were 114.3 mm diameter and manufactured to 265 mm length steel tubes. Two different tube thicknesses (5.63 mm, and 3.15 mm) were purchased to establish two different (D/t) ratio. Two steel reinforcement bar's diameter (ϕ8 mm, and ϕ 12 mm) were used with a different number of bars for each specimen as a longitudinal reinforcement to specify different steel reinforcement ratio. The longitudinal reinforcements were applied within the concrete core by two methods, by using the naming system as shown in Table 4.3. In group A, the longitudinal reinforcement bars were welded to the inner surface of the steel tube with 60 mm distance between welding points; while in group B, the longitudinal reinforcement bars were centered in the middle of the steel tube, and a stirrup (ϕ 5.5 @ 60 mm) was used with a distance between the steel tube's inner surface and the longitudinal reinforcement bar as 16 mm on average. This difference in specimens properties was selected to increase the knowledge about the reinforcing bars effects on the column specimen's behavior.

Table 4.3 Properties of column specimens

Column Specimens	Section Shape	t (mm)	D/t	D of Rei. Bar	# of Bar	A_s mm^2	A_c mm^2	A_{sr} mm^2	Method of using bars
Group A1									
20-Control		5.63	20.3	-	0	1922.0	8338.7	0	-
20-3-12-S		5.63	20.3	12	3	1922.0	7999.4	339.3	Stirrups
20-3-8-S		5.63	20.3	8	3	1922.0	8187.9	150.8	Stirrups
20-4-12-S		5.63	20.3	12	4	1922.0	7886.3	452.4	Stirrups
20-4-8-S		5.63	20.3	8	4	1922.0	8137.6	201.1	Stirrups
Group A2									
20-3-12-W		5.63	20.3	12	3	1922.0	7999.4	339.3	Weld
20-3-8-W		5.63	20.3	8	3	1922.0	8187.9	150.8	Weld
20-4-12-W		5.63	20.3	12	4	1922.0	7886.3	452.4	Weld
20-4-8-W		5.63	20.3	8	4	1922.0	8137.6	201.1	Weld
20-6-12-W		5.63	20.3	12	6	1922.0	7660.1	678.6	Weld
20-6-8-W		5.63	20.3	8	6	1922.0	8037.1	301.6	Weld

Group B1										
36-Control		3.15	36.28	-	0	1099.9	9160.8	0	-	
36-3-12-S		3.15	36.28	12	3	1099.9	8821.6	339.3	Stirrups	
36-3-8-S		3.15	36.28	8	3	1099.9	9010.0	150.8	Stirrups	
36-4-12-S		3.15	36.28	12	4	1099.9	8708.5	452.4	Stirrups	
36-4-8-S		3.15	36.28	8	4	1099.9	8959.8	201.1	Stirrups	
Group B2										
36-3-12-W		3.15	36.28	12	3	1099.9	8821.6	339.3	Weld	
36-3-8-W		3.15	36.28	8	3	1099.9	9010.0	150.8	Weld	
36-4-12-W		3.15	36.28	12	4	1099.9	8708.5	452.4	Weld	
36-4-8-W		3.15	36.28	8	4	1099.9	8959.8	201.1	Weld	
36-6-12-W		3.15	36.28	12	6	1099.9	8482.3	678.6	Weld	
36-6-8-W		3.15	36.28	8	6	1099.9	8859.3	301.6	Weld	

For the first group, the longitudinal reinforcements were cut to the same length of steel tube before machining process, and welded to the inner surface of the steel tube by SEM weld at a distance of 60 mm between the weld points, so, both steel and reinforcement bars will be machined together to ensure the flat surface of the column specimens in final stage. As well as, for the second group, the longitudinal reinforcements were cut (2-3 mm) less than the steel tube to purchase a flat surface after casting and ensure that the reinforcement will not exceed the steel tube length while vibrating process during casting. And so, two different shapes of stirrups, rectangular stirrups (65 x 65 mm), and equilateral triangle stirrups with 75 mm leg length were prepared as shown in Figure 4.1.

Figure 4.1 Rectangular and equilateral triangle steel bar reinforcement

All the steel tubes were manufactured for the desired length and the ends of the steel tube and reinforcement bars were machined to ensure the maximum flatness of the column's ends and maximum uniformity of contact with the loading heads of the testing machine. Figures 4.2 and 4.3 shows the machining process of the steel tube. Thick mica plates were used at the bottom ends of columns to restrict the fresh concrete. Silicon was used to connect the bottom of steel tube with mica plate without any bond between the mica plate and concrete. So, after removing the mica plate a flat surface will be obtained easily.

a) Machining process b) Specimens after machining process

Figure 4.2 Machining process for welded reinforcement specimens

a) Machining process b) Specimens after machining process

Figure 4.3 Machining process for stirrups reinforcement specimens

4.3 Casting Procedure

A total of twenty-two specimens were casted, two of them were a control specimens. The inner surfaces of all steel tubes were cleaned from dust several times to ensure obtaining maximum bond strength between the steel tube and concrete core. All materials of concrete mix were weighted according to the mix design and the steel tubes were prepared with reinforcement bars. The coarse aggregate was put in the mixer and cleaned again by air to minimize the dust as possible, after 2 minutes of cleaning, the normal sand was added and mixed with the coarse aggregate for 3 minutes, then the crushed aggregate with a particle size of (2 - 0.3 mm) were added and mixed for 3 min, after that the crushed aggregate with a particle size of (≤0.3 mm) were added, after 3 minutes of mixing the cement then the fly ash were added and mixed a little for each one. The mix's water was weighted and divided for two equal amounts and the superplasticizer (Glenium 51) was added to one amount and motivated well till it comminuted completely with water. After adding the water with superplasticizer, all the material were mixed for 5 minutes.

In the casting process, the specimens were divided into two groups as thick and thin tube specimens and each group had 11 specimens to make the casting easier and more accurate. Each specimen was filled with concrete in three layers and a vibration process were activated after each layer. All the specimens were filled with concrete to 3-5 mm less than the steel tube length to give a sufficient space for epoxy and for the curing process. For each group casting, 4 cylinders (100 × 200 mm) concrete compressive specimens were cast from concrete batch to evaluate the compressive strength of concrete. The specimens were cured daily for (28 days) after casting by adding water in the space of (3-5 mm) in the top of the columns as shown in Figure 4.4. After 28 days, the curing stopped and the specimens were dried and cleaned.

Figure 4.4 Curing process of the specimens

After accomplishing a flat surface at the bottom by using the mica plate, a high strength leveling epoxy was used to level the top of the columns to accomplish the goal of flat surface at the two ends of the columns to ensure a maximum uniformity of contact with the loading heads of the test machine, as shown in Figure 4.5.

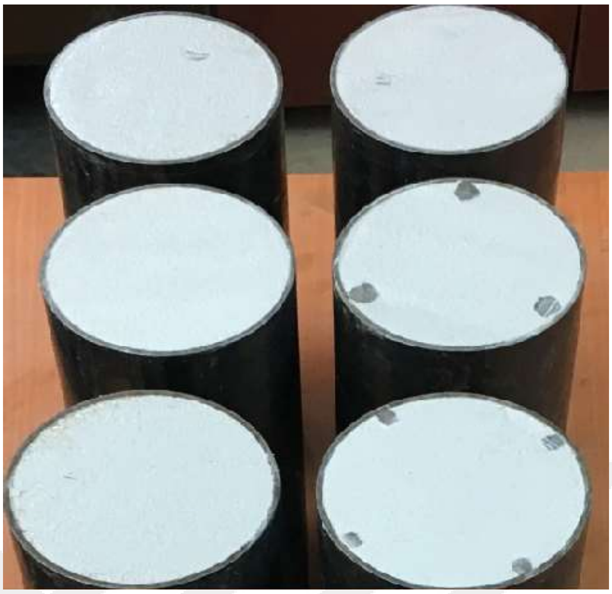
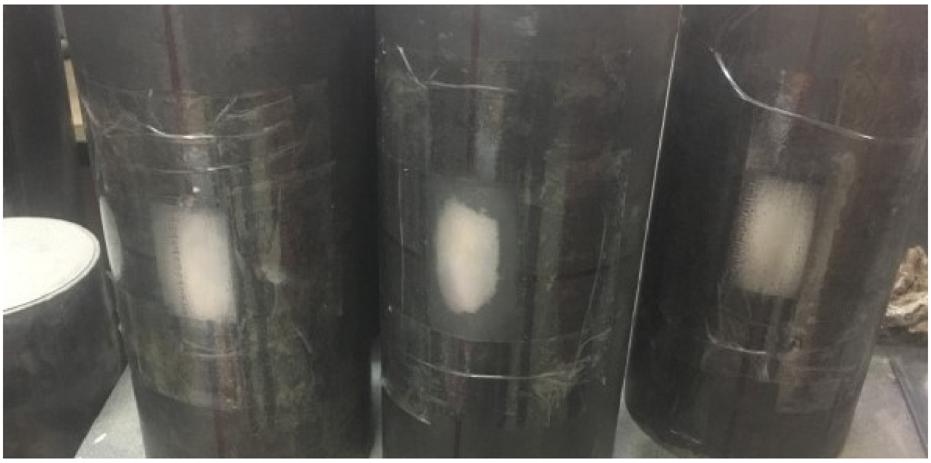


Figure 4.5 Sample of specimens after epoxy process

4.4 Instrumentation

The following types of instrumentations were used to record data of specimens during compression loading to monitoring the behavior of column specimens.

- A strain gages with 2% strain capacity, were affixed by strong glue on each column on the opposite side of the seam weld line of steel tube in the middle length of the columns and if the affix area come on the welded bar's area, the strain gage was affixed between the two welded bars, before affixing the strain gage, the affix place was cleaned by an abrasive paper with strong alcohol. The strain gauges were affixed in the longitudinal direction of column specimens to measure the alternative shortening of columns with the corresponding compressive load. Figure 4.6 shows the strain gages affix process. After this step, all the specimens were ready to test.



a) Cleaning of the strain gage affix area

b) Specimens after affixing strain gages

Figure 4.6 Strain gage affix process

- A linear variable differential transducer (LVDT), two LVDT was used in the test to record the axial deformation of the specimens under eccentric compressive load, the average of the two LVDT will obtain to draw the load-shortening curves. The arrangements of the specimens with LVDT and test device are shown in Figure 4.7.

4.5 Test Procedure

All column specimens were prepared for the test after strain gage affixing process. LVDTs were installed to the test device. Figure 4.7 shows the arrangement of specimens with the test device. The axial load was applied with a displacement rate of 1 mm/min, the applied force with the corresponding deformation was recorded as well, the strain from strain gage and the shortening from LVDTs. Each group of cylinder concrete compressive specimens was tested on the same day of column specimens test to evaluate the compressive strength of the concrete core. Load-shortening curve and load-strain curve will be drawn for each specimen to make a comparison between the specimens with stirrups and the specimens with welded steel bars, this comparison will give us a total image of columns behavior under axial compression and the effect of the steel bar reinforcement on compressive strength, ductility, toughness. Furthermore, to know the best way of using the steel bar reinforcement with its maximum benefits in composite columns.

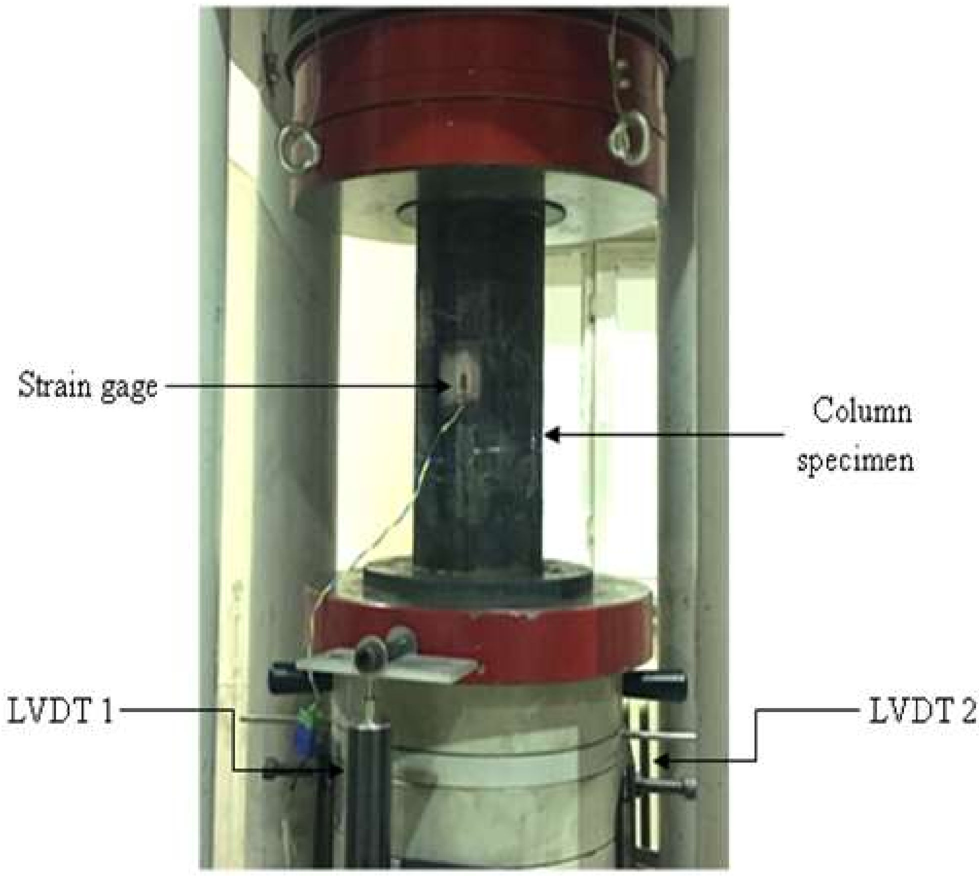


Figure 4.7 Column test

CHAPTER 5

TEST RESULTS AND DISCUSSION

5.1 General

Twenty-two specimens were tested under axial compression force and all data were recorded. The specimens were divided into two group, according to steel tube thickness, and also, each group were divided into another two group, according to the method of using reinforcing bars in column specimens (welded or with stirrups) to simplified the comparison and to conduct a detailed knowledge about the column specimens behavior with different parameters and the influence of reinforcing bars on the compressive strength capacity, ductility, toughness and failure mode of the column specimens.

5.2 Compressive Load Capacity

The short columns with 265 mm length considered as stub columns and the typical failure mechanism of these columns were crushing of concrete core and yielding of steel. In this section, the maximum compressive strength capacity will be investigated with its corresponding deformation capacity of the column specimens. All specimens of welded reinforcing bars had opposite specimens of reinforcing bars with stirrups with the same parameters to compare with, only the specimens with 6 welded reinforcing bars have been added to increase the knowledge of its behavior.

5.2.1 Thinner Steel Tube Column Specimens

In this section, only thinner steel tube column specimens with a thickness of 3.15 mm with 460 MPa yield strength will be taken. A total of 11 specimens, 10 specimens were the reinforcing steel bars used in, and 1 specimen was a control specimen without any reinforcing bars. The cylinder concrete compressive specimens have been tested on the test day and gave a 49.46 MPa on average. For the specimens with the same number and diameter of reinforcing bars used, the comparison will be carried out between the specimens of welded bars against the specimens with stirrups depending on the load-shortening curves of selected specimens with control specimen. For the control specimen the maximum load with the corresponding shortening were 1063.5 kN and 4.86 mm respectively, all thinner steel tube column specimens with different parameter will be compared with the control specimens at first to evaluate the enhancement percentage of the column specimens. The load was taken from the test device's recorder and the shortening from the two LVDTs as average.

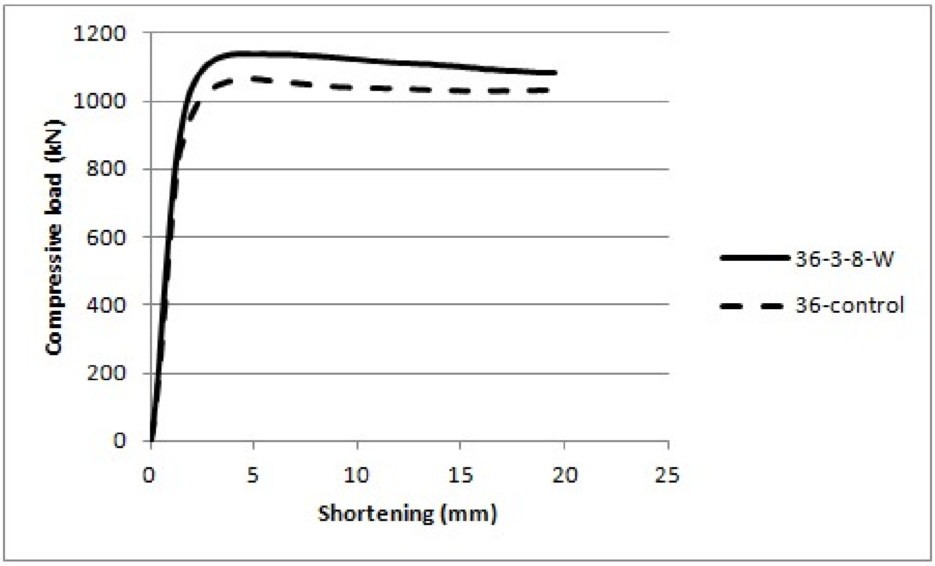


Figure 5.1 Load-shortening curve for 36-3-8-W specimen

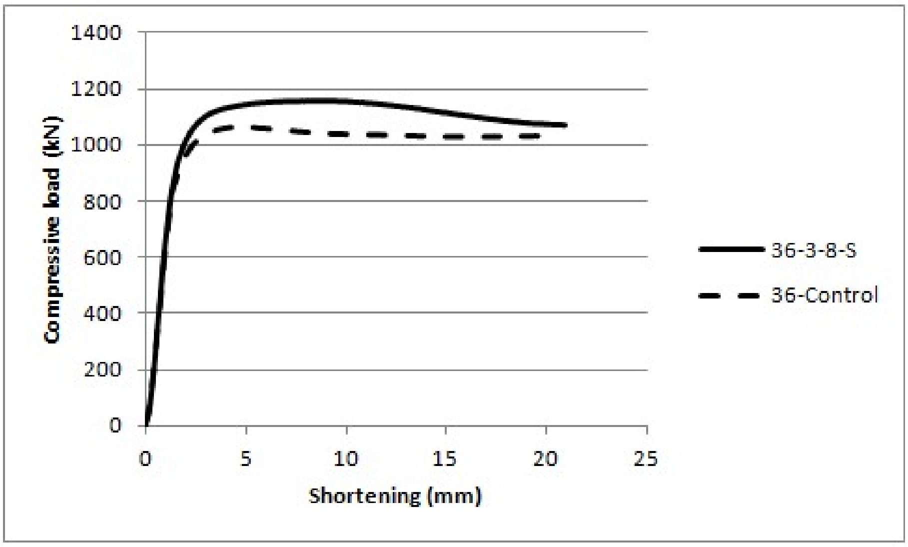


Figure 5.2 Load-shortening curve for 36-3-8-S specimen

The maximum loads for 36-3-8-W and 36-3-8-S specimens were 1140.825 kN and 1154.892 kN with corresponding shortening equal to 4.56 mm and 8.71 mm respectively. Specimen 36-3-8-W was enhanced in compressive load capacity by 7.3%. The welded reinforcing bars worked as a stiffener and gave a good resistance against the longitudinal deformation, in another hand, the specimen 36-3-8-R enhanced the compressive capacity by 8.6%, with increasing the shortening amount by 7.9%, and this increase in shortening is slightly high compared with the corresponding increase in compressive load capacity. Thought, from Figure 5.2, a good deformation capacity can be observed from the smoothness in the load-shortening curve due to the reinforcing bars with stirrups.

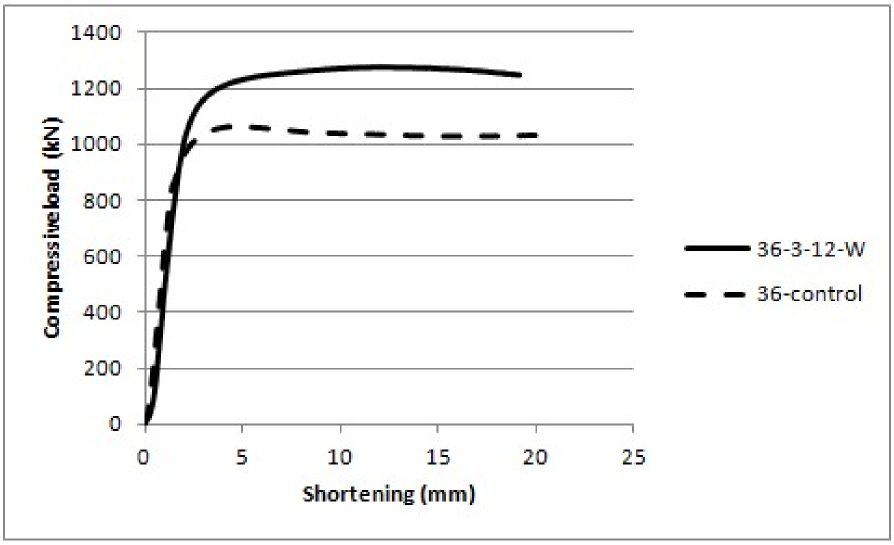


Figure 5.3 Load-shortening curve for 36-3-12-W specimen

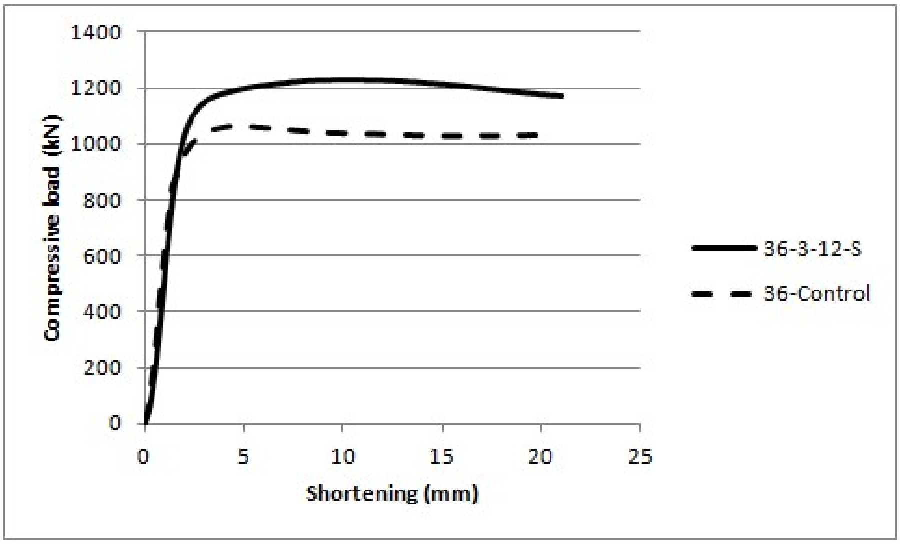


Figure 5.4 Load-shortening curve for 36-3-12-S specimen

The maximum loads for 36-3-12-W and 36-3-12-S specimens were 1278.906 kN and 1230.087 kN with corresponding shortening equal to 12.03 mm and 10.21 mm respectively. The compressive load capacity of the specimen 3-12-W enhanced by 20.25%, the load-shortening curve of this specimen is very smooth as shown in Figure 5.3 which implies a superior deformation capacity. For the 36-3-12-R specimen shown in Figure 5.4, the compressive load capacity increased by 15.67%. However, the load-shortening curve also was smooth and gave a good deformation capacity. For the same number and diameter of reinforcing bars (3 bars ϕ12 mm), the welded reinforcing specimens are better than the reinforced one with stirrups in compressive load capacity enhancement.

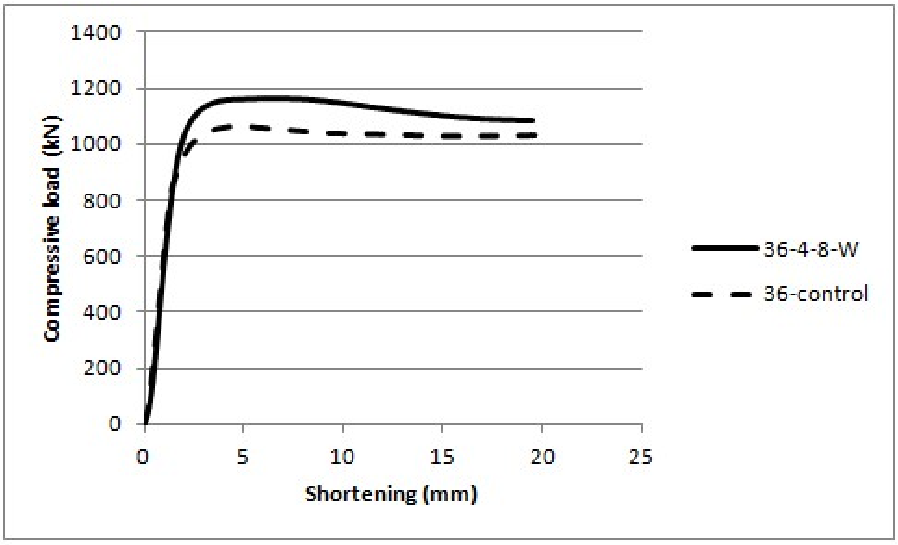


Figure 5.5 Load-shortening curve for 36-4-8-W specimen

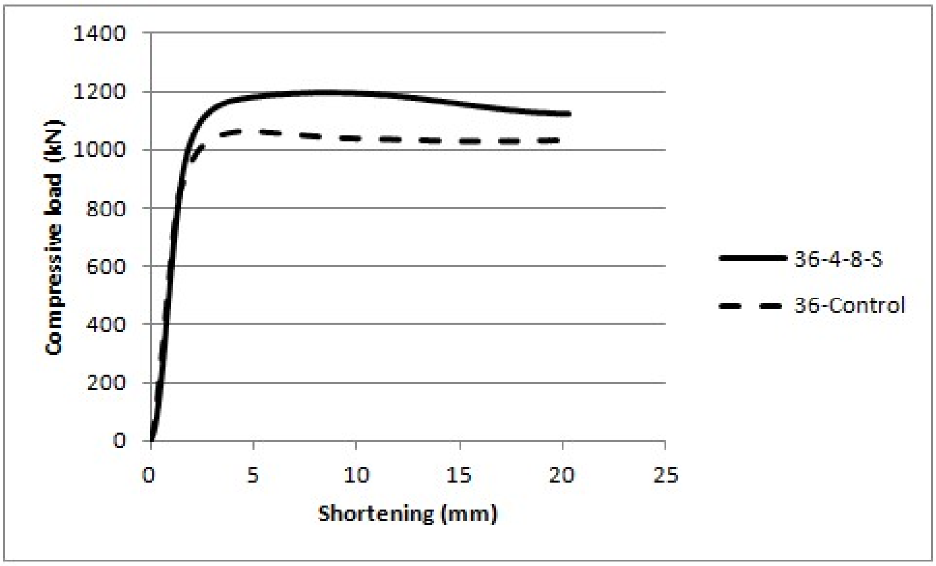


Figure 5.6 Load-shortening curve for 36-4-8-S specimen

The maximum loads for 36-4-8-W and 4-8-S specimens were 1161.975 kN and 1195.332 kN with corresponding shortening equal to 6.658 mm and 8.87 mm respectively. The compressive load capacity of the 36-4-8-W specimen increased by 9.26% due to the welded reinforcing bars with slightly good deformation capacity as shown in Figure 5.5. The specimen 36-4-8-S had a 12.4% enhancement in compressive load capacity due to the reinforcing bars with stirrups, but showed a higher deformation capacity than the specimen 36-4-8-W, as shown in Figure 5.6.

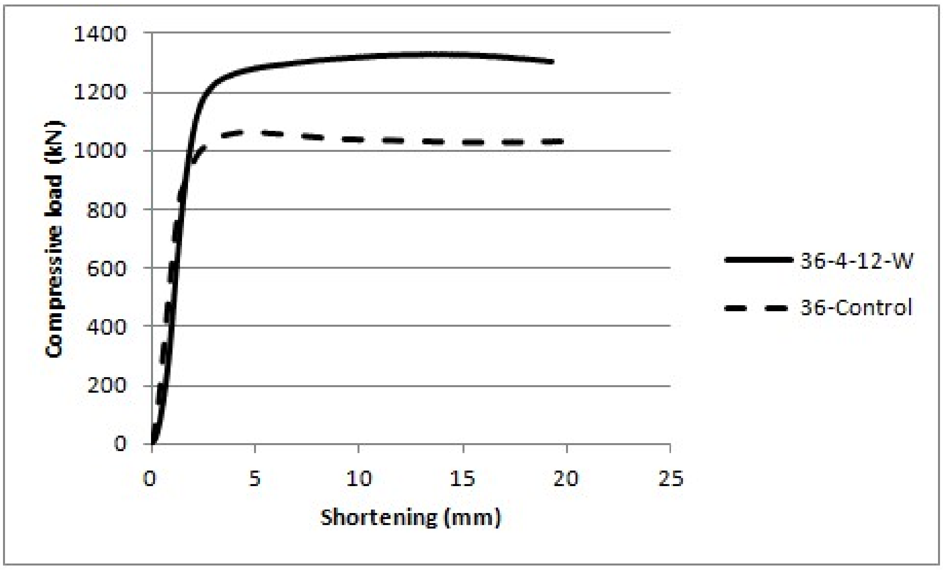


Figure 5.7 Load-shortening curve for 36-4-12-W specimen

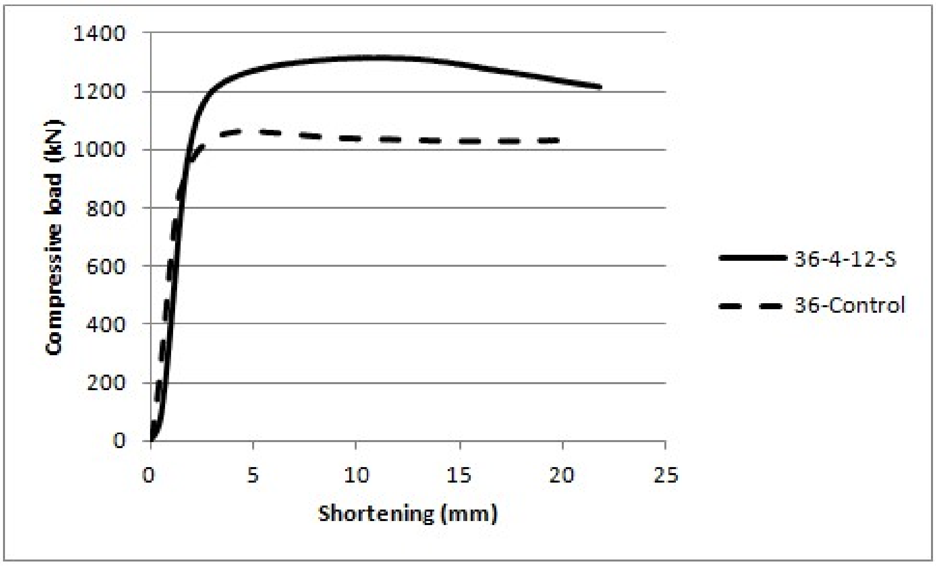


Figure 5.8 Load-shortening curve for 36-4-12-S specimen

The maximum loads for 36-4-12-W and 36-4-12-S specimens were 1329.681 kN and 1316.718 kN with corresponding shortening equal to 13.67 mm and 11.56 mm respectively. Specimen 36-4-12-W gained an increase in compressive load capacity equal to 25% with a high deformation capacity due to the welded reinforcing bars as shown in Figure 5.7, the specimen resists the increase in load by more deformation, which gave a smooth load-shortening curve better than the specimens 36-4-12-S who resist the increase in compressive load capacity 23% by less deformation capacity as observed in the load-shortening curve shown in Figure 5.8.

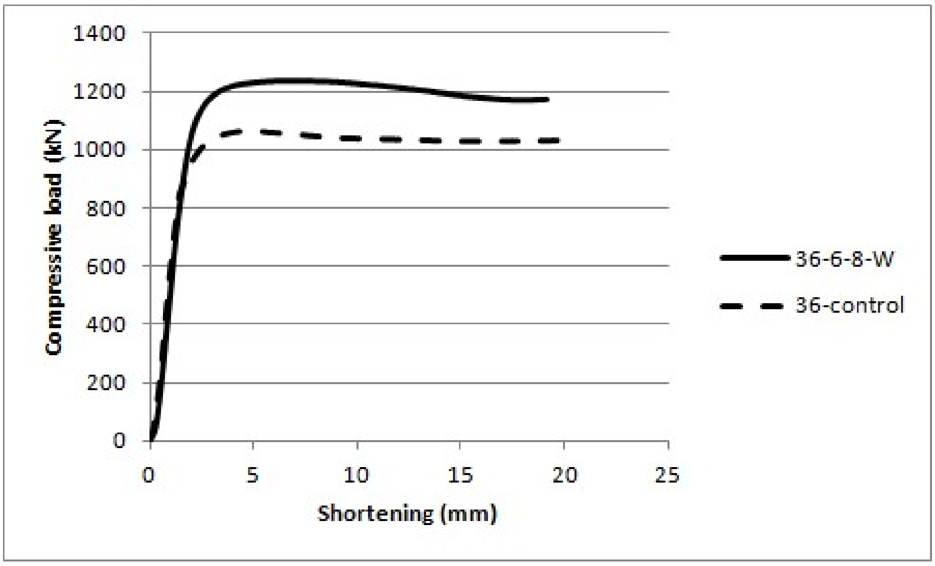


Figure 5.9 Load-shortening curve for 36-6-8-W specimen

This specimen showed an enhancement in compressive load capacity by 16.29% of 1236.77 kN with the corresponding shortening of 6.71 mm. This increase is more than the increase in compressive load capacity of the specimens with the same reinforcing bar's diameter (36-3-8-W, and 36-4-8-W). However, the deformation capacity also was more than the mentioned specimens as shown in Figure 5.9.

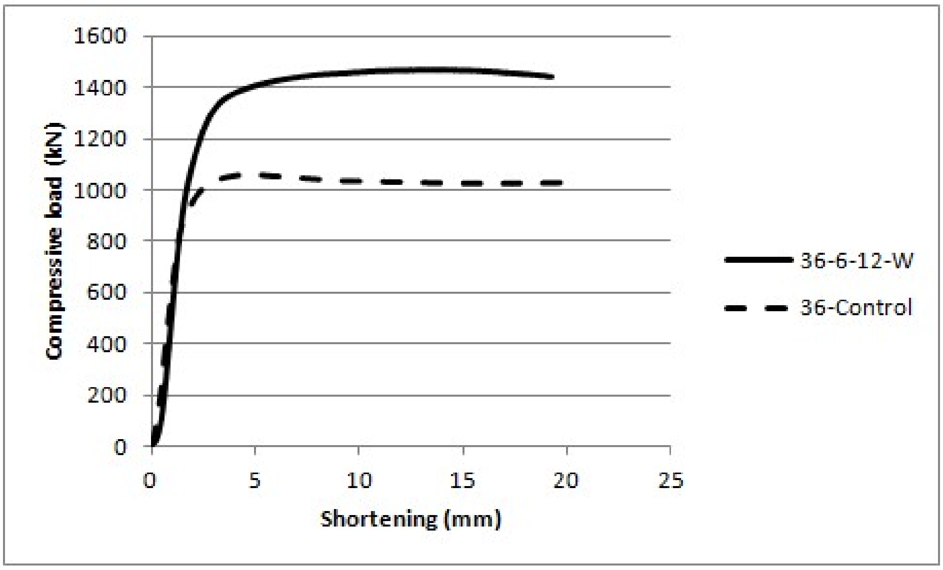


Figure 5.10 Load-shortening curve for 36-6-12-W specimen

The maximum load of the 36-6-12-W specimen was 1467.87 kN which gives an increase of 38% of compressive load capacity from the control specimen, the corresponding shortening was 13.84 mm. The increase of compressive load capacity is more than other specimens with same reinforcing bar's diameter (36-3-12-W, and 36-4-12-W). As well as, the deformation capacity shown in Figure 5.10 was more than the mentioned specimens and also the load-shortening curve was smoother.

5.2.2 Thicker Steel tube Column Specimens

In this section, only thicker steel tube column specimens with a thickness of 5.63 mm will be taken. A total of 11 specimens, 10 specimens were the reinforcing steel bars used with, and 1 specimen was a control specimen without any reinforcing bars. The cylinder concrete compressive specimens have been tested on the test day and gave a 49.46 MPa in average. The same procedure of comparison that used with thinner steel tube column specimens will be carried out for thicker steel tube column specimens. For the control specimen, the maximum load with the corresponding shortening were 1362.75 kN and 18.93 mm respectively.

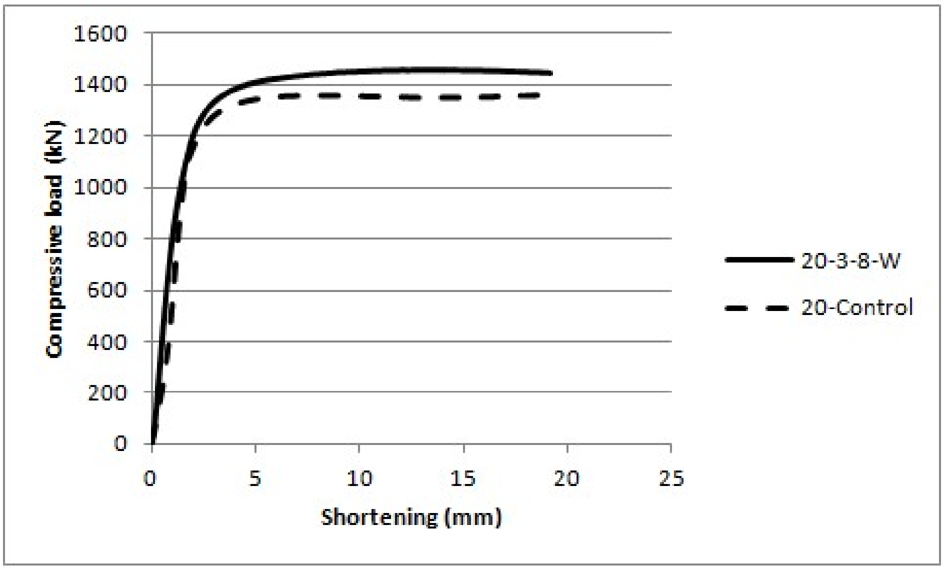


Figure 5.11 Load-shortening curve for 20-3-8-W specimen

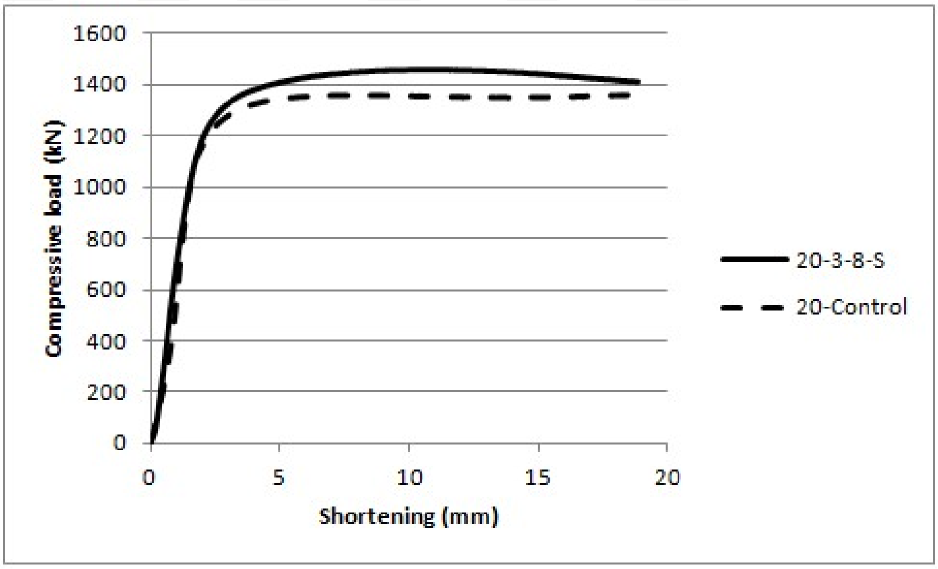


Figure 5.12 Load-shortening curve for 20-3-8-S specimen

The maximum compressive load capacities of the specimens 20-3-8-W and 20-3-8-S were 1455.88 kN and 1455 kN with the corresponding shortening of 13.31 mm and 10.99 mm respectively. The increases of compressive load capacity for the two specimens 20-3-8-W and 20-3-8-S were approximately equal to 6.83%, and 6.78% respectively. These increases are attributed to the reinforcing bars as welded or stirrups. Anyway, by observing the load-shortening curves of the two specimens Figures 5.11-12, the 20-3-8-W shows a deformation capacity slightly higher than the 20-3-8-S specimen.

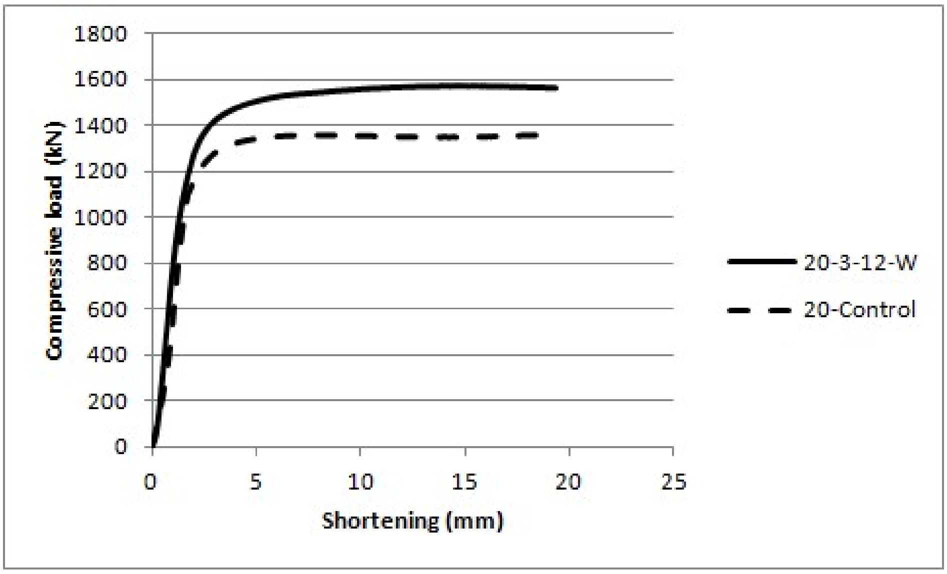


Figure 5.13 Load-shortening curve for 20-3-12-W specimen

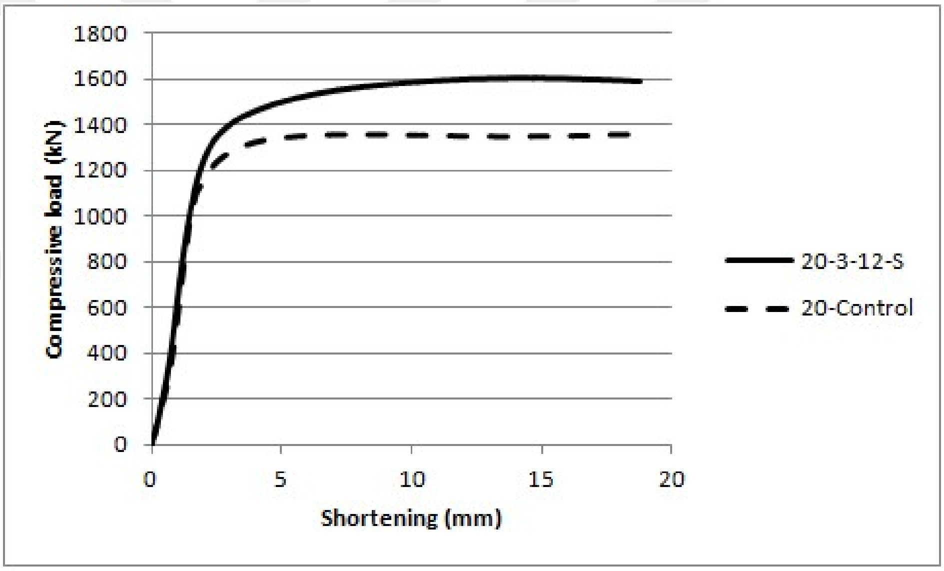


Figure 5.14 Load-shortening curve for 20-3-12-S specimen

The maximum compressive load capacities of the specimens 20-3-12-W and 20-3-12-S were 1573.2 kN and 1609.63 kN with the corresponding shortening of 14.58 mm and 14.08 mm respectively. The specimen 20-3-12-W gained an increase of compressive capacity up to 15.44% with a good deformation capacity as shown in Figure 5.13. The compressive load capacity enhancement in specimen 20-3-12-S was 18.11%, the load-shortening curve of this specimen was smooth and implied a good deformation capacity approximately the same of that of the specimen 20-3-12-W.

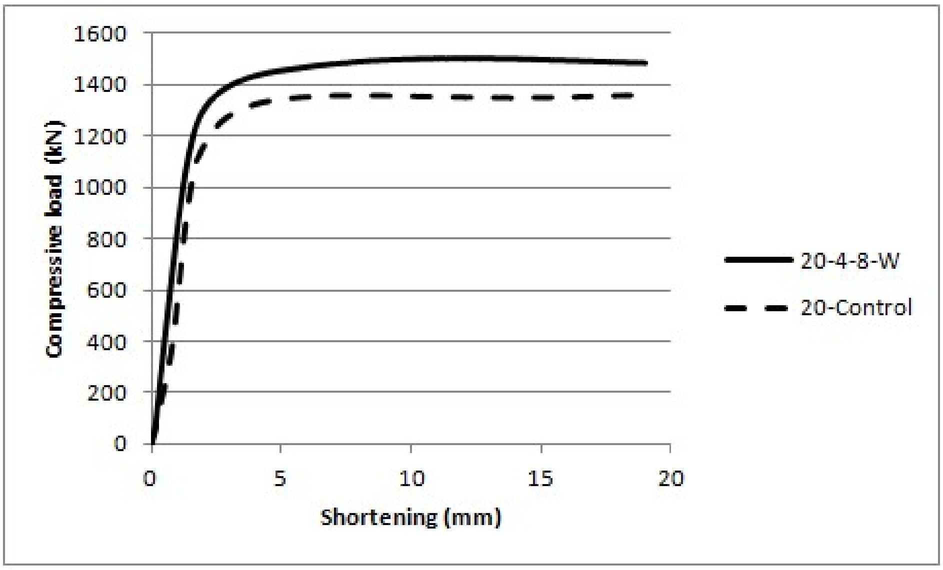


Figure 5.15 Load-shortening curve for 20-4-8-W specimen

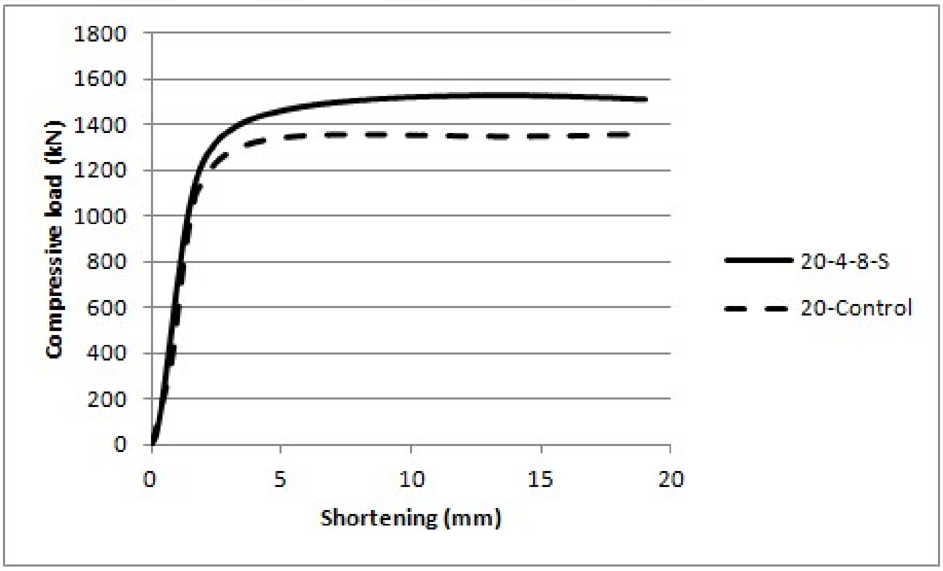


Figure 5.16 Load-shortening curve for 20-4-8-S specimen

The maximum compressive load capacities of the specimens 20-4-8-W and 20-4-8-S were 1504.45 kN and 1529.5 kN with the corresponding shortening of 11.74 mm and 13.75 mm respectively. The increase of the compressive load capacity for the 20-4-8-W specimen was 10.4%, and 11.7% for 20-4-8-S specimen. However, by observing the load-shortening curves for both specimens, the curves were smooth and implies a good deformation capacity against the increase of compressive load, but specimen 20-4-8-S has a slightly more enhanced than the specimen 20-4-8-W as shown in Figures 5.15-16.

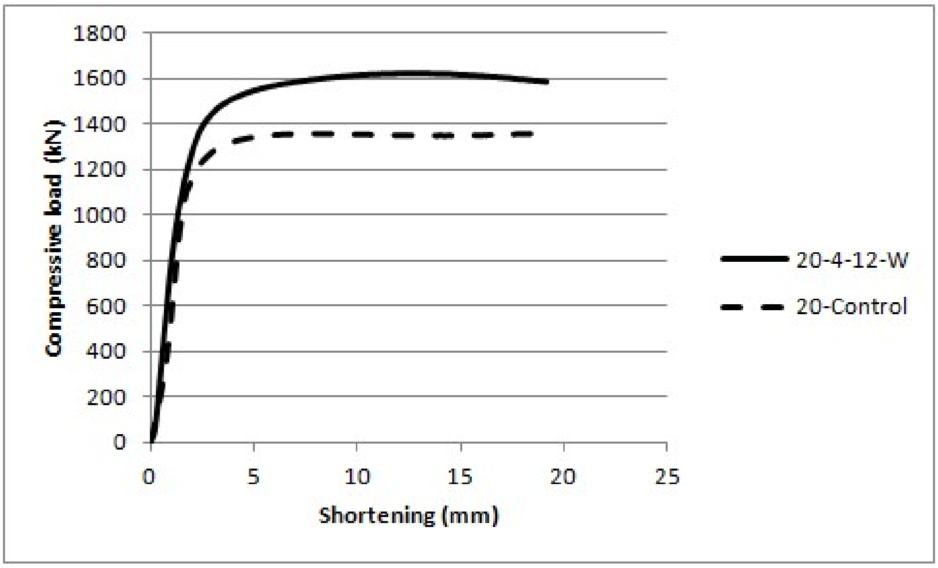


Figure 5.17 Load-shortening curve for 20-4-12-W specimen

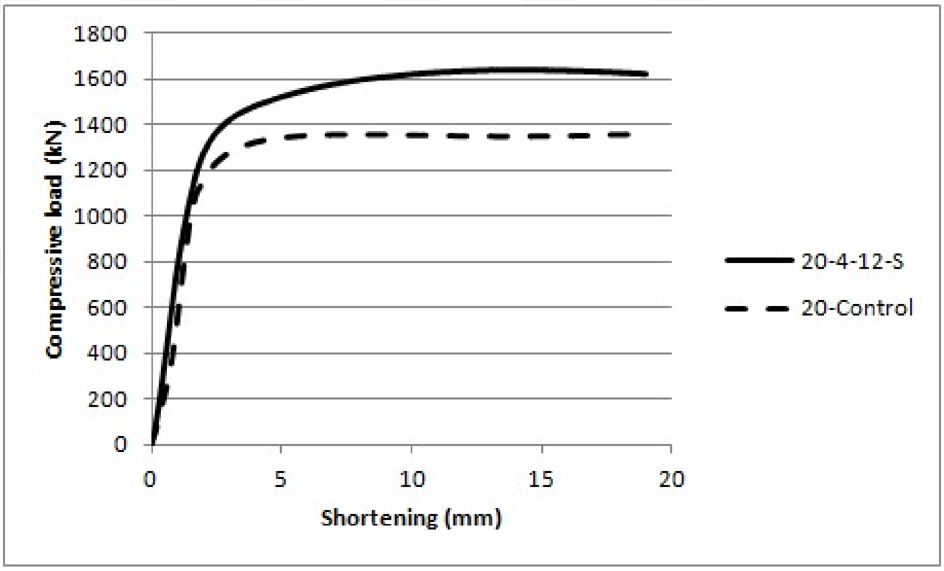


Figure 5.18 Load-shortening curve for 20-4-12-S specimen

The compressive behaviors of these two specimens were approximately the same, the maximum compressive load of 1625.31 kN for the 20-4-12-W specimen with the corresponding shortening of 12.6 mm, and for the 20-4-12-S specimen the maximum load capacity was 1641.75 kN with corresponding shortening of 13.99 mm which is a slightly bigger than the opposite specimen. However, the enhancements of the two specimens in compressive load capacity were 19.26% and 20.47% respectively. The load-shortening curves show a smooth behavior which implies a good load deformation capacity for both specimens as shown in Figures 5.17-18.

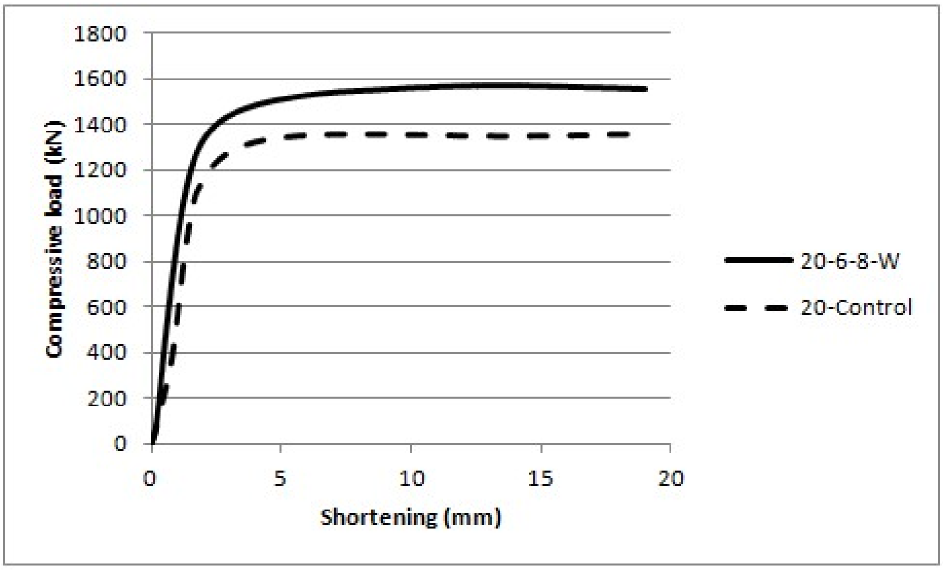


Figure 5.19 Load-shortening curve for 20-6-8-W specimen

The maximum load of the 20-6-8-W specimen was 1572.74 kN, which gave an increase of 15.4% in compressive load capacity with the corresponding shortening of 13.67 and had a good deformation capacity as shown in the curve of load-shortening in Figure 5.19. This specimen gave the best behavior of the corresponding specimens with the same welded reinforcing bars (20-3-8-W and 20-4-8-W with an increase of 6.83% and 10.4% in compressive load capacities respectively).

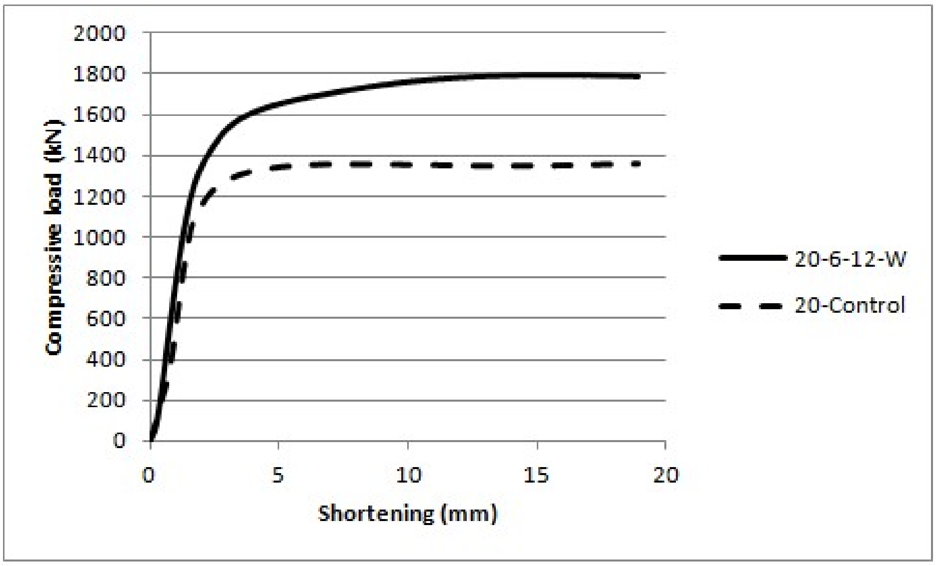


Figure 5.20 Load-shortening curve for 20-6-12-W specimen

1790.78 kN was the maximum compressive load capacity of the specimen 20-6-12-W with the corresponding shortening of 15.79 mm. the enhancement percentage in compressive strength was 31.4% compared with the control specimen with a superior deformation capacity as shown in Figure 5.20. This increase was more than the corresponding specimens with the same welded reinforcing bars (20-3-12-W and 20-4-12-W with percent enhancement 15.44% and 19.26%).

5.2.3 Load versus Strain

The measured axial load is plotted against the axial strain for all tested column specimens in Appendix A. The plotted axial load is measured by the test machine while the axial strain is measured from affixed strain gauges reading.

5.2.4 Compressive Behavior Summary

In this section, a new dividing system will be produced, and so, new curves can be drawn to gain more knowledge about the effect of reinforcing bar's number and diameter on the composite action and concrete core as well. Also, the behavior of these parameters with steel tube thickness will be investigated. However, by using the equation 4.1 to determine the strength index, which gives a good indication of the concrete core behavior and also used to compare the performance of the columns, the comparison between column specimens will be more accurate, as this equation has been used by several researchers to measure the composite action and to compare the performance of the columns [45-47].

Table 5.1 Compressive behavior summary of the column specimens

Comparison	Maximum compressive load (kN)	Corresponding shortening (mm)	Enhancement in compressive capacity (%)	f_c MPa	f_y MPa	f_{ysr} MPa	SI
Thinner specimens (3.15 mm thickness) with ϕ8 mm reinforcing bars, A1							
36-Control	1063.5	4.86	-	49.46	460	0	1.193
36-3-8-W	1140.825	4.56	7.27	49.46	460	534.2	1.181
36-3-8-S	1154.82	8.71	8.59				1.196

36-4-8-W 36-4-8-S	1161.97 1195.33	6.65 8.87	9.26 12.4	49.46	460	534.2	1.173 1.207
36-6-8-W	1236.77	6.71	16.29	49.46	460	534.2	1.189
Thinner specimens (3.15 mm thickness) with ϕ12 mm reinforcing bars, A2							
36-3-12-W 36-3-12-S	1278.90 1230.08	12.03 10.21	20.25 15.67	49.46	460	470.5	1.233 1.18
36-4-12-W 36-4-12-S	1329.68 1316.71	13.69 11.56	25 23.8	49.46	460	470.5	1.225 1.213
36-6-12-W	1467.87	13.84	38	49.46	460	470.5	1.242
Thicker specimens (5.63 mm thickness) with ϕ8 mm reinforcing bars, B1							
20-Control	1362.75	18.93	-	49.46	410	0	1.196
20-3-8-W 20-3-8-S	1455.88 1455	13.31 10.99	6.83 6.78	49.46	410	534.2	1.200 1.199
20-4-8-W 20-4-8-S	1504.45 1529.5	11.74 13.75	10.4 11.7	49.46	410	534.2	1.215 1.235
20-6-8-W	1572.74	13.67	15.4	49.46	410	534.2	1.221
Thicker specimens (5.63 mm thickness) with ϕ12 mm reinforcing bars, B2							
20-3-12-W 20-3-12-S	1573.2 1609.63	14.58 14.08	15.44 18.11	49.46	410	470.5	1.225 1.253
20-4-12-W 20-4-12-S	1625.31 1641.75	12.6 13.99	19.26 20.47	49.46	410	470.5	1.219 1.232
20-6-12-W	1790.78	15.79	31.4	49.46	410	470.5	1.252

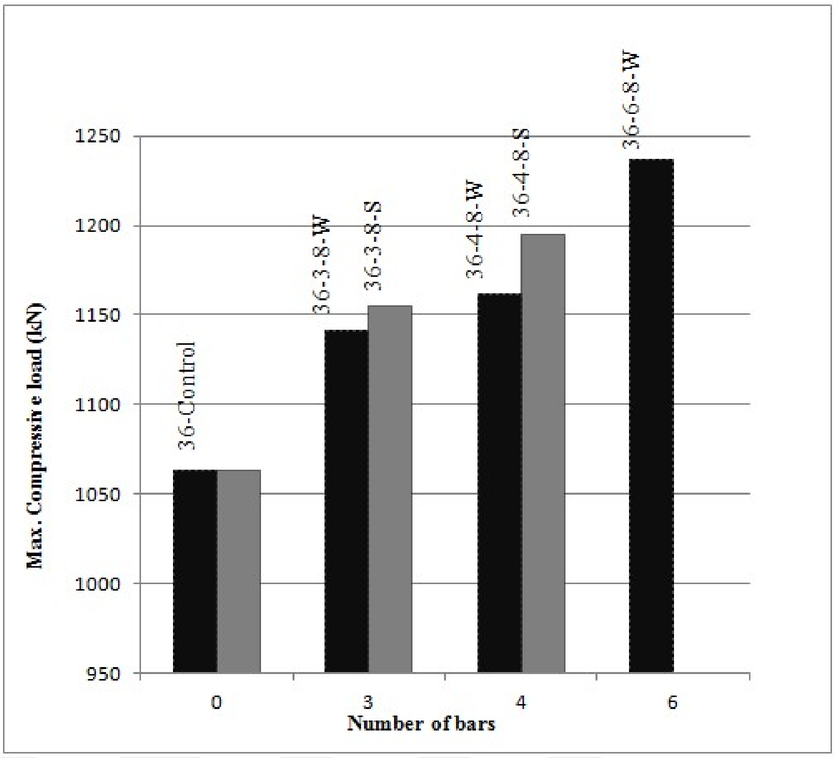


Figure 5.21 Maximum load for thinner steel tube with ϕ8 mm reinforcing bars

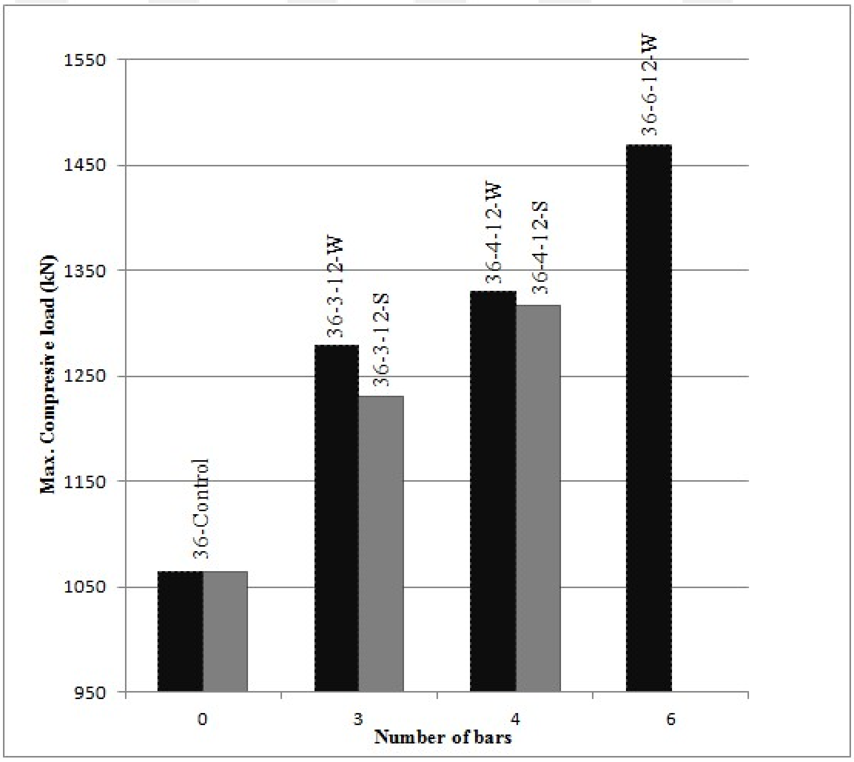


Figure 5.22 Maximum load for thinner steel tube with ϕ12 mm reinforcing bars

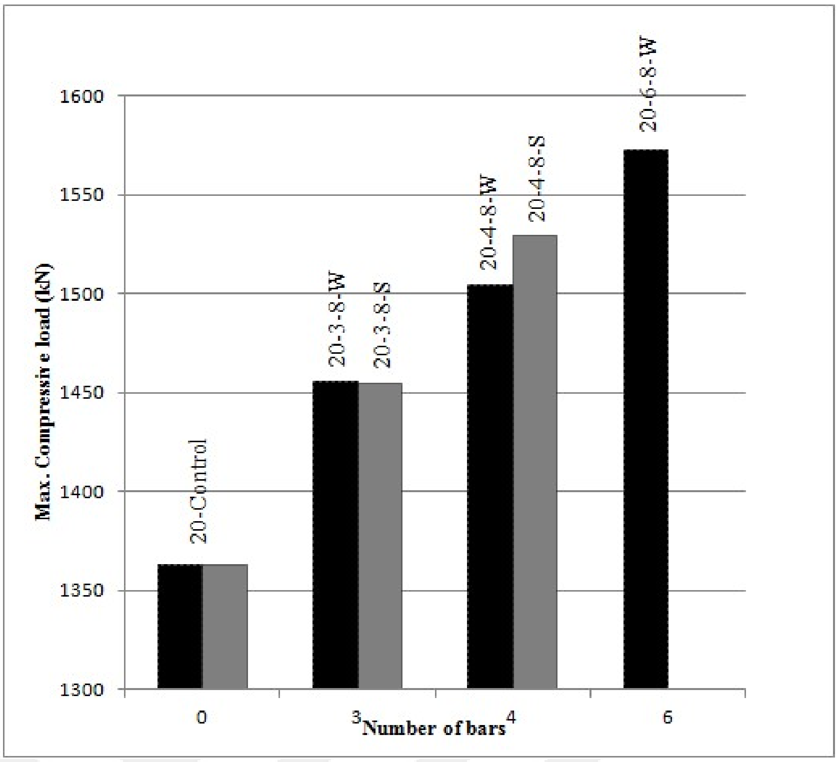


Figure 5.23 Maximum load for thicker steel tube with ϕ8 mm reinforcing bars

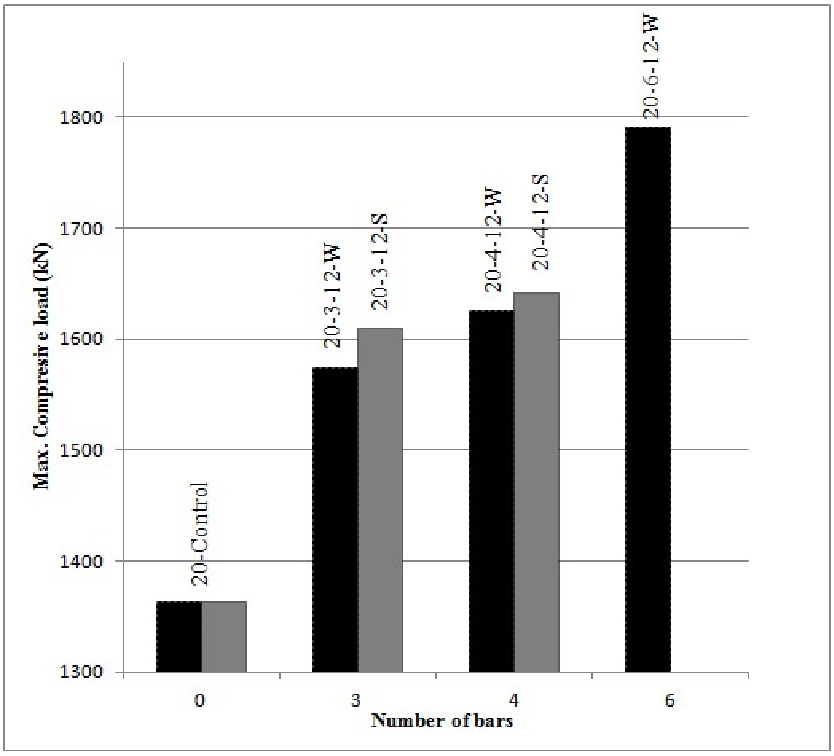


Figure 5.24 Maximum load for thicker steel tube with ϕ12 mm reinforcing bars

When the column specimen become under compression load, the axial force starts a try to deform the specimen longitudinally and the column specimen resists that force by the steel tube and the concrete core till the point of maximum strength of the weakest component in the composite is reached, as known the steel strength is much more than concrete strength which increased by the confinement effect of steel tube. When the maximum strength of concrete core reached, the concrete will crush and a transfer strain will generate laterally causing an addition force effecting on the steel tube laterally which cause outward buckling and specimen fail at final. However, the use of the reinforcing bars as stirrups or welded bars is to delay that failure as possible.

For specimens in group A1 in Table 5.1, Figure 5.21, which the steel tubes were 3.15 mm in thickness and have a 460 MPa yield strength, the reinforcing steel bars ϕ8 mm with stirrups shows more enhancement in compressive strength and had a slight increase in strength index (SI in the eighth column) than that of welded steel bars by resisting the transfer lateral force of concrete core which delayed the crushing of concrete and the outward buckling of the steel tube and gave more compressive load capacity as a result, so the best effect of steel reinforcing bars in this group was improving in the concrete core more than the improving in steel tube because, the ϕ8 mm welded reinforcing bars with a thinner steel tube was not enough to resist the outward buckling as the stirrups did. While for the group A2, Figure 5.22, with the same steel tube characteristics but with reinforcing steel bars of 12 mm diameter. By observing the ultimate load and strength index (SI), it was concluded that the enhancement in the thinner steel tube by welded reinforcing bars was more than the enhancement in concrete core by stirrups. So, the concrete core was crushed, causing a lateral force on the steel tube and because of the resistance of the steel tube and welded reinforcing bars, the outward buckling was delayed which gave a more compressive load capacity than the specimens with stirrups.

For group B1, and B2 in Table 5.1, shown in Figure 5.23-24, which have a 5.63 mm in thickness and 410 MPa yield strength steel tube and a strength index (SI) as shown in the eighth column of table 5.1. Generally, the steel tube with these properties already has a good resistance to the outward buckling caused by the transfer force of concrete core than the thinner tubes. Both groups show a superior enhancement in compressive load with a slight increase for the specimens with stirrups more than that with welded steel reinforcing bars. So, for thicker steel tube the best effect of reinforcing bars was as stirrups, which resist the lateral force of concrete core, causing a delay in the concrete crush and the outward buckling of the 5.63 mm thickness steel tube giving more compressive load capacity as a result.

5.3 Ductility

Ductility is a mechanical property that describes the extent in which solid materials can be plastically deformed without fracture. However, the ductile behavior could be easily indicated by observing the load-deformation curve for any structural member. The observed relationship of the load-deformation curve exhibited three distinct stages. In the initial stage of loading the curves exhibit linear load-shortening till a point of yield strength reached and the stage of elasto-plastic behavior occur up to ultimate load, then the failure mechanisms start to take a place. By observing these regions of the curve for each column specimen in Figures 5.26-29, the ductility can be generally compared between the selected specimens with the same parameter of tube thickness and the method of using reinforcing bars (welded or with stirrups).

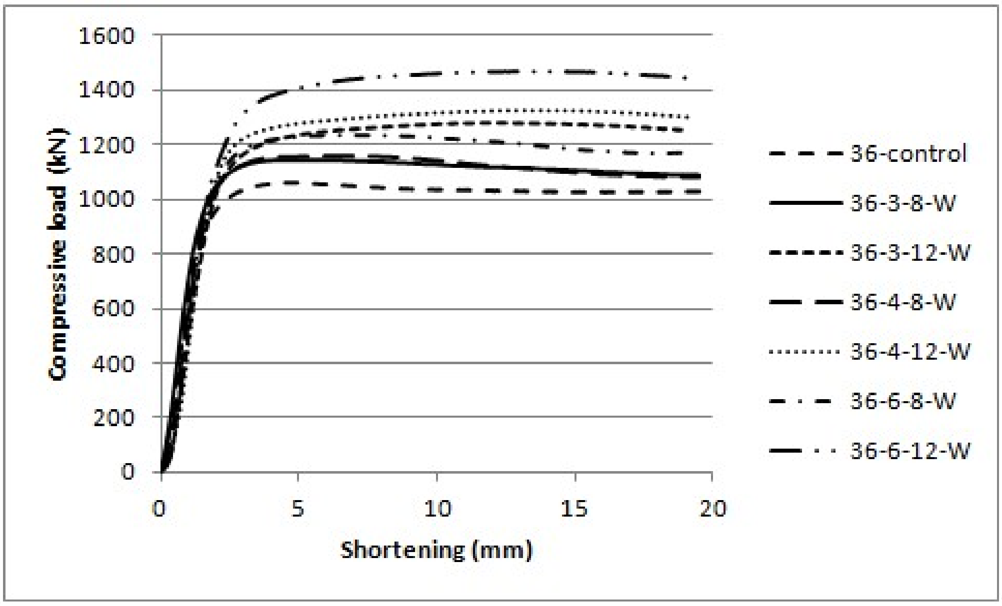


Figure 5.25 Load-deformation curves for thinner steel tube with welded reinforcing bars

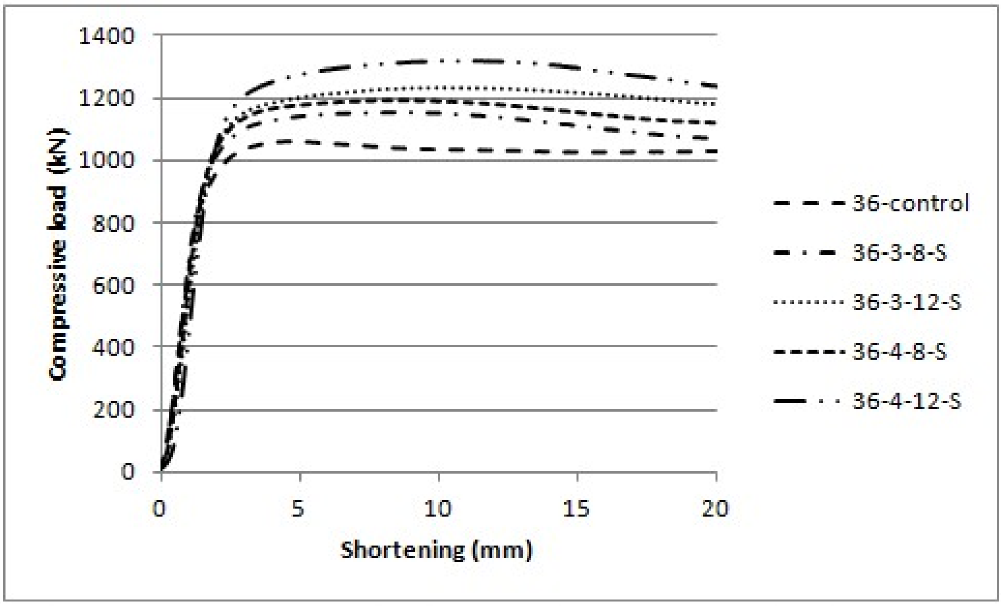


Figure 5.26 Load-deformation curves for thinner steel tube with stirrups

For the 3.15 mm thickness and 460 MPa yield strength steel tube, The welded reinforcing bars specimens show more ductile behavior than these with stirrups, Figures 5.25 and 5.26. However, the ductility behavior was increased by increasing the number and diameter of reinforcing bars as the strength index (SI) was increased, which increased the stiffness as well. This is signified by elongated Elasto-plastic plates, and smoother curve after ultimate load for specimens of welded reinforcing bars than the specimens of reinforcing bars with stirrups. The effect of reinforcing bars on ductility was more obvious for ϕ12 mm reinforcing bars than ϕ 8 mm reinforcing bars.

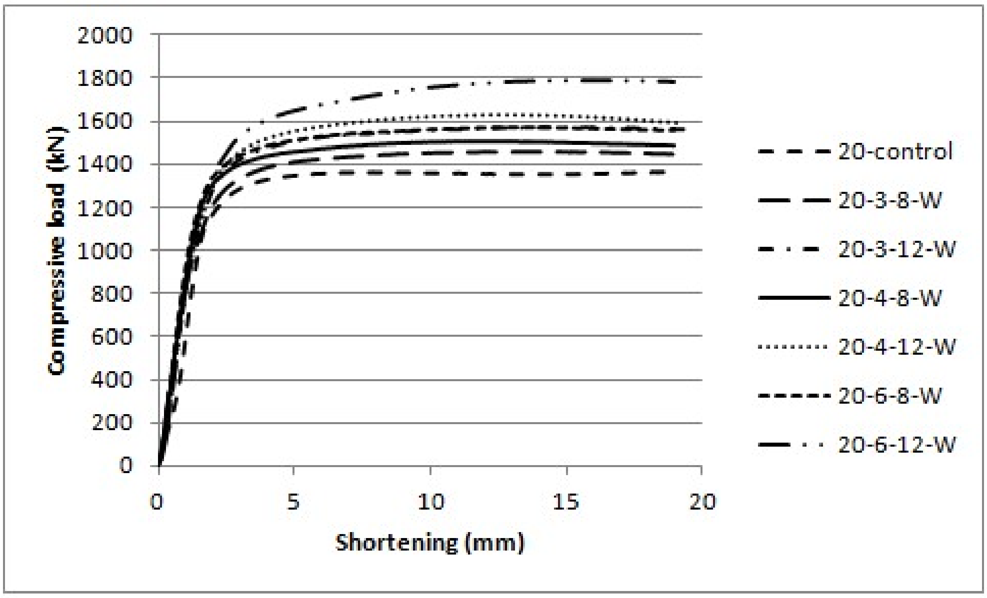


Figure 5.27 Load-deformation curves for thicker steel tube with welded reinforcing bars

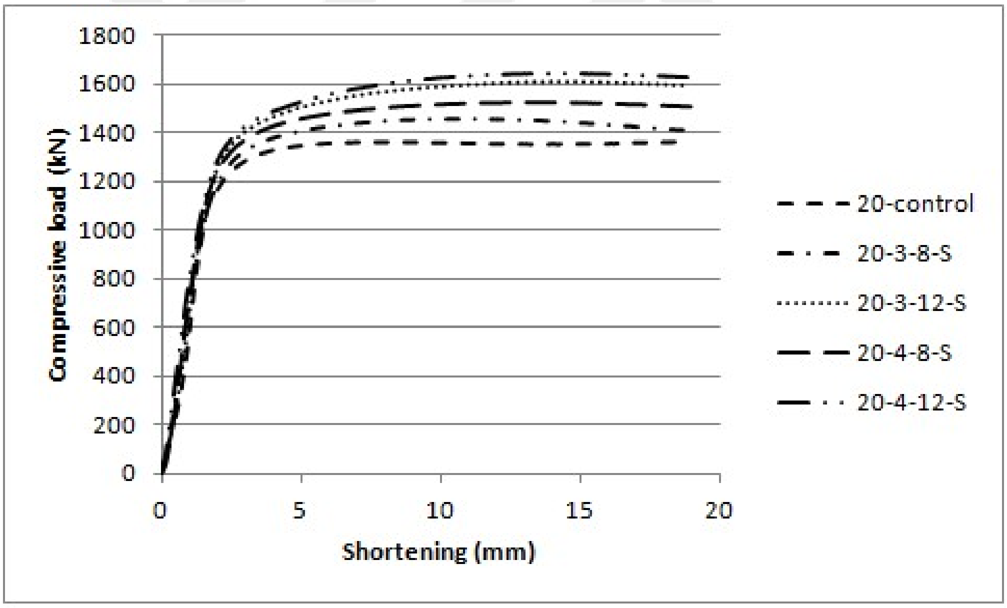
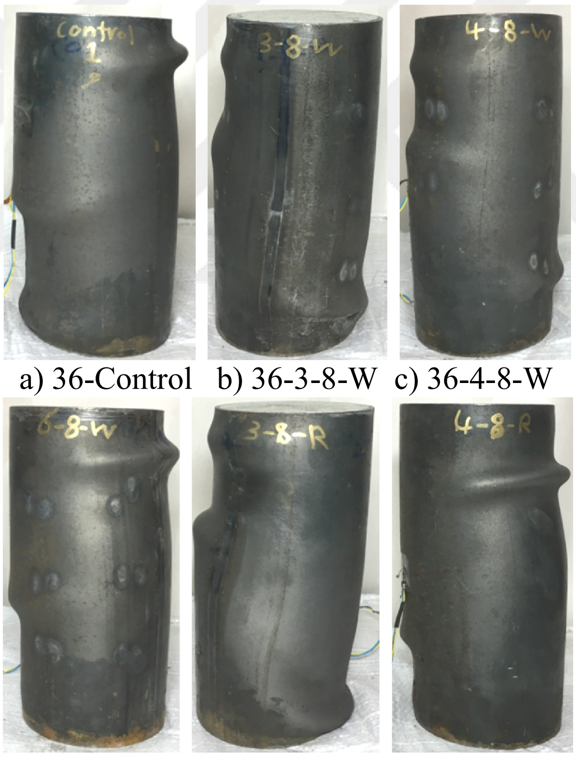


Figure 5.28 Load-deformation curves for thicker steel tube with stirrups

For the thicker steel tube with a 5.63 mm thickness, 410 MPa yield strength, and strength index (SI) in Table 5.1, the ductile behavior of both groups (with stirrups and welded reinforcing bars) was approximately the same with a slight increase in ductility for the welded reinforcing bars specimens. Figures 5.27 and 5.28 shows the load-deflection curves for the mentioned groups, it can be observed from the figures the elongated elasto-plastic region with not much loss in compressive strength after the maximum load was reached which is an indication of a good ductile behavior. In general, the steel tube of 5.63 mm thickness already has a good ductility with 410 MPa yield strength and can produce a sufficient confinement strength and load absorption which results a good ductile behavior. So, the $\phi8$ mm reinforcing bars with a smaller number of bars will not affect like $\phi12$ mm reinforcing bars on the ductile behavior of column specimens.

5.4 Failure Mode

The 265 mm long with a diameter of 114.3 mm composite columns, are considered as a stub column and the failure mode of such a column is crushing in concrete core and yielding in steel tube. By using the same dividing system that used in section 5.3.2, the failure mode of column specimens can be arranged as in Figures 5.29-32.



a) 36-Control b) 36-3-8-W c) 36-4-8-W

d) 36-6-8-W e) 36-3-8-S f) 36-4-8-S

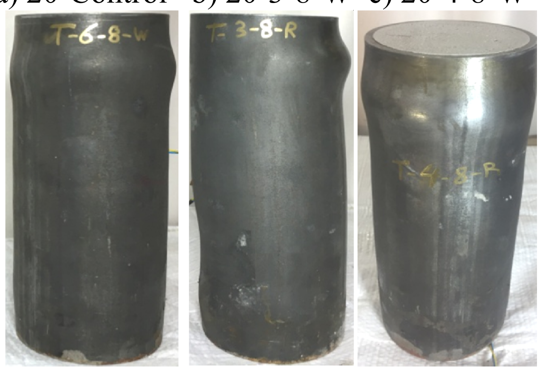
Figure 5.29 Failure mode of 3.15 thickness steel tube specimens with $\phi8$ mm reinforcing bars

a) 36-Control b) 36-3-12-W c) 36-4-12-W

d) 36-6-12-W e) 36-3-12-S f) 36-4-12-S

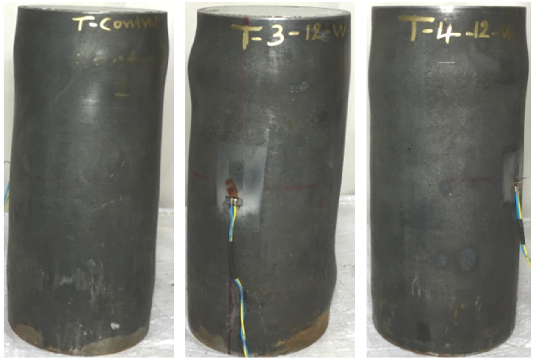
Figure 5.30 Failure mode of 3.15 thickness steel tubes with ϕ12 mm reinforcing bars specimens

a) 20-Control b) 20-3-8-W c) 20-4-8-W

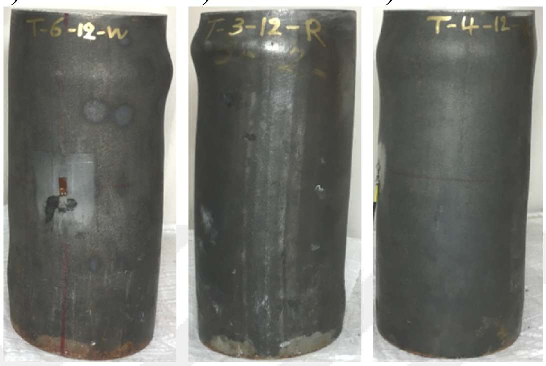


d) 20-6-8-W e) 20-3-8-S f) 20-4-8-S

Figure 5.31 Failure mode of 5.63 thickness steel tubes with ϕ8 mm reinforcing bars specimens



a) 20-Control b) 20-3-12-W c) 20-4-12-W



d) 20-6-12-W e) 20-3-12-S f) 20-4-12-S

Figure 5.32 Failure mode of 5.63 thickness steel tubes with ϕ12 mm reinforcing bars specimens

In general, it was observed from load-shortening curves, that the initial stiffness was enhanced for the specimens with welded bars. Initially, when the axial load was increasing, the axial strain increased linearly, and the slope of this portion was the elastic stiffness (modulus of elasticity) of the column specimens. However, such a behavior is attitude of resistance of the welded bars and steel tube against the axial load in the elastic region when the specimens were loaded at the earlier time of the test. At this time the confining pressure wasn't developed yet and the biggest part of the load was carried by the steel tube. So, the existence of welded bars increased the initial stiffness. As the load increased further, the curve would present elasto-plastic behavior, which mean the development of micro-cracks in concrete core causing an expansion in concrete core. This expansion would be restrained by the steel tube and thus the confining pressure would be developed. Using the welded bars will increase the resistance of the steel tube against the transformed lateral force that developed by concrete core's micro cracks. Furthermore, using reinforcing bars with stirrups within the concrete core will provide a confining pressure to the concrete core in addition to that of steel tube by resisting the micro-cracks in concrete core.

Nevertheless, while the inward buckling of the steel tube was prevented by the concrete core, at the larger strain, the strain-hardening of the steel tube could be fully developed, causing an enhancement in the strength of steel tube hence the confining pressure. Then, the curve increased to a smaller extent in the plastic stage.

CHAPTER 6

CONCLUSION

6.1 Conclusion

The uses of reinforcing bars with concrete filled steel tube columns has a few popularity in spite of its benefits in enhancing the column behavior, thus, this study was carried out to investigate and increase the knowledge about using the reinforcing bars and its effect on the compressive behavior, ductility, stiffness, and failure mode as well. Total of twenty-two column specimens (two were a control specimens) with 265 mm length and outer diameter of 114.3 mm were tested under axial compressive load with different parameters, two different steel tube thickness (5.63 mm, and 3.15 mm), two different reinforcing bar diameter (ϕ8 mm, and ϕ12 mm), different number of bars, and the method of using reinforcing bars (welded or stirrups).

It was observed that the reinforcing bars increased the compressive strength capacity, ductility, and stiffness of the twenty column specimens and that enhancement was increased by increasing the number and diameter of reinforcing bars comparing with the two control specimens.

By applying a comparison between the welded reinforcing bars column specimens and the specimens with stirrups, it was observed that for both the D/t ratio (36.28, and 20.3), the strength index (SI) for column specimens with stirrups was slightly increased more than the specimens of welded bars, which increased the ultimate load capacity by increasing the confining pressure. In other hand, by observing the load-shortening curve of the specimens, the initial stiffness, and ductility were a slightly more enhanced for the specimens with welded bars.

The failure mode of the specimens with welded bars was more enhanced than the specimens with stirrups, because the welded bars worked as a stiffener to the steel tube, which gave a good resistance to the lateral transformed force that caused after the micro-cracks develop in concrete core in early time of loading (elastic region), and the crush of concrete core at the plastic region.

From all above, it was concluded that the welded reinforcing bars is a superior alternative to stirrups, which give approximately the same behavior of stirrups with easier construction process, since the construction process of stirrups with in-filled composite column in real life is harder than welded bars which can be manufactured in origin, because the in-filled steel tube column have no formwork and the steel tube work as a frame for the column, which need to complete the tying process of reinforcing bars with stirrups outside the column and then the reinforcing bars with stirrups should be moved as one part and centered inside the column steel tube, which is approximately impossible for the long columns without any bent in longitudinal reinforcing bars, hence the reinforcing bars will lose its effect on the composite behavior.

6.2 Recommendations of future works

This work showed the effect of using reinforcing bars on the compressive load capacity, ductility, stiffness, and failure mod of short columns under axial compressive load. It is recommended for the future work to study the effect of reinforcing bars with a different column length or different loading condition or different concrete core properties.

REFERENCES

[1] Porsch, M., Hanswille, G. (2006). Load introduction in composite columns with concrete filled hollow sections. In *Composite Construction in Steel and Concrete v*, 402-411.

[2] Shakir-Khalil, H. (1988). Steel-concrete composite columns I. *Steel-concrete composite structures: stability and strength*, 163-193.

[3] Ellobody, E., Young, B., Lam, D. (2006). Behaviour of normal and high strength concrete-filled compact steel tube circular stub columns. *Journal of Constructional Steel Research*, **62(7)**, 706-715.

[4] Han, L. H., Li, W., Bjorhovde, R. (2014). Developments and advanced applications of concrete-filled steel tubular (CFST) structures: Members. *Journal of Constructional Steel Research*, **100**, 211-228.

[5] Giakoumelis, G., Lam, D. (2004). Axial capacity of circular concrete-filled tube columns. *Journal of Constructional Steel Research*, **60(7)**, 1049-1068.

[6] Wardenier, J., Packer, J. A., Zhao, X. L., Van der Vegte, G. J. (2002). *Hollow sections in structural applications*. Rotterdam. The Netherlands: Bouwen met Staal.

[7] Chen, J., Ni, Y. Y., Jin, W. L. (2015). Column tests of dodecagonal section double skin concrete-filled steel tubes. *Thin-Walled Structures*, **88**, 28-40.

[8] Chitawadagi, M. V., Narasimhan, M. C., Kulkarni, S. M. (2010). Axial strength of circular concrete-filled steel tube columns—DOE approach. *Journal of Constructional Steel Research*, **66(10)**, 1248-1260.

[9] Eggemann, H. (2003, January). Development of composite columns. Emperger's effort. In *1st Int. Congress on Construction History, Madrid*, 787-797.

[10] Dutta, B., Bhattacharyya, S. K. (1995). An experimental study on the behaviour of concrete-filled light gauge steel tubular columns. *Indian Concrete Journal*, **69**, 477-480.

[11] O'Shea, M. D., Bridge, R. Q. (1995). Circular thin walled concrete filled steel tubes. *PSSC*, **95**, 53-60.

[12] Schneider, S. P. (1998). Axially loaded concrete-filled steel tubes. *Journal of structural Engineering*, **124(10)**, 1125-1138.

[13] O'Shea, M. D., Bridge, R. Q. (2000). Design of circular thin-walled concrete filled steel tubes. *Journal of Structural Engineering*, **126(11)**, 1295-1303.

[14] Uy, B. (2001). Strength of short concrete filled high strength steel box columns. *Journal of Constructional Steel Research*, **57(2)**, 113-134.

[15] Johansson, M., Gylltoft, K. (2002). Mechanical behavior of circular steel–concrete composite stub columns. *Journal of structural engineering*, **128(8)**, 1073-1081.

[16] Giakoumelis, G., Lam, D. (2004). Axial capacity of circular concrete-filled tube columns. *Journal of Constructional Steel Research*, **60(7)**, 1049-1068.

[17] Sakino, K., Nakahara, H., Morino, S., Nishiyama, I. (2004). Behavior of centrally loaded concrete-filled steel-tube short columns. *Journal of Structural Engineering*, **130(2)**, 180-188.

[18] Zeghiche, J., Chaoui, K. (2005). An experimental behaviour of concrete-filled steel tubular columns. *Journal of Constructional Steel Research*, **61(1)**, 53-66.

[19] Liu, D. (2005). Tests on high-strength rectangular concrete-filled steel hollow section stub columns. *Journal of Constructional Steel Research*, **61(7)**, 902-911.

[20] Han, L. H., Liu, W., Yang, Y. F. (2008). Behaviour of concrete-filled steel tubular stub columns subjected to axially local compression. *Journal of Constructional Steel Research*, **64(4)**, 377-387.

[21] Yu, Z. W., Ding, F. X., Cai, C. S. (2007). Experimental behavior of circular concrete-filled steel tube stub columns. *Journal of Constructional Steel Research*, **63(2)**, 165-174.

[22] Tao, Z., Han, L. H., Wang, D. Y. (2008). Strength and ductility of stiffened thin-walled hollow steel structural stub columns filled with concrete. *Thin-walled structures*, **46(10)**, 1113-1128.

[23] de Oliveira, W. L. A., De Nardin, S., de Cresce El, A. L. H., El Debs, M. K. (2009). Influence of concrete strength and length/diameter on the axial capacity of CFT columns. *Journal of Constructional Steel Research*, **65(12)**, 2103-2110.

[24] Uy, B., Tao, Z., Han, L. H. (2011). Behaviour of short and slender concrete-filled stainless steel tubular columns. *Journal of Constructional Steel Research*, **67(3)**, 360-378.

[25] Hu, Y. M., Yu, T., Teng, J. G. (2011). FRP-confined circular concrete-filled thin steel tubes under axial compression. *Journal of Composites for Construction*, **15(5)**, 850-860.

[26] Dundu, M. (2012). Compressive strength of circular concrete filled steel tube columns. *Thin-Walled Structures*, **56**, 62-70.

[27] Abed, F., AlHamaydeh, M., Abdalla, S. (2013). Experimental and numerical investigations of the compressive behavior of concrete filled steel tubes (CFSTs). *Journal of Constructional Steel Research*, **80**, 429-439.

[28] Ferhoune, N. (2014). Experimental behaviour of cold-formed steel welded tube filled with concrete made of crushed crystallized slag subjected to eccentric load. *Thin-Walled Structures*, **80**, 159-166.

[29] Huang, F., Yu, X., Chen, B., Li, J. (2016). Study on preloading reduction of ultimate load of circular concrete-filled steel tubular columns. *Thin-Walled Structures*, **98**, 454-464.

[30] Ding, F. X., Fu, L., Liu, X. M., Liu, J. (2016). Mechanical performances of track-shaped rebar stiffened concrete-filled steel tubular (SCFRT) stub columns under axial compression. *Thin-Walled Structures*, **99**, 168-181.

[31] Ekmekyapar, T. (2016). Experimental performance of concrete filled welded steel tube columns. *Journal of Constructional Steel Research*, **117**, 175-184.

[32] Ge, H., Usami, T. (1992). Strength of concrete-filled thin-walled steel box columns: experiment. *Journal of structural engineering*, **118(11)**, 3036-3054.

[33] Claeson, C., Gylltoft, K. (2000). Slender concrete columns subjected to sustained and short-term eccentric loading. *ACI Structural Journal*, ***97*(1)**.

[34] Chen, B. L., Wang, L. G., Qin, G. P. (2011). Bearing Capacity Calculating of GFRP Tube Filled with Steel-Reinforced Concrete Composite Column Subjected to Eccentric Loading. In *Advanced Materials Research*, **163**, 191-195.

[35] Lai, M. H., Ho, J. C. M. (2013). Uni-axial compression test of concrete-filled-steel-tube columns confined by tie bars. *Procedia Engineering*, **57**, 662-669.

[36] Xu, F., Chen, J., Jin, W. L. (2014). Experimental investigation of thin-walled concrete-filled steel tube columns with reinforced lattice angle. *Thin-Walled Structures*, **84**, 59-67.

[37] Brown, N. K., Kowalsky, M. J., Nau, J. M. (2014). Impact of diameter to thickness ratio on the seismic behavior of reinforced concrete filled steel tubes.

[38] Cai, J., Pan, J., Wu, Y. (2015). Mechanical behavior of steel-reinforced concrete-filled steel tubular (SRCFST) columns under uniaxial compressive loading. *Thin-Walled Structures*, **97**, 1-10.

[39] Lu, Y., Li, N., Li, S., Liang, H. (2015). Behavior of steel fiber reinforced concrete-filled steel tube columns under axial compression. *Construction and Building Materials*, **95**, 74-85.

[40] Zhou, X., Liu, J., Wang, X., Chen, Y. F. (2016). Behavior and design of slender circular tubed-reinforced-concrete columns subjected to eccentric compression. *Engineering Structures*, **124**, 17-28.

[41] Ding, F. X., Lu, D. R., Bai, Y., Zhou, Q. S., Ni, M., Yu, Z. W., Jiang, G. S. (2016). Comparative study of square stirrup-confined concrete-filled steel tubular stub columns under axial loading. *Thin-Walled Structures*, **98**, 443-453.

[42] EN1994, B. S. 1-1.(2004). Eurocode 4: Design of composite steel and concrete structures-Part 1-1: General rules and rules for buildings. *British Standard Institution, London*.

[43] AISC Committee. (2010). Specification for Structural Steel Buildings (ANSI/AISC 360-10). *American Institute of Steel Construction, Chicago-Illinois*.

[44] Standard, A. S. T. M. (2003). C33: Specification for concrete aggregates. *ASTM, International, West Conshohocken, PA*.

[45] Han, L. H., Li, W., Bjorhovde, R. (2014). Developments and advanced applications of concrete-filled steel tubular (CFST) structures: Members. *Journal of Constructional Steel Research*, **100**, 211-228.

[46] Han, L. H., Yao, G. H., Zhao, X. L. (2005). Tests and calculations for hollow structural steel (HSS) stub columns filled with self-consolidating concrete (SCC). *Journal of Constructional Steel Research*, **61(9)**, 1241-1269.

[47] Ding, F. X., Liu, J., Liu, X. M., Yu, Z. W., & Li, D. W. (2015). Mechanical behavior of circular and square concrete filled steel tube stub columns under local compression. *Thin-Walled Structures*, **94**, 155-166.

APPENDIX A

Load versus Strain curves for tested column specimens

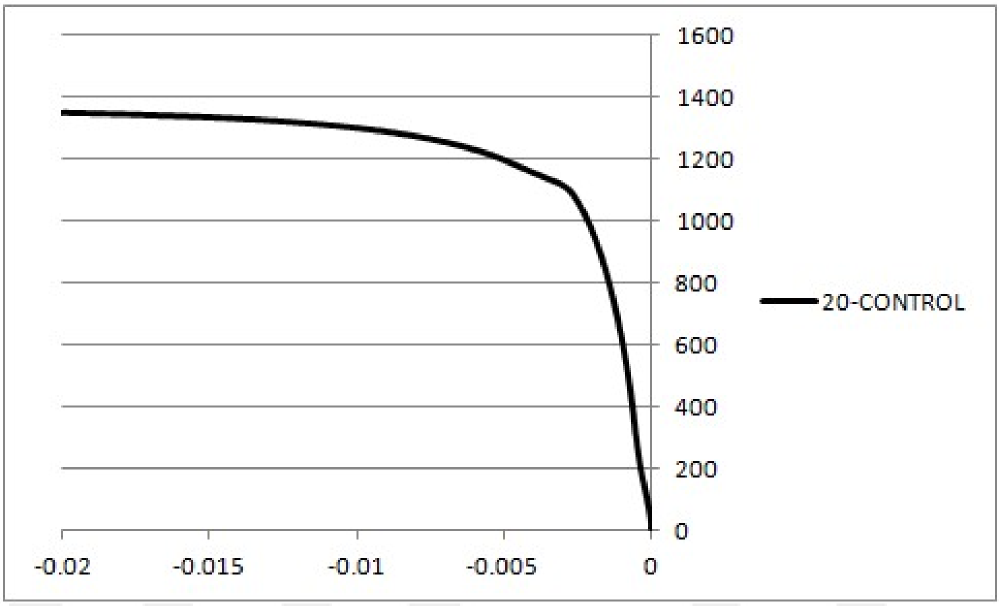


Figure A.1 Load-Strain curve for specimen 20-Control

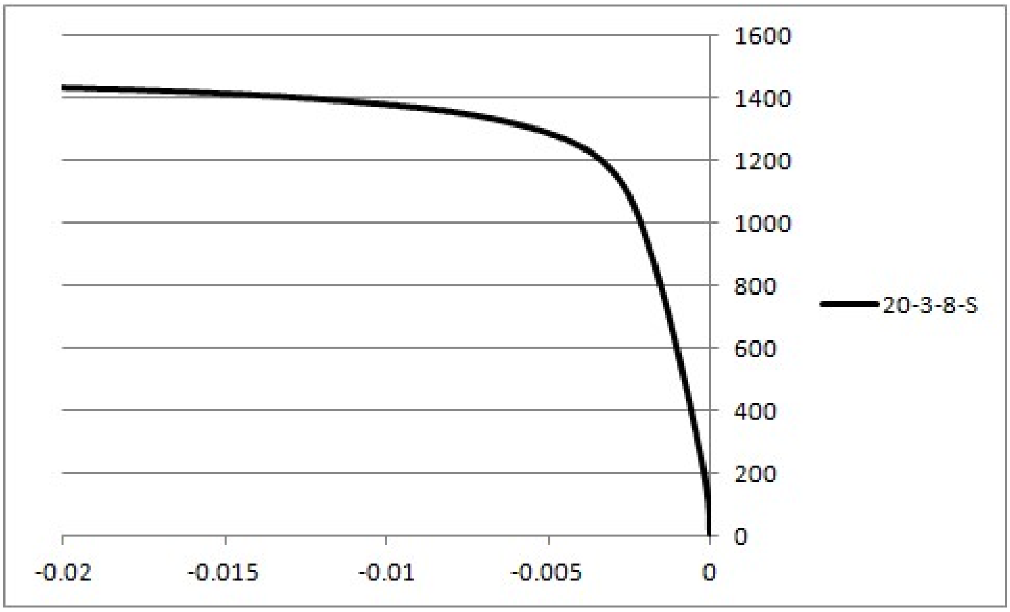


Figure A.2 Load-Strain curve for specimen 20-3-8-S

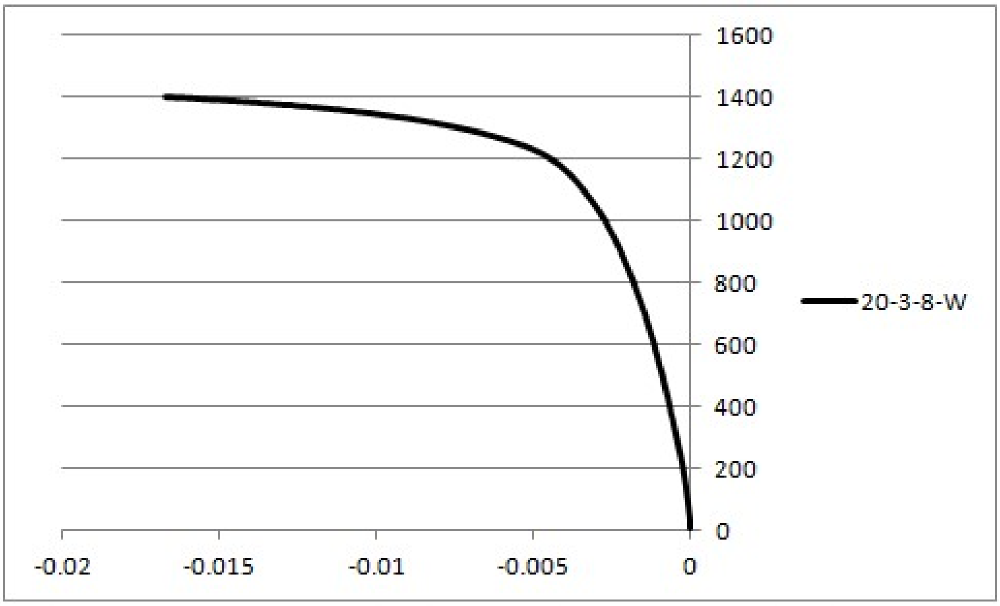


Figure A.3 Load-Strain curve for specimen 20-3-8-W

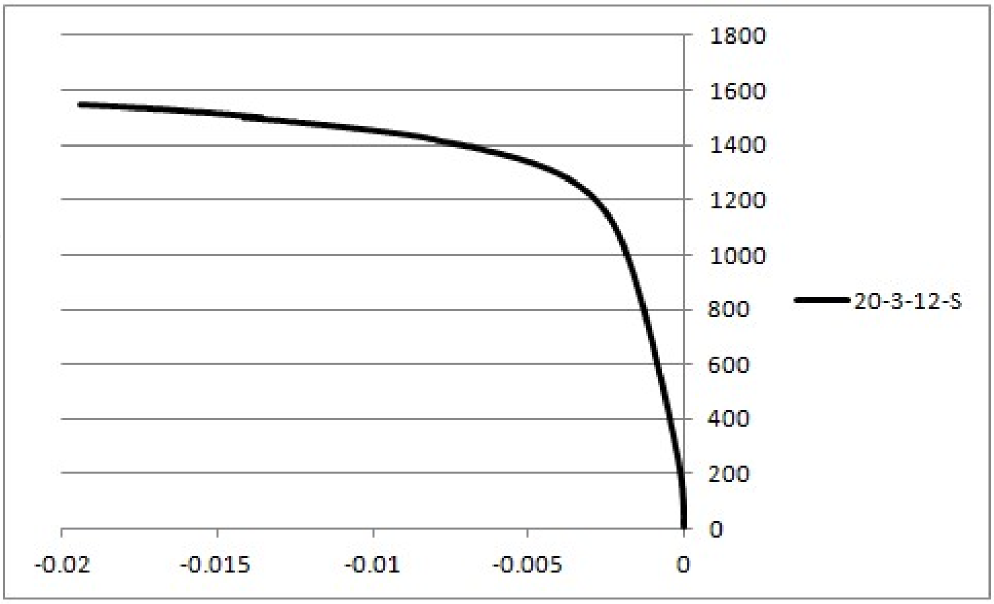


Figure A.4 Load-Strain curve for specimen 20-3-12-S

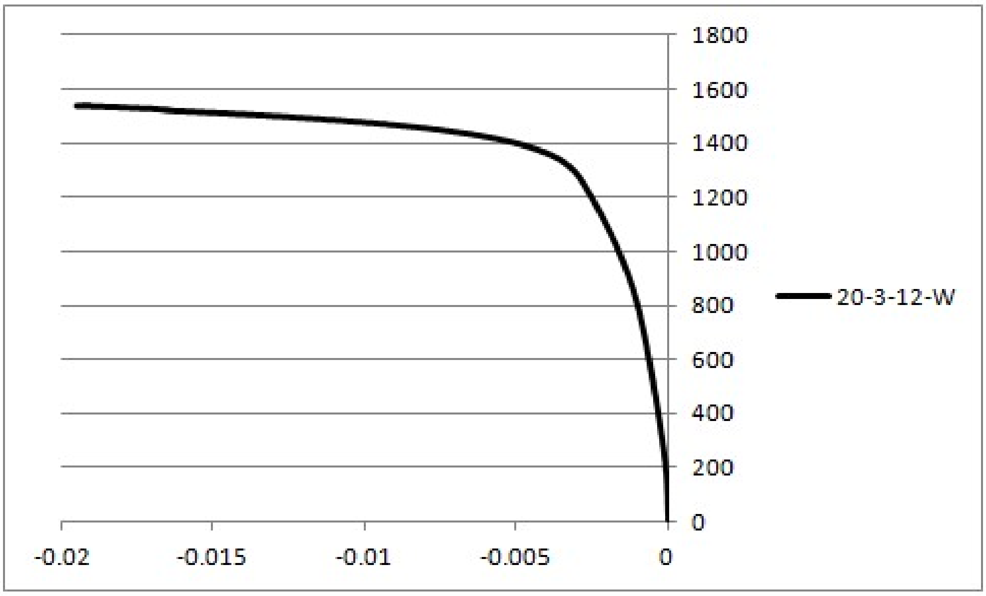


Figure A.5 Load-Strain curve for specimen 20-3-12-W

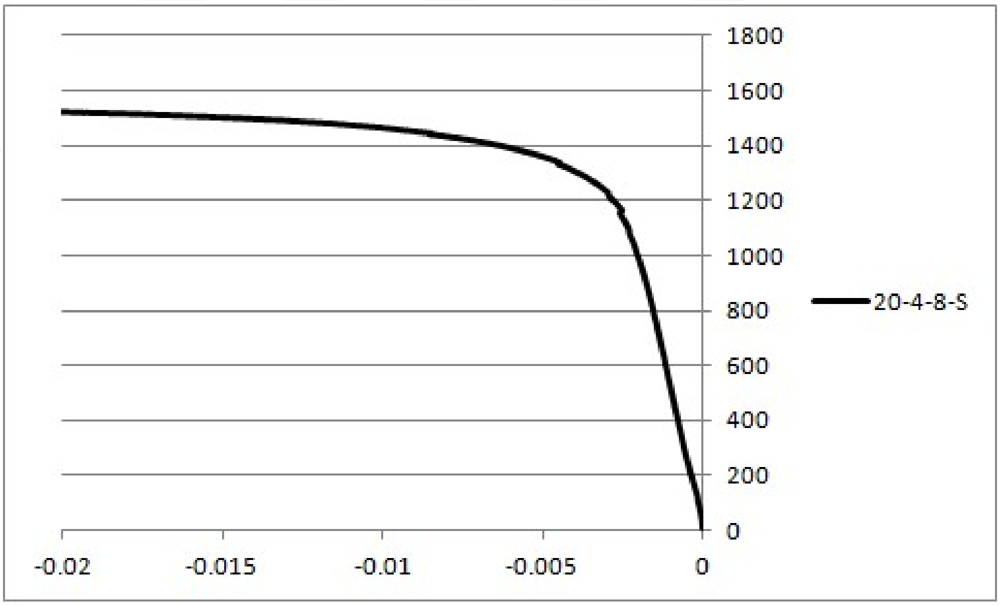


Figure A.6 Load-Strain curve for specimen 20-4-8-S

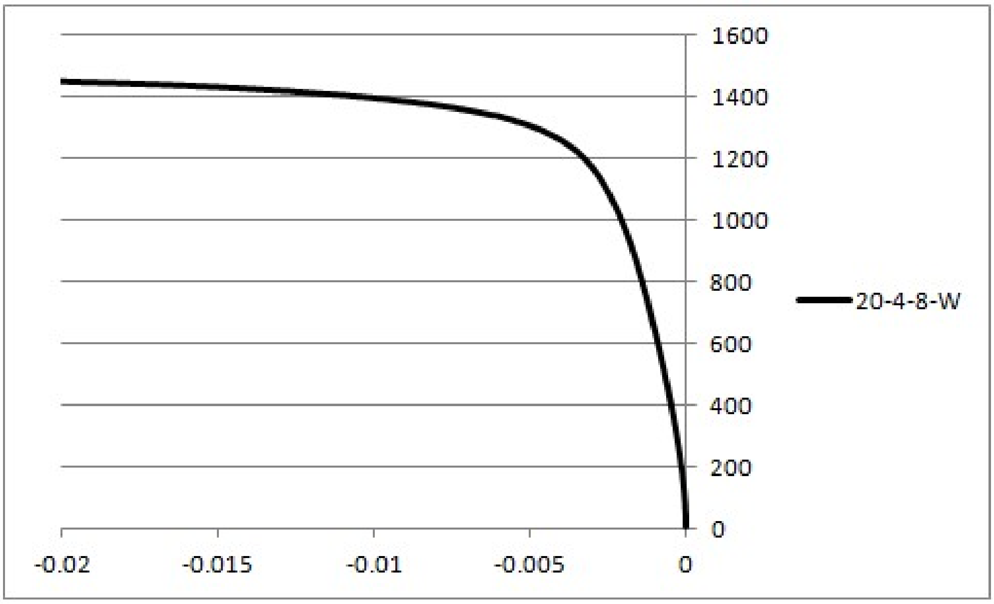


Figure A.7 Load-Strain curve for specimen 20-4-8-W

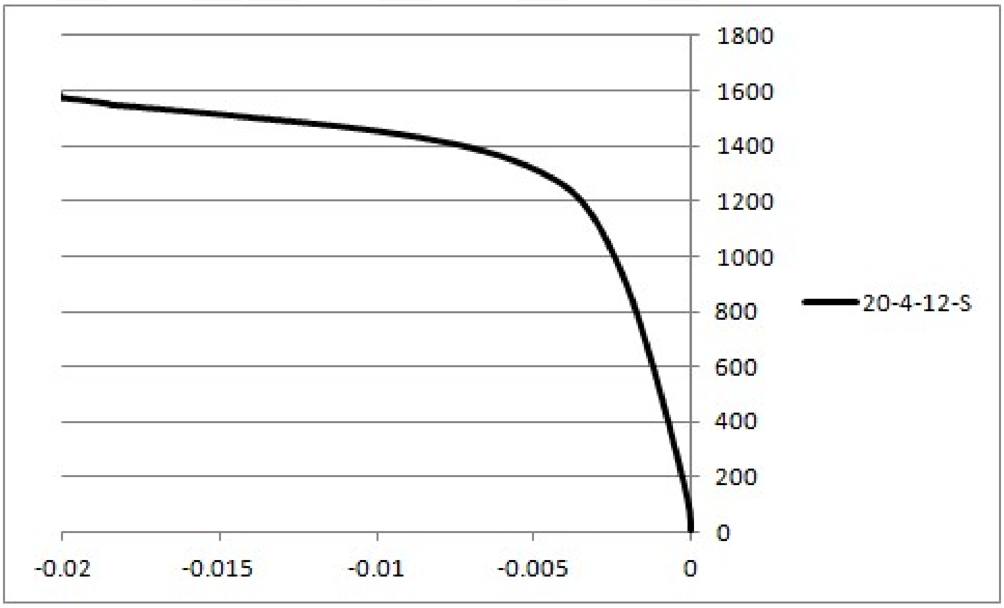


Figure A.8 Load-Strain curve for specimen 20-4-12-S

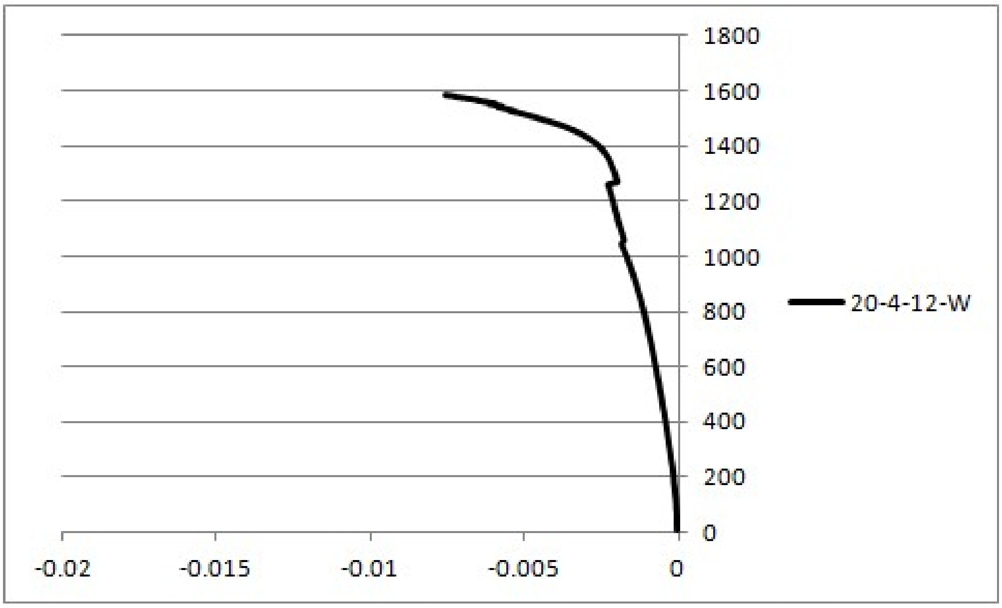


Figure A.9 Load-Strain curve for specimen 20-4-12-W

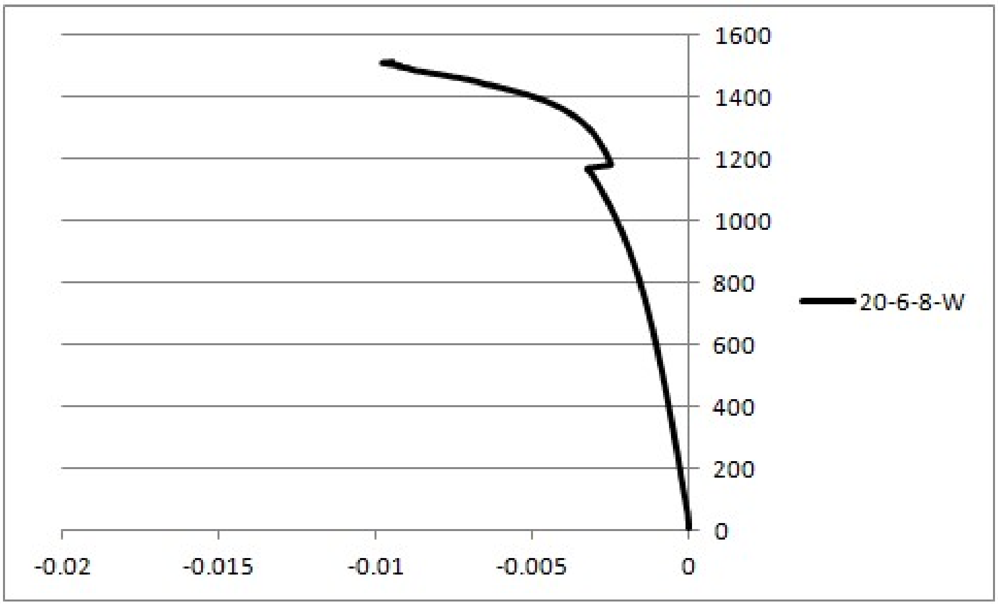


Figure A.10 Load-Strain curve for specimen 20-6-8-W

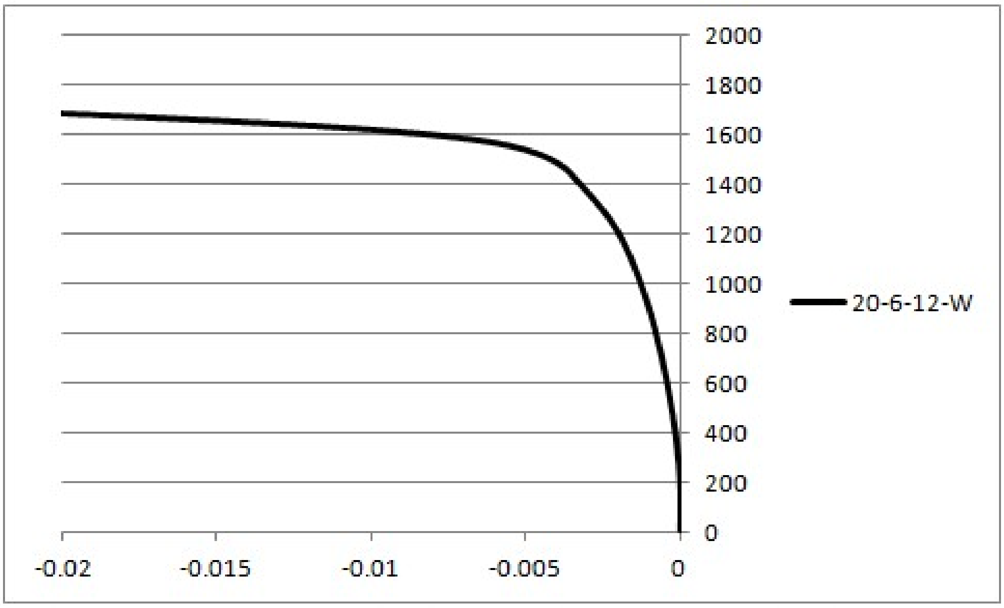


Figure A.11 Load-Strain curve for specimen 20-6-12-W

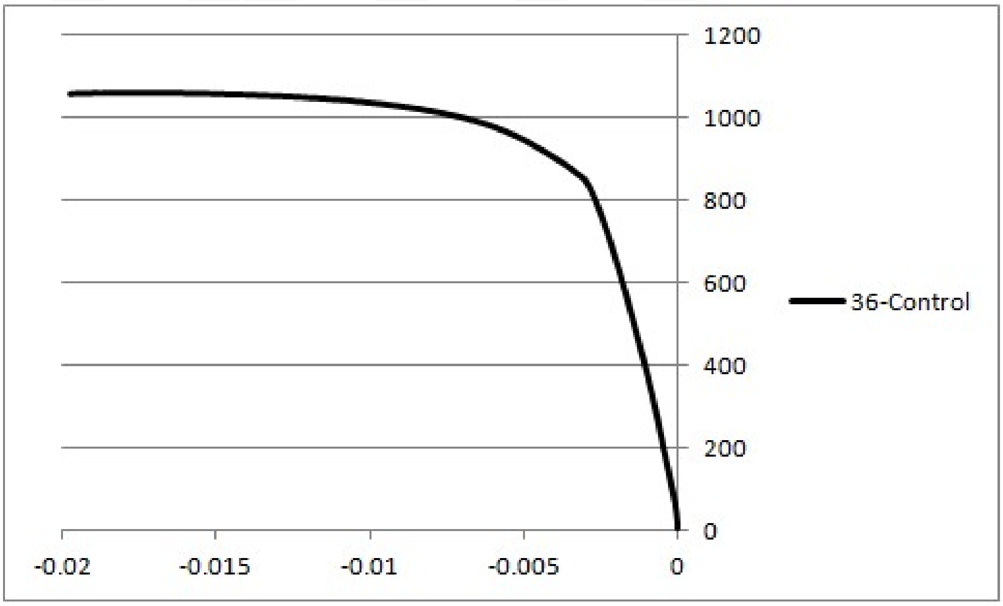


Figure A.12 Load-Strain curve for specimen 36-Control

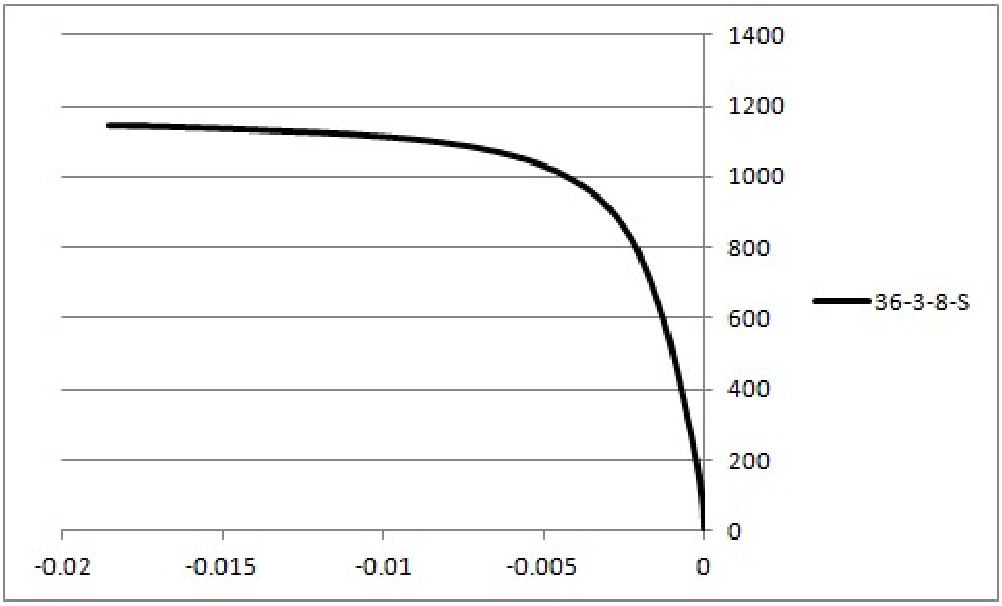


Figure A.13 Load-Strain curve for specimen 36-3-8-S

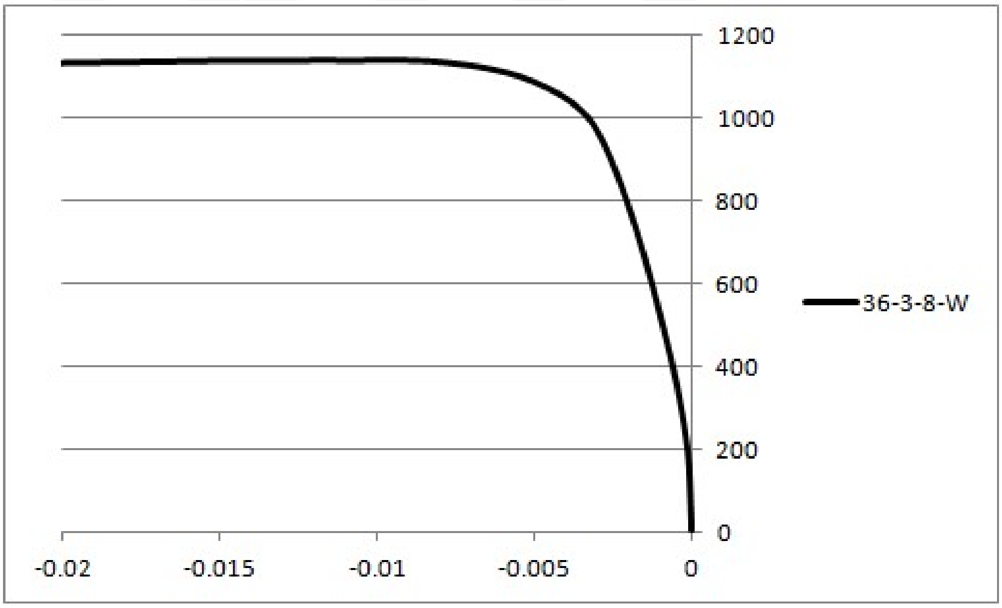


Figure A.14 Load-Strain curve for specimen 36-3-8-W

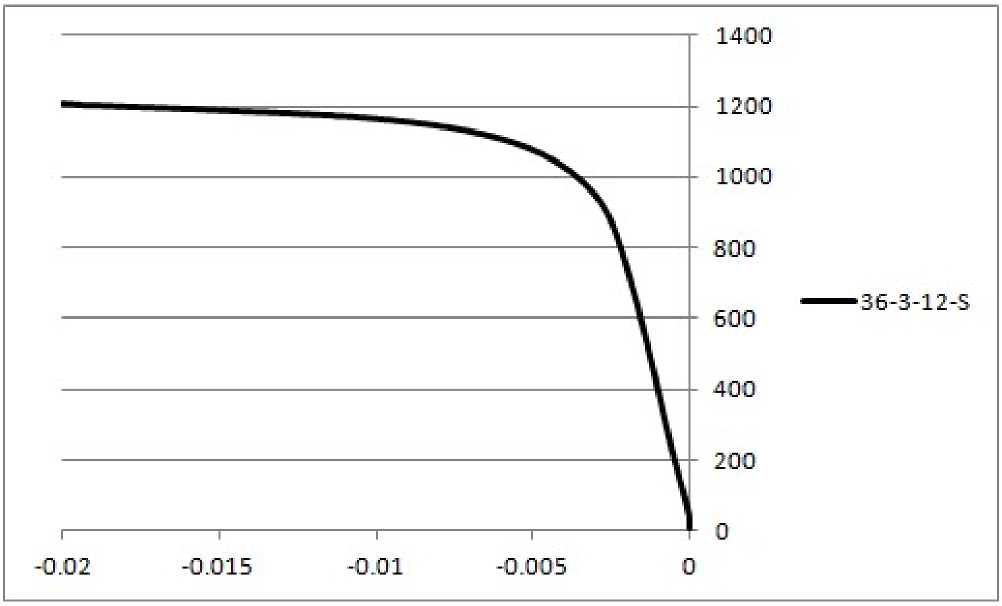


Figure A.15 Load-Strain curve for specimen 36-3-12-S

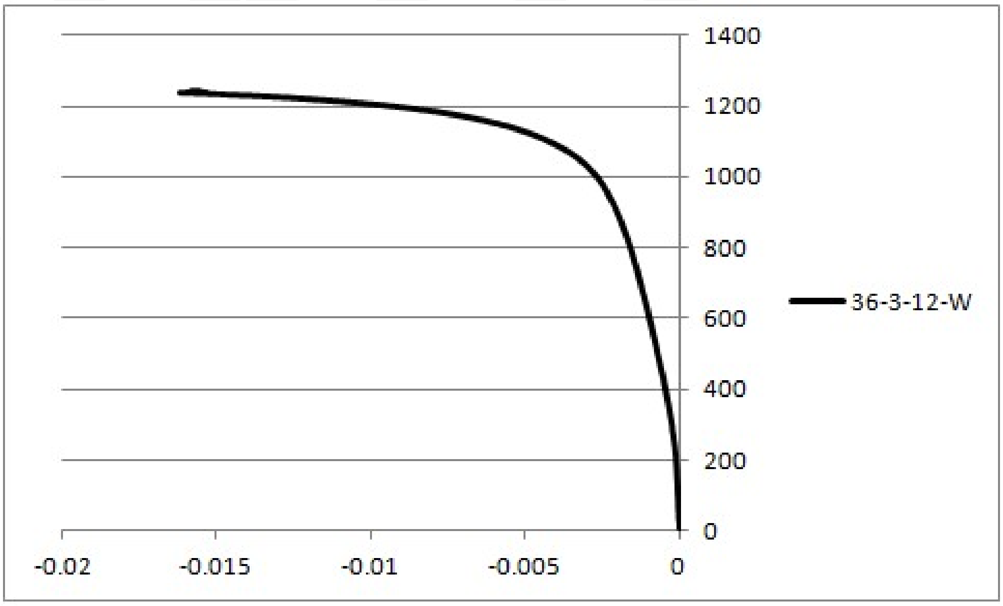


Figure A.16 Load-Strain curve for specimen 36-3-12-W

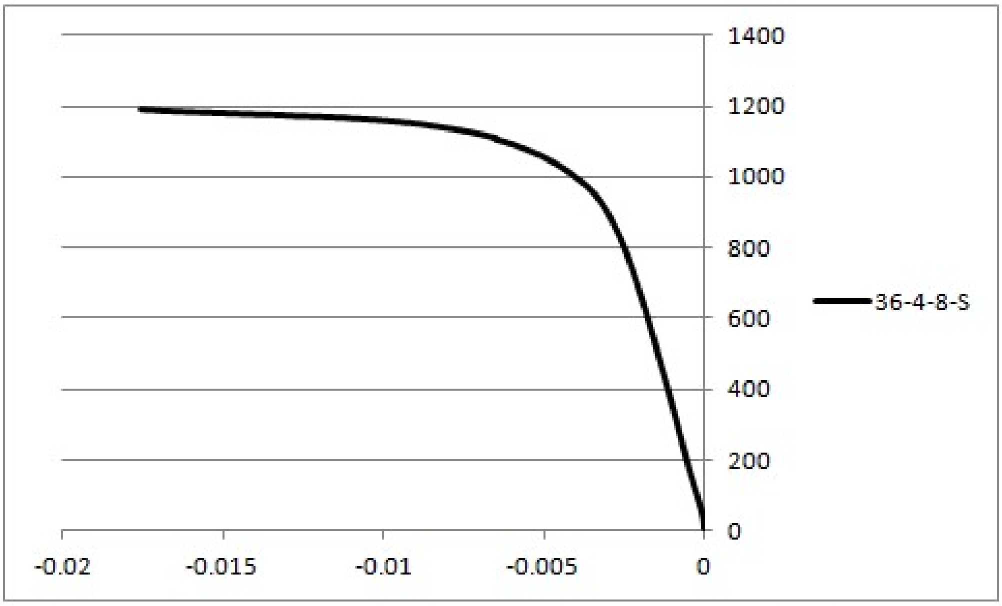


Figure A.17 Load-Strain curve for specimen 36-4-8-S

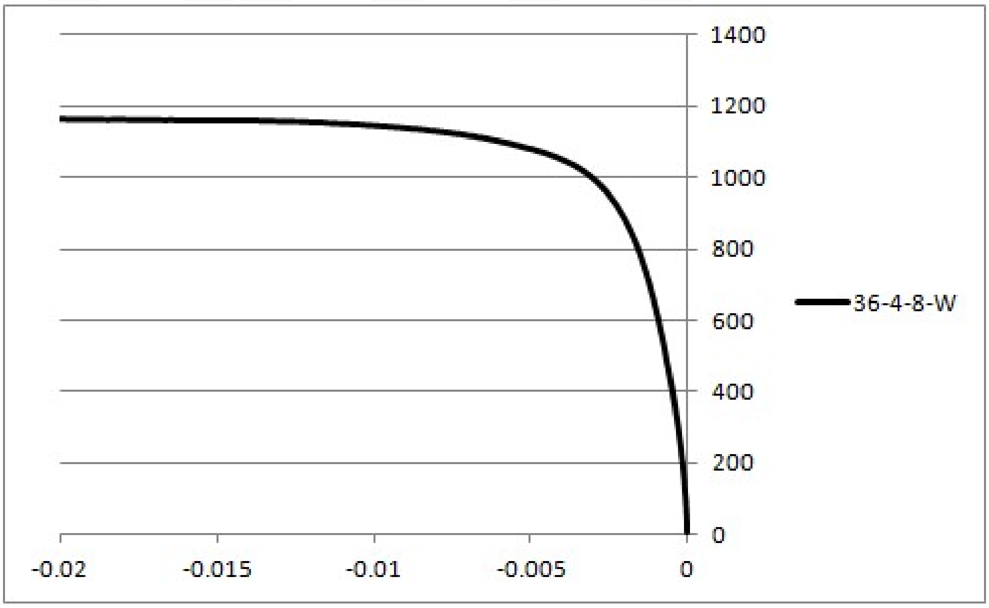


Figure A.18 Load-Strain curve for specimen 36-4-8-W

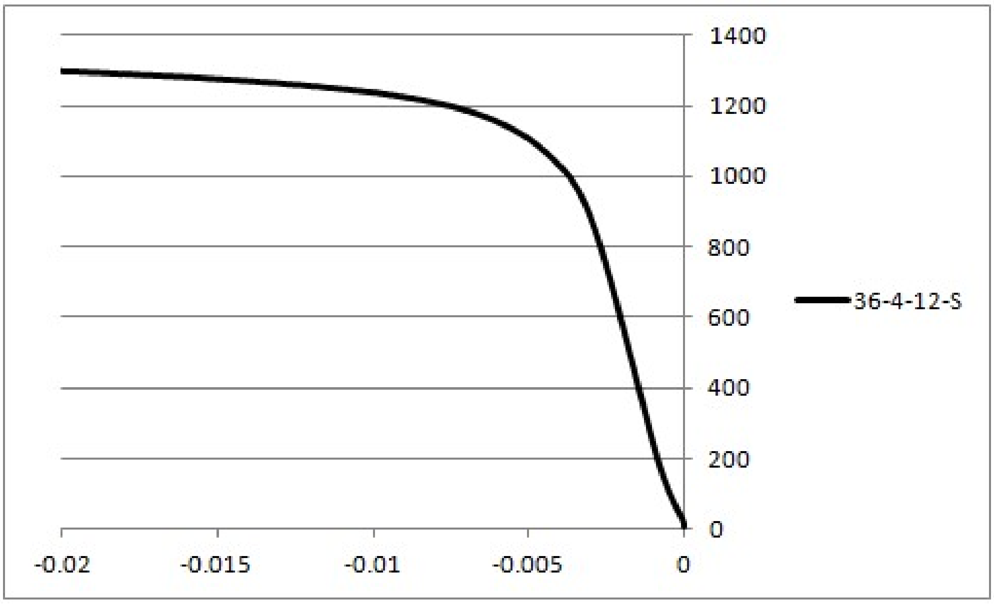


Figure A.19 Load-Strain curve for specimen 36-4-12-S

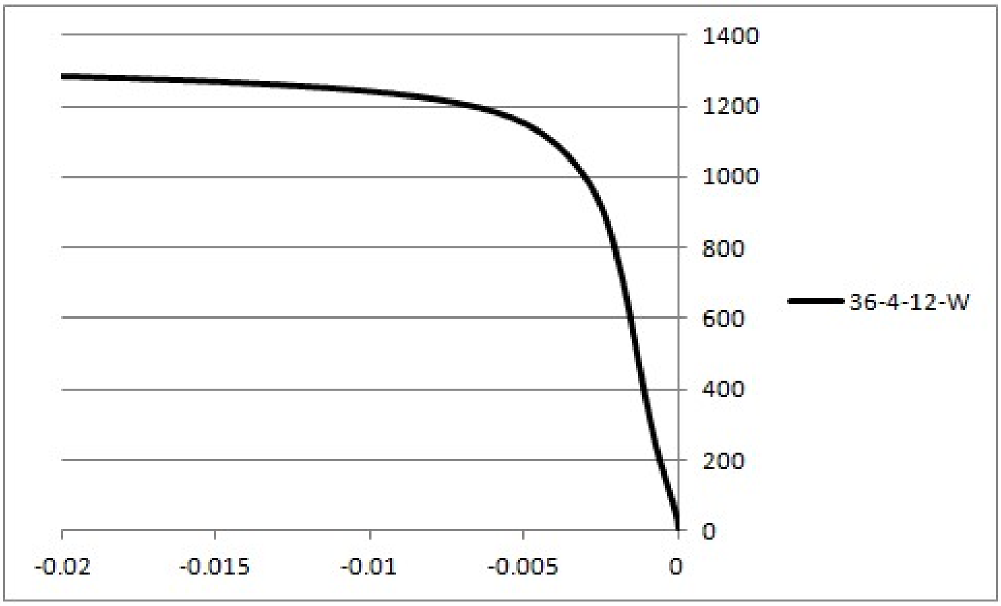


Figure A.20 Load-Strain curve for specimen 36-4-12-W

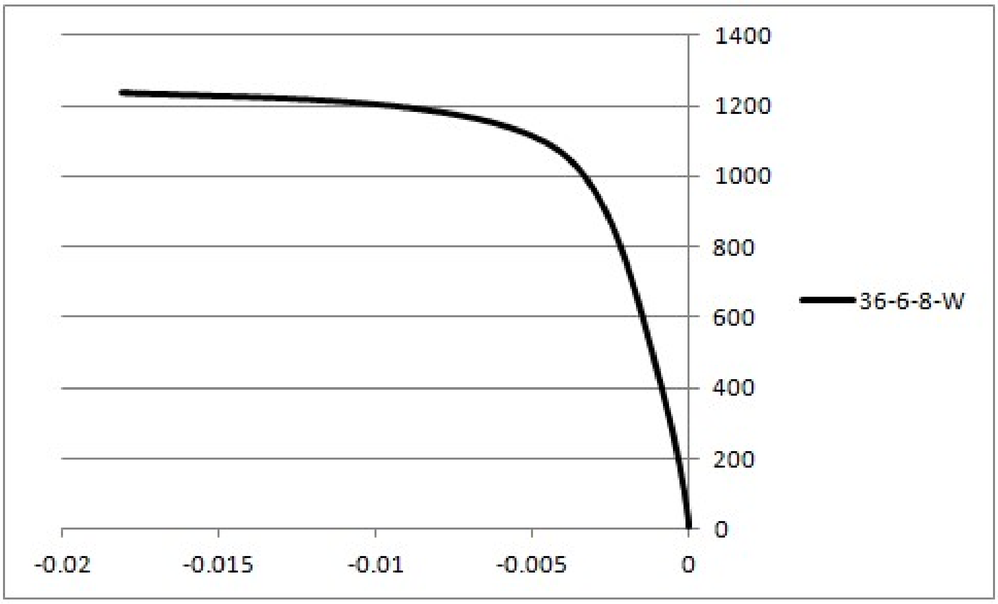


Figure A.21 Load-Strain curve for specimen 36-6-8-W

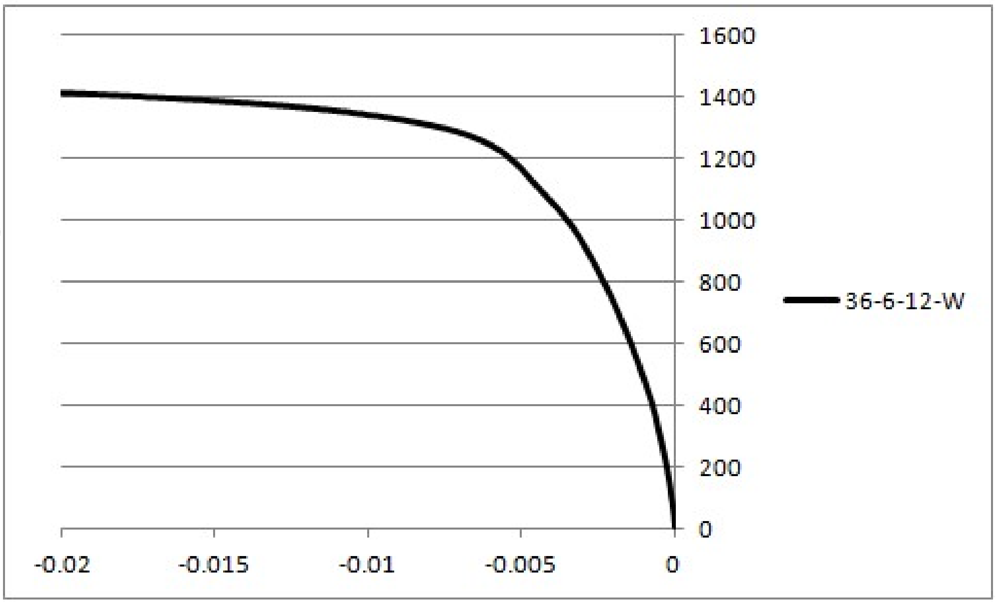


Figure A.22 Load-Strain curve for specimen 36-6-12-W