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AISC 360-16 AND EC4 APPROACHES FOR CONCRETE FILLED STEEL TUBE STUB COLUMNS

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Hussein Alaa Mohammed Saeed AL-JUBOORI

ABSTRACT

AISC 360-16 AND EC4 APPROACHES FOR CONCRETE FILLED STEEL TUBE STUB COLUMNS

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Composite members are used widely in civil engineering structures. The steel and concrete act together to resist the loads. Composite columns are a significant application of composite construction, and the Concrete-Filled Steel Tube (CFST) columns are the most common type of composite columns. Circular CFST column provides much more effective confinement to the core concrete than other types of column sections under axial load due to an enhancement of composite action between steel tube and core concrete. Many design specifications are used to predict the capacity of CFST columns, such as AISC 360–16 and the Eurocode 4. The AISC 360–16 is the specification for steel structures in the United States; the Eurocode 4 is the European code for composite structure design, respectively. The objective of this thesis is to investigate the differences between the AISC 360-16 and the Eurocode 4 approaches of circular CFST columns under axial load and to evaluate how well they model the actual column behavior through a series of statistical comparisons. Also, the parameters which are used in design specification calculations steps will be assessed. The important parameters in calculations will also be specified to underline the appropriate way in the design field. In present study, A total of 256 column models, where various structural parameters were varied to investigate their combined effect: concrete compressive strength f_c' taken as 30 MPa, 50 MPa, 70 MPa and 90 MPa were considered as ranged to cover normal and high strength concrete, steel tube yield strength f_y varied as 235 MPa, 315 MPa, 395 MPa and 475 MPa to cover mild and high tensile strength steel, D/t ratio taken as 20, 40, 60 and 80, and L/D ratio taken as 3, 6, 9 and 12 to cover short and long column with constant diameter of the composite column as 200 mm.

Keywords: Composite columns, CFST column, compression capacity, AISC 360-16, EC4.

ÖZET

İÇİ BETON DOLDURULMUŞ ÇELİK TÜP KOLONLAR İÇİN AISC 360-16 VE EC4 YAKLAŞIMLARI

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Tez Yöneticisi: Yard. Doç. Dr. Talha EKMEKYAPAR
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Kompozit elemanlar inşaat mühendisliği yapılarında yaygın olarak kullanılmaktadır. Bu elemanlarda çelik ve beton yüklere karşı direnç göstermek için birlikte çalışırlar. Kompozit yapılarda kompozit kolon uygulamaları önemli bir yer tutar ve içi beton doldurulmuş çelik tüp kolonlar kompozit kolon uygulamarının en yaygın türüdür. Çelik ile beton arasında daha iyi bir etkileşim ortaya çıkardığı için dairesel tüpler ile oluşturulan kompozit kolonlar daha iyi bir sargı etkisi sağlarlar. İçi beton doldurulmuş çelik tüp kolonların kapasitesini tahmin etmek için AISC 360-16 ve Eurocode 4 gibi birçok tasarım şartnamesi kullanılır. AISC 360-16 Amerika'da çelik yapılar için kullanılan bir şartname olup, Eurocode 4 ise Avrupa'da kompozit yapılar için kullanılan bir şartnamedir. Bu tez çalışmasının amacı eksenel basınç yüklerine maruz kalan kompozit kolonlar için AISC 360-16 ve Eurocode 4 kapasite tahminlerini karşılaştırmalı olarak araştırmak ve istatiksel olarak kolon davranışlarını nasıl modellediklerini ortaya koymaktır. Ayrıca, tasarım şartnameleri hesaplamalarında kullanılan parametreler de değerlendirilmektedir. Uygun tasarımlar elde etmek için Önemli parametreler belirlenecektir. Kolon çapı 200 mm olarak sabit tutulurken, beton mukavemeti f_c' normal ve yüksek mukavemetli betonları kapsayacak şekilde 30 MPa, 50 MPa, 70 MPa ve 90 MPa seviyelerinde değişken, çelik tüp akma mukavemeti f_y normal ve yüksek mukavemetli çelikleri temsil etmek için 235 MPa, 315 MPa, 395 MPa ve 475 MPa seviyelerinde değişken, D/t oranı 20, 40, 60 ve 80 seviyelerinde değişken ve son olarak kısa ve uzun kolonları kapasayacak şekilde L/D oranları 3, 6, 9 ve 12 olarak değişken alınmıştır.

Anahtar kelimeler: Kompozit kolonlar, İçi beton doldurulmuş çelik tüp kolonlar, Basınç Kapasitesi, AISC 360-16, EC4.

To all my dears, especially:

My parents

My wife and son

My sister

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TABLE OF CONTENTS

LIST OF FIGURES

Pages

LIST OF TABLES

Pages

LIST OF SYMBOLS/ABBREVIATIONS

CFST	Concrete-filled steel tube
CFDST	Concrete-filled Double Skin steel tube
AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
EC4	Eurocode 4
ASD	allowable stress design and plastic design
LRFD	Load Resistance Factor Design
ACI	American Concrete Institute
AIJ	Architectural Institute of Japan
AS	Australian Standards code
SANS	South African code
NBR	Brazilian Code
CAN/CSA	Canadian Standards Association
A1085/A1085M	Standard Specification for Cold-Formed Welded Carbon Steel
A_c	Cross-sectional area of concrete
A_s, A_a	Cross-sectional area of steel
A_{sr}	area of continuous reinforcing (in present study equal to zero).
A_g	gross area of composite member
σ_{sz}	The axial stress of steel tube in yield condition
σ_{ccB}	the strength of Confined concrete
σ_{scR}	ultimate compressive stress of square steel tubes

Υ_u	the strength reduction factor for concrete
CHS	Circular hollow specimen
HSS	Hollow structural sections
f_c'	Compressive strength of standard concrete cylinders
f_{ck}	Compressive strength of prism
Fy	Yield strength of steel
$\lambda, \lambda_p, \lambda_r$	slenderness ratio
D	Outer diameter of the column
L	Length of the column
t	Wall thickness of the steel tube
P_n	the compressive load capacity
P_{no}	the Nominal compressive capacity of composite section
P_p	the plastic strength of the composite section
P_y	the yield strength of the composite section
P_{uo}	the sectional capacity or squash load
P_e	the Euler critical load
F_{cr}	the critical local buckling stress
E_c,E_{cm}	Elastic modulus of concrete
E_s,E_a	Elastic modulus of steel
EI_{eff}	The effective flexural stiffness
Lc	effective length of the member
K	effective length factor
I_c	moments of inertia of the concrete section
I_s	moments of inertia of the structural steel shape
C_1	coefficient for calculation of effective rigidity of encased

C_2	Edge distance increment (in present study equal to 0.95)
C_3	Coefficient for calculation of effective rigidity of filled
δ	the steel contribution ratio
$\bar{\lambda}$	relative slenderness ratio
$N_{pl,RK}$	Characteristic value of the plastic resistance of the composite section to compressive normal force
η_a	Coefficient of confinement for the steel tube
η_c	Coefficient of confinement for the concrete
Ncr	Elastic critical normal force
K_e	Correction factor = 0.6
X	reduction factor
$\varnothing$	Parameter
ξ	Confinement factor
SI	strength index
NSC	normal strength concrete
HSC	High-strength concrete
MSS	mild steel strength
HSS	high tensile steel strength

CHAPTER 1

INTRODUCTION

1.1 General

A composite steel-concrete column can be defined as a compression member, which may be either a concrete encased steel section or a concrete filled steel tubular section. Composite columns have been used for high-rise buildings as an alternative to ordinary reinforced concrete during the past decades. Recently, composite columns have been increasingly used throughout the world.

The concept, which is the reason for economical composite steel-concrete manufacturing, is utilization of best quality of each and every item i.e., steel and concrete. The major quality of composite construction is good merging of steel with concrete. The connection between steel and concrete makes flow of forces allows composite material to behave uniquely.

1.2 Concrete Filled Steel Tube (CFST) Columns

A concrete-filled steel tube (CFST) column is a structural member that uses a combination of steel tubes and concrete to provide adequate load carrying capacity to sustain either axial or combined loadings. This is partly due to their excellent earthquake resistant properties such as high strength, high ductility and higher fire resistance than conventional steel or concrete columns of the same size. The steel tube of column provides confinement so enhances the axial load capacity as well as the energy absorption capacity or ductility; and the concrete core ensures a restraining force for the steel tube against buckling. Moreover, steel tube also employs as a formwork, so reduces construction cost and speeds up the construction works. Because of these exclusive features of the CFST columns, they are still one of the most important research issues in civil engineering field, to deceive their behavior under different types of loading. The geometrical parameters e.g. diameter – thickness ratio, length – diameter ratio, steel tube shape, material's properties and axial load applied on the column are the most important factors affecting response of these columns (Sakino et al.2004, Hu et al. 2005).

1.3 Applications of CFST Columns

The CFST column has a long history. In China, concrete filled steel tubes as columns, were first used in the construction of the Beijing Underground Railway Station in 1963. Later, they were used in some other projects as columns under heavy loading such as bridges and viaducts. Large arch bridges of this type have been constructed over rivers with the through span of 140 m and a rise of 22 m, pipes of diameter 114 x 4 mm filled with concrete were used as stanchions.

Several research since 1970, have proved that CFST column based farming architecture and H-shaped beams are more beneficial than normal protected concrete and steel system. The mentioned system is being widely used in sky-scrappers and mid-range buildings in Japan. Japan's Ministry of Construction offer was gained by five common contractors and builders. This offer was given to build urban houses in 21^{st} century. An experimental project is initiated by Building Research Institute (BRI) for five-year duration. The project was named as New Urban Housing Project (NUHP) which ultimately paced up the analysis of the system. In 1993, another project was initiated which was continued for five years. This project focused on hybrid structures as a 5^{th} phase of cooperative earthquake research program of US-Japan. This project also included CFST column system. Results of the mentioned project used to present current designs for CFST column system.

Steel-concrete composite columns have several obvious advantages in terms of architecture and structure point of view. These advantages are highly admired by modern constructor, architect, and engineers. These feature have been utilized since decades in the construction of buildings. However, recent advancements show that applications have prominently increased. Below are some features proposed by architects and builders:

- Hollow sections filled with concrete have more firmness and load bearing capacity. Because of this, esthetic slender columns may endure greater loads without incrementing exterior dimensions. Reinforcing bars may be utilized to improve the load bearing strength.

- As the exterior surface of the columns is relatively smaller, therefore, cost of painting to avoid corrosion may be reduced in a large amount. Corrosion avoidance may include sprays etc. steel architectures are available in different shades and colors.
- Sophisticated technique of structural engineering may reduce the several problems. Such assemble techniques allows fabrication process in workshops.

In ancient times, structures were built with stone, timber and brick masonry. The use of reinforced concrete, modern structural steel, pre stressed concrete and recently developed composite (CFST) sections give the opportunity for construction and development of long span sections. In long span bridges the natural vibration response of the arch bridge plays an important role in the seismic design of arch bridge, which may include natural frequencies of vibration and mode shape of arch bridges in structural dynamic analysis. For longer span bridges the lateral direction vibration is more and hence the arch rib is required to be stiffer, stronger and durable. This is satisfied by the concrete filled steel tubular circular sections.

many codes are designed from numerical analysis and experimental results to propose a design principles of composite elements. The major two guides are Eurocode 4 and AISC360. Eurocode is European integrated design standard supporting composite structures. However, AISC360 is the American based code for structures of steel.

1.4 Scope of Thesis

The aim of this thesis study is, to investigate the differences between the AISC 360-16 and the EC4 approaches of circular CFST columns under axial load and to evaluate how well they model the actual column behavior through a series of statistical comparisons. Also, the parameters which are used in design specification calculations steps will be assessed. The important parameters in calculations will also be specified to underline the best way in design field.

1.5 Thesis Organization

This thesis will be organized with the following sections:

Chapter 1: introduces the problem and the aim of the study.

Chapter 2: presents a brief review for the previous researches, which dealt with the problem.

Chapter 3: Design Specifications & Limitations According to AISC 360-16 & EC4 codes for circular concrete filled steel tube columns are presented in this chapter.

Chapter 4: in this chapter the axial load carrying capacity of CFST columns is calculated according to AISC 360-16 & EC4 codes and compared it.

Chapter 5: The conclusions drawn from the analytical study presented in this thesis are summarized in this chapter.

CHAPTER 2

LITERATURE REVIEW

2.1 General

Major part of this chapter deals with an objective review of the previous research work on concrete filled tube columns. Firstly, we will discuss the development and the advantages of using composite columns, the buildings made from composite columns will also be presented. Subsequently, for a comparison purpose the existing design codes of the axial capacity of concrete filled steel tube (CFST) will later be discussed.

2.2 Composite Columns

Composite columns are defined as a combination of concrete and one or more steel sections in a compression member, in such a way that the resulting arrangement operates as a single item (Berge, 1998). The major aim of the composite construction can be expressed as concrete is very good in compression and steel is very good in tension and by joining the two materials together structurally these strengths can capitalize to result in a light weight and very highly efficient design. Moreover, they exhibit a higher level of performance compared when the two materials functioned separately (Nethercot, 2004).

2.2.1 History of Composite Columns

The first application of composite columns in construction extends back to year 1940. But the year 1970 is marked the evolution of concrete filled steel tubes in many countries around the world. And most of those countries have developed a design code and specifications for use of CFST. These specifications have created either from the steel or concrete design approaches (Nethercot, 2004). Since its introduction, the usage of composite action has been recognized as an effective way of enhancing structural performance.

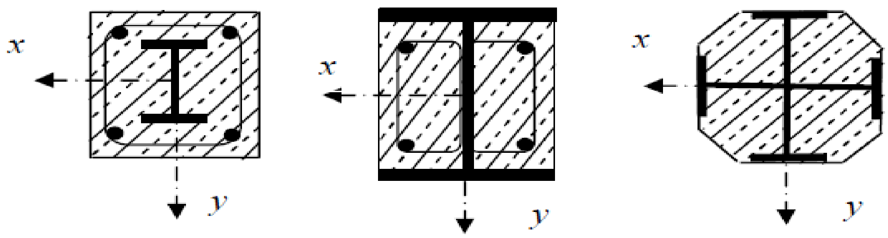
Over the last 40 years a significant amount of work has been aimed for better understanding of composite columns especially concrete filled steel tube columns. What attracts the attention of research workers all over the world is that CFST columns can replace the standard structural columns such as reinforced concrete structures, structural steel that contain reinforced concrete and the structural steel alone with an improved performance and at the same time reducing construction cost to the minimum. And that is why it is particularly very useful in tall buildings where high strength is necessary and the flexibility of the open space is needed for the maximum range of applications (Ketema, 2005).

At the present time, the composite structural elements are extensively used in tall buildings, bridges and other types of structures all around the world, because of its speed of construction and a light weight it has, which also reduces the size and cost of structural foundations. Moreover, when we use steel with concrete we take the advantage of both steel and concrete properties since concrete supplies its inherent mass, stiffness, and economy, while steel contributes its, strength and light weight (Hull, 1998).

2.2.2 Types of Composite Columns

The two major types of composite columns are concrete-encased (either partially or fully) and concrete filled steel tube (CFST) (Berge, 1998).

Concrete encased columns have a structural steel component that can be either one or more rolled steel sections. The concrete encasement improves the behavior of the structural core by stiffening it, and making it more effective against both the local and overall buckling (Ketema, 2005). The concrete encasement can be full encased or partial encased as shown in Figures 2.1a and 2.1b.



a) b)

Figure 2.1 Concrete encased columns; a) Fully encased and b) Partially encased (Ketema, 2005).

Concrete filled steel tube contains a steel hollow tube of different shape (circular, square, rectangular, etc) filled with concrete or concrete with transverse and longitudinal reinforcement (Ketema, 2005). Figure 2.2a and 2.2b shows the cross-section of CFSTs.

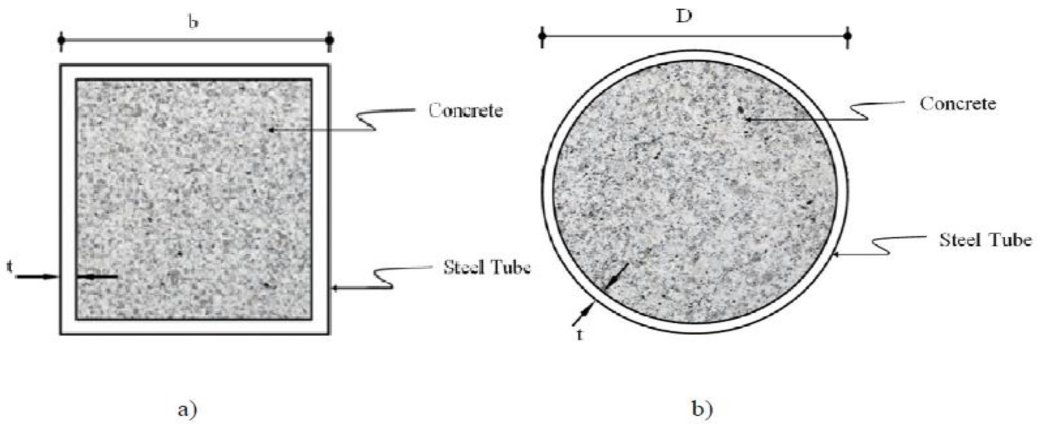


Figure 2.2 Concrete filled steel tube; a) Square shape without reinforcement and b) Circular shaped CFST without reinforcement (Ajel and Abass, 2015).

In this thesis, only circular concrete filled steel tubes (CFST) columns without any reinforcement bars will be discussed.

Several experimental researches about CFST have been carried out. The main study parameters of these experiments include the slenderness ratio (length/diameter), the section slenderness ratio (diameter/thickness) the strength in filled concrete (high, normal or lightweight concrete), yield strength of the steel, the manner of loading application (loading the entire system, the steel section or the concrete core only), the loading condition (axial compression, concentric or eccentric, cyclic loading, pure bending, etc).

2.2.3 Merits of CFST Column

CFST have a several distinct advantages rather than using other types of columns**.** The main advantages are listed below:

1. In CFST columns, the steel tube confines the concrete and while the concrete delays the incident of local buckling of steel tube leading to enhance the axial and flexural capabilities. The steel tube also confines the entire concrete core in the

circumferential direction that leads to an excellent energy absorption capacity (Hu, 2011).

2. Steel tube and concrete core are fully compatible and complementary to one another as they have more or less the same thermal expansion (Ketema. 2005).
3. CFST members are much more economical than the steel members due to the reduced usage of steel. The steel tube can be used as a formwork for casting the concrete and that leads to cost savings in the construction process (Hu, 2011).
4. Composite construction, particularly those uses CFSTs, allows very rapid construction as there is no need for a long-term formwork construction. Moreover, waiting time for curing of concrete is not necessary as the constructions of the upper storey can continue before the curing of concrete in the lower storey (Ketema, 2005).
5. Concrete inside the steel tube can enhance the fire resistance of the structure so that the need of fire proof material can be reduced (Morino and Tsuda, 2003).
6. The steel section can replace the function of transverse confinement and longitudinal reinforcement by carrying both longitudinal and transverse loads and at the same time providing a confinement. In conclusion, the usage of transverse and longitudinal reinforcement may be eliminated (Ketema, 2005).
7. By using CFST the ductility requirement for earthquake resistant design is easily achieved when compared with the common reinforced concrete columns. The vibrations that earthquakes and winds caused can also be reduced due to its higher rigidity (Ketema, 2005).

2.2.4 Building Constructed with CFST Columns

In recent decades, the construction of the CFST structures has increased rapidly. Several buildings have been constructed with using CFST columns around the world. Some examples are presented below. Figures 2.3 to 2.10 also show such examples.

1. Millennium Tower in Vienna - Austria (Choudhary, www.Civildigital.com).
2. Parking deck “DEZ” in Innsbruck - Austria (Choudhary, www.Civildigital.com).
3. Redevelopment of the Hilton Hotel - Cheung Kong Center in Hong Kong-China (Wong, 2003).
4. International Finance Center Tower 2 in Hong Kong - China (Wong, 2003).

5. Composite multi-storey steel-framed constructions (Building used for commercial located in London and car industry factory building in Germany) (Şamhâl, 2005).
6. Ruifeng building in Hangzhou in China (Han et al, 2014).
7. SEG plaza in Shenzhen in China (Han et al, 2014).
8. Wangcang East River Bridge in China (Han et al, 2014).

Figure 2.3 Millennium Tower in Vienna - Austria (Choudhary, www.civildigital.com)

Figure 2.4 Parking deck “DEZ” in Innsbruck – Austria (Choudhary, www.civildigital.com)

Figure 2.5 Redevelopment of the Hilton Hotel Cheung Kong Center in Hong Kong – China (Wong, 2003)

Figure 2.6 International Finance Center Tower 2 in Hong Kong – China (Wong, 2003)

Figure 2.7 Composite multi-storey steel-framed construction (Building used for commercial located in London and car industry factory building in Germany) (Şamhâl, 2005).

Figure 2.8 Ruifeng building in Hangzhou in China (Han et al, 2014)

Figure 2.9 SEG plaza in Shenzhen in China (Han et al, 2014)

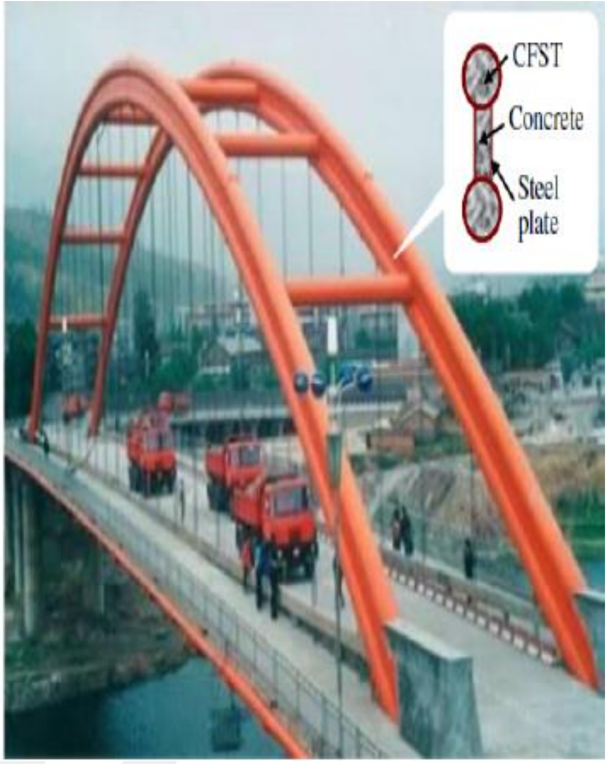


Figure 2.10 Wangcang East River Bridge in China (Han et al, 2014)

2.3 Behaviour of the CFST Column

The part the primarily loads in compression is a structural element which is called a column, it loads in compression along with its length. A column can be combined with a bending moment or eccentric load; it also can be subjected to compression forces. The compression of axial force results in a failure of short compression columns, compression members in a longer columns case are affected by the material strength and the material stiffness, also the geometry of the members affects it too.

In the terms of resisting compression loads a section is known to be very efficient, this section is called the steel hollow section, in industrial buildings the steel hollow section has been widely used in case of framed structures, in addition the development technology progresses, the capacity of the structures is not only expected to increase by the concrete infill in the hollow section, but also to increase its overall fire resistance.

The ultimate strength of the CFT columns may have considerably effected by several factors, thickness, cross-section shape, slenderness ratio, and the steel and concrete mechanical properties (Zeghiche and Shakir-Khalil, 1989, Romero et al., 2005). The CFST members which are compression axially loaded, in general can fail in main ways: in terms of slenderness and material properties. In the short columns case, an important part is played based on the behavior of the mechanical properties. When the limit state of the steel tube is achieved about yielding and concrete crushing then the failure state is achieved, this case is known as a strength criterion. Furthermore, the slender CFT columns ultimate load capacity will be governed essentially by stability. As a result of second-order critical effects and buckling the members has more chance to fail (Gourley et al., 2001, Sakino, 2006); accordingly, the load of the critical buckling, which it represents the load for the column at which slender column buckles is more crucial. In slender columns the influence of the stiffness is noticed to be more than the usual, but that this is not the short columns case where the strength depends mainly on cross-sectional area and materials strength.

The most efficient section for compression members is the circular hollow section, as it has in all directions stiffness and an equal second-moment of area. Schneider (1998), Huang et al., (2002), shams and Saadeghvaziri (1997) researches carried out to acknowledge the ultimate axial strength of CFT columns is effected by the cross-section shape and thickness. The circular CFT columns that are short axially loaded was concluded to have a plastic behavior of elastic-perfect based on the studies, and also when compared to rectangular or square CFT column it offered more post-yield axial ductility. This was exhibited further by the circular column significant strain-hardening behavior, however many output behavior depending on the tube column wall thickness, were made in the rectangular and square tubes.

2.3.1 Diameter to Thickness (D/t) Ratio

The ratio of diameter to thickness (D/t) of the steel tube section is a significant parameter for composite columns. Circular CFST columns with a high value of the D/t ratio provide inadequate confinement for the concrete. On the other hand, circular CFST columns with a small value of the D/t ratio provide remarkable confinement for the concrete (Hatzigeorgiou, 2008).

2.3.2 Confinement

In the term of the column`s ultimate strength beside providing a huge advantages the ductility of CFT columns has a noteworthy relation to confinement. In the columns, the hollow steel tube provides passive confinement which produce a compressive confining stress (Johansson and Akesson, 2002). Furthermore, the effect of the confining, is basically controlled by a number of factors, like the steel tube dimension, the strength of the concrete, thickness of tube, the resulted stress of the steel tube, the column cross section overall shape (Young and Ellobodoy, 2006, Ellobodoy et al., 2006). The steel concrete interface confinement development is partly initiated by the poisson`s ratios difference in the structure between the materials. The smaller poisson`s ratio is found in the concrete, V_{conc} (0.15-0.25) than steel, during the elastic stage is (0.3), which therefore leads to lateral separation of the concrete core and steel tube. Furthermore, it’s noticeable that, V_{conc} get higher than steel, especially when the crack and crush starts in the concrete core, in addition, an interactive contact with the steel tube is reinitiated by the side distortion (Gourley et al., 2001). Development of radial pressure is resulted by the further loading and a hoop tension setting up in the tube. In this stage, confining pressure is provided by the steel shell to the expansion of the concrete in the concrete core, then it puts the steel in biaxial, and under the tri-axial stress the concrete. In like manner, the CFT steel tubes is expected to enhance by its confinement the concrete core found in the axial compressive strength. High stress and strains can be withstanding by the tri-axial compression concrete in the columns (Grauers, 1993).

Fujimoto et al., (2004) and El Debs and Nardin (2007) investigated the confinement effected by the section`s shape. The study also confirmed, in the octagonal and circular CFT columns the confinement was found superior, however it was not found in square steel section CFT columns. This is due to lateral pressure uniform in a circular tube that encloses the concrete core; in the square section case, the corner and the center of the section has a higher confining pressure compared to the other side. Furthermore, consideration shows the ductility behavior of the square section is influenced by the thickness of the tube.

Lakshmi and Shanmugam (2001) carried out an investigation on rectangular tubes, the results of the investigation shows that tension hoop developed along the side of the tube was not stable, and this defines the reason why the confinement was only present in the corner of the steel tube and nowhere else. The analysis conducted by (Hu et al., 2003) shows the same conclusion about the tension hoop. The program of the finite nonlinear element, *ABAQUS* in their circular section studies that have been performed, also on square section, reinforcing ties stiffening on square section. Then in the circular CFT columns it is considered that the confinement is more significant rather than square CFT columns, where the column axial strength shows a small increment (Fujimoto et al., 2004). Debs and Nardin (2007) studies determine that in the square section the confinement was noticed to have more effect rather than the rectangular section, in the post-peak behavior a greater ductility represented it. Notably hence the circular section is more ductile than other shapes, it was still established. A good confining effect is provided by the CFT columns that has circular sections. Yet, when the thickness-to-width (D/t<40) is a small ratio, it was established a large confining effect was not provided by the square section specifically when the thickness-to-width (B/t>30) ratio was large. The columns with a large plate slenderness ratio the concrete confinement effects also were found to be less obvious, and may local buckling occur (Uy, 1998).

It was noted that the confining effect have extremely enhanced the short column strength, yet, in case of slender columns the effect was little (Bradford and Oehlers 1995). It`s mainly because of that a large L/D ratio column was found in general, fail because of the columns buckling before it achieved the indispensable strain to bring about an increase to the volume of the concrete (Han, 2002). Neogi et al. (1969) carried out an investigation that established due to a triaxial effect there was a strength gain for the L/D less than 15 ratio columns. A full interaction between concrete and steel in that study was presumed: so, both biaxial effects and triaxial effects were not considered. Using a tangent-modulus approach the columns were analyzed thence, and with experimental results comparison.

2.3.3 Axial Buckling

It`s an instability and one of the structural behavior form, Which should be considered essential for compression members, for specimen, columns. Its existence clearly reduce the structure capacity. Eurocode 4 (EC4), for specimen, has forced a few constraints on the permissible diameter-to-thickness ratio, D/t with a specific end goal to keep local buckling from affecting structure capacity.

2.3.3.1 Local Buckling

Local buckling and overall buckling are the two types of the buckling. When the thin steel components and compacted in their planes the local buckling happens (Bradford and Oehlers, 1995). On account of this, also with a specific end goal to completely utilize the strength of steel, before the yield stress is reached failure mode should be blocked. In a thin walled steel tubes case, the local buckling influence is important to its strength. Bulges growth, ripples or waves indicates this type of failure. In the CFT columns case, the concrete infill presence has an effect in elongating the local buckling of the tube wall (Schneider, 1998, Baig et al., 2006). Further, in a local buckling of steel the resistance owing to the concrete presence was firstly discovered by Matsui (Uy, 1998). Beside the delaying the local buckling presence, the CFT concrete core has been established also to increase the structure`s ductility significantly. By preventing the inward buckling it also joins stiffness to the steel tube, as the buckling modes is forced outward by the concrete core, based on that, the strength and stability of the structure is increased (El Debs and Nardin, 2007, Hu et al., 2003, Hu et al., 2005). As an outcome of the inward buckling prevention by the concrete core, the buckling capacity increment was found almost 50% more when a comparison with the hollow steel column is made, as mentioned in (Lakshmi and Shanmugam, 2001). The concrete core contribution in the inward local buckling occurrence delaying could ensure that the steel would achieve its longitudinal yield strength before buckling happens (Chaoui and Zighiche, 2005). Owning only outward mode failure mechanism grants another advantage, which is significant decrease prevention in the modulus section. This is due to the reality of the distance between the top flanges and the bottom flanges of the steel not decrease, instead it increases, as without when local buckling occurs in the concrete core. The confinement in concrete filled columns is affected by the local buckling presence when concrete crushing causes failure where the confinement effect is limited due to the prevention of the tube sections from providing a continuous restrain on the concrete for confinement. Yet, the confinement of concrete can develop if there is an inelastic local buckling (Uy, 1998).

Schneider (1998) conducted an experiment that proved in square and rectangular section a very similar local buckling behavior is established. Buckling effects, the occure equally on each face of the square tube, while in rectangular CFT due to the fact that the rectangular CFT has a wider face, it owns an extensive local buckling. Furthermore, the wall buckling in Circular CFT has been found to be due to a radial expansion of the tube. In general, a larger D/t ratios tube has additional capability of local buckling with clear high distortion in comparison with small D/t ratios sections.

2.3.3.2 Overall Buckling

In general, overall buckling occurs in slender or long columns. The sideways bending illustrates failure where, for columns specially, global stability is at greater risk, as owing to flexural buckling the columns tends to failure (Bradford and Oehlers, 1995). When there is between the internal and external forces a condition of a stable equilibrium the failure is no longer possible (Lakshmi and Shanmugam, 2001). the slenderness ratio influences these columns behavior, with the contribution of slenderness to the seemingly to be called second order effects.

2.3.4 Modes of Failure

The crushing of concrete and yielding of steel characterize the failure mechanism of a stocky or short columns, however the CFT columns with a medium length has inelasticity behavior and thus due to the partial yielding of steel it fails, in compression, the concrete crushing, and concrete cracking in tension (Lakshmi and Shanmugam, 2001). In the end, by the elastic limit the slender columns are bounded. As the strength limit of the material dominates the short columns capacities, the structures is identified by material-dependent.

The method of loading conditions significantly influences the stub CFST columns buckling mode. Gylltoft and Johansson (2002) studies on the various different loading conditions on columns subjected to the axial loading, namely steel tube loading, concrete core loading, and concrete and steel simultaneously loading. Based on the study, for the columns with concrete core load application and columns with the entire section load application, the stability of the columns was found affected by the crushing and local buckling combination. When simultaneous loading is applied, there was no effect by the bond strength on the behavior of the columns; but, the confinement effects was found to be significantly affected by the bond strength in the case of the load where applied on the concrete core. The columns overall behavior and overall strength was very influenced in this condition.

Long axially-loaded CFT columns with between 12.5 and 25.0 ratio of L/D, and 2-4 m length were tested by Chaoui and Zeghiche (2005). After reaching the steel yield strain with mid-length small-lateral deflections the columns failed as the results showed. Moreover, no local buckling sign was showed, no sign of mid-length sections concrete crushing was revealed by some specimen's steel envelope removal. The enhancement due to concrete crushing was highlighted also in this study is essentially more significant for short columns.

2.4 Composite Column Materials and Structure

In a material, strength and stiffness are the essential properties at most. Seward's (1998) study indicates that the strength of the material overwhelms the assurance of the collapse load of a structure, while its stiffness ensures that the structure is not going to deflect significantly, when it is under the load.

2.4.1 Concrete Infill

The strength for the concrete infill can be in the form of normal concrete having compressive strength < 60 MPa, on the other hand high strength concrete with a compressive strength > 60 MPa. Prominently, failure through concrete splitting may occur in case of concrete core with lower concrete strength, while on account of superior concrete strength, failure may be caused by concrete sliding. From the study between low and high concrete strength it shows that as far as stress-strain curves demonstrates that the type of concrete with higher strength has large elastic rigidity as the initial elastic modulus of concrete increase parallel with the increase in the concrete strength.

High-strength concrete (HSC) utilization presents a few preferences over normal strength concrete because of its greater stiffness. Study of Nardin and El Debs (2007) shows that increasing the concrete strength likewise brings about more strength capacity and decrease in the external dimension of the composite columns. As per Lue (2007) in most studies on CFT columns focus is put on circular and square sections having lower concrete strength. Therefore, for such study, the HSC is incorporated for investigating experimentally the elliptical CFST column behavior and it is caused to axial loading. Brittleness is real concern in regards to utilization of HSC and it is related to ductility. Hence, in high seismic zones, utilization of the HSC column is limited. In any case, by encasing the concrete core in a steel shell the HSC's ductility in columns can be essentially enhanced. As explained by Aboutaha and Machado (1998), it can give shear resistance and changes in overall ductility. Their study highlights different shortcomings in relation to the utilization of CFST columns; nevertheless, such contemplations are not addressed here.

Compaction is required to accomplish the end goal of satisfactory strength and durability of the concrete material. In general sense, it is noted that air void presence in concrete can reduce its strength and in this manner increases its permeability, which thus decreases durability of concrete. To dispensing the air voids that are caught in loose concrete compaction is used. In (2004), Han and Yao led a study on compaction method's influence on CFST columns, which demonstrates that the overall strength of CFST members can be upgraded by superior compaction. Additionally, it showed compaction's significance for concrete core in CFT columns. This way it is given importance for any fresh concrete to satisfy the properties of good concrete, for instance keeping up its homogeneity.

2.4.2 Steel

Steel can be sorted into two types as per Zhang and Shahrooz's (1999) literature: normal strength tubes with < 400 MPa of yield strength, and high-strength tubes with more than 400 MPa yield strength. Steel properties are measured from the stress-strain curves of coupon analysis in tension.

2.5 Other Studies and Theories of Concrete Filled Steel Tube

Sakino et al. (2004) completed a five-year examine on CFST column system. 114 samples in total were arranged and examined for the study of centrally loaded hollow and CFST short columns. Four distinct parameters have been examined; the steel tube shape (circular and square), tensile strength which ranges from 400 to 800 MPa, diameter to-thickness ratio, and concrete compressive strength which different from 20 MPa to 80 MPa. Design formulas were proposed by them for computing ultimate capacities of CFST columns with both square and circular samples. For an ultimate strength (Nu) of axially loaded circular CFST short columns, following formula has been proposed.

$$N_u = A_s \sigma_{sz} + A_c \sigma_{ccB} \quad (2.1)$$

Where A_c and A_s are the area of concrete and the area of steel tube at cross section, separately. In yield condition, σ_{sz} and σ_{ccB} are the axial stress of steel tube and the confined concrete strength respectively.

And the axial load capacity of square CFT short columns could be assessed by following equation: -

$$N_u = A_s \sigma_{scR} + A_c \Upsilon_u f'_c \quad (2.2)$$

Here σ_{scR} is square steel tubes' ultimate compressive stress, the minimum yeild value is taken, Υ_u is the strength reduction factor for concrete ($\Upsilon_u = 51.67D_c^{-20.11}$) and fc is the cylinder strength of concrete.

They additionally demonstrated that the distinction between an ultimate strength and the nominal squash load circular CFST columns could be surveyed as a linear function of the steel tube's yield strength.

Kuranovas et al. (2009) completed a study of load bearing capacity of CFST. Analysis of about 1303 samples of various types of CFST collected from existing test information was done. Comparison of test outcomes was done with the EC4 design strategy to calculate the bearing capacity of those composite samples. It was discovered that the behavior of hollow CFST samples was bigly more unpredictable than the behaviour of solid ones, because of complicated stress states that none of those stresses in the hollow concrete cores are equally dispersed throughout the thickness of its cross-section. They additionally expressed that Eurocode 4 was a protected indicator for capacity for all kinds of circular CFST and it could be safely utilized for concrete cylinder strength up to 100 MPa. In any case, for rectangular CFST columns, Eurocode 4 must be utilized carefully, when the concrete strength is more than 75 MPa, the experimental value turns out to be less than that anticipated by the EC4 prediction and a factor of 0.85 was recommended to utilize.

O'Shea and Bridge (2000) developed several design methods that could be used to evaluate the strength of circular thin-walled CFST under the different loading conditions. These loading conditions that were investigated were axial loading of the concrete, axial loading of the steel tube and instantaneous loading of the concrete core and the steel tube both axially and at a small eccentricity. Short test specimens were used with a length/diameter ratio of 3.5 and with a diameter/thickness ratio between 60 and 220. The concrete had nominal unconfined cylinder strength of 50, 80, and 120 MPa. It was discovered that the strength of unfilled circular tubes was extremely influenced by local buckling. In spite of the fact that the buckling of the square tubes could be upgraded by giving internal lateral restraint, this was not found in the circular samples that were inspected. Rather, the most outward buckle proceeded with unaltered by the internal concrete. They likewise found that the loading condition influenced the degree of confinement provided by a thin-walled circular steel tube to the internal concrete. The most extreme concrete confinement happens for the axially loaded thin-walled tubes with just the concrete loaded and the steel tube utilize as pure circumferential restraint. However, if there was a sufficient bond between the concrete and steel for axially loaded thin walled steel tubes, the local buckling of the steel tube did not take place. For a concrete strength up 80 MPa, Eurocode 4 arrangement could be utilized with no reduction for local buckling. What is more, for concrete strength that were more than 80 MPa, Eurocode 4 could be utilized, without having any improvement of the internal concrete from confinement and no reduction in the steel strength from local buckling and bi-axial effects. Subsequently, Eurocode 4 must be utilized for the design calculations of thin walled steel tubes filled with high strength concrete if care was given in the formuale of the design equation.

An experimental study was provided by Yu et al. (2007) regarding the circular CFST stub columns behavior with normal concrete (NC) and self-compacting concrete (SCC) that were centrically loaded in the axial compression up to the failure. 17 samples were investigated under concentrically loaded axial compression. The concrete strength effect, slots or notched holes, and furthermore the different loading conditions on the axial load capacity and the load deformation performance of the columns were investigated. All of the samples used in the study were three times the diameter in length to make sure that the samples would be stub columns and to reduce the end effects. It was found out that an expansion in the concrete compressive strength (SCC or NC) brought about an impressive increment in load capacity, however practically managed a consistent value in residual capacity after failure. It was likewise discovered that under the different loading conditions the sample's succession of the confinement effect changed. In contrast to samples where the load was applied to the whole section, the initial application of load to just the steel sections, the confinement effects became visible earlier but reduced, and the ultimate load capacity was roughly the same. Then again, when the load was applied initially to just the concrete section, the confinement effect appeared later yet especially enhanced, with a little bit expansion in the ultimate capacity. In any case, the residual capacity of the sample columns under the four loading conditions was hardly affected. Moreover, Eurocode 4 gave better prediction than the axial capacity of the unnotched CFST stub columns with SCC and NC when the loading of the entire section was done.

An experimental study was presented by Gupta et al. (2006) on circular concentrically loaded CFST columns' performance until failure. Almost 81 samples were checked to look at the diameter effect and diameter to thickness ratio on the axial capacity of the CFST columns. The concrete grade's effect and the fly ash volume in concrete were also inspected. The diameter to thickness ratio changed between 25 to 39 and the length to diameter ratio was in the range of 3 to 8. A concrete strength of 30 and 40 MPa were utilized as a part of the test. Also, composition percentage of 15%, 20% and 25% of a fly ash volume were utilized. The strength consequences of CFST were contrasted against the discoveries of Mander et al. (1988), Giakoumelis and Dennis (2004). To analyze the load carrying capacity of CFSTs, a nonlinear finite element model (ANSYS) was utilized. It was watched that the real and the numerically computed methods of the failure of various CFSTs were practically like one other. The load compression curves for the failure modes are shown in Figure 2.11. The curves depicted that the concrete strength was lessened with the expanding rate of fly ash up to 20% but at 25% of fly ash the strength was seen to be more than that at 15% and 20%. Toward the beginning of the failure procedure, a variation was identified amongst computational and the experimental deformations of CFSTs. The failure method of those samples with 50 mm diameter across CFST was seen to be Euler buckling. Moreover, the experimental deformed shape got matched with the computed one's deflected shape.

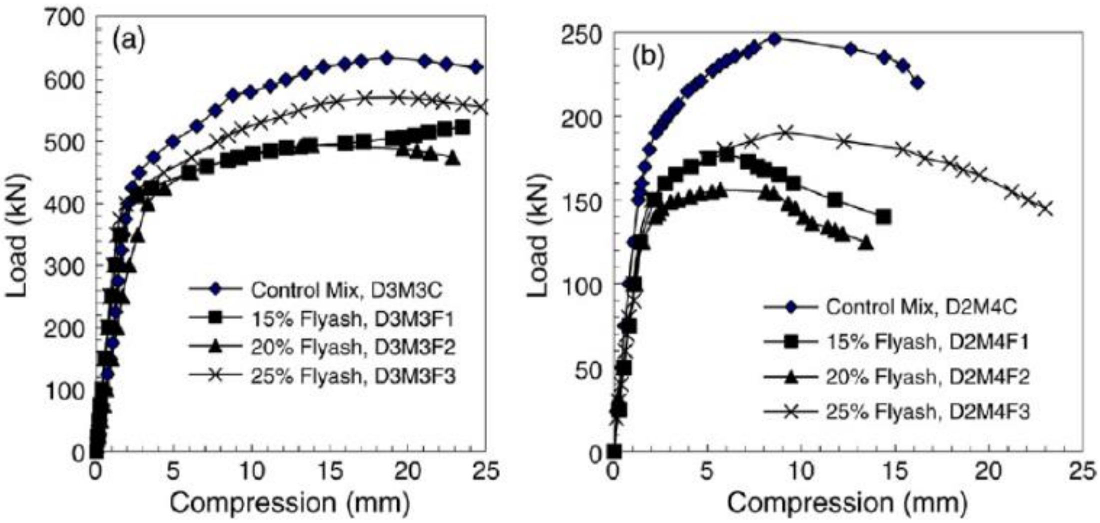


Figure 2.11 Load–compression curves of CFST for various concretes (a) 75 mm diameter and (b) 50 mm diameter CFST (Gupta et al., 2006).

Besides, for smaller diameter to thickness ratio, it was watched that steel tube gave an incredible confinement effect to concrete. In any case, from the bare steel tube outcomes, it was discovered that the axial capacity of the steel tube per unit volume decreases with an increase in diameter to thickness ratio, as given in Figure 2.12. They proposed that it was essential to fix the exact diameter to thickness ratio to make most ideal use of the material.

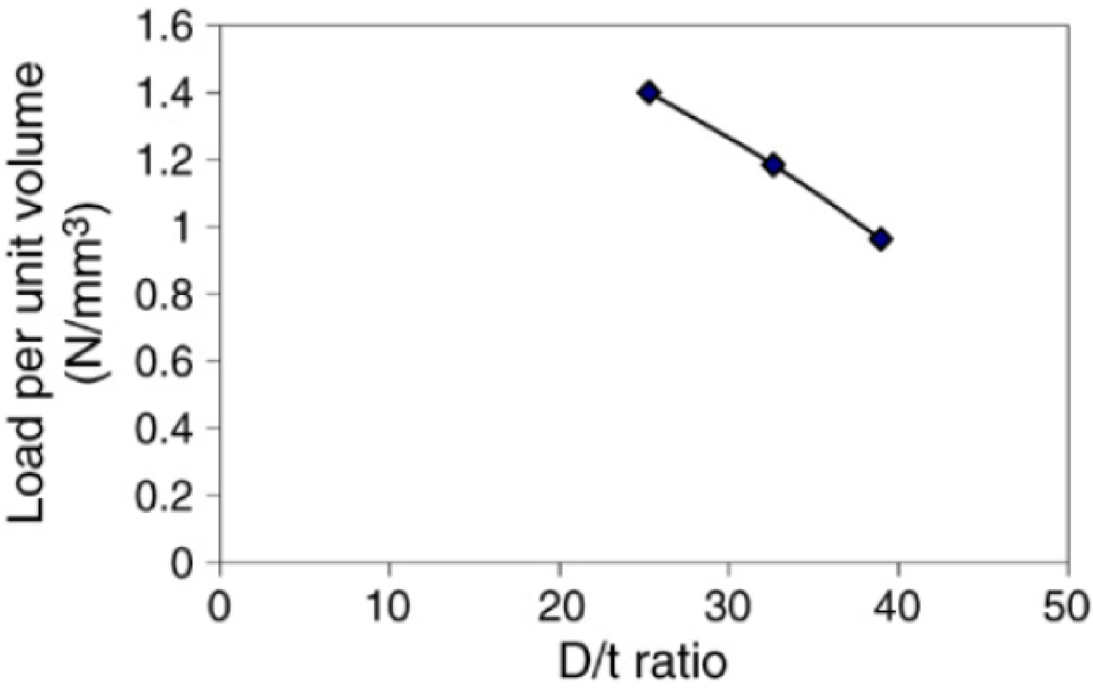


Figure 2.12 Variation of specific load with diameter to thickness ratio (Gupta et al., 2006)

Giakoumelis and Lam (2004) carried out an investigation on the behavior of circular CFST with different concrete compressive strengths under axial loading. The parameters of the study were the confinement effect, the bond strength between the steel tube, the steel tube thickness and the concrete core. 15 stub column samples in total were examined utilizing a concrete strength of 30 MPa, 60 MPa and 100 MPa and with a diameter to thickness ratio running from 22.9 to 30.5. Comparison of the test outcomes of column strengths was done with Eurocode 4 (EC4, 1998), Australian Standards (AS4100, 1998) and American Codes (ACI Committee 318, 1995) predicted values. It was observed that the concrete shrinkage effect was important for the samples that contain high concrete strength, however negligible for those samples with normal concrete strength. All of the codes predicted less values than those calculated during the experiments, however EC4 gave the best estimation of the axial capacity with respect to both CFST with normal and high concrete strength. The maximum difference between the predicted values on ultimate capacity of EC4 and the experimental value was 17%. However, for ACI and AS the anticipated axial capacity were 35% which was less than the experimental values. Additionally, the effects of the steel tube and concrete core bond turned out to be more critical with increase in the concrete strength. For the normal strength concrete, there was a very small reduction in the ultimate capacity of the column. In any case, for a high-strength concrete, there lies a difference of 17% between non-greased and greased.

Chitawadagi et al. (2010) studied the parameters that influenced the ultimate capacity and corresponding axial shortening of the CFST utilizing the tests' design (DOE) approach. The principle highlights considered were the diameter, steel thickness, the in-filled concrete strength and length of the steel tube. As much as 243 circular CFST samples were examined to approve the correctness of these models. To reduce the quantity of the analyses, Taguchi's method with L9 orthogonal array was utilized. The analysis of the outcomes of the experiments was done utilizing variance analysis to inspect the most affecting component on strength and axial shortening of CFST samples. It was found out that the steel tube's diameter had the most essential effect on the ultimate capacity and the consequent axial shortening of the CFSTs.

Abd el Fattah et al. (2012) explored the behavior of confined concrete strength components with confinement steel reinforcement or confining steel tubes as the instance of CFST have been attempted. They conducted an inclusive assessment of confined models for concrete columns under concentric axial compression that are accessible in the literature. The models investigated are sequentially introduced and looked at by an arrangement of criteria that survey thought of various factors in building up the models, for specimen, effectively confined area, yielding strength and ductility.

In 2016, Ekmekyapar and Al-Eliwi displayed test conduct of circular concrete filled steel tube columns and specifications for design. 18 samples were conducted on small, medium and long circular CFST column utilizing a concrete strength of 56, 66 and 107 MPa. Also, 239 test information were gathered from literature for the assessing AISC 360-10 and EC4 predictions within and out of the application limits. To investigate the effect of column parameters and effect of confinement, three L/D proportions, two D/t proportions, 3 concrete classes and 2 steel qualities were utilized. This paper looks at the column's Strength index (SI) diminishes as the core concrete strength increments. Along these lines, performance of confinement for columns is increased with decreased concrete compressive strength. Contrasting 107 MPa concrete, 56 MPa and 66 MPa concretes had better deformation capacity and ductility. Additionally, the confinement of thicker steel tubes with superior yield strength is better for 107 MPa concrete. Keeping in mind the end goal to acquire more ductile and smooth behaviors, this sort of concrete ought to be utilized with much thicker steel tubes. The Contribution of 107 MPa concrete to the compression load capacity was observed to be more for columns with thinner steel tubes. Un conservative expectations of EC4 for the most part were watched for columns with relative slenderness ratios < 0.4. Past the comparative slenderness estimation of roughly 0.4, conservative results were given by EC4. Nonetheless, aside from 5 tests, AISC 360-10 forecasts for every analysis information are conservative. With increase in column slenderness, predicted behaviors of AISC 360-10 and EC4 draw nearer. Albeit the two codes give conservative outcomes past a relative slenderness ratio of 1.0, AISC 360-10 plays out somewhat improved in this district and has predictions nearer to trial comes about for the samples considered in this discussion. As shown in this paper, L/D ratio and relative slenderness ($\bar{\lambda}$) are essential parameters for CFST columns and the column capacity is directly affected by them. Then again, D/t and confinement factor do not have straight effect on the behavior of CFST column.

Dundu et al. (2012) in this study explored the axial load capacity of 24 CFST columns and their behavior with various cross-sections utilizing a concrete compressive strength of 30 MPa and 40 MPa were researched. The comparison of test outcomes of column strengths was done with predicted values of Eurocode 4 (EC4 - 2004) and South African code (SANS 10162-1). The CFST columns' failure mode in Series (1) tests was to a great extent flexural buckling with no indication of local buckling. This mode of failure was caused by the columns' extensive relative slenderness ratio. In Series (2) tests, the 1.0 m (L/D = 5.16 to 6.56) and 1.5 m (L/D = 7.74 to 9.84) samples failed by the concrete core crushing and the steel tube's yielding. The 193.70 mm diameter composite columns accomplished superior ultimate load capacities in comparison to other diameters of similar lengths. This variance in ultimate load capacities is ascribed to the hoop or circumferential stress. A higher hoop or circumferential stress brings about huge expanded load carrying capacity of the CFST columns. All things considered, loads investigated by EC4 are conservative by 13.6 per cent, and loads anticipated by SANS 10162-1 are conservative by 8.4 per cent for Series (1) tests. The average expectations of test failure load by SANS 10162-1 and EC4 - 2004 are moderate by 10.1 and 12.2 for Series (2), individually.

Abed et al. (2013) presented an experimental study to investigate the compressive behavior of circular concrete filled steel tubes (CFSTs) when subjected to pure axial loading at a low rate of 0.6 kN/s. CFSTs of three different diameter-to-thickness (D/t) ratios of 54, 32, and 20 are considered in this study. Total 16 specimens were tested under pure axial loading using a concrete compresive strength of 44 MPa and 60 MPa. The test outcomes of compressive axial load capacities are contrasted with their corresponding hypothetical values anticipated by four distinctive international codes and standards: The Australian Standard (AS), the American Institute of Steel Construction (AISC 360), Eurocode 4 (EC4) and the American Concrete Institute (ACI 318). In this exploration, the compressive behaviour of the CFST under axial compressive loads is experiential examined. In this analysis the impacts of two principle parameters are investigated, specifically; the concrete compressive strength (fc′) and the diameter to thickness (D/t) ratio. As recommended by tried and true way of thinking and instinct, the expansion in fc′ expands the axial load capacity. This concrete compressive strength increment, nonetheless, has negligible impact on the stiffness of element. It is likewise noticed that the column's ductility reduces with increase in the concrete filling material's strength for superior D/t ratios, yet the inverse is valid for lower D/t proportions. The expansion in the D/t ratio decreases the CFST part's stiffness as well as its axial strength because of the lessening in the confinement. Besides, in this paper it was watched that the ACI, AS, Eurocode 4 and AISC codes have thought little of the compression capacity of CFST columns up to 43%, consequently these codes are considered completely conservative. Additionally, the theoretical equation, which does not represent the concrete confinement, thought little of the CFST capacity yet not as much as the other four codes since its predicted values ranges from 5% to 27%. Then again Mander et al. proposed the techniques, here sometimes the section's capacity furthermore overrated by Giakoumelis and Lam. The two proposed drew nearer ought to be closely utilized as it might bring about unconservative designs.

Uy et al. (2011) in this study. carried out a study where a progression of tests were done on slender and short concrete filled stainless steel tube like columns to investigate their execution under axial compression or combined action of axial force and bending moment. In total 117 samples were done. The test samples were arranged into 4 distinct groups. The axial load capacity against axial strain curves for CFSST stub columns under pure compression is ordered into 3 sorts related with a strain-softening or strain-hardening reaction. As contrasted with traditional carbon steel CFST columns, the stainless steel composite columns observe behavior that is more ductile and have a more superior remaining strength. 6 circular and 3 square stub columns were researched to assess the impact of various loading techniques. Contrasted with empty tubes, a strength increment was found for the infilled columns despite the fact that lone the steel tubes were loaded. The short composite columns under combined activities of axial force and bending moment showed exceptionally ductile behavior. The CFST columns' overall strength and stability were improved by the in-filled concrete. There is no obvious distinction between conventional carbon steel CFST columns and CFSST columns as far as test perceptions and failure modes for the slender columns. An arranged out-of-straightness ought to be proposed later on for slender CFSST columns. All the 4 codes utilized as a part of this paper fairly think little of the CFSST columns' load-carrying capacities under axial compression as well as combined activities. More researches to be carried out to build up a more exact outline way to deal with better use the material behavior of stainless steel.

De Oliveira et al. (2009) introduced a test study of the confinement effects in steel-concrete composite columns with respect to 2 parameters: column slenderness and concrete compressive strength. 16 samples of CFST column utilizing a concrete strength of 30, 60, 80, and 100 MPa, and the length to diameter across ratios of 3, 5, 7, and 10. The trial estimations of the columns' ultimate load were contrasted with the expectations of 4 code arrangements: The Eurocode 4 (EN 1994-1-1:2004), Brazilian Code NBR 8800:2008, CAN/CSA S16 01:2001 and AINSI/AISC 360:2005. Considering similar estimations of concrete compressive strength, the samples with (L/D=3) demonstrated a higher increment of load axial capacity because of the confinement index, up to the crushing of the concrete core and the local buckling of the tube. The samples with (L/D=10) introduced a lower strain, since the global buckling happened before the concrete center could build up its full axial capacity, lessening the radial distortion of the concrete core and staying away from the assembly of the confinement effect of the tube. The near study of the standard codes displayed acceptable results, for the most part for the samples with higher estimations of L/D ratio. Both ANSI/AISC and NBR codes indicated comes about 10.4% and 10.7% below the acquired outcomes, while the ultimate capacities of the 16 samples were higher than the ones anticipated by the ANSI/AISC Code. The outcomes of CAN/CSA and EC4 were definitely un conservative, in spite of having lower differences between the anticipated and the estimated ultimate load results of on average 2.4% and 2.3%, respectively. In general, every standard code demonstrated great understanding outcomes for columns with higher estimations of L/D, yet for short columns (L/D=3) the codes overestimated the expansion of load capacity of CFT columns because of the confinement index, particularly the CAN/CSA and EC4 codes.

Evirgen et al. examined the compressive behavior of CFST section in 2014, considering different cross-sectional shapes like hexagonal, rectangular, circular and square. Experimental study was done by differing concrete grades and B/t (Breadth to thickness) proportion. The results found of test study were contrasted with software outcomes computed by building up a finite element model utilizing software ABAQUS. Scientists watched in this study that the CFST's concrete core opposes the internal buckling of steel tube. The steel tube gives better confinement to concrete core, which comes about increment in the strength of CFST section. Additionally, the study demonstrates that the circular CSFT section's ductility is more than hexagonal, rectangular and square CFST columns.

In 2014, the compressive conduct of double skin concrete filled steel tube circular short columns with differing diameter to thickness (D/t) proportion was considered by (Hassanein and Kharoob). A lot of already created calculations were alluded in this analysis to ascertain the strength of Concrete Filled Double Skin Tube (CFDST). The acquired consequences of those conditions were contrasted and trial comes about and by building up a finite element model utilizing ABAQUS software. Following variance, it was observed to be a less conformity in the middle of both diagnostic and test result values. Consequently, new condition was determined by scientists for discovering strength of the CFDST short column.

CHAPTER 3

METHODOLOGY

3.1 AMERICAN INSTITUTE OF STEEL CONSTRUCTION AISC 360-16

3.1.1 General

The specification for structural steel buildings American National Standards Institute and American Institute of Steel Construction (ANSI/AISC), are adopted to the design and fabrication of structural steel constructions and other structures.

The 2016 Specification supersedes the 2010 version (ANSI/AISC 360-10). It has been approved by the AISC Committee on Specifications and is ANSI-accredited.
The AISC – 360 – 2016 satisfies the complete solution and treatments with the allowable strength design (ASD) and with Load Resistance Factor Design (LRFD).
In this chapter discuss the methodology that adopted by AISC 360-16 to design the composite members.

3.1.2 Analysis Method and Assumptions

The analysis adopted by AISC 360-16 (LRFD) was Direct Analysis Method (DAM) which applied either as elastic or inelastic analysis. The general requirements of the analysis summarized below:

1. The analysis for flexural, shear and axial deformation as (deflection, diagonal cracks and displacements) respectively.
2. Reduction in stiffness of composite column.
3. Second order analysis with both P-Δ and P-δ effects.
4. All gravity loads must have considered in the analysis.

3.1.3 Design of Composite Members

The design strength of composite column in all load cases equal or greater than the strength required based on the AISC 360-16 (LRFD).

1. The methods of analysis are elastic or inelastic which gave the worst case must adopt.
2. The selection of hollow structural sections (HSS) for preliminary design, the wall thickness matches the requirements of the standards of ASTM A1085/A1085M.
3. Provided stability for composite column.

The composite columns as mentioned above composite from two different materials as steel and filled by concrete without reinforcements. The two differ materials by modulus of elasticity and Poisson's ratio acts together as unity and the design formula are derived under the assumption of full interaction theory that rely on there are no slips between the interface of steel and concrete.

Composite columns provide economies solution by using slender lightweight members and high strength materials. The behavior of CFTs relies on many parameters such as the stress-strain of the composite material and the chemical adhesion, friction between concrete and surrounding steel tube, and volumetric.

3.1.3.1 Nominal Strength of Composite Sections

The nominal strength of composite column based on the method of analysis such as:

1. Plastic stress distribution.
2. Strain compatibility.
3. Elastic stress distribution.
4. The effective stress-strain.

The first and second methods provide a general method to calculate the geometry compressive capacity for composite members in case of compact cross sections. The third method for calculate the geometry compressive capacity of composite members in case of non-compact. The fourth method provides guidance for calculating the cross-section strength (axial force-moment strength interaction) for composite members with non-compact or slender cross sections.

Base on the AISC – 360 – 2016, the tensile strength in composite column ignored that is mean the nominal strength of composite members not resist tensile load.

3.1.3.1.a Plastic Stress Distribution Method

The main assumption to calculate the nominal strength of composite member that the steel sections have reached a stress equal or greater than *fy* in tension or compression because of the steel is ductile material, and the concrete section in compression only (ignored tension zone because of the concrete is brittle material) in case of axial force and/or flexure have reached a stress of 0.85 $f_c^{'}$, where $f_c^{'}$ is the specified compressive strength of concrete. For round HSS filled with concrete, a stress of 0.95 $f_c^{'}$ *is* acceptable.

3.1.3.1.b Strain Compatibility Method

The strain assumed to be linear along the cross section up to the maximum strain of concrete in compression zone equal to 0.003. The real stress-strain behavior of steel and concrete obtain from laboratory tests.

3.1.3.1.c Elastic Stress Distribution Method

The nominal strength determined from the superposition of elastic stresses for the limit state of yielding (as tension) or concrete crushing (as compression).

3.1.3.1.d Effective Stress-Strain Method

The nominal strength computed assuming strain compatibility, so that the local buckling determined from the effective stress-strain relationships for steel and concrete.

3.1.3.2 Design Code Limitations of CFST

The geometrical properties of the steel tube, and mechanical properties of steel and concrete sections are summarized in Table (3.1) based on AISC – 360 – 2016.

Table 3.1 AISC 360-16 limitations of Design Specifications

Codes	Yield strength of steel (MPa)	Compressive strength of concrete (MPa)
AISC 360-16	$f_y \leq 525$	$21 \leq f_c' \leq 70$ (for normal weight concrete) $21 \leq f_c' \leq 41$ (for lightweight concrete)
Slenderness	Steel amount	Width to thickness ratio for Round HSS
$KL/r \leq 200$	$A_s \geq 1\% A_g$	$D/t \leq 0.31(E_s/f_y)$

The limitations of filled composite column, first, the steel section geometry as cross section comprise at least (1%) of the total cross sectional area of the composite column. Second, the longitudinal reinforcement not required, finally, checking of local buckling of the composite column.

3.1.3.3 Classification of Filled Composite Sections for Local Buckling

The local stability of the individual filled composite section not same for hollow steel section because of the inner concrete gave more stable because of stiffness and strength increased in composite action. The concrete infill changes the buckling mode of the steel HSS (both within the cross section and along the length of the member) by preventing it from deforming inwards as shown in Figures 3.1 and 3.2.

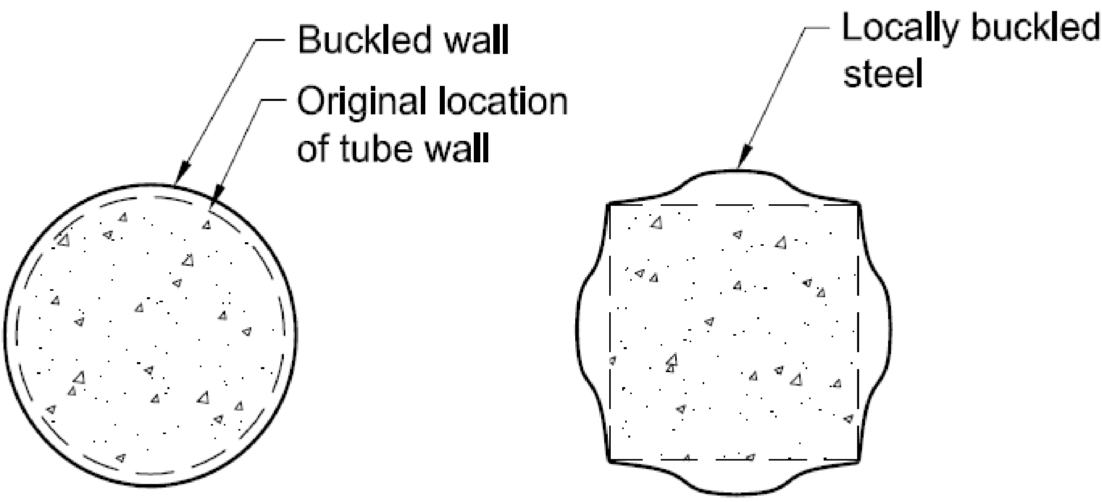


Figure 3.1 Cross-sectional buckling mode with concrete infill (AISC 360-16)

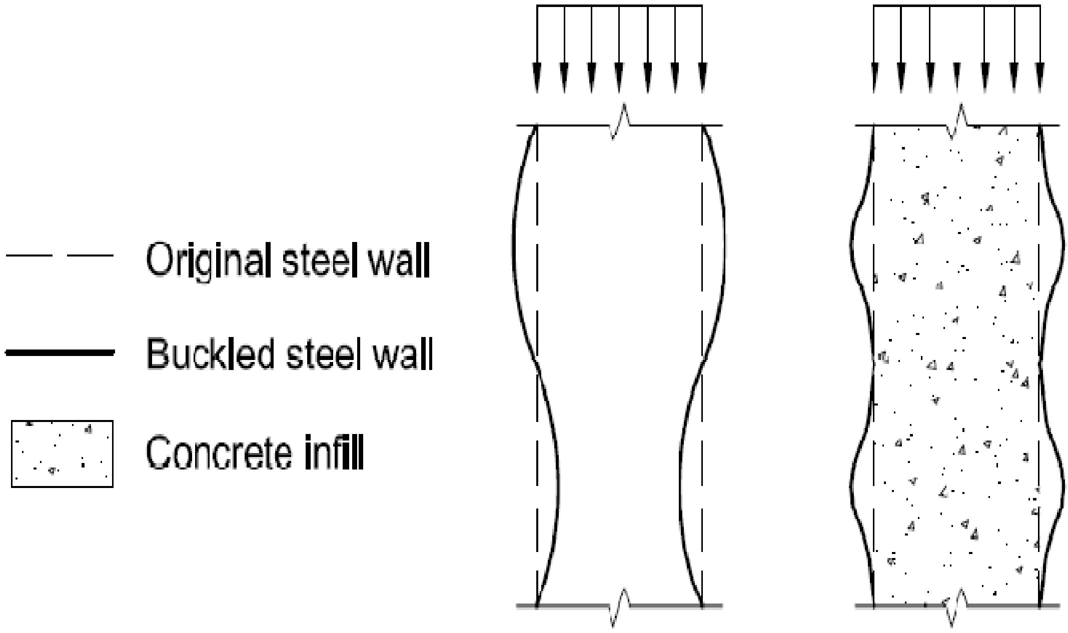


Figure 3.2 Changes in buckling mode with length due to the presence of infill (AISC 360-16)

The nominal strength of composite columns mainly has two types of applied loading as flexure and axial compression that rely on the classifications of section as compact, non-compact, slender, or too slender. Compact sections are capable of developing the full plastic strength before local buckling occurs. non-compact sections can develop partial yielding in compression, and buckle in elastically before reaching a fully plastic stress distribution. The composite column as slender sections buckle elastically before any of the elements yield under compression. Based on AISC 360-16, considers local buckling classifications according to the value of λ_p and λ_r for axial or flexural.

The classification of sections as follow:

1. Compact section if $D/t \leq \lambda_p$ (from table 3.2).
2. Non-compact section if $\lambda_p \leq D/t \leq \lambda_r$ (from table 3.2).
3. Slender section if $D/t \geq \lambda_r$ (from table 3.2).

Table 3.2 Classifications of section base on diameter to thickness ratio

Description of the element	Width to thickness ratio	λ_p compact/ non-compact	λ_r non-compact/ slender	Maximum permitted
Composite members subjected to axial compression				
Round HSS	*D/t*	*(0.15E/fy)*	*(0.19E/fy)*	*(0.31E/fy)*
Composite members subjected to flexure				
Round HSS	*D/t*	*(0.09E/fy)*	*(0.31E/fy)*	*(0.31E/fy)*

3.1.4 AXIAL FORCE

The model that adopt by AISC – 360 – 2016 similar as adopted by previous LRFD specifications to determine the section strength of composite column under the effects of axial force only.

3.1.5 Filled Composite Sections

3.1.5.1 Calculation of Nominal Compressive Strength

A compact hollow structural section (HSS) that adopt in present study with design thickness according to AISC – 360 – 2016, so that to develop yielding of the steel in the longitudinal compression, and to offer more confinement to the inner concrete to develop its compressive strength up to (0.85 or 0.95 f_c') to resists the axial force and flexural that came from external loading as dead and live loading. In case of composite columns classified as non-compact, the confinement of surrounding steel tube to the filled concrete up to 0.70f ′c and after that point the full behavior different and become due to inelasticity and volumetric dilation. The composite column as slender, neither develops yielding of the steel HSS in the longitudinal direction and gave little confines to the concrete after it reaches 0.70f ′c and after that the composite column starts to undergoing inelastic strains and significant volumetric dilation pushing against the HSS.

Figure 3.3 represent the variations of the nominal axial compressive strength, P_{no}, vs. HSS wall slenderness. The compact sections P_p represent the full plastic strength in compression when the tube slenderness equal to λ_P. In case of non-compact sections, the nominal strength P_{no} determined by assuming a quadratic interpolation between the plastic strength, P_p, and the yield strength, P_y. Slender sections are limited to developing the critical buckling stress, F_{cr}, of the steel HSS and 0.70 f_c' of the concrete infill (AISC – 360 – 2016).

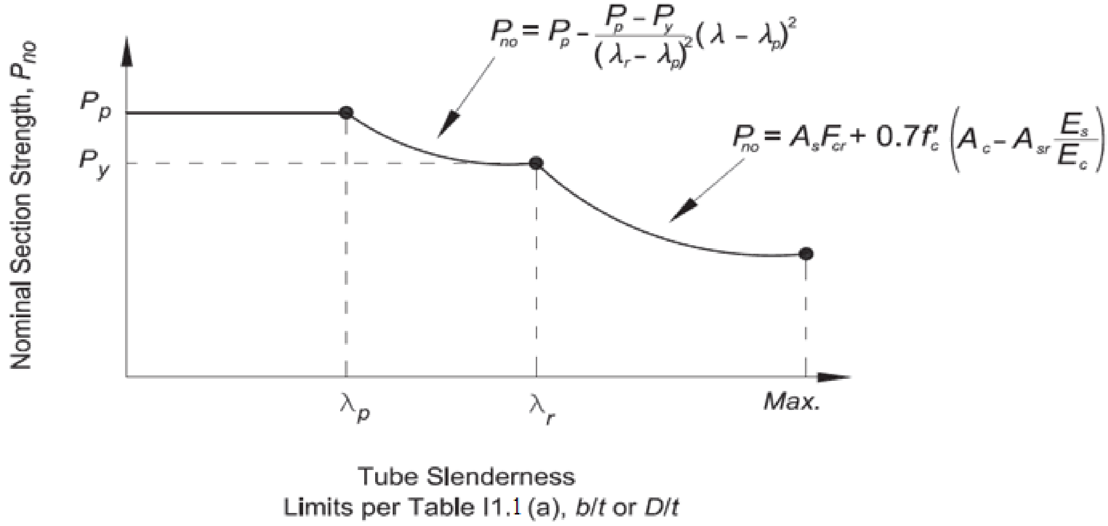


Figure 3.3 Nominal axial strength, P_{no}, versus HSS wall slenderness (AISC 360-16)

The design compressive strength, $\phi_c P_n$, as LRFD and the allowable compressive strength, $P_n/\Omega c$, as ASD of member's subject to axial compression that resistance and safety factors to determine the design and allowable compressive strengths are:

$$\phi_c = 0.75((LRFD) \qquad and \qquad \Omega_c = 2.00(ASD) \tag{3.1}$$

The nominal compressive strength of doubly symmetric axially loaded CFST shall be determined for the limit state of flexural buckling based on member slenderness as follows:

$$P_n = P_{no}\left[0.658^{\frac{P_{no}}{P_e}}\right] \qquad if \quad \frac{P_{no}}{P_e} \leq 2.25 \tag{3.2}$$

$$P_n = 0.877\,P_e \qquad if \quad \frac{P_{no}}{P_e} > 2.25 \tag{3.3}$$

Where;

P_{no}= the nominal strength of the composite section.

P_e = the Euler critical load, which is calculated using effective stiffness (EI_{eff}):

$$EI_{eff} = E_S\,I_S + C_3\,E_C\,I_C \tag{3.4}$$

$$P_e = \frac{\pi^2\,(EI_{eff})}{(LC)^2} \tag{3.5}$$

Where;

Lc = KL = effective length of the member

I_c and I_s = the moments of inertia of the concrete and structural steel sections, respectively.

E_s = the elastic modulus of steel.

E_C = the elastic modulus of concrete and can be calculated as below:

$$E_c = 4700\sqrt{f_c^{'}} \tag{3.6}$$

C_3 = the coefficient for effective rigidity of the filled composite column:

$$C_3 = 0.45 + 3\left(\frac{A_S + A_{Sr}}{A_g}\right) \leq 0.9 \tag{3.7}$$

Where;

A_s= area of the steel cross-section.

A_{sr} = area of continuous reinforcing (in present study equal to zero).

A_g= gross area of composite member.

(a) In case of compact sections, the nominal axial capacity is calculated as:

$$P_{no} = P_p \tag{3.8}$$

and the plastic strength of the section is given by:

$$P_p = F_y A_s + C_2 f_c' A_c \tag{3.9}$$

Where;

C_2 = 0.95 for circular CFST columns

AISC 360 – 16 adopts the confinement effect of circular section by the coefficient of C_2 of 0.95, which gives an 11% constant improvement due to confinement.

(b) For non-compact sections, the nominal axial capacity is evaluated as:

$$P_{no} = P_p - \frac{P_p - P_y}{(\lambda_r - \lambda_p)^2}(\lambda - \lambda_p)^2 \tag{3.10}$$

Where;

λ, λ_p and λ_r are slenderness ratios.

P_y = the yield strength of the composite section, and is given by:

$$P_y = F_y A_s + 0.7 f_c' A_c \tag{3.11}$$

(c) For slender sections, the nominal compressive capacity is:

$$P_{no} = F_{cr} A_s + 0.7 f_c' A_c \tag{3.12}$$

Where;

F_{cr} = the critical local buckling stress of the filled circular section, and is calculated as follows:

$$F_{cr} = \frac{0.72 F_y}{\left((\frac{D}{t}) \frac{F_Y}{E_s} \right)^{0.2}} \tag{3.13}$$

Based on AISC – 360 -2016, the slender sections up to critical local buckling stress F_{cr} of steel and 0.7 f_c' of concrete core.

3.1.5.2 Calculation of Nominal Tensile Strength

The design tensile strength, $\phi_t P_n$, as LRFD and the allowable tensile strength, P_n/Ω_t as ASD in case of the gross section under tension only as follows.

$$P_n = F_y A_s + F_{ysr} A_{sr} \tag{3.14}$$

$$\phi t = 0.90(LRFD) \qquad and \qquad \Omega t = 1.67(ASD) \tag{3.15}$$

3.2 Eurocode 4 (BS EN 1994-1-1:2004): Design of Composite and Concrete Structures

3.2.1 General

Eurocode is comprehensive code that contains many parts that deal with the structural elements such as concrete, steel and composite structures. The application fields of Eurocode adopt in many purposes such as stability and mechanical resistance, contracts for construction works and framework for all type of drawings to matching the construction specifications and requirements.

In this chapter discuss the methodology that adopted by EC4 to design the composite members.

3.2.2 Method of Analysis and Assumptions

First order analysis is acceptable for buildings analysis for composite members. The assumptions of the analysis are summarized below based on Eurocode 4 as follows:

1. Cracking of concrete allowed but within limit.
2. First order analysis with suitable amplification give results around of second order analysis.
3. The individual analysis of member to check the stability not necessary if the global analyses of the structure are fully accounted.
4. The flexural stability for composite column checking either by global analysis, second order analysis or by buckling curves.

3.2.3 Composite Columns

The Eurocode – 4 1994-1-1 (EC4, 2004) specialized for design of composite compression members covers the design criteria for encased, partially encased and concrete infill columns both with and without reinforcements as shown in Figure 3.4. The code applies to the column with steel mechanical properties as grades S235 - S460 and with the normal concrete of compressive strength classes C20/25 - C50/60 that classified and lists in Tables 3.3 and 3.4. The design procedure for composite columns adopted by Eurocode was behaved toward by an amalgamation of both steel and concrete design approaches.

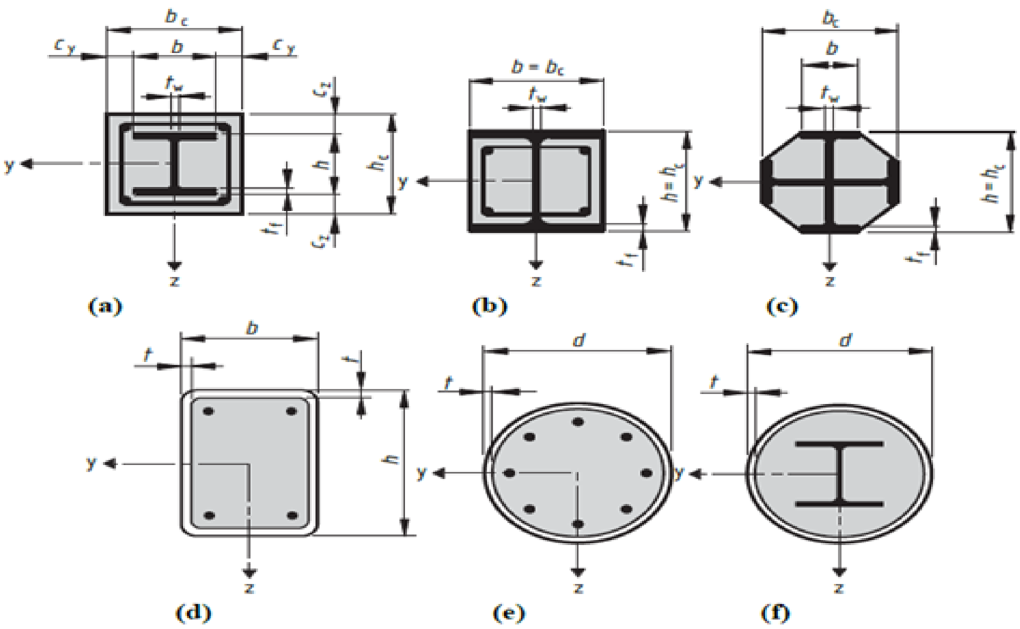


Figure 3.4 Typical cross-sections of composite columns and notation (EC 4 – 2004)

Table 3.3 Nominal (characteristic) values of yield strengths and modulus of elasticity for structural steel

Steel grades	yield strength f_y (MPa)	modulus of elasticity E_a (MPa)
S235	235	210000
S275	275	
S355	355	
S460	460	

Table 3.4 Strength classes of concrete, characteristic cylinder strength and modulus of elasticity for normal weight concrete

Strength class of concrete $f_{ck.cyl}/f_{ck.cub}$	Cylinder strength f_{ck} (MPa)	Secant modulus of elasticity E_{cm} (MPa)
C20/25	20	29000
C25/30	25	30500
C30/37	30	32000
C35/45	35	33500
C40/50	40	35000
C45/55	45	36000
C50/60	50	37000

The first step in design by Eurocode 4 is has to attain the equation (3.16) below:

$$S_d \leq R_d = R[\frac{f_y}{\gamma_{ma}}, \frac{f_{ck}}{\gamma_c}, \frac{f_{sk}}{\gamma_s}] \quad (3.16)$$

Where;

S_d = the combinations with the load factor.

Υ_F and R_d = the combinations of the resistances depending on the different partial safety factors for the materials Υ_M. The advised values for the factors that mentioned above are lists in Table 3.5.

Table 3.5 Partial safety factor for resistances and material properties for fundamental combinations

Structural steel	concrete	reinforcement
Υ_a = 1.1	Υ_c = 1.5	Υ_s = 1.15

3.2.4 Design of Composite Members

Composite column is a compression member like reinforced concrete column and carried mainly axial force and flexural. The Eurocode 4 gave a guide for design of composite columns that adopt hollow circular cross section filled by concrete with and without longitudinal reinforcements.

The general method of design for the composite columns summarized as below:

1. Second order analysis taking into account to check the stability for design purpose. The analysis considered the effects of residual stresses, geometrical defects, individual instability, the cracking that developed in concrete, creep and shrinkage of concrete as long term effects and yielding of steel.
2. Second-order analysis applies in all directions of expected failure.
3. Internal forces calculated based on the elasto-plastic stage.
4. Plane sections remain plane before and after applied loading.
5. Full interaction theory applying up to failure so that there is no slip between steel section and inner concrete.
6. In the analysis of composite section, the tensile strength of concrete ignores.

7. For long term analysis, the effects of shrinkage and creep taking into account in case of these values effects on the stability of composite column. These considerations may be ignored in case of increase in the first-order bending moments because of the presence of creep deformations and longitudinal force due to permanent loads not more than 10%.

3.2.5 Design Code Limitations of CFST

Based on the Eurocode 4 for design of CFST those rely on the mechanical properties of concrete and steel section that lists in Table 3.6. Mainly parameters in design of composite column classified according to relative slenderness ratio because of effect on the column stability. Also, the cube characteristic compressive strength is the important parameter in addition of yielding of steel in tension.

Table 3.6 Eurocode limitations of Design Specifications

Codes	Yield strength of steel (MPa)	Compressive strength of concrete (MPa)
EC4 (BS EN 1994-1-1:2004)	$235 \leq f_y \leq 460$	$20 \leq f_{ck} \leq 60$
Relative slenderness ratio	Steel contribution ratio	Width to thickness ratio for Round HSS
$\bar{\lambda} \leq 2.0$	$0.2 \leq \delta \leq 0.9$	Max $(d/t) = 90\ (235/f_y)$

3.2.6 Local Buckling

Design of the structural elements based on Eurocode 4 by the ultimate limit state method that depends on the loadings magnifications. The preliminary design of a composite column starting with a certain limit ratio of depth / thickness as follow:

- For concrete filled circular hollow sections

$$d/t \leq 90\varepsilon^2 \tag{3.17}$$

In which ε account for different yield limits:

$$\varepsilon = \sqrt{235/fy} \tag{3.18}$$

The limit ratio as wall geometry to the thickness of wall to satisfies the local buckling are lists in Table 3.7 with respect to steel grade in which they consider that the buckling of the walls of concrete filled sections is only possible in the outer direction.

Table 3.7 Limit ratio of wall dimensions to wall thickness for which local buckling is prevented

Steel grade	S235	S275	S355	S460
Circular HSS	99	77	60	46

3.2.7 Resistance of A Section to Axial Loads

The plastic analysis of compression member composite column against the applied external axial force by steel section and filled concrete, so the resistances of composite section calculate as follow:

$$N_{pl,RK} = A_a\, f_y + 1.0\, A_c\, f_c \tag{3.19}$$

Where;

A_c = Area of the concrete cross-section.

A_a = Area of the steel tube cross-section.

The steel contribution ratio δ is defined as:

$$\delta = \frac{A_a\ f_y}{N_{pl,RK}} \tag{3.20}$$

This value has to fulfill following requirement:

$$0.2 \leq \delta \leq 0.9 \tag{3.21}$$

In case of the value of $\delta < 0.2$, then there is no contribution from steel section to resists the applied loading and the column designed as concrete column. This recommended by Eurocode 2, but when when $\delta > 0.9$, the column shall be designed as a steel column recommended by Eurocode 3.

Hollow sections rather than circular (i.e square and rectangular) has little confinement because of the walls in resist the concrete pressure by plate bending, but in case of circular that generated hoop stress.

If the confinement affect taking into account, the EC4 suggested a formula for the circular CFST columns as follows:

$$N_{PL,RK} = \eta_{ao}\ A_a \frac{f_y}{y_a} + A_c \frac{f_c}{y_c}(1+\eta_{co}\frac{t}{d}\frac{f_y}{f_c}) \tag{3.22}$$

Where;

$\eta_a =$ steel reduction factor, where the yield stress decreased due to the hoop stress.

$\eta_c =$ concrete enhancement factor, where the concrete strength increased under triaxial stress state. when eccentricity is smaller than 10% of the outer diameter of the steel tube D, the values $\eta_a = \eta_{ao}$ and $\eta_c = \eta_{co}$ are evaluated as follows:

$$\eta_{ao} = 0.25(3+2\vec{\lambda}) \leq 1.0 \tag{3.23}$$

$$\eta_{co} = 4.9 - 18.5\overline{\lambda} + 17\overline{\lambda}^2 \geq 0 \tag{3.24}$$

The EC4 considers the confinement effect in circular CFST columns when the relative slenderness ratio ($\bar{\lambda}$) does not exceed 0.5. The relative slenderness $\bar{\lambda}$ can be calculated by:

$$\bar{\lambda} = \sqrt{\frac{N_{PL,RK}}{N_{cr}}} \leq 0.5 \tag{3.25}$$

Where;

$N_{PL,RK}$ = the plastic resistance of column.

N_{cr} = the elastic buckling load of the member which is calculated using effective stiffness $(EI)_{eff}$:

$$N_{cr} = \frac{\pi^2 (EI)_{eff}}{(KL)^2} \tag{3.26}$$

$$(EI)_{eff} = E_a I_a + K_e E_{cm} I_c \tag{3.27}$$

Where;

K_e = correction factor equal to 0.6

I_s and I_c = the moment of inertia of steel tube section and concrete section, respectively.

E_{cm} = the elastic modulus of concrete, as below:

$$E_{cm} = 22000 \left(\frac{fck + 8\,MPa}{10}\right)^{0.3} \tag{3.28}$$

EC4 considered the effect of imperfections that might be caused second order moments by multiplying the column plastic resistance by a reduction factor χ :

$$\chi = \frac{1}{\phi + [\phi^2 - \bar{\lambda}^2]^{0.5}} \leq 1.0 \tag{3.29}$$

The reduction factor χ is calculated using European column curves and the parameter ϕ is calculated as:

$$\phi = 0.5[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2] \quad (3.30)$$

Where, α is an imperfection factor, equal to 0.21 for circular CFST columns.

$$N_{Rd} = \chi N_{PL,RK} \quad (3.31)$$

3.3 Comparisons between AISC – 360-16 and Eurocode 4

The differences between AISC – 360 and Eurocode 4 as follows:

AISC 360

1. Local buckling classifications according to the value of λ_p and λ_r for axial or flexural**.**
2. The nominal compressive strength depends on the nominal compressive capacity of the composite section and the Euler critical load.
3. The methods of analysis as ASD and LRFD adopted for composite column design.
4. The composite column first order analysis used direct design method that included the effects of P – delta.

Eurocode 4

1. First order analysis is acceptable for buildings analysis for composite members.
2. Second order analysis taking into account to check the stability for design purpose.
3. Internal forces calculate based on the elasto-plastic stage.
4. For long term analysis, the effects of shrinkage and creep taking into account in case of these values effects on the stability of composite column.
5. Confinement and non - confinement affect taking into account.

3.4 Summary

The AISC – 360 – 2016 are summarized for analysis and design for circular hollow steel section filled by concrete without reinforcement above with recommended formulas for each code. Based on the diameter to thickness ratio, the section type classified as compact, non-compact and slender section according to AISC – 360 – 2016. There are no main reinforcements of composite columns. The steel section surrounding the inner concrete working as permanent casing and increasing the concrete confinements. There are no shear connections inside the steel section to connect the steel section with the inner concrete. Full interaction theory applied to design the composite members so that there are no slips between the two different materials. The direct design method that suggested by the code is the moderate procedure is the first order analysis that contained the effect of P – delta effects on the vertical element such as columns.

The Eurocode 4 suggested formulas that deal with the design of composite columns that mentioned in second paragraphs. The methodology that suggested included the effects of second order analysis applies in all directions of expected failure. In the analysis of composite section, the tensile strength of concrete ignores. The cracking of concrete allowed but within limit that not greater than the allowable tensile strength in the concrete. First order analysis with suitable amplification give results around of second order analysis.

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 Introduction

The parameters that effects on the axial capacity of the Concrete Filled Steel Tube (CFST) column such as height to diameter ratio, diameter to thickness ratio, compressive strength of concrete and yield strength of the steel section were considered in present study. The comparisons between two global codes as axial capacity of composite column as AISC – 360 – 2016 and EC 4 – 2004 were adopted. The study consisted of many classifications of columns such as compact, non-compact and slenderness rely on the diameter to thickness ratio.

4.2 Material Properties

The mechanical properties for concrete and steel were selected within the range and the limits of the adopted codes. Four different compressive strengths of concrete f_c' taken as (30, 50, 70 and 90 MPa) were considered as ranged to cover normal and high strength concrete. The yield strength of steel tube f_y varied as (S235, S315, S395 and S475 MPa) to cover mild and high tensile strength steel. The main function of the selected mechanical properties, length to diameter ratio and diameter to thickness ratio are to evaluate the axial capacity of the composite columns. To evaluate the axial capacity of the composite columns as a function of mechanical properties, The confinement index is a parameter has been adopted to specify the confinement capability of the CFST column roughly (Han *et al.*, 2014; Han *et al.*, 2005).

$$\xi = \frac{A_s\ f_y}{A_c\ f_c^{'}} \tag{4.1}$$

In which, A_s and A_c are the cross-sectional areas of the steel tube and core concrete, respectively, f_c' is the compressive strength of concrete, and f_y is the steel yield strength.

The studies proved that the circular steel tube could provide more effective confinement to the core concrete than other types of steel tube sections. Large experimental studies focused on the performance of circular CFST column under axial load were carried out over the last decades. In addition to experimental works, several design specifications have been published to enhance the applications and design of the CFST columns.

The strength index (*SI*) is the most important parameter to compare the performance of the column. The (*SI*) useful for CFST to check out the full behaviors that compute as follow and (Ekmekyapar and Al-Eliwi,2016; Han *et al.*, 2014; Portolés *et al.*, 2011; Yang *et al.*, 2008; Yu *et al.*, 2008):

$$SI = \frac{P_u}{P_{uo}} \tag{4.2}$$

Where, P_u is the axial capacity of a CFST column predicted by AISC 360 – 16 and EC4 codes. And P_{uo} is the sectional capacity or squash load:

$$P_{uo} = A_s f_y + 0.85 A_c f'_c \tag{4.3}$$

4.3 Column Specimens

The outer diameter for all CFST columns is (200 mm). The lengths to diameters L/D ratio taken as (3, 6, 9 and 12) to cover short and long columns with different height as (600, 1200, 1800 and 2400 mm). The diameters to thickness D/t ratio taken as (20, 40, 60 and 80) so that the thicknesses varied are (10, 5, 3.33 and 2.5 mm).

In definition of short and long CFST columns the AISC 360 – 16 and EC4 codes are completely different, therefore, the term "short column" and "long column" are classified according to L/D ratio, where the "short column" is defined as specimen with L/D ratio less than or equal 4, while, "long column" is defined as specimen with L/D ratio more than 4 (Han *et al.*, 2014; Le Hoang and Fehling, 2017; Li *et al.*, 2015). The modulus of elasticity of steel tube is 200 GPa, and modulus of elasticity of concrete is determined according to the corresponding codes.

The selection of different ratio to examined the slenderness ratio and take into account many type of CFST as compact, non-compact and slenderness section that lists in Appendix A. the ended of all columns are assumed as pin – pin, so that the (k) value for all columns is equal to unity. In contact surface between the inner concrete and the surrounding steel section assumed that there are no shear connectors and there is no frictional force between them. The applied external loadings resisted by the strength capacity of composite column by steel section and unreinforced concrete. The presence of steel section makes the composite column more confinement and provide permanent casing to the column so that the axial capacity increased.

4.4 Design Specifications

The two international codes that adopted here in present work as AISC – 360 – 2016 and EC 4 – 2004, in which the AISC concerned on the steel design in two different methods such as limit state design and allowable strength design in addition to load resistance factor design that combined the advantages of the mentioned methods. In EC 4, the limit state design. A total of (256) specimens of CFST that lists in Appendix A are analyzed by the two international codes and make comparisons between then as full column axial capacity.

In EC 4, the plastic resistance of CFST by inner concrete and surrounding steel section and the circular composite column section gave more confinements because of generated hoop stress.

In AISC – 360, the axial capacity based on the slenderness of composite column. The slenderness ratio for circular composite columns classified based on the local buckling that relay on the diameter to wall thickness ratio as compact, non – compact and slender that mentioned in chapter three. AISC 360 equations slender sections are limited to developing critical local buckling stress f_{cr} of steel tube and $0.7 f_c$ of concrete core.

4.5 The Effect of Parameters on Axial Capacity of CFST Columns

To study the behavior of the CFST columns, there are materials and geometrical parameters are effect on the axial capacity of column, (1) concrete compressive strength f_c', (2) steel tube yield strength f_y, (3) diameter – to – thickness D/t ratio, and (4) length – to – diameter L/D ratio.

To investigate which parameter has more effect on the axial capacity of the CFST column, the results of analysis of variance showed that the D/t ratio and concrete compressive strength have the more effects than other parameters and the maximum interaction is between D/t ratio and f_y for both AISC 360 – 16 and EC4.

The results show that the CFST column of (D/t = 20, L/D = 3, $f_c' = 90$ MPa, and f_y = 475 MPa) gives the maximum axial capacity of 4.959 MN, and 6.037 MN for both AISC 360 – 16 and EC4 respectively by difference about 21.7% where the EC4 takes the confinement effect on its consideration. While the CFST column of (D/t = 80, L/D = 12, $f_c' = 30$ MPa, and f_y = 235 MPa) gives the minimum axial capacity of 1.084 MN, and 1.163 MN for both AISC 360 – 16 and EC4 respectively by difference about 7.3%.

Figure 4.1 to 4.24 present the interaction plots for both AISC 360 – 16 and EC4. Use an interaction plots to show how the relationship between one parameter and the mean of the axial capacity depends on the value of the second parameter.

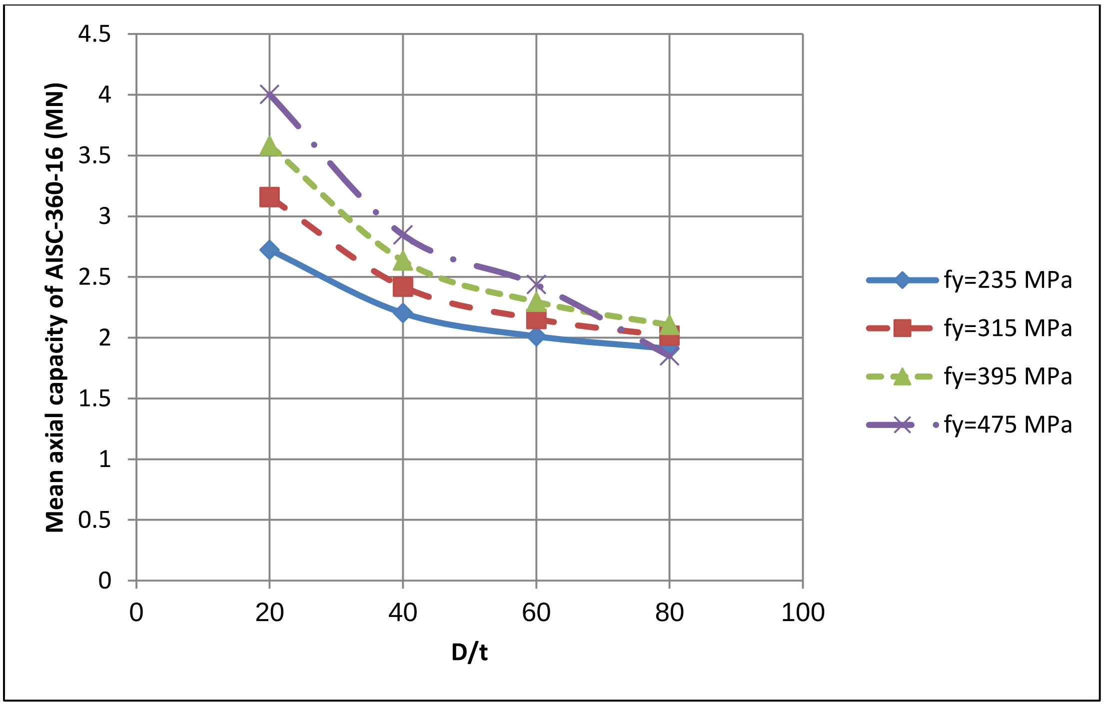


Figure 4.1 Axial Capacity of CFST Columns vs. D/t Ratio with different steel yield strength

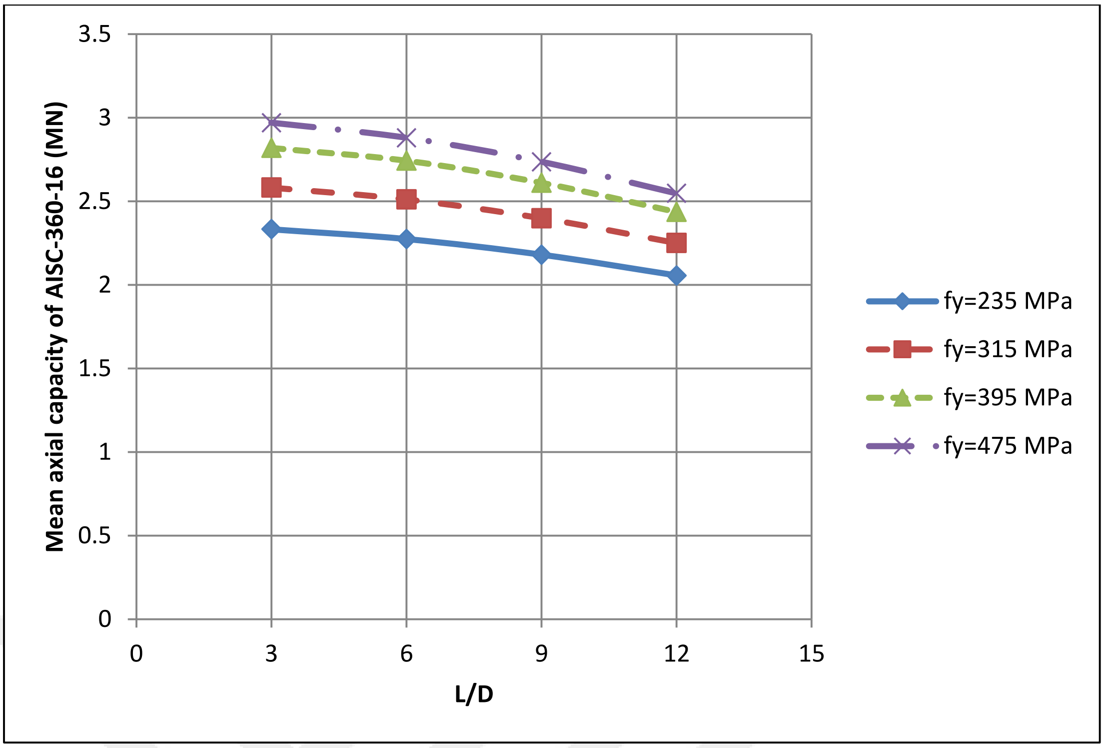


Figure 4.2 Axial Capacity of CFST Columns vs. L/D Ratio with different steel yield strength

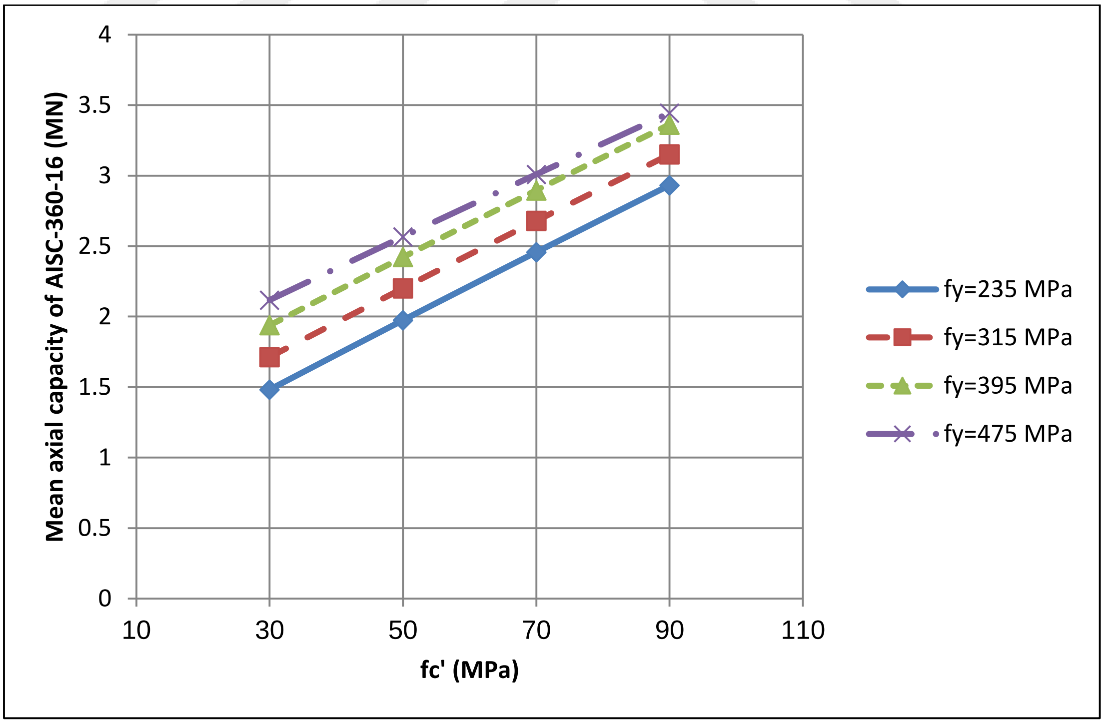


Figure 4.3 Axial Capacity of CFST Columns vs. compressive strength of concrete with different steel yield strength

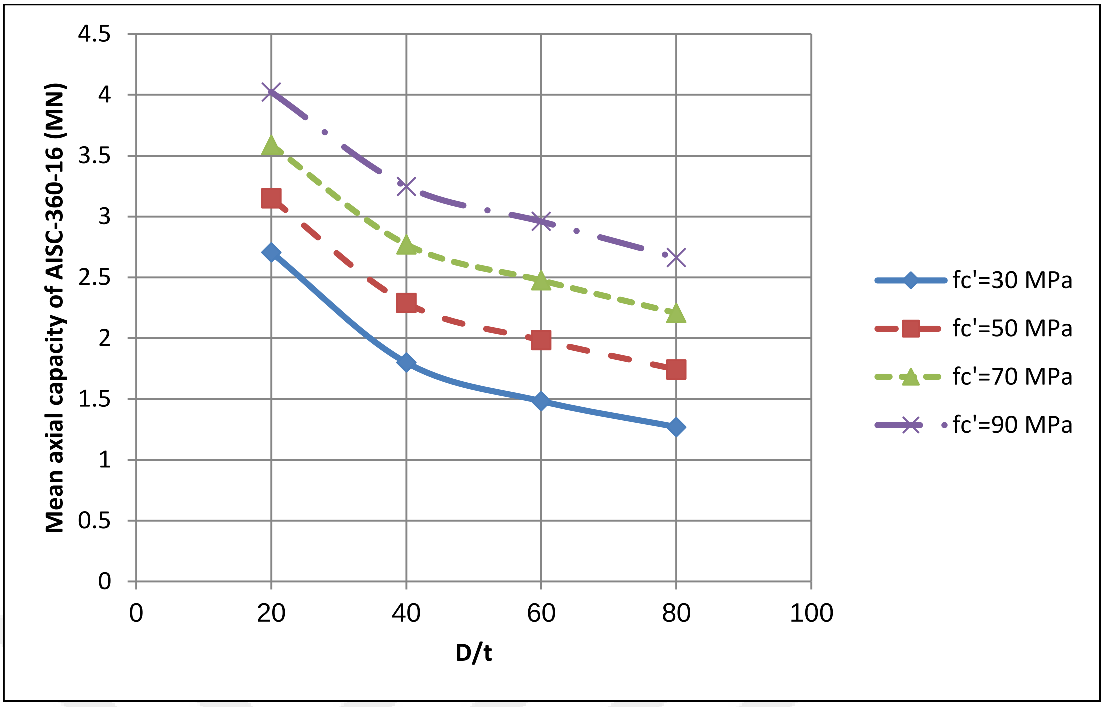


Figure 4.4 Axial Capacity of CFST Columns vs. D/t Ratio with different concrete compressive strength

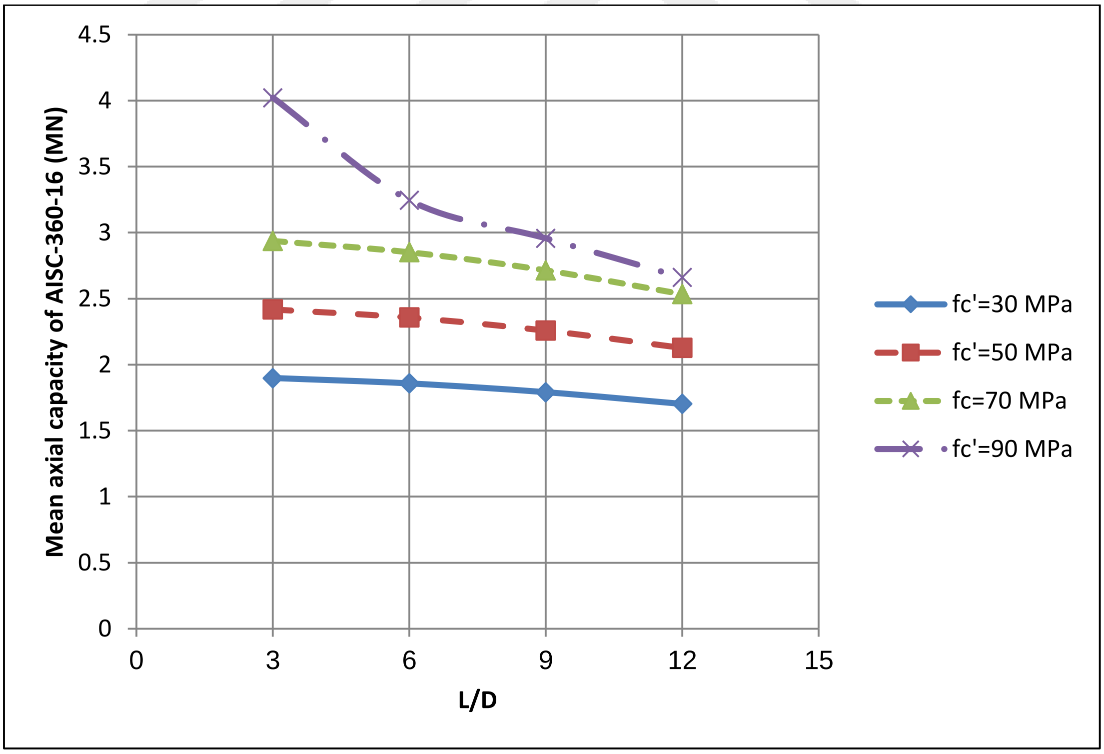


Figure 4.5 Axial Capacity of CFST Columns vs. L/D Ratio with different concrete compressive strength

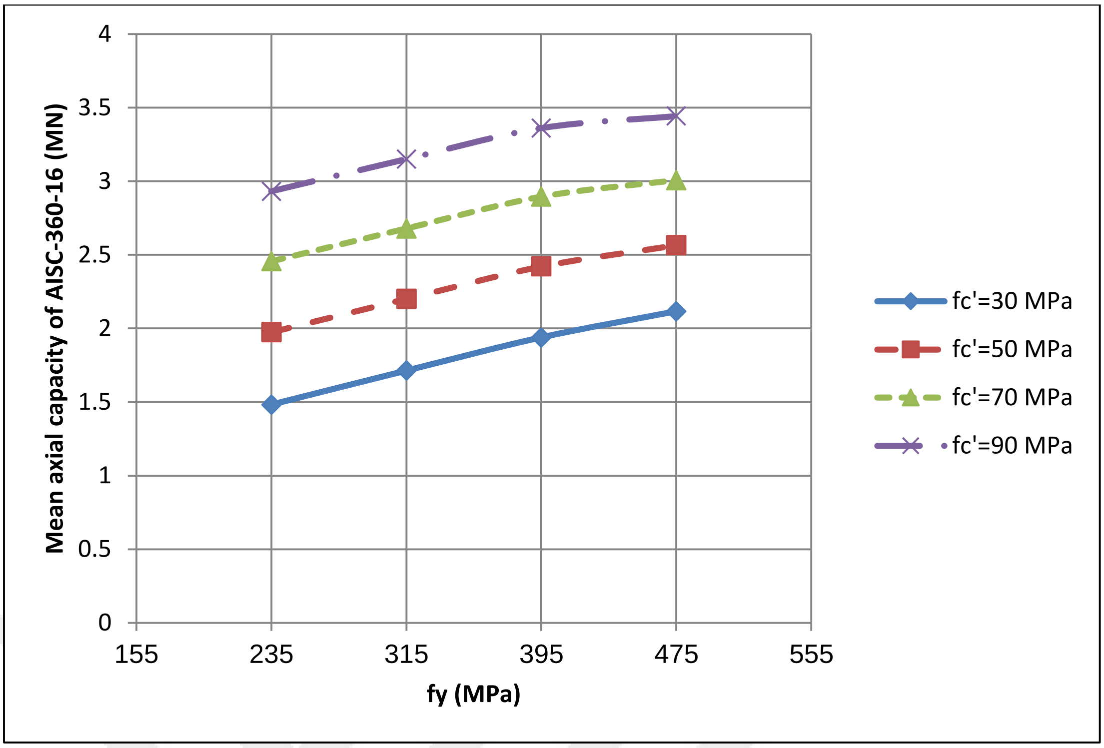


Figure 4.6 Axial Capacity of CFST Columns vs. steel yield strength with different concrete compressive strength

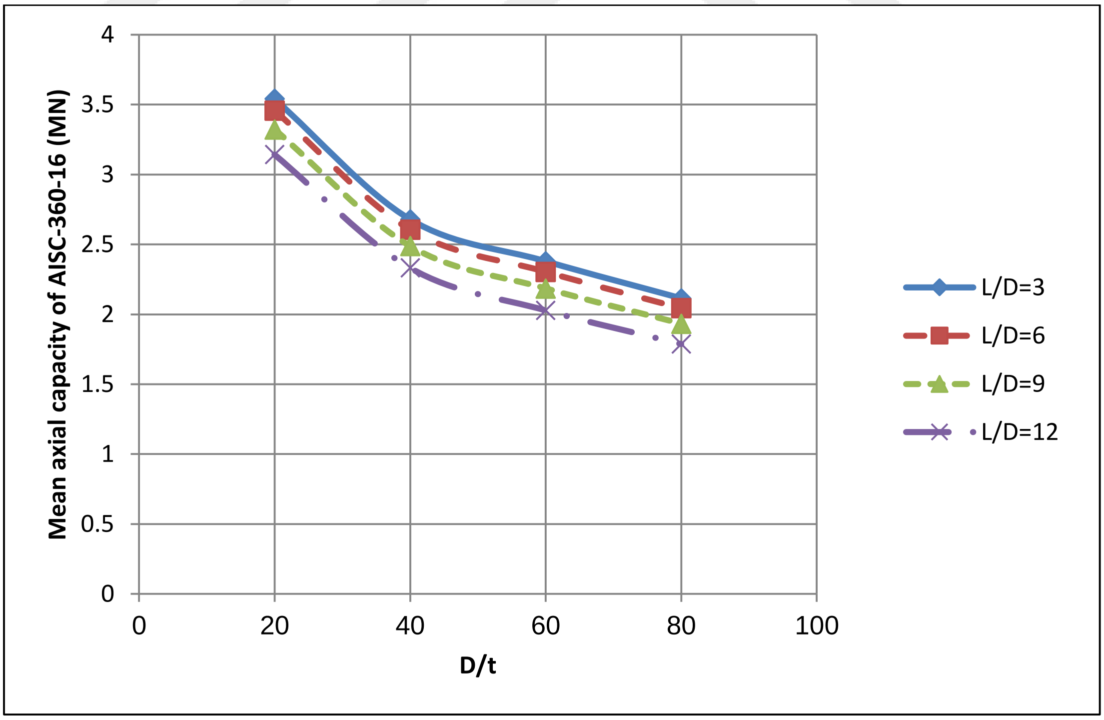


Figure 4.7 Axial Capacity of CFST Columns vs. D/t Ratio with different L/D Ratio

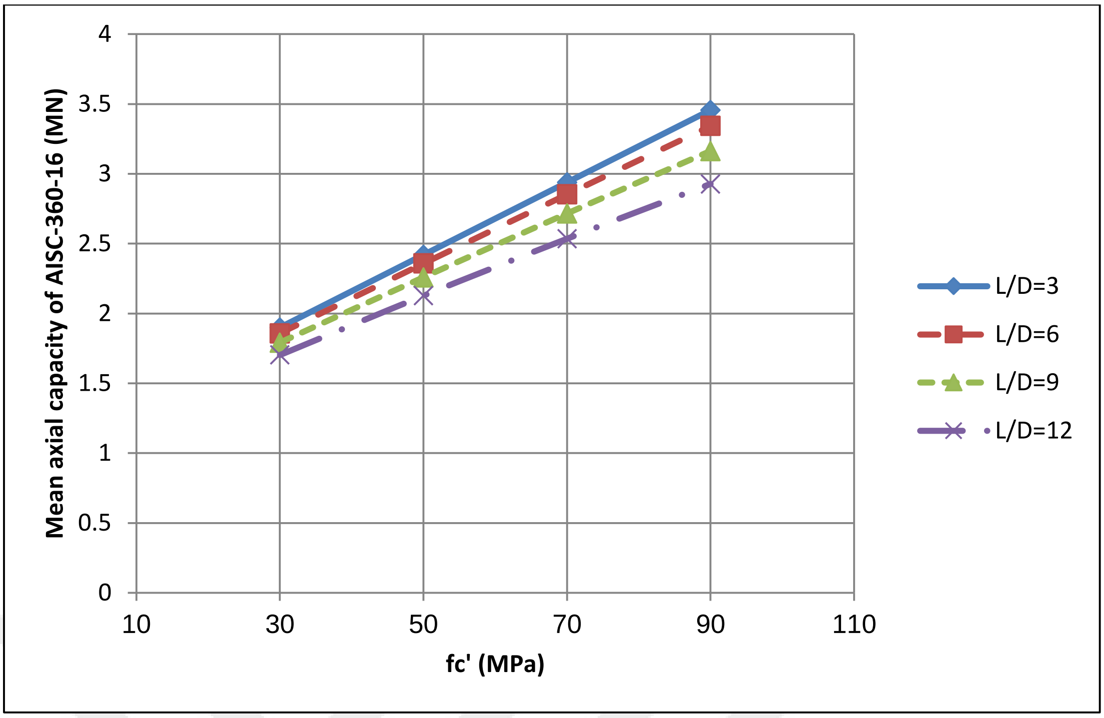


Figure 4.8 Axial Capacity of CFST Columns vs. concrete compressive strength with different L/D Ratio

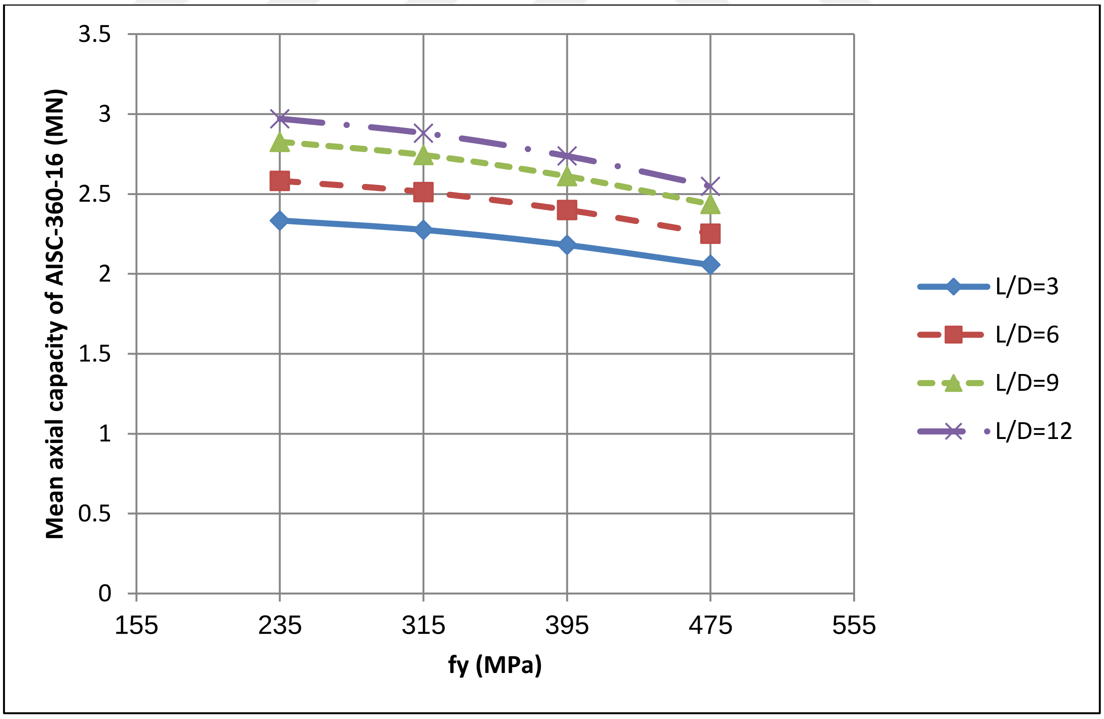


Figure 4.9 Axial Capacity of CFST Columns vs. steel yield strength with different L/D Ratio

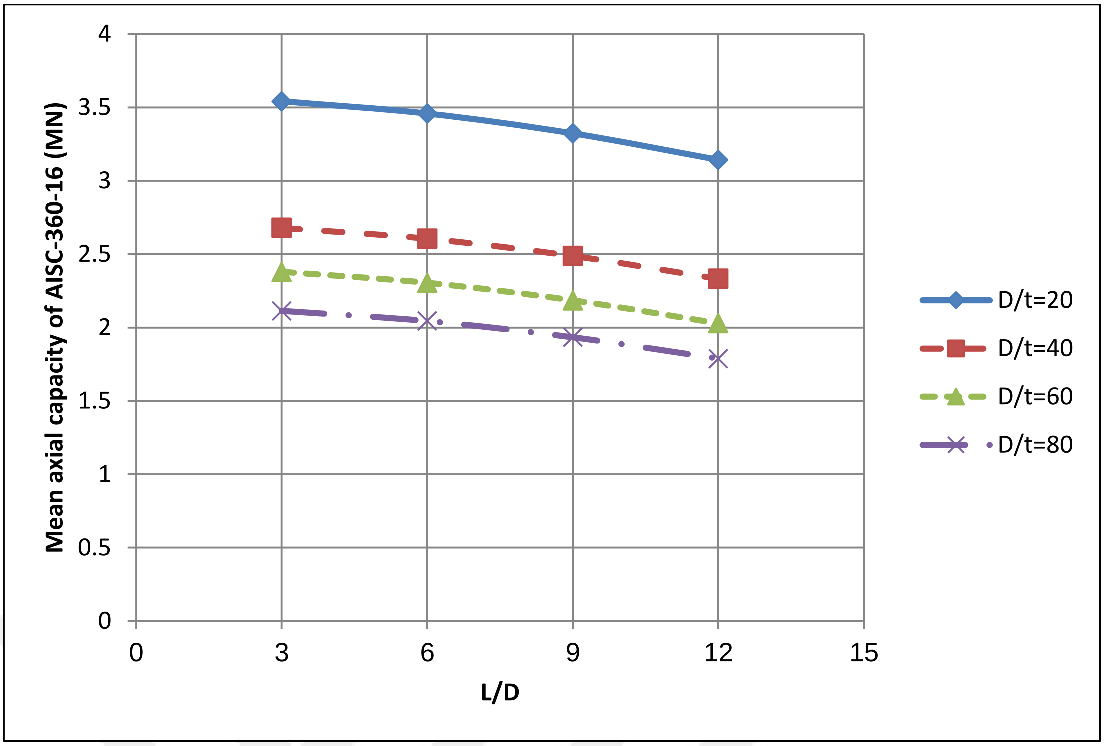


Figure 4.10 Axial Capacity of CFST Columns vs. L/D Ratio with different D/t Ratio

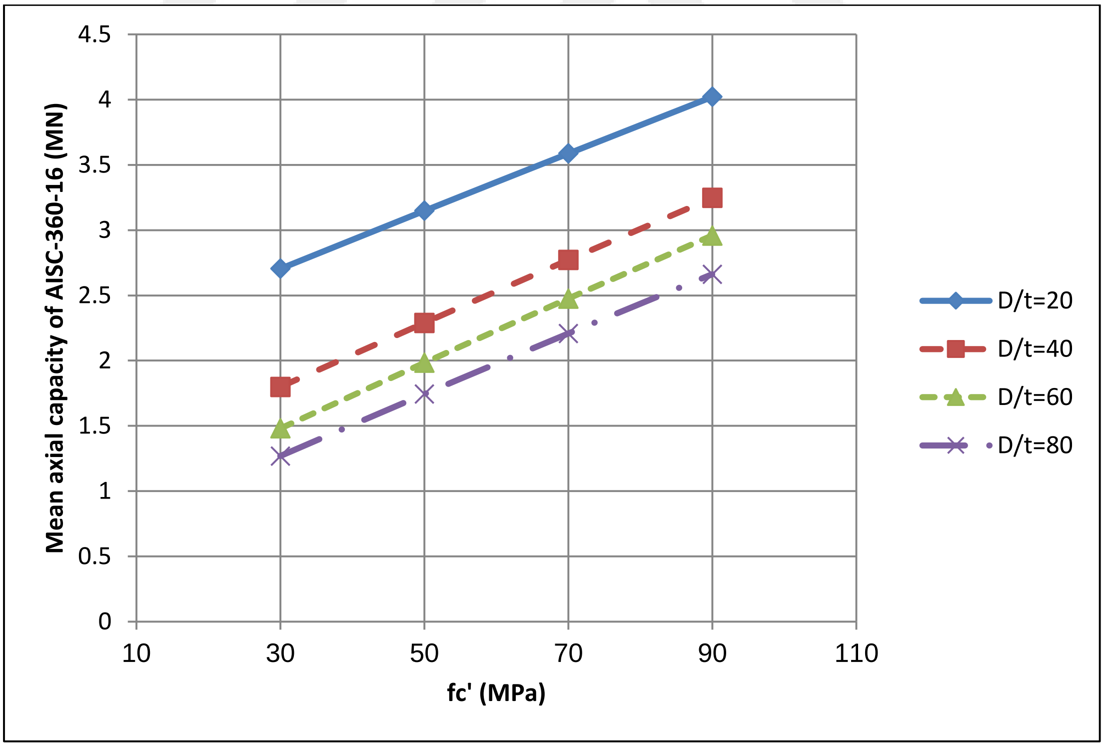


Figure 4.11 Axial Capacity of CFST Columns vs. Concrete compressive strength with different D/t Ratio

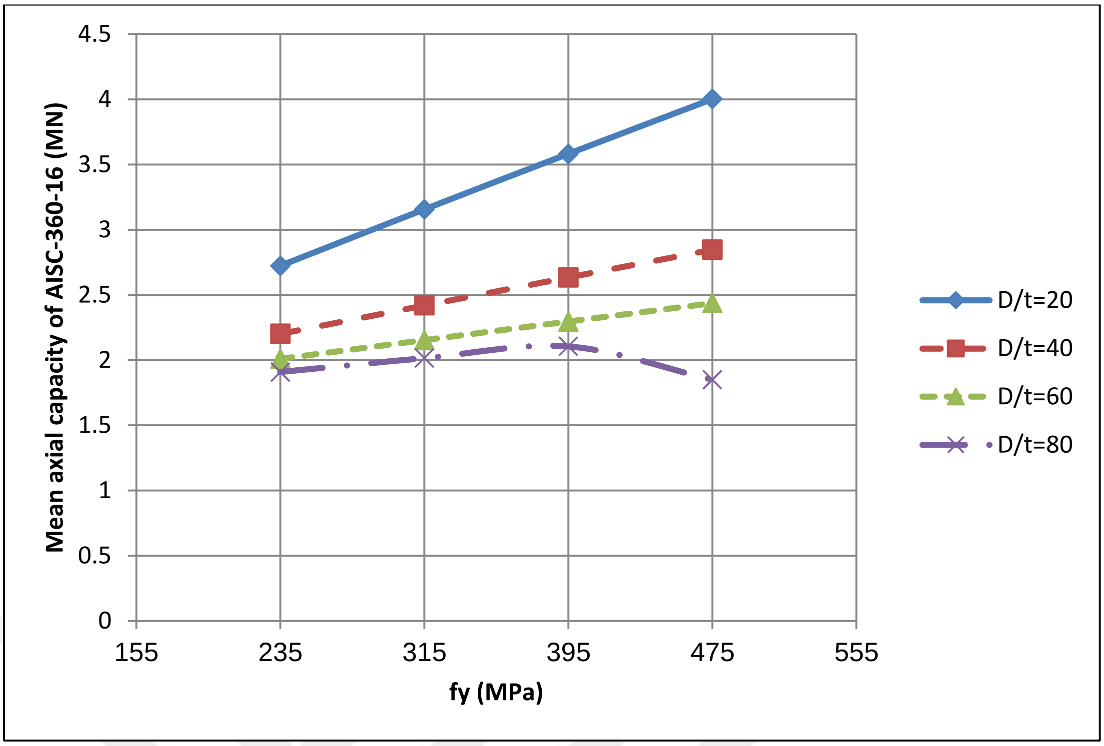


Figure 4.12 Axial Capacity of CFST Columns vs. Steel yield strength with different D/t Ratio

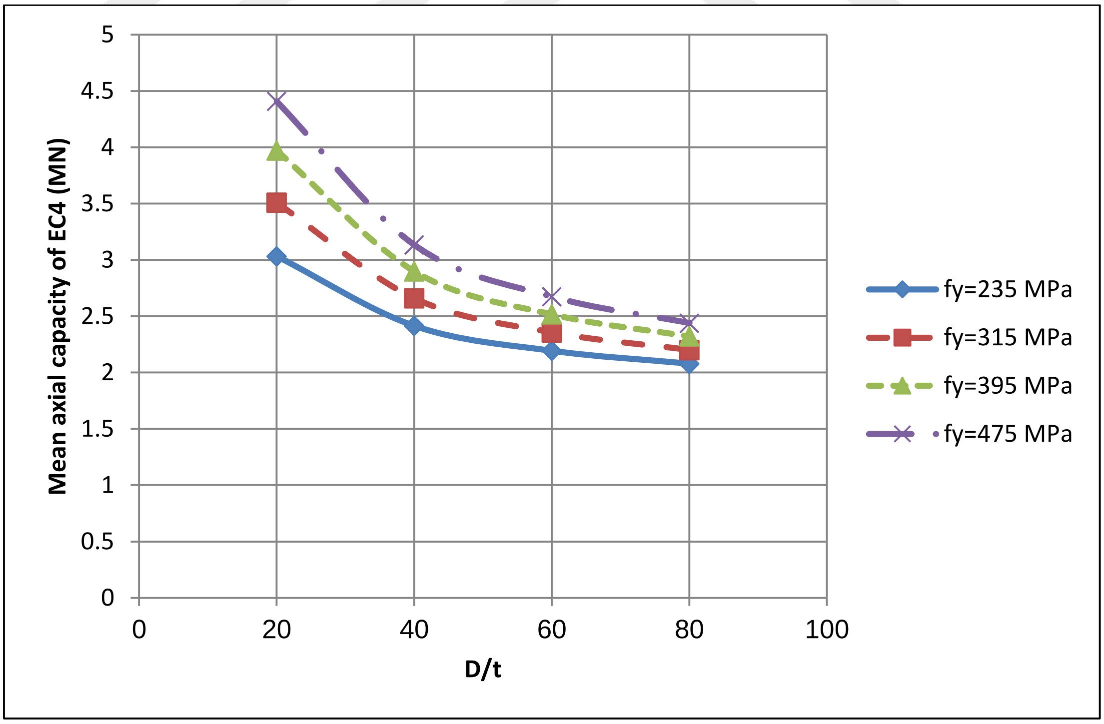


Figure 4.13 Axial Capacity of CFST Columns vs. D/t Ratio with different steel yield strength

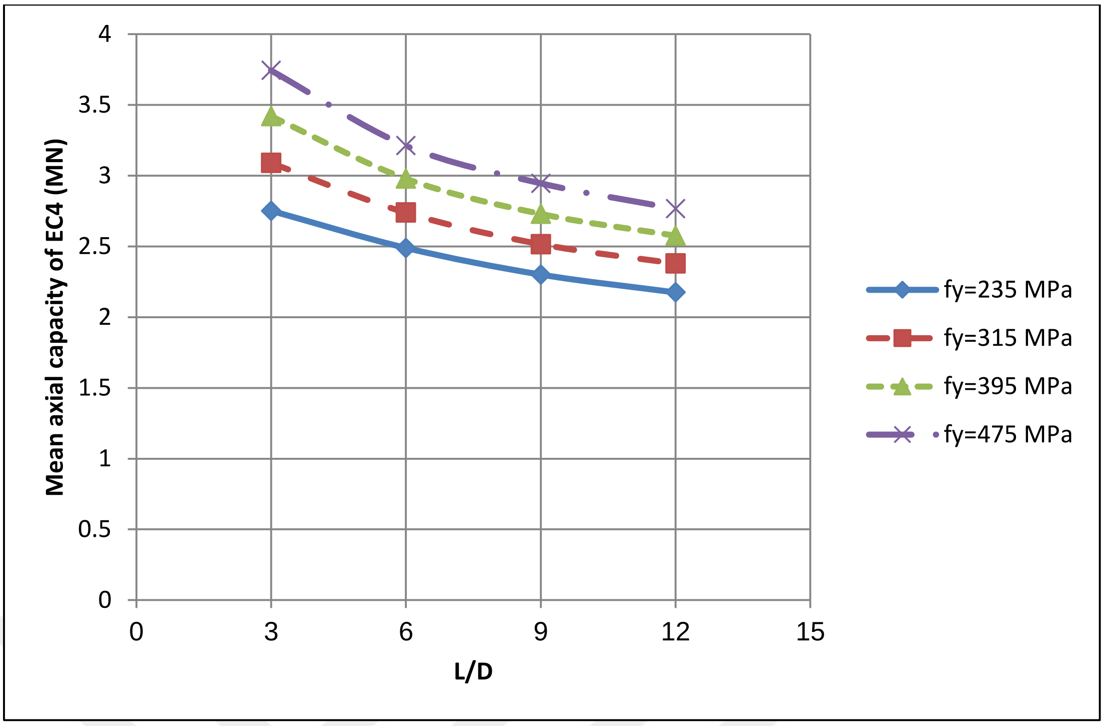


Figure 4.14 Axial Capacity of CFST Columns vs. L/D Ratio with different steel yield strength

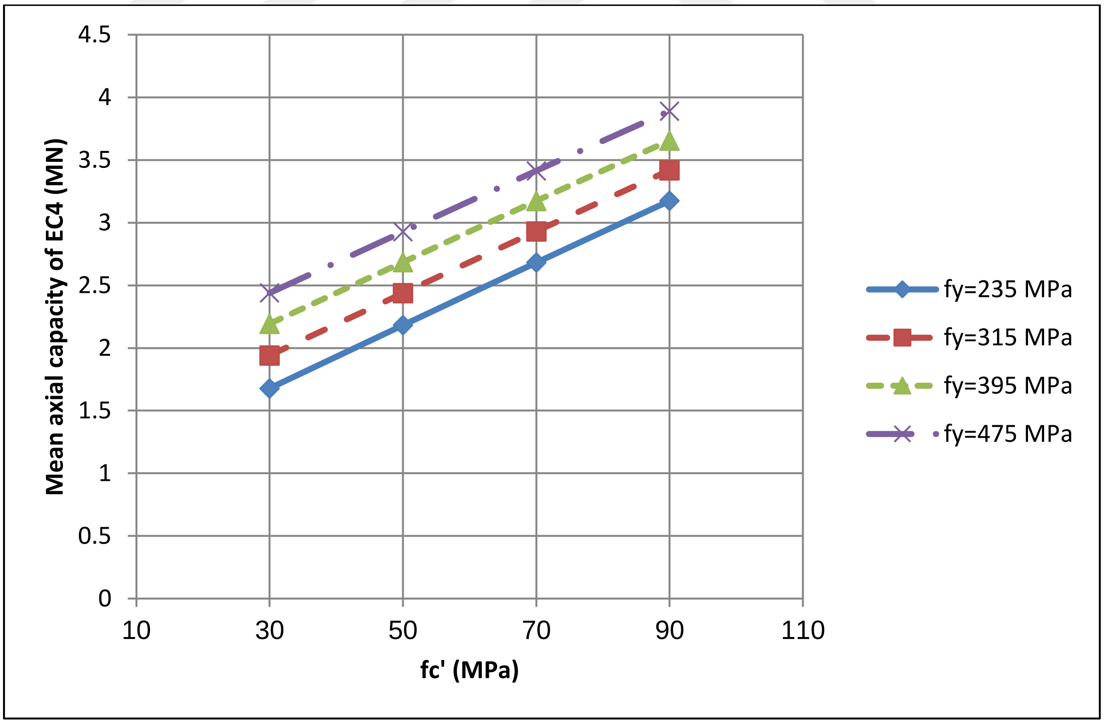


Figure 4.15 Axial Capacity of CFST Columns vs. compressive strength of concrete with different steel yield strength

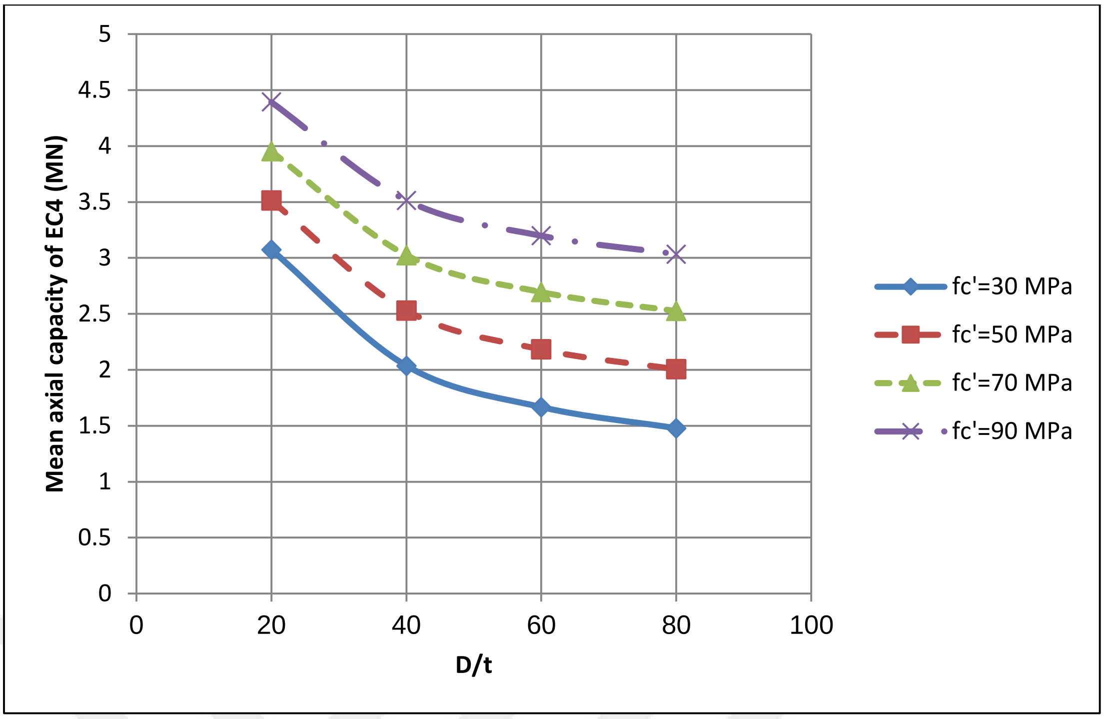


Figure 4.16 Axial Capacity of CFST Columns vs. D/t Ratio with different concrete compressive strength

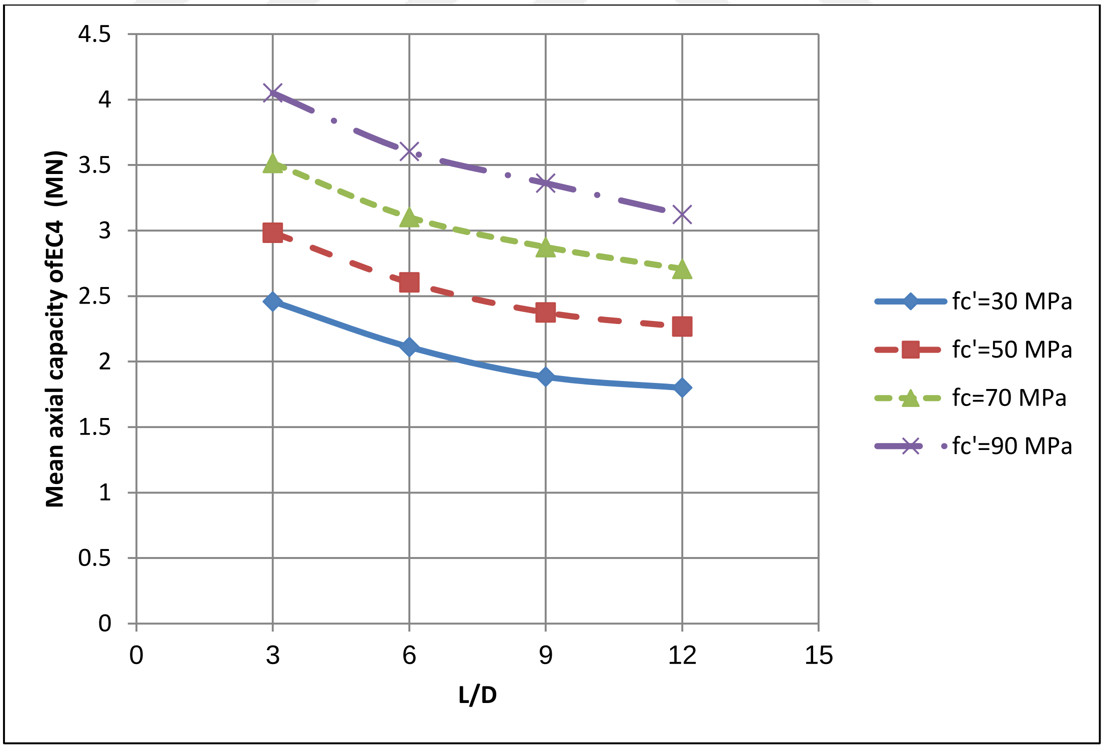


Figure 4.17 Axial Capacity of CFST Columns vs. L/D Ratio with different concrete compressive strength

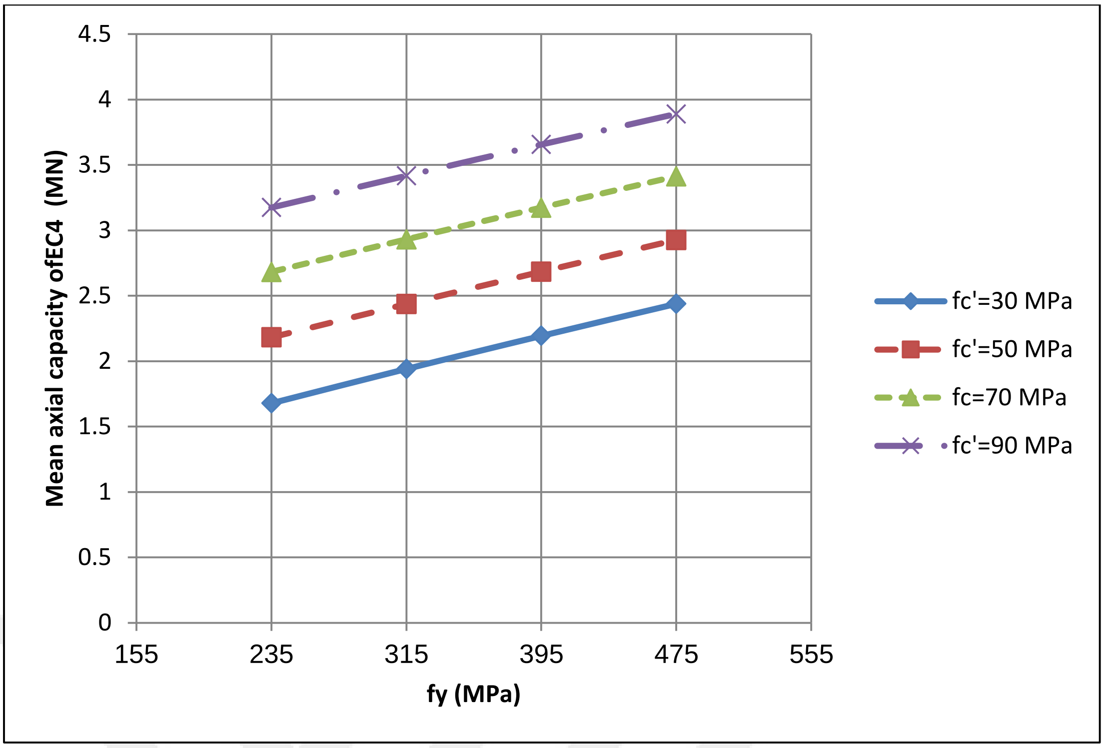


Figure 4.18 Axial Capacity of CFST Columns vs. steel yield strength with different concrete compressive strength

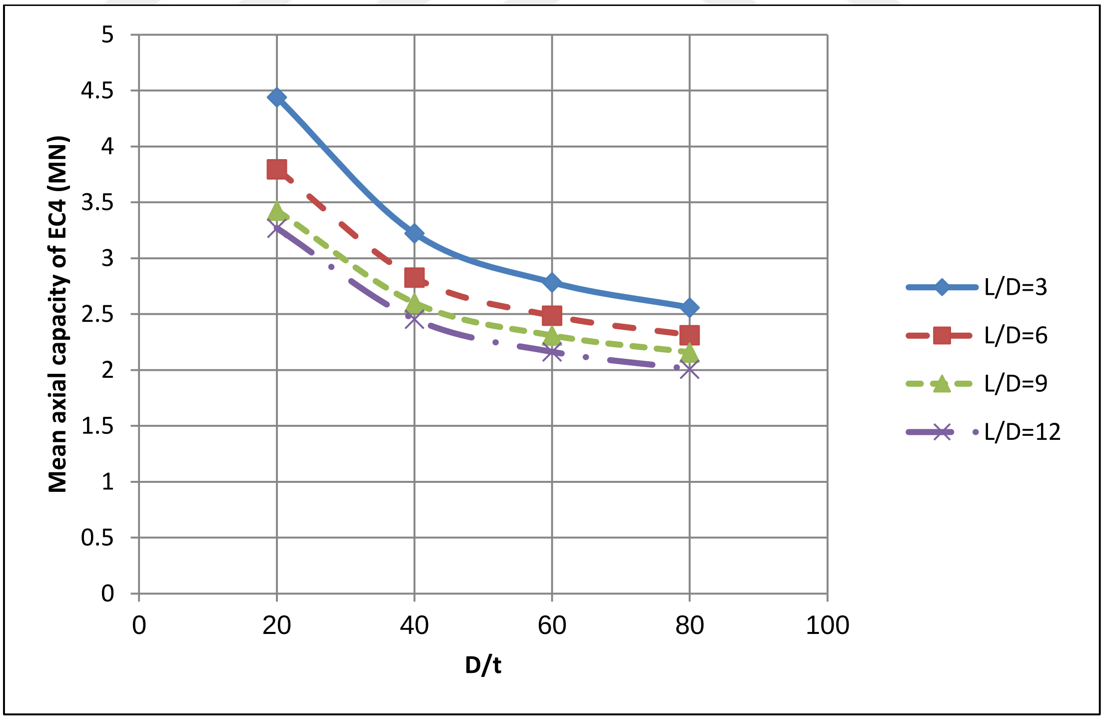


Figure 4.19 Axial Capacity of CFST Columns vs. D/t Ratio with different L/D Ratio

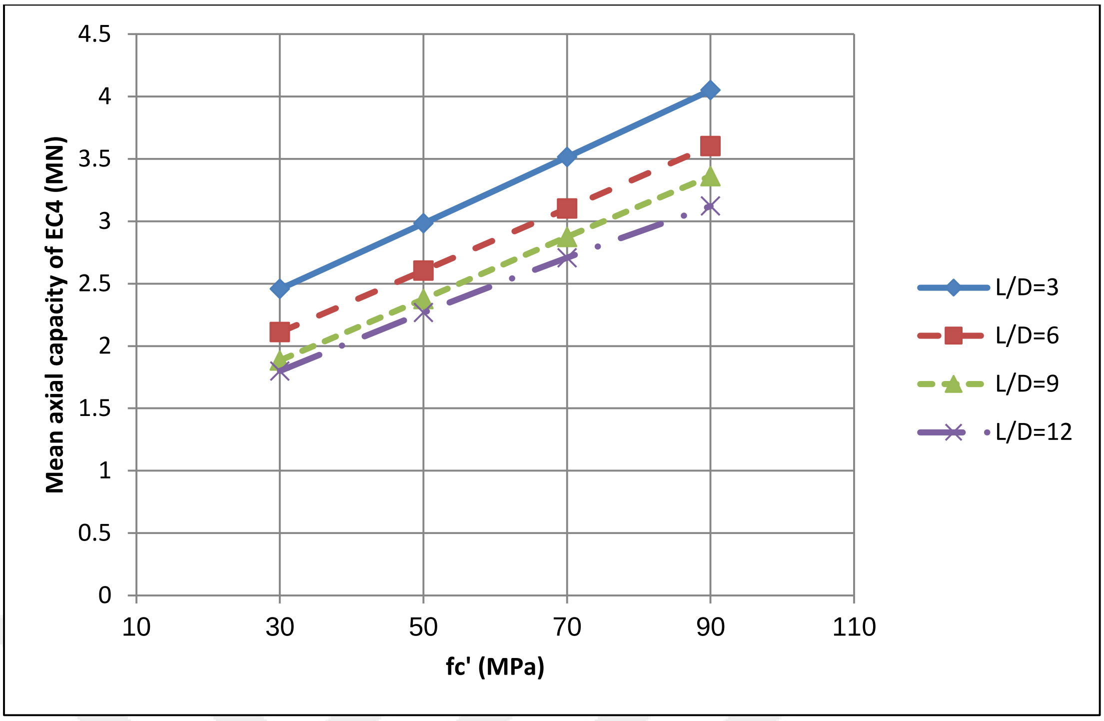


Figure 4.20 Axial Capacity of CFST Columns vs. concrete compressive strength with different L/D Ratio

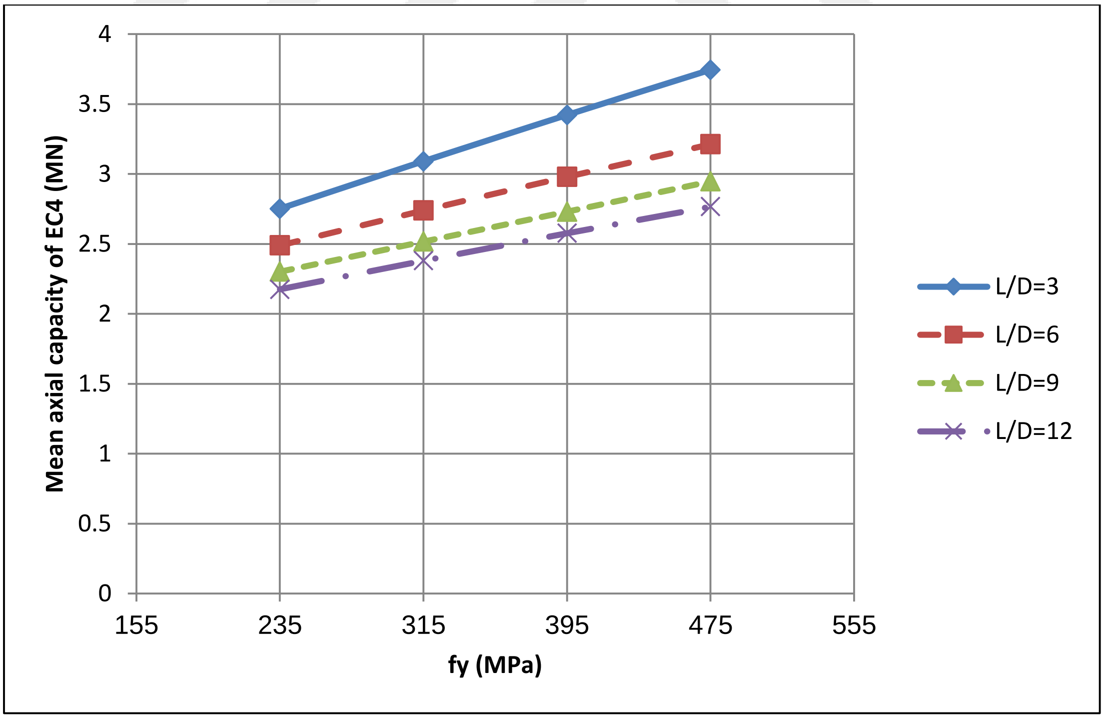


Figure 4.21 Axial Capacity of CFST Columns vs. steel yield strength with different L/D Ratio

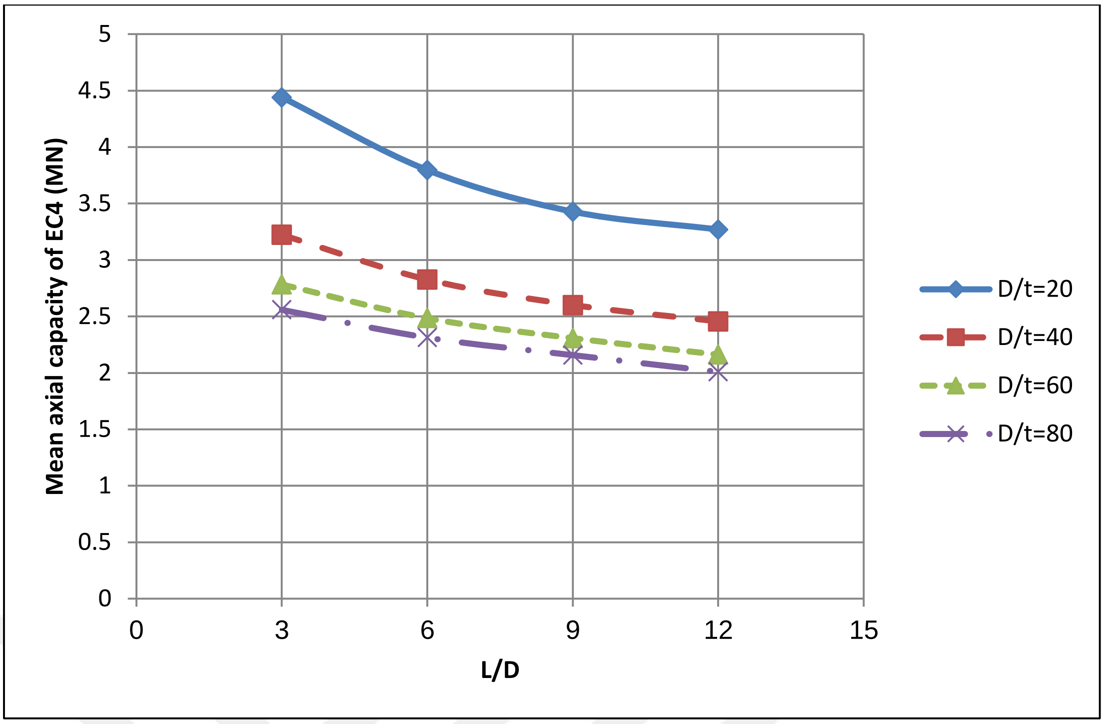


Figure 4.22 Axial Capacity of CFST Columns vs. L/D Ratio with different D/t Ratio

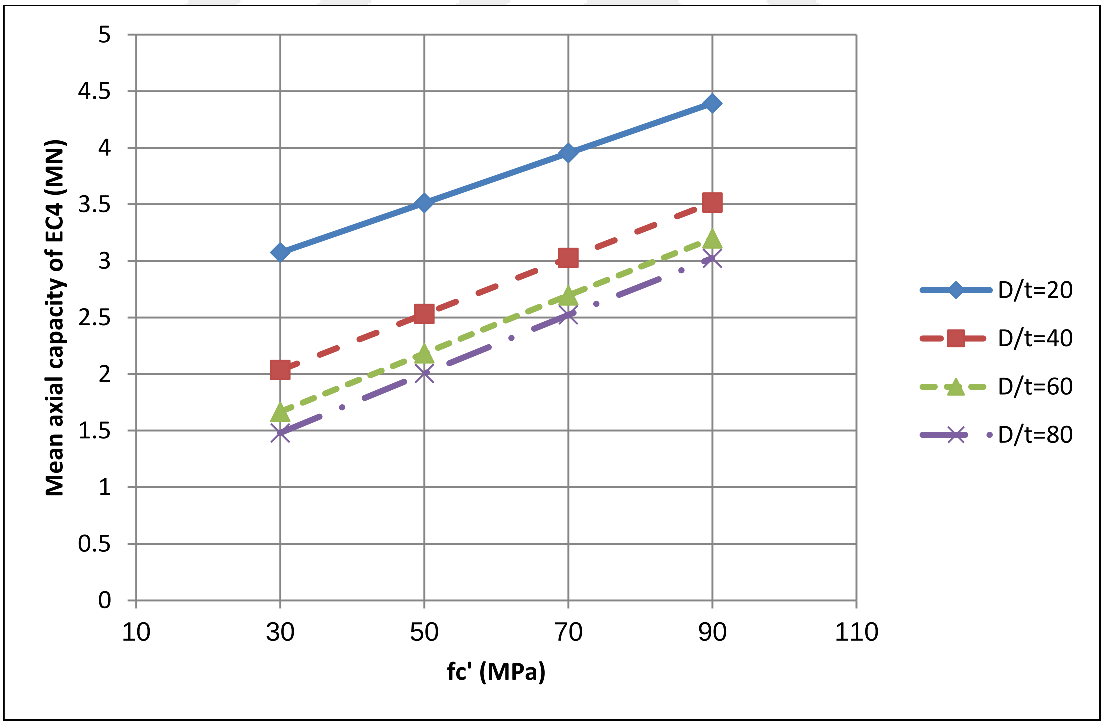


Figure 4.23 Axial Capacity of CFST Columns vs. Concrete compressive strength with different D/t Ratio

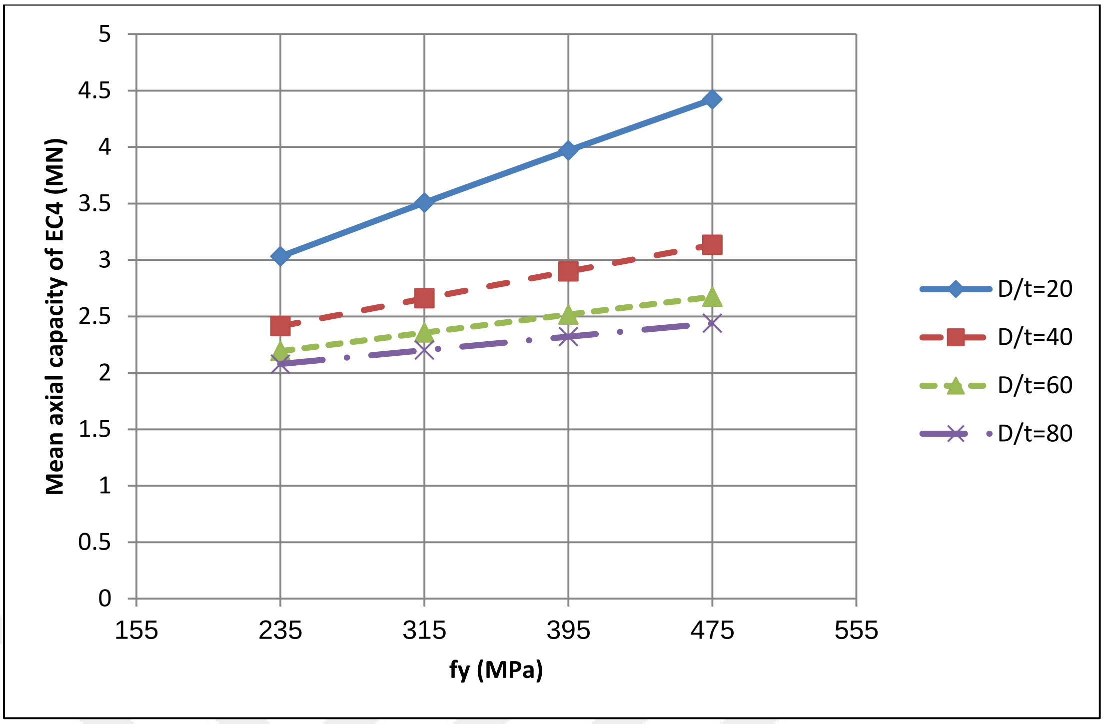


Figure 4.24 Axial Capacity of CFST Columns vs. Steel yield strength with different D/t Ratio

4.5.1 Effect of Concrete Compressive Strength f_c'

The effect of concrete compressive strength f_c' is shown in Figure 4.25 for both AISC 360 – 16 and EC4 codes that indicate that the axial capacity of columns increase as f_c' increases because of the composite columns have more confining tube in increasing the infill concrete compressive strength f_c'. As shown in Figures 4.1 to 4.24, the maximum axial capacity gives when f_c' = 90 MPa with D/t = 20 compared with others parameters.

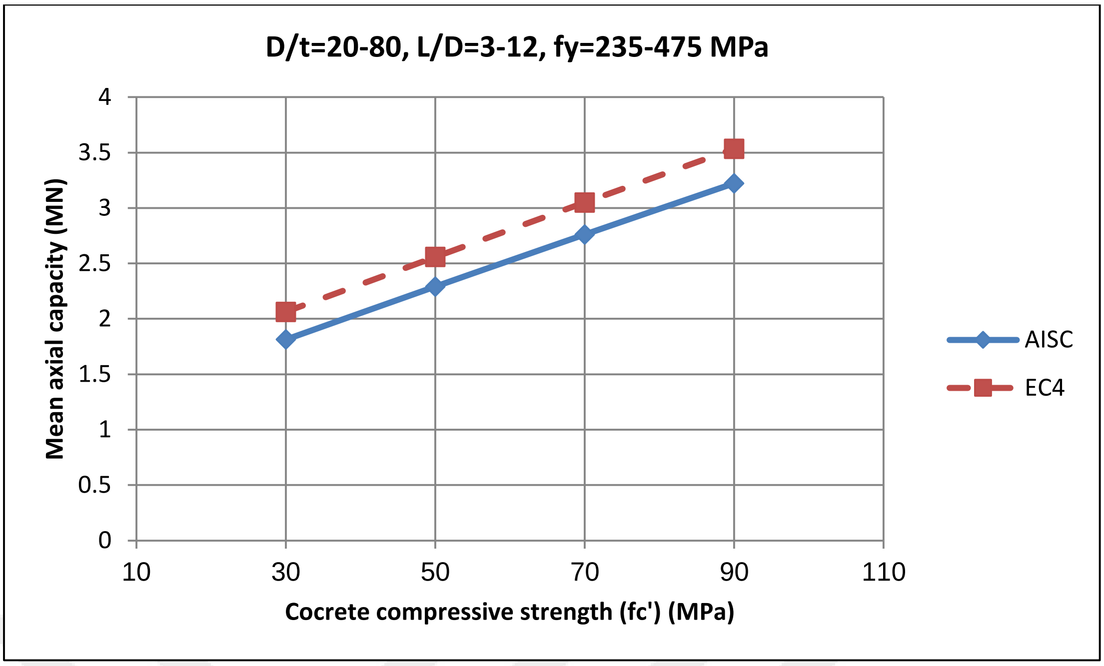


Figure 4.25 Effect of f_c' on Axial Capacity of CFST Columns

4.5.2 Effect of Steel Yield Strength f_y

Figure 4.26 shows the influence of f_y, for both the AISC 360 – 16 and EC4 codes the axial capacity of the CFST column increased as increase in f_y increases. Figures 4.1 to 4.24, present the maximum axial capacity gives when f_y = 475 MPa with D/t = 20 compared with others parameters.

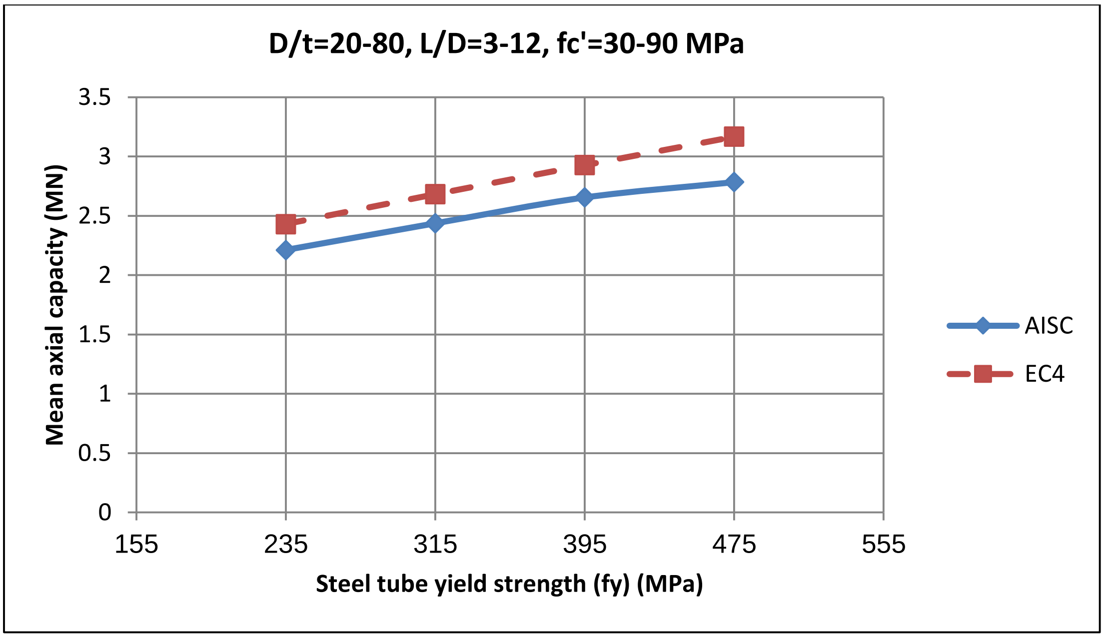


Figure 4.26 Effect of f_y on Axial Capacity of CFST Columns

4.5.3 Effect of Diameter – to – Thickness *D/t* Ratio

D/t ratio classified as slenderness ratio that significantly effect on the local buckling stability of the CFST columns, however, the for AISC 360 – 16 the local buckling accounted according to the classification of cross section as compact, non-compact and slender, while the EC4 code the local buckling occurs when this ratio passed the maximum value. The axial capacity of the CFST column decrease when D/t increases due to the reduction in confinement provided by small thickness. Figure 4.27 shows the influence of D/t ratios on the axial capacity of the CFST columns. However, the D/t = 20 gives the maximum effect with compared with other values (D/t = 40, 60 and 80).

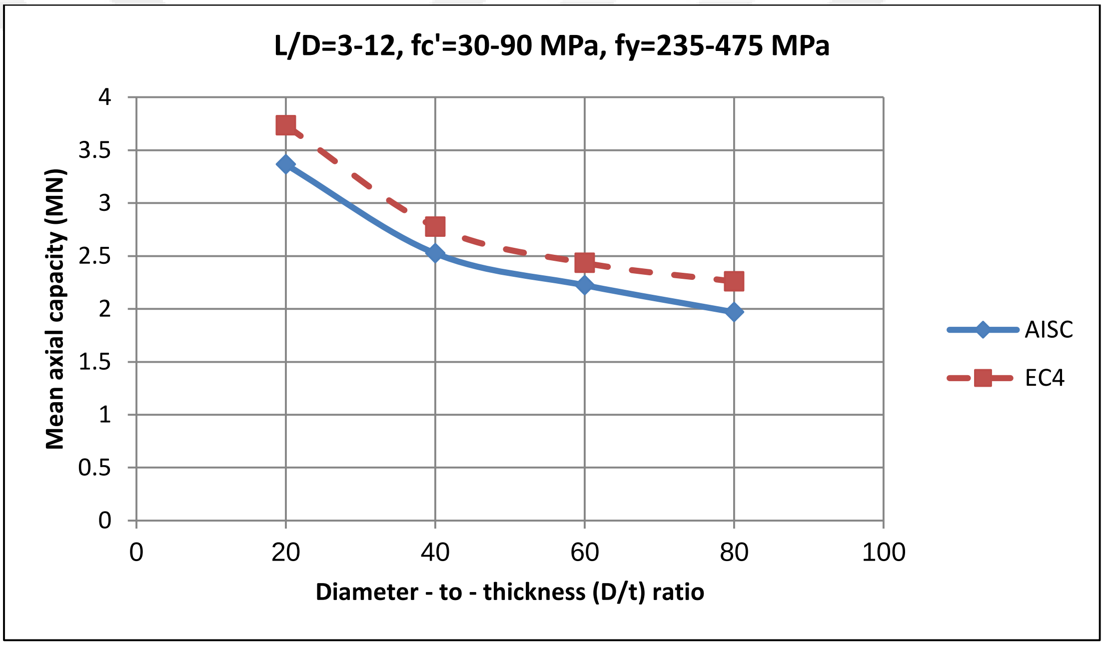


Figure 4.27 Mean Effect Plot of D/t Ratio

4.5.4 Effect of Length – to – Diameter *L/D* Ratio

The L/D ratio affects the axial capacity and the confinement effect of the CFST columns, where, both decrease when L/D ratio increased (de Oliveira *et al.*, 2009; Ekmekyapar and Al-Eliwi, 2016). Figure 4.28 shows the mean effect of L/D ratio on the axial capacity of the CFST columns for both AISC 360 – 16 and EC4. Where the axial capacity of short columns (L/D = 3) greater than long columns (L/D =6,9 and 12). Also from Figures 4.1 to 4.25 the L/D = 3 with D/t ratio = 20 gives the maximum axial capacity in compared with other parameters.

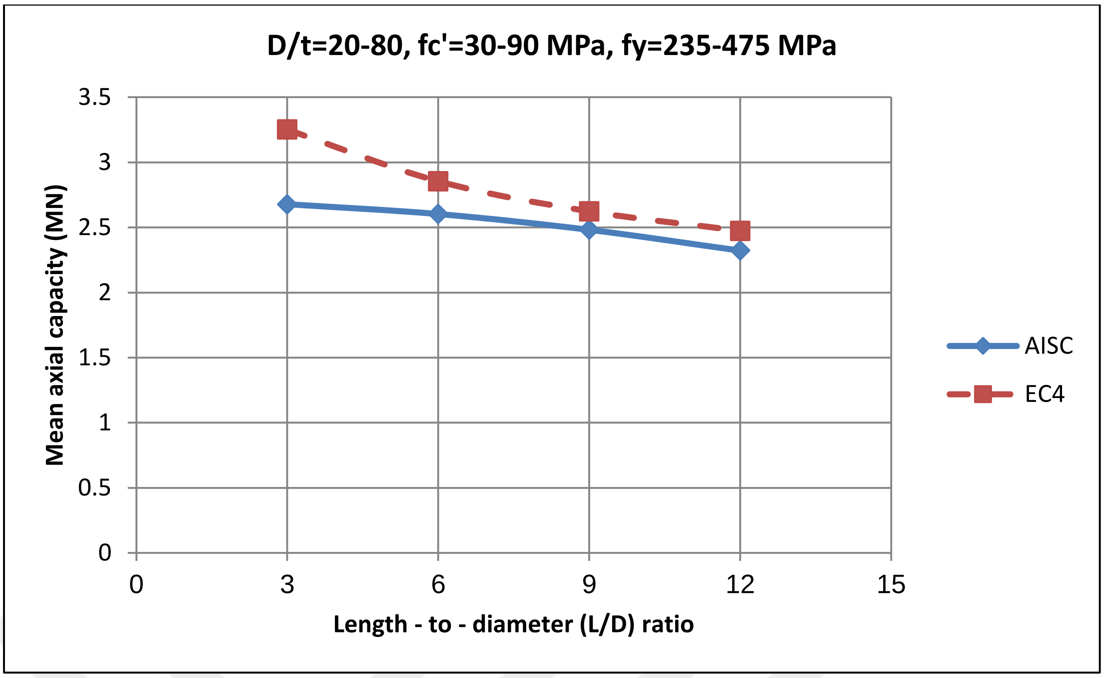


Figure 4.28 Mean Effect Plot of L/D Ratio

4.5.5 The Strength Index (*SI*) and the Confinement Index (ξ)

In the design of the CFST columns, most codes recognize the effect of the "composite action" especially. Therefore, the strength of the composite member is enhanced. The Strength index *SI* and the confinement index ξ are very useful measures for composite action and confinement assessments in CFST columns. where ξ is defined in equation 4.1 and *SI* is defined in equation 4.2.

The following Figures 4.29 to 4.32 show the effect of parameters of the parametric study on the strength index. For AISC 360 – 16 *SI* ranges from 0.718 to 1.087 and for EC4 ranges from 0.92 to 1.368 by increasing in mean about 11.43%, this in difference is due to the EC4 code take the confinement effect in its consideration. Figures 4.29 and 4.30 show that *SI* for normal strength concrete (f_c' = 30 and 50 MPa) greater than high strength concrete (70 and 90 MPa) and for mild steel strength (f_y = 235, 315 and 395 MPa) greater than higher strength steel strength (475 MPa) because the squash load of the CFST column depends on cross – section and the materials properties f_c' and f_y.

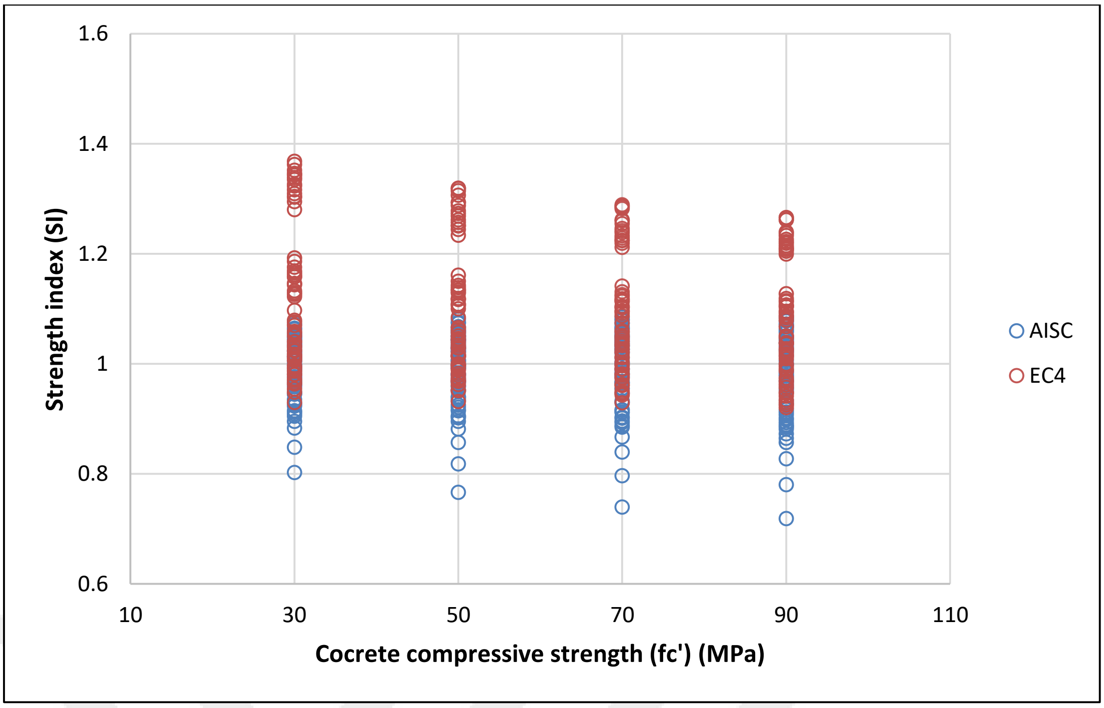


Figure 4.29 Effect of f_c' on the Strength Index *SI*

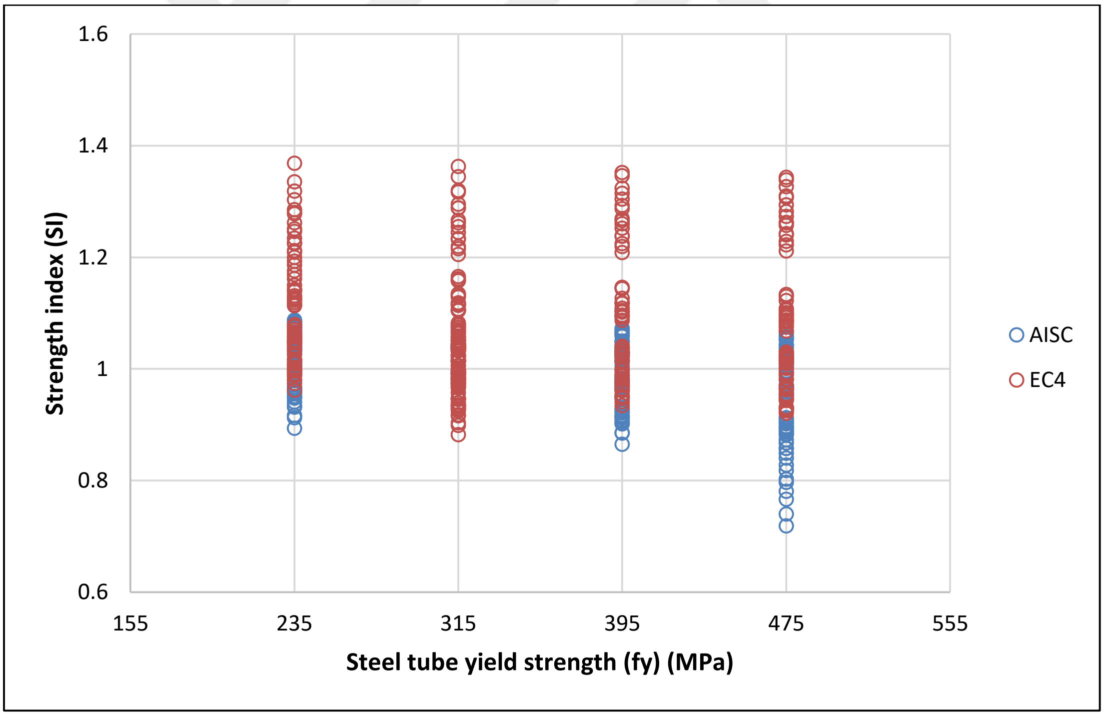


Figure 4.30 Effect of f_y on the Strength Index *SI*

Figure 4.31 shows the effect of D/t ratio on the strength index for both AISC 360 – 16 and EC4 codes. The strength index decreased when the D/t ratio increases this means the thicker tube provides confinement more than, the thinner tube. The column's ductility decreases as the concrete compressive strength increases for higher D/t ratios, but for smaller D/t ratios the opposite is true (Abed *et al.*, 2013).

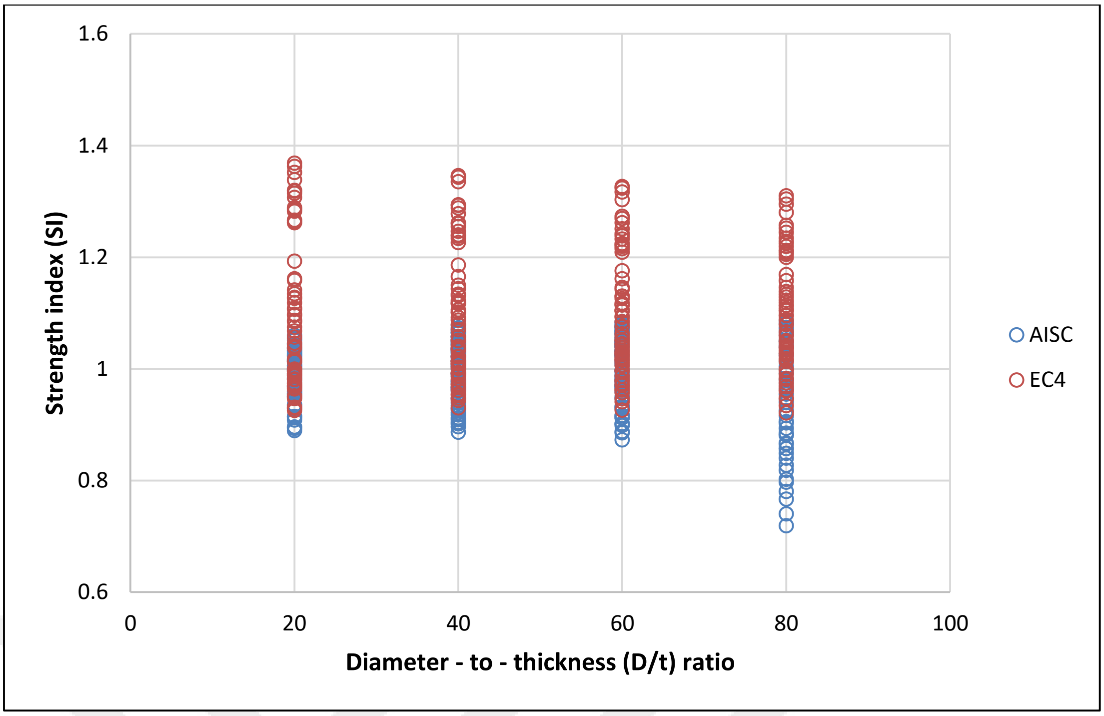


Figure 4.31 Effect of D/t ratio on the Strength Index *SI*

As shown in Figure 4.28, the L/D ratio has a direct impact on the axial capacity of the CFST column. The short columns have an axial capacity greater than long columns for both AISC 360 – 16 and EC4. This also clear in Figure 4.32 where the short columns have strength index more than unity particularly for EC4 due to the effect of the L/D ratio on the confinement index, where the confinement decrease when L/D ratio increases. For a column with small L/D ratio, the failure is recognized by material yielding while for high L/D ratio the failure is characterized by global instability with small deformation before the facing the confinement (de Oliveira *et al.*, 2009).

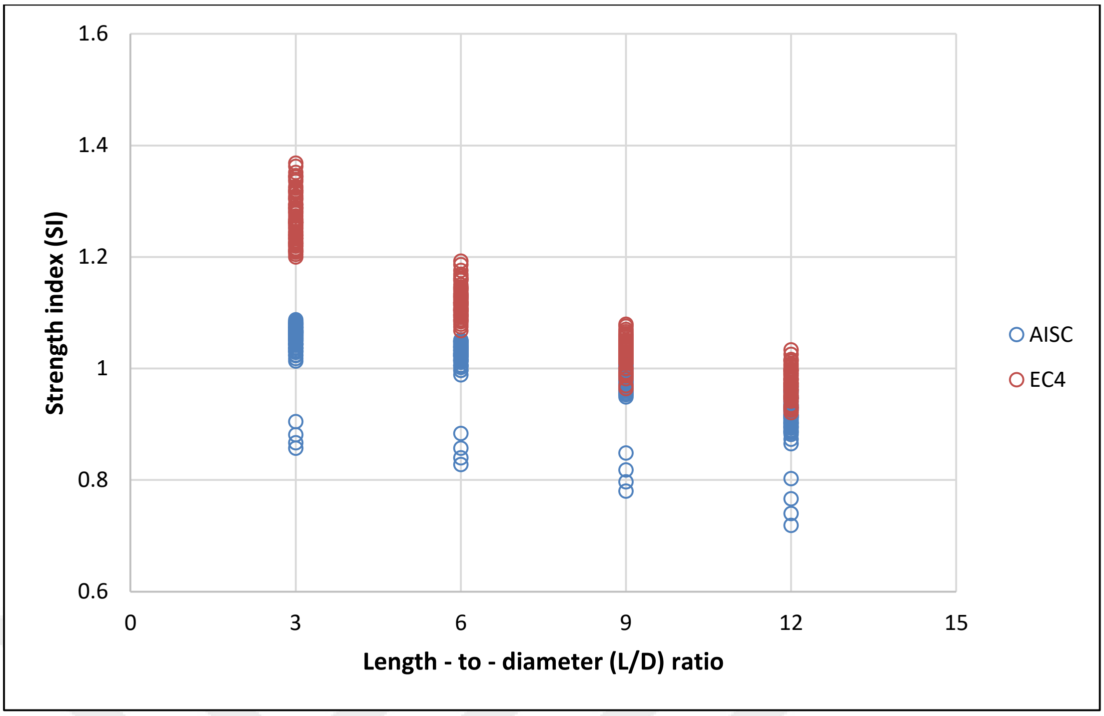


Figure 4.32 Effect of L/D ratio on the Strength Index *SI*

The confinement index ξ is a function of D/t ratio, as well as the material properties f_c' and f_y, for this parametric study the confinement index ranges from 0.135 to 3.713. Figures 4.33 to 4.36 show the relation between the D/t ratios and the confinement index ξ with different values of f_c' and f_y. It is observed that the samples of D/t ratios = 20 with f_c' = 30 MPa and f_y = 475 MPa are more affected on the confinement index.

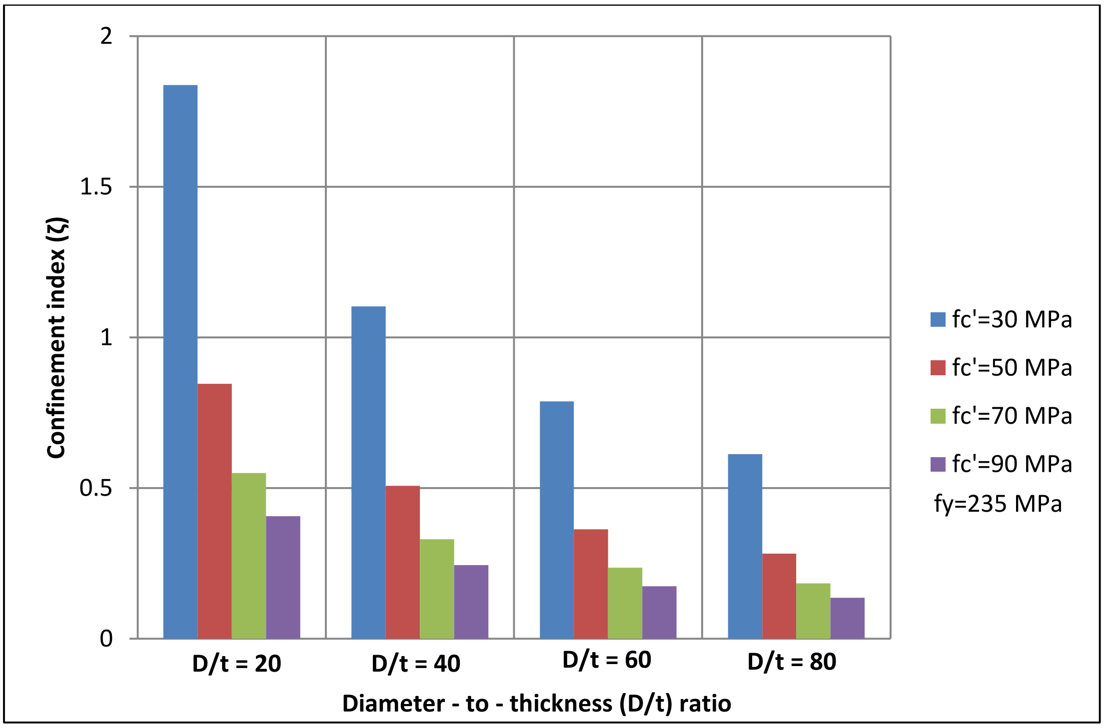


Figure 4.33 D/t Ratio Versus the Confinement Index ξ, f_y = 235 MPa

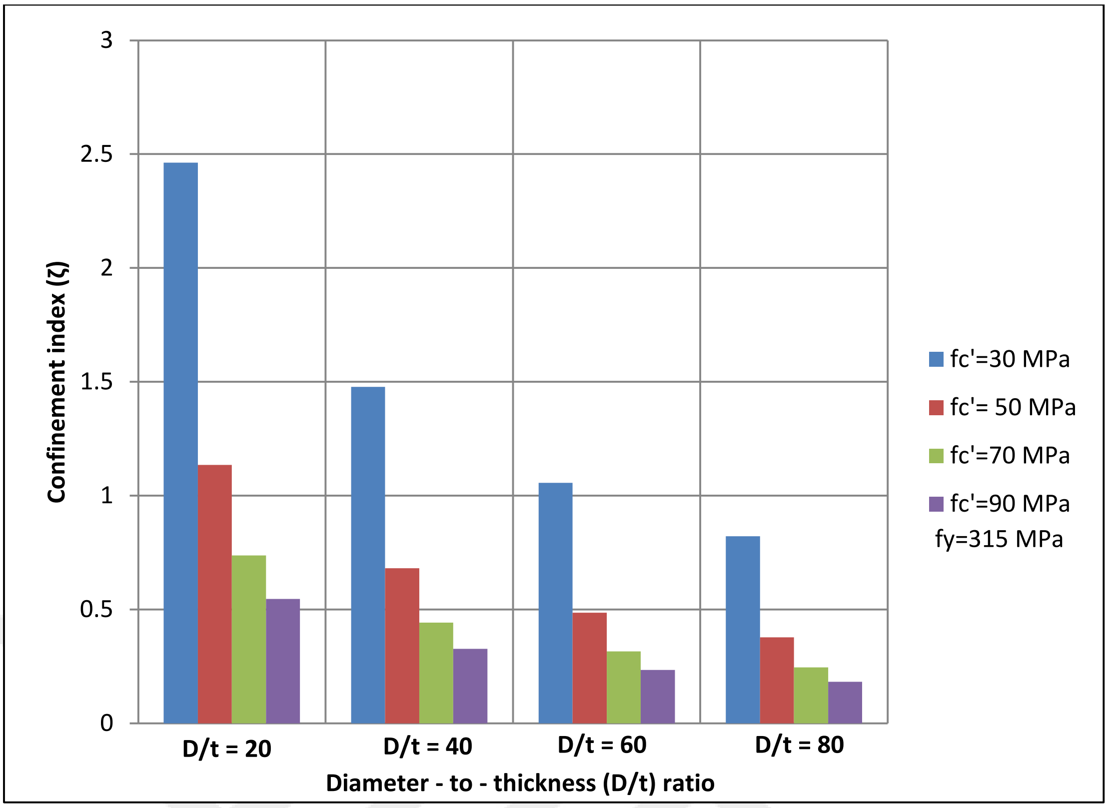


Figure 4.34 D/t Ratio Versus the Confinement Index ξ, f_y = 315 MPa

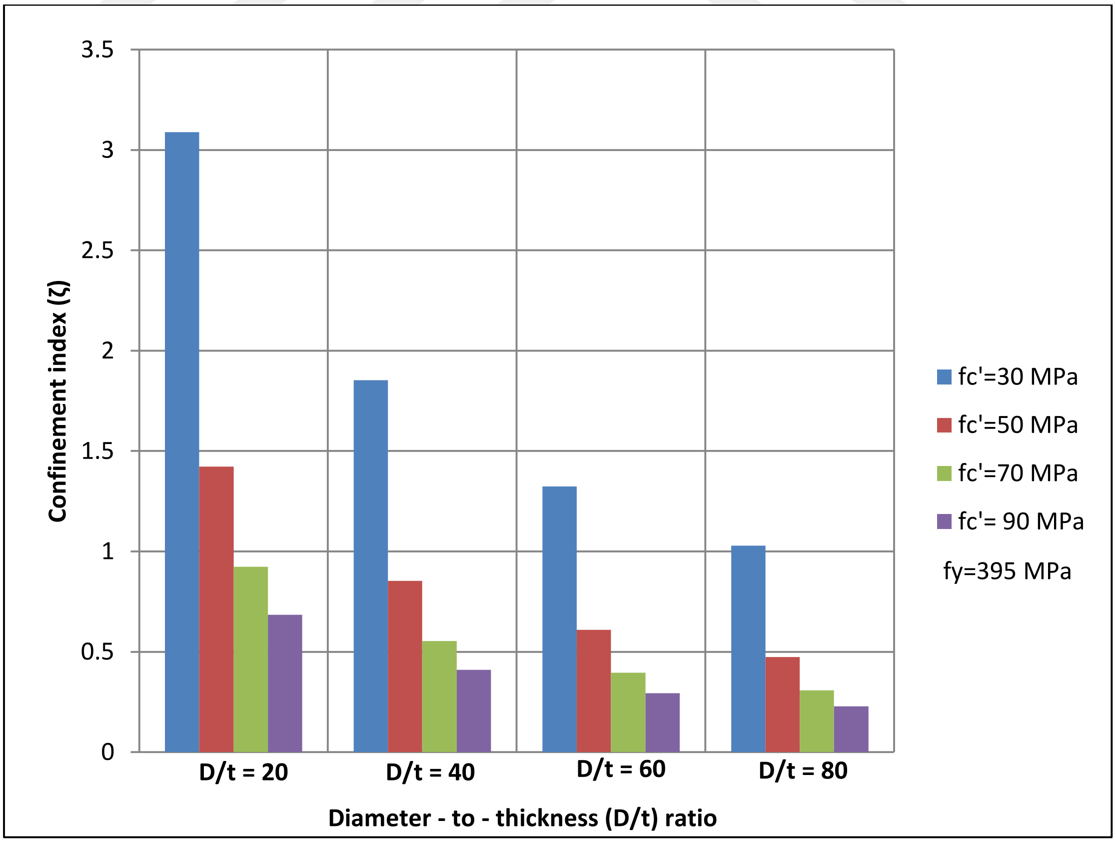


Figure 4.35 D/t Ratio Versus the Confinement Index ξ, f_y = 395 MPa

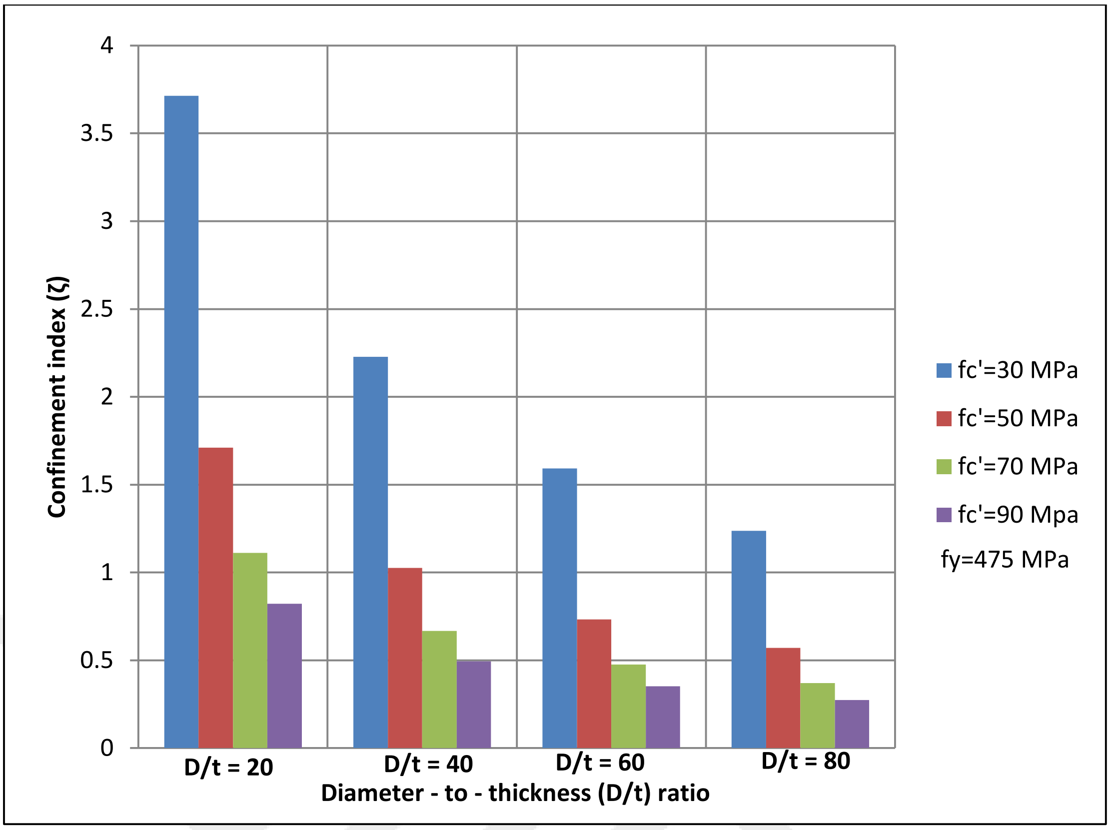


Figure 4.36 D/t Ratio Versus the Confinement Index ξ, f_y = 475 MPa

4.6 Discussions

The analysis results are lists in Table (4.1), that represents the comparisons in predictions between AISC 360 – 16 and EC4 codes. The percentage of the difference of the axial capacity for total columns is 11.09%, this difference due to the confinement effect and the variation between the limitations of codes for mechanical and geometrical properties, when classify the columns specimens according to length, the difference is 21.43% this exactly due to the confinement where EC4 takes in its consideration the confinement effect for short column while for long columns the difference decreased to 7.36% because no confinement effect for long column in EC4 calculations. The differences in strength index between short and long columns are 21.62% and 7.76% for the same reasons that mentioned above. The compressive strength of concrete classified as normal and high strength concrete, in case of (NSC) and (HSC), the difference is little as 12.59% and 10.07% but in general the confinement is more effective when the infilled concrete is NSC due to its higher deformation capacity in comparison with the HSC (de Oliveira *et al.*, 2009). this result is clear to observe in mean of *SI*EC4 between NSC and HSC due to confinement effect while it is slight difference in *SI*AISC because no big effect of confinement in this code. The same conclusion observed when comparisons between the mean of the confinement index ξ between NSC and HSC. While there is no effect of length on the mean of ξ because the equation 4.1 do not take the length in calculations. conversely for steel tube strength, where, the mild steel strength (MSS) ($f_y \leq 460$ MPa) gives lower confinement than high tensile steel strength (HSS) ($f_y > 460$ MPa).

Table 4.1 Summary of the Results

	Mean of P_{AISC} (MN)	**Mean of P_{EC4} (MN)**	**P_{EC4}/P_{AISC} (%)**	**Mean of SI_{AISC}**	**Mean of SI_{EC4}**	**SI_{EC4}/SI_{AISC} (%)**	**Mean of ζ**
Total columns	2.521	2.801	11.09	0.985	1.098	11.43	0.812
Short columns	2.678	3.252	21.43	1.046	1.272	21.62	0.812
Long columns	2.469	2.651	7.36	0.966	1.041	7.76	0.812
NSC	2.052	2.310	12.59	0.988	1.114	12.74	1.100
HSC	2.991	3.292	10.07	0.983	1.083	10.13	0.524
MSS	2.434	2.679	10.07	1.001	1.104	10.32	0.720
HSS	2.784	3.167	13.78	0.939	1.081	15.01	1.086

CHAPTER 5

CONCLUSIONS

5.1 Introduction

In this research, theoretical approach to calculate the axial load capacity of circular composite columns filled by concrete with any reinforcements inside. The two internal codes as AISC – 360 – 16 and EC4 - 04 are adopted in present study to evaluate such columns. Different parameters are considered such as length to diameter ratios, diameter to thickness ratios, mechanical properties of concrete and steel.

Many parameters were founded to evaluate the axial load capacity of the circular composite columns such as confinement factor (ξ) and the strength index (SI) of the CFST.

The comparisons between two global codes as axial load capacity of composite column as AISC – 360 – 2016 and EC 4 – 2004 were adopted. The study consisted of many classifications of columns such as compact; non-compact and slenderness rely on the diameter to thickness ratio.

5.2 Discussions

The comparisons between two international codes that adopted in present study as AISC – 360 – 2016 and EC4-2004. The axial load capacity of CFST in case of different mechanical properties of concrete and steel are considered in addition of different (L/D), (D/t) ratios. Based on the analysis results, the EC4 gave axial capacity greater than AISC – 360 and become conservative in design.

Based on the analysis results according to the procedure adopted by AISC – 360 – 2016 and EC4-2004, followings are important points that noticed from analysis results of CFST columns:

1. The variation of geometrical and mechanical properties of specimens suggested covering the practical cases in this field.

2. These specimens were within and out of the limitations of the AISC 360 – 16 and EC4 codes.
3. The AISC 360 – 16 and EC4 codes depend on different functions to estimate the axial load capacity of CFST columns. Therefore, there is the difference in results between them.
4. The EC4 takes the confinement effect within its consideration that represents by the term $(1+\eta_{co}\frac{t}{d}\frac{f_y}{f_c})$, while for in case of AISC 360 – 16 code was assumed to be constant.
5. The diameter of the steel tube has the most significant effect on the ultimate axial load capacity of the CFST columns as the results of parametric study taking into account because of the axial capacity affected by other parameters.
6. The parameters of geometrical and mechanical properties of specimens are the effect on the predictions of both codes with different percentages.
7. The analysis of variance showed that the D/t ratios and f_c' have the more effective parameters than others and the maximum interaction occurred for D/t ratio and f_y.
8. The CFST column of (D/t = 20, L/D = 3, f_c' = 90 MPa, and f_y = 475 MPa) gives the maximum axial capacity for AISC 360 – 16 and EC4 respectively by difference about 21.7% where the EC4 takes the confinement effect on its consideration. While the CFST column of (D/t = 80, L/D = 12, f_c' = 30 MPa, and f_y = 235 MPa) gives the minimum axial capacity for AISC 360 – 16 and EC4 respectively by difference about 7.3%.
9. The axial capacity increased when f_c' and f_y increase, while decreased when D/t ratio and L/D ratio increase.
10. Strength index *SI* and confinement index ξ are very useful measures for composite action and confinement assessments in CFST columns. *SI* for NSC and short column is greater than HSC and long column, while the HSS gives confinement index more than MSS.
11. The strength index *SI* of the CFST columns increased as axial load capacity increased because of the strength index *SI* represent the ratio of column loading to the contributions of concrete and steel to resist the loading. For a certain values of compressive strength, the strength index *SI* increase in case of decreased in mechanical properties of concrete and steel.

12. The confinement index ξ increased when the compressive strength of concrete decreased and the yield strength of steel increased. Also, the confinement index ξ increased as the (D/t) decreased because in this case the steel thickness of the section increased.
13. The axial load capacity of CFST by applied EC4 have values greater than AISC – 360 for the same mechanical properties of the materials with the same (L/D) and (D/t) ratios because of the EC4 factors are greater than AISC – 360.
14. In case of the column slenderness increases, the difference in predictions of the axial capacity between AISC 360-16 and EC4 become closer.

5.3 Recommendations

Based on the analysis results and literature review, followings are recommendations for design of CFST columns:

1. CFST is used in the buildings because it gives the high axial capacity.
2. Analysis of CFST taking into account partial interactions theory.
3. Discover the frictional forces between concrete and steel effect on the axial capacity of CFST.
4. Weld shear stud connectors inside steel section to provide more connections between concrete and steel.

5.4 Suggestions for Future Works

Refer to the approach and results obtained from the present work, future works suggested to continue along the following lines:

1. Behavior and axial capacity of circular concrete-filled steel tube columns under dynamic loading.
2. Nonlinear analysis of axially loaded circular concrete filled steel tube columns under combined axial loading and moment.
3. Ultimate capacity prediction of axially loaded CFST filled by normal concrete with steel fiber.

REFERENCES

Abd el Fattah, Ahmed Mohsen. (2012). Behavior of Concrete Columns Under Various Confinement Effects, Ph. D. thesis, Kansas, USA.

Abed, F., Al Hamaydeh, M., and Abdalla, S. (2013). Experimental and Numerical Investigations of the Compressive Behavior of Concrete Filled Steel Tubes (CFSTs). *Journal of Constructional Steel Research Engineering Structures*, **80**, 429–439.

Aboutaha, R.S., and Machado, R. (1998). Seismic Resistance of Steel Confined Reinforced Concrete (SCRC) Columns. *The Structural Design of Tall Building*, **7**, 251-260.

ACI. American Concrete Institute. (1995). ACI Committee 318 – 95. Building code requirements for structural concrete and commentary, Detroit. USA.

AISC 360-16. Specification for Structural Steel Buildings, in, Alsc, Chicago, Illinois, 2016.

Ajel, A. H., Abbas, M. A. (2015). Experimental and Analytical Investigations of Composite Stub Columns. *International Journal of Engineering Trends and Technology*, **4**, 185-200.

AS. Australia Standards. (1998). AS4100. Steel structures. Sydney, Australia.

Associação Brasileira de Normas Técnicas. NBR 8800:2008. Projeto e execução de estruturas de aço e de estruturas mistas aço-concreto de edifícios: Procedimento. Rio de Janeiro, Portuguese.

ASTM A1085/A1085M – 15. Standard Specification for Cold-Formed Welded Carbon Steel Hollow Structural Sections (HSS).

Baig, E. M., Jiansheng F., Jianguo, N. (2006). Strength of Concrete Filled Steel Tubular Columns. *Tsinghua science and technology***, 11**, 657-666.

Berge M. (1998). Composite columns. Master‘s Thesis, New Jersey Institute of Technology, USA.

CAN/CSA. Canadian Standards Association. (2001). S16-01. Limit states design of steel structures. Ontario, Canada.

Chitawadagi, M. V., Narasimhana, M. C. and Kulkarni, S. M. (2010). Axial strength of circular concrete-filled steel tube columns DOE approach. *Journal of Constructional Steel Research*, **66**, 1248-1260.

Design, A. S. (1989). Specification for structural steel buildings. *AISC, June, 1*.

De Oliveira, W. L. A., De Nardin, S., de Cresce El Debs, A. L. H. and 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**, 2103-2110.

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

EC4. Euro code 4. (2004). Design of composite steel and concrete structures. Part 1.1, General rules and rules for buildings. EN 1994-1-1. European Committee for Standardization. British Standards Institution.

Ekmekyapar, T. and Al-Eliwi, B.J.M. (2016). Experimental behavior of circular concrete filled steel tube columns and design specifications. Thin Walled Structure, **105**, 220-230.

Ellobody, E. and Young, B. (2006). Nonlinear analysis of concrete-filled steel SHS and RHS columns. *Thin-Walled Structures*, **44**, 919-930.

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

Eurocode 2: Design of concrete structures - Part 1-1. (2004). General rules and rules for buildings. EN 1992-1-1. European Committee for Standardization. British Standards Institution.

Eurocode 3: Design of steel structures - Part 1-1. (2004). General rules and rules for buildings. EN 1993-1-1. European Committee for Standardization. British Standards Institution.

Evirgen, B., Tuncan, A. and Taskin, K. (2014). Structural behavior of concrete filled steel tubular sections (CFT/CFST) under axial compression. Thin Walled Structure, **80**, 46-56.

Fujimoto, T., Mukkai, A, Nishiyama, I. and Sakino, K. (2004). Behavior of eccentrically loaded concrete-filled steel tubular columns. *Journal* of *Structural Engineering*, **130**, 203-212.

Giakoumelis, G. and Lam, D. (2004). Axial capacity of circular concrete-filled tube Columns. *Journal of Constructional Steel Research,* **60**, 1049–1068.

Gourley, B. C., Tort, C., Hajjar, J. F. and Schiller, P. H. (2001). A Synopsis of studies of the monotonic and cyclic behaviour of concrete-filled steel tube beam-columns. *Structural Engineering Report No. ST-01-4,* Institute of Technology, University of Minnesota.

Grauers, M. (1993). Composite Columns of Hollow Steel Sections Filled with High Strength Concrete. PhD, Thesis, Division of Concrete Structures, *Chalmers University of Technology,* Sweden.

Gupta,P. K., Sarda, S. M. and Kumar, M. S. (2007). Experimental and computational study of concrete filled steel tubular columns under axial loads. *Journal of Constructional Steel Research,* **63**, 182–193.

Han, L.H. (2002). Tests on stub columns of concrete-filled RHS sections. *Journal of Constructional Steel Research,* **58**, 353-372.

Han, L.-H., Yao, G.-H., and 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**, 1241-1269.

Han, L. H. and Yao, G. H. (2004). Experimental behaviour of thin-walled hollow structural steel (HSS) columns filled with self-consolidating concrete (SCC). *Thin-Walled Structures*, **42**, 1357 -1377.

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

Hassanein, M. F. and Kharoob, O. F. (2014). Compressive strength of circular concrete-filled double skin tubular short columns. Thin Walled Structure, **77**, 165-173.

Hatzigeorgiou, G.D. (2008). Numerical model for the behavior and capacity of circular CFT columns, Part I: Theory. *Engineering Structures*, **30**, 1573-1578.

Hu, H. T., Huang, C.S. and Chen, Z.L. (2005). Finite element analysis of CFT columns subjected to an axial compressive force and bending moment in combination. *Journal of Constructional Steel Research*, **61**, 1692–1712.

Hu, H.T., Huang, C.S., Wu, M.H. and Wu, Y.M. (2003). Nonlinear analysis of axially loaded concrete-filled tube columns with confinement effect. *Journal* of *Structural Engineering*, **129**, 1322-1329.

Hu, M. Y. (2011). Behaviour and modeling of FRP confined hallow and concrete filled steel tubular columns. PHD thesis, Hong-Kong Polytechnic university, China.

Huang, C. S., Yeh, Y. K., Liu, G. Y., Hu, H. T., Tsai, K. C., Weng, Y. T., Wang, S. H. and Wu, M. H. (2002). Axial load behaviour of stiffened concrete-filled steel columns. *Journal* of *Structural Engineering*, **128**, 1222-1230.

Hull K. Bradford. (1998). Experimental behavior of high strength concrete filled tube Columns. Master's Thesis, Lehigh University, USA.

Johansson, M. and Akesson, M. (2002). Finite element study on concrete-filled steel tubes using a new confinement sensitive concrete compression model. *Nordic Construction Research*, **27**, 43-62.

Johansson, M. and Gylltoft, K. (2002). Mechanical behaviour of circular steel concrete composite stub columns. *Journal of Structural Engineering*, **128**, 1073-1081.

Ketema E. (2005). Design aid for composite columns. Master's Thesis, Addis Ababa University, Ethiopia.

Kuranovas, A., Goode, D., Kvedaras, A. K. and Zhong, Sh. (2009). Load-bearing capacity of concrete-filled steel columns, *Journal of Civil Engineering and Management*, **15**, 21-33.

Le Hoang, A. and Fehling, E. (2017). Numerical study of circular steel tube confined concrete (STCC) stub columns. *Journal of Constructional Steel Research*, **136**, 238-255.

Li, N., Lu, Y. Y., Li, S. and Liang, H.-J. (2015). Statistical-based evaluation of design codes for circular concrete filled steel tube columns. *Steel and Composite Structures*, **18**, 519-546.

LRFD, A. (1994). Manual of steel construction, load and resistance factor design. *Chicago: American Institute of Steel Construction*.

Lue, D. M., Liu J.L. and Yen, T. (2007). Experimental study on rectangular CFT columns with high-strength concrete. *Journal of Constructional Steel Research*, **63**, 37-44.

Mander, J. B., Priestley M. J. N., Park, R. (1988). Theoretical stress–strain model for confined concrete. *Journal of Structural Engineering*, **114**, 1804–1826.

Morino, S. And Tsuda, K. (2003). Design and Construction of Concrete-filled Steel Tube Column System in Japan. *Earthquake Engineering and Engineering Seismology*, **4**, 51-73.

Nardin, S. D. and EL Debs, A. L. H. C. (2007). Axial load behaviour of concrete-filled steel tubular columns. *Proceeding of the Institution of Civil Engineers, Structures and Building*, **160,** 13-22.

Neogi, P. K., Sen, H.K, and Chapman, J.C (1969). Concrete-filled tubular steel columns under eccentric loading. *The Structural Engineer*, **47**, 187-195.

Nethercot, DA. (2004). Composite Construction: 1st Edition. London EC4P 4EE: Taylor & Francis.

Oehlers, D. J. and Bradford, M. A. (1995). Composite steel and concrete structural members: *Fundamental behaviour*. Pergamon.

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

Portolés, J. M., Romero, M. L., Bonet, J. L. and Filippou, F. C. (2011). Experimental study of high strength concrete-filled circular tubular columns under eccentric loading. *Journal of Constructional Steel Research*, **67**, 623-633.

Romero, M. L., Bonet, J. L. and Ivorra, S. (2005). A Review of Nonlinear Analysis Models for Concrete Filled Tubular Columns. In: B.HV. Topping, eds. 2005. *Innovation in Civil and Structural Engineering Computing*. Stirling, UK: Saxe-Coburg Publications, 119-142.

Samhâl E. (2005). Composite Construction. European Steel Computer Aided Learning (Lecture 1.1).

Sakino, K. (2006) Confined concrete in concrete-filled steel tubular columns. American Concrete Institute, ACI, 238 (Special Publication), 267-287.

Sakino, K., Nakahara, H., Morino, S. and Nishiyama, I. (2004). Behavior of Centrally Loaded Concrete-Filled Steel-Tube Short Columns. *Journal of Structural Engineering*, **130**, 180-188.

SANS. South Africa National Standards. (2005). 10162-1. The structural use of steel, limit-state design of hot- rolled steelwork. Pretoria, South Africa.

Schneider, S. (1998). Axially loaded concrete-filled steel tubes. *Journal* of *Structural Engineering*, **124**, 1125-1138.

Seward, D. (1998). Understanding structures: Analysis, materials, design, Basingstoke: Macmillan.

Shakir-Khalil, H. and Zeghiche, J. (1989). Experimental behaviour of concrete-filled rolled rectangular hollow-section columns. *The Structural Engineer*, **67**, 346-353.

Shams, M. and Saadeghvaziri, M. A. (1997). State of the art of concrete-filled steel tubular columns. *ACI Structural Journal*, **94**, 558-571.

Shanmugam, N. E. and Lakshmi, B. (2001). State of the art report on steel concrete composite columns. *Journal of Constructional Steel Research*, **57**, 1041-1080.

Uy, B. (1998). Concrete-filled fabricated steel box columns for multi-storey buildings: behaviour and design. *Progress in Structural Engineering and Materials*, **1**, 150-158.

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

Wong, R. (2003). The construction of super high-rise composite structures in Hong Kong. *2nd International Structural Engineering and Construction Conference at Rome, Italy*. *(Presentation*).

Yang, H., Lam, D. and Gardner, L. (2008). Testing and analysis of concrete-filled elliptical hollow sections. *Engineering Structures*, **30**, 3771-3781.

Yu, Q., Tao, Z. and Wu, Y.-X. (2008). Experimental behaviour of high performance concrete-filled steel tubular columns. *Thin-Walled Structures*, **46**, 362-370.

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

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

Zhang, W. and Shahrooz, B. M. (1999). Comparison between ACI and AISC for concrete-filled Tubular Columns. *Journal* of *Structural Engineering*, **125**, 1213-1223.

APPENDIX A

Results of Axial Load Capacity of CFST Design Codes

Column	*fy* (Mpa)	f_c' (Mpa)	*L/D*	*D/t*	*Section*	N_{AISC} (kn)	N_{EC4} (kn)
G 01							
1	235	30	3	20.00	Compact	2118	2807
2	315	30	3	20.00	Compact	2590	3446
3	395	30	3	20.00	Compact	3061	4065
4	475	30	3	20.00	Compact	3531	4665
5	235	50	3	20.00	Compact	2596	3276
6	315	50	3	20.00	Compact	3068	3908
7	395	50	3	20.00	Compact	3539	4522
8	475	50	3	20.00	Compact	4008	5119
9	235	70	3	20.00	Compact	3075	3749
10	315	70	3	20.00	Compact	3546	4375
11	395	70	3	20.00	Compact	4015	4984
12	475	70	3	20.00	Compact	4484	5577
13	235	90	3	20.00	Compact	3552	4227
14	315	90	3	20.00	Compact	4022	4846
15	395	90	3	20.00	Compact	4492	5449
16	475	90	3	20.00	Compact	4960	6038

APPENDIX A (continued)

Column	*fy* (Mpa)	f_c' (Mpa)	*L/D*	*D/t*	*Section*	N_{AISC} (kn)	N_{EC4} (kn)
G 02							
17	235	30	6	20.00	Compact	2086	2447
18	315	30	6	20.00	Compact	2543	2930
19	395	30	6	20.00	Compact	2996	3387
20	475	30	6	20.00	Compact	3445	3824
21	235	50	6	20.00	Compact	2552	2885
22	315	50	6	20.00	Compact	3006	3360
23	395	50	6	20.00	Compact	3456	3813
24	475	50	6	20.00	Compact	3903	4247
25	235	70	6	20.00	Compact	3015	3329
26	315	70	6	20.00	Compact	3466	3796
27	395	70	6	20.00	Compact	3914	4243
28	475	70	6	20.00	Compact	4358	4675
29	235	90	6	20.00	Compact	3475	3777
30	315	90	6	20.00	Compact	3924	4237
31	395	90	6	20.00	Compact	4369	4679
32	475	90	6	20.00	Compact	4810	5107

APPENDIX A (continued)

Column	*fy* (Mpa)	f_c' (Mpa)	*L/D*	*D/t*	*Section*	N_{AISC} (kn)	N_{EC4} (kn)
G 03							
33	235	30	9	20.00	Compact	2036	2168
34	315	30	9	20.00	Compact	2468	2574
35	395	30	9	20.00	Compact	2891	2968
36	475	30	9	20.00	Compact	3306	3358
37	235	50	9	20.00	Compact	2480	2599
38	315	50	9	20.00	Compact	2906	3003
39	395	50	9	20.00	Compact	3323	3400
40	475	50	9	20.00	Compact	3733	3795
41	235	70	9	20.00	Compact	2918	3036
42	315	70	9	20.00	Compact	3338	3439
43	395	70	9	20.00	Compact	3750	3837
44	475	70	9	20.00	Compact	4154	4269
45	235	90	9	20.00	Compact	3350	3476
46	315	90	9	20.00	Compact	3764	3879
47	395	90	9	20.00	Compact	4171	4304
48	475	90	9	20.00	Compact	4570	4738

APPENDIX A (continued)

Column	*fy* (Mpa)	*f* f_c' (Mpa)	*L/D*	*D/t*	*Section*	N_{AISC} (kn)	N_{EC4} (kn)
G 04							
49	235	30	12	20.00	Compact	1967	2012
50	315	30	12	20.00	Compact	2366	2443
51	395	30	12	20.00	Compact	2750	2852
52	475	30	12	20.00	Compact	3121	3240
53	235	50	12	20.00	Compact	2382	2475
54	315	50	12	20.00	Compact	2771	2884
55	395	50	12	20.00	Compact	3146	3273
56	475	50	12	20.00	Compact	3508	3648
57	235	70	12	20.00	Compact	2787	2914
58	315	70	12	20.00	Compact	3166	3304
59	395	70	12	20.00	Compact	3532	3680
60	475	70	12	20.00	Compact	3885	4043
61	235	90	12	20.00	Compact	3183	3334
62	315	90	12	20.00	Compact	3552	3711
63	395	90	12	20.00	Compact	3909	4075
64	475	90	12	20.00	Compact	4254	4425

APPENDIX A (continued)

Column	*fy* (Mpa)	f_c' (Mpa)	*L/D*	*D/t*	*Section*	N_{AISC} (kn)	N_{EC4} (kn)
G 01-1							
65	235	30	3	40.00	Compact	1519	1927
66	315	30	3	40.00	Compact	1761	2269
67	395	30	3	40.00	Compact	2003	2602
68	475	30	3	40.00	Compact	2244	2926
69	235	50	3	40.00	Compact	2052	2460
70	315	50	3	40.00	Compact	2294	2795
71	395	50	3	40.00	Compact	2535	3123
72	475	50	3	40.00	Compact	2775	3442
73	235	70	3	40.00	Compact	2584	2999
74	315	70	3	40.00	Compact	2825	3328
75	395	70	3	40.00	Compact	3065	3650
76	475	70	3	40.00	Compact	3305	3965
77	235	90	3	40.00	Compact	3114	3543
78	315	90	3	40.00	Compact	3354	3866
79	395	90	3	40.00	Compact	3594	4182
80	475	90	3	40.00	Compact	3834	4493

APPENDIX A (continued)

Column	f_y (Mpa)	f_c' (Mpa)	*L/D*	*D/t*	*Section*	N_{AISC} (kn)	N_{EC4} (kn)
G 02-1							
81	235	30	6	40.00	Compact	1494	1711
82	315	30	6	40.00	Compact	1727	1967
83	395	30	6	40.00	Compact	1959	2211
84	475	30	6	40.00	Compact	2189	2444
85	235	50	6	40.00	Compact	2009	2214
86	315	50	6	40.00	Compact	2240	2461
87	395	50	6	40.00	Compact	2469	2698
88	475	50	6	40.00	Compact	2697	2927
89	235	70	6	40.00	Compact	2520	2722
90	315	70	6	40.00	Compact	2748	2961
91	395	70	6	40.00	Compact	2975	3192
92	475	70	6	40.00	Compact	3201	3417
93	235	90	6	40.00	Compact	3026	3232
94	315	90	6	40.00	Compact	3252	3464
95	395	90	6	40.00	Compact	3477	3690
96	475	90	6	40.00	Compact	3701	3911

APPENDIX A (continued)

Column	*fy* (Mpa)	*f_c'* (Mpa)	*L/D*	*D/t*	*Section*	N_{AISC} (kn)	N_{EC4} (kn)
G 03-1							
97	235	30	9	40.00	Compact	1452	1544
98	315	30	9	40.00	Compact	1672	1756
99	395	30	9	40.00	Compact	1887	1961
100	475	30	9	40.00	Compact	2099	2163
101	235	50	9	40.00	Compact	1940	2035
102	315	50	9	40.00	Compact	2154	2243
103	395	50	9	40.00	Compact	2364	2448
104	475	50	9	40.00	Compact	2572	2661
105	235	70	9	40.00	Compact	2417	2530
106	315	70	9	40.00	Compact	2626	2741
107	395	70	9	40.00	Compact	2832	2959
108	475	70	9	40.00	Compact	3035	3177
109	235	90	9	40.00	Compact	2885	3035
110	315	90	9	40.00	Compact	3089	3249
111	395	90	9	40.00	Compact	3291	3454
112	475	90	9	40.00	Compact	3489	3658

APPENDIX A (continued)

Column	*fy* (Mpa)	*f_c'* (Mpa)	*L/D*	*D/t*	*Section*	N_{AISC} (kn)	N_{EC4} (kn)
G 04-1							
113	235	30	12	40.00	Compact	1396	1456
114	315	30	12	40.00	Compact	1597	1676
115	395	30	12	40.00	Compact	1791	1880
116	475	30	12	40.00	Compact	1980	2079
117	235	50	12	40.00	Compact	1847	1951
118	315	50	12	40.00	Compact	2039	2150
119	395	50	12	40.00	Compact	2225	2344
120	475	50	12	40.00	Compact	2406	2533
121	235	70	12	40.00	Compact	2281	2413
122	315	70	12	40.00	Compact	2464	2602
123	395	70	12	40.00	Compact	2643	2786
124	475	70	12	40.00	Compact	2816	2963
125	235	90	12	40.00	Compact	2699	2853
126	315	90	12	40.00	Compact	2875	3031
127	395	90	12	40.00	Compact	3046	3203
128	475	90	12	40.00	Compact	3213	3370

APPENDIX A (continued)

Column	*fy* (Mpa)	*f_c'* (Mpa)	*L/D*	*D/t*	*Section*	N_{AISC} (kn)	N_{EC4} (kn)
G 01-2							
129	235	30	3	60.00	Compact	1312	1606
130	315	30	3	60.00	Compact	1475	1840
131	395	30	3	60.00	Compact	1637	2068
132	475	30	3	60.00	Compact	1799	2291
133	235	50	3	60.00	Compact	1863	2166
134	315	50	3	60.00	Compact	2025	2393
135	395	50	3	60.00	Compact	2187	2616
136	475	50	3	60.00	Compact	2348	2834
137	235	70	3	60.00	Compact	2412	2731
138	315	70	3	60.00	Compact	2573	2953
139	395	70	3	60.00	Compact	2735	3171
140	475	70	3	60.00	Compact	2896	3385
141	235	90	3	60.00	Compact	2960	3300
142	315	90	3	60.00	Compact	3121	3517
143	395	90	3	60.00	Compact	3281	3730
144	475	90	3	60.00	Compact	3442	3941

APPENDIX A (continued)

Column	*fy* (Mpa)	f_c' (Mpa)	*L/D*	*D/t*	*Section*	N_{AISC} (kn)	N_{EC4} (kn)
G 02-2							
145	235	30	6	60.00	Compact	1287	1449
146	315	30	6	60.00	Compact	1443	1623
147	395	30	6	60.00	Compact	1598	1790
148	475	30	6	60.00	Compact	1752	1951
149	235	50	6	60.00	Compact	1817	1978
150	315	50	6	60.00	Compact	1971	2144
151	395	50	6	60.00	Compact	2124	2304
152	475	50	6	60.00	Compact	2276	2460
153	235	70	6	60.00	Compact	2341	2511
154	315	70	6	60.00	Compact	2493	2670
155	395	70	6	60.00	Compact	2643	2825
156	475	70	6	60.00	Compact	2793	2976
157	235	90	6	60.00	Compact	2859	3043
158	315	90	6	60.00	Compact	3009	3197
159	395	90	6	60.00	Compact	3158	3348
160	475	90	6	60.00	Compact	3306	3496

APPENDIX A (continued)

Column	*fy* (Mpa)	f_c' (Mpa)	*L/D*	*D/t*	*Section*	N_{AISC} (kn)	N_{EC4} (kn)
G 03-2							
161	235	30	9	60.00	Compact	1246	1326
162	315	30	9	60.00	Compact	1392	1469
163	395	30	9	60.00	Compact	1535	1609
164	475	30	9	60.00	Compact	1676	1746
165	235	50	9	60.00	Compact	1744	1840
166	315	50	9	60.00	Compact	1884	1980
167	395	50	9	60.00	Compact	2023	2120
168	475	50	9	60.00	Compact	2160	2267
169	235	70	9	60.00	Compact	2227	2359
170	315	70	9	60.00	Compact	2364	2503
171	395	70	9	60.00	Compact	2498	2645
172	475	70	9	60.00	Compact	2631	2783
173	235	90	9	60.00	Compact	2699	2869
174	315	90	9	60.00	Compact	2832	3006
175	395	90	9	60.00	Compact	2962	3141
176	475	90	9	60.00	Compact	3091	3274

APPENDIX A (continued)

Column	*fy* (Mpa)	f_c' (Mpa)	*L/D*	*D/t*	*Section*	N_{AISC} (kn)	N_{EC4} (kn)
G 04-2							
177	235	30	12	60.00	Compact	1191	1264
178	315	30	12	60.00	Compact	1323	1406
179	395	30	12	60.00	Compact	1451	1542
180	475	30	12	60.00	Compact	1575	1675
181	235	50	12	60.00	Compact	1645	1760
182	315	50	12	60.00	Compact	1769	1890
183	395	50	12	60.00	Compact	1890	2018
184	475	50	12	60.00	Compact	2007	2142
185	235	70	12	60.00	Compact	2077	2224
186	315	70	12	60.00	Compact	2194	2346
187	395	70	12	60.00	Compact	2308	2464
188	475	70	12	60.00	Compact	2419	2579
189	235	90	12	60.00	Compact	2490	2659
190	315	90	12	60.00	Compact	2601	2771
191	395	90	12	60.00	Compact	2709	2880
192	475	90	12	60.00	Compact	2814	2985

APPENDIX A (continued)

Column	*fy* (Mpa)	*f_c'* (Mpa)	*L/D*	*D/t*	*Section*	N_{AISC} (kn)	N_{EC4} (kn)
G 01-3							
193	235	30	3	80.00	Compact	1207	1442
194	315	30	3	80.00	Compact	1329	1619
195	395	30	3	80.00	Non-compact	1443	1792
196	475	30	3	80.00	Slender	1356	1963
197	235	50	3	80.00	Compact	1767	2016
198	315	50	3	80.00	Compact	1888	2188
199	395	50	3	80.00	Non-compact	1995	2356
200	475	50	3	80.00	Slender	1768	2522
201	235	70	3	80.00	Compact	2324	2596
202	315	70	3	80.00	Compact	2445	2762
203	395	70	3	80.00	Non-compact	2546	2927
204	475	70	3	80.00	Slender	2180	3089
205	235	90	3	80.00	Compact	2880	3179
206	315	90	3	80.00	Compact	3001	3341
207	395	90	3	80.00	Non-compact	3095	3502
208	475	90	3	80.00	Slender	2590	3660

APPENDIX A (continued)

Column	*fy* (Mpa)	f_c' (Mpa)	*L/D*	*D/t*	*Section*	N_{AISC} (kn)	N_{EC4} (kn)
G 02-3							
209	235	30	6	80.00	Compact	1182	1316
210	315	30	6	80.00	Compact	1298	1448
211	395	30	6	80.00	Non-compact	1406	1575
212	475	30	6	80.00	Slender	1324	1698
213	235	50	6	80.00	Compact	1718	1860
214	315	50	6	80.00	Compact	1833	1984
215	395	50	6	80.00	Non-compact	1933	2105
216	475	50	6	80.00	Slender	1719	2223
217	235	70	6	80.00	Compact	2247	2405
218	315	70	6	80.00	Compact	2360	2524
219	395	70	6	80.00	Non-compact	2453	2640
220	475	70	6	80.00	Slender	2112	2754
221	235	90	6	80.00	Compact	2769	2948
222	315	90	6	80.00	Compact	2881	3063
223	395	90	6	80.00	Non-compact	2967	3175
224	475	90	6	80.00	Slender	2501	3286

APPENDIX A (continued)

Column	*fy* (Mpa)	f_c' (Mpa)	*L/D*	*D/t*	*Section*	N_{AISC} (kn)	N_{EC4} (kn)
G 03-3							
225	235	30	9	80.00	Compact	1140	1216
226	315	30	9	80.00	Compact	1248	1324
227	395	30	9	80.00	Non-compact	1347	1430
228	475	30	9	80.00	Slender	1272	1534
229	235	50	9	80.00	Compact	1640	1742
230	315	50	9	80.00	Compact	1744	1847
231	395	50	9	80.00	Non-compact	1834	1956
232	475	50	9	80.00	Slender	1641	2066
233	235	70	9	80.00	Compact	2124	2270
234	315	70	9	80.00	Compact	2224	2375
235	395	70	9	80.00	Non-compact	2306	2479
236	475	70	9	80.00	Slender	2003	2582
237	235	90	9	80.00	Compact	2594	2776
238	315	90	9	80.00	Compact	2691	2877
239	395	90	9	80.00	Non-compact	2766	2977
240	475	90	9	80.00	Slender	2358	3076

APPENDIX A (continued)

Column	*fy* (Mpa)	f_c' (Mpa)	*L/D*	*D/t*	*Section*	N_{AISC} (kn)	N_{EC4} (kn)
G 04-3							
241	235	30	12	80.00	Compact	1085	1164
242	315	30	12	80.00	Compact	1182	1268
243	395	30	12	80.00	Non-compact	1269	1370
244	475	30	12	80.00	Slender	1202	1470
245	235	50	12	80.00	Compact	1536	1659
246	315	50	12	80.00	Compact	1626	1756
247	395	50	12	80.00	Non-compact	1704	1850
248	475	50	12	80.00	Slender	1538	1942
249	235	70	12	80.00	Compact	1962	2121
250	315	70	12	80.00	Compact	2047	2209
251	395	70	12	80.00	Non-compact	2115	2296
252	475	70	12	80.00	Slender	1860	2380
253	235	90	12	80.00	Compact	2367	2548
254	315	90	12	80.00	Compact	2446	2628
255	395	90	12	80.00	Non-compact	2506	2706
256	475	90	12	80.00	Slender	2172	2782