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EKTIFAA SALIH KHUDHUR

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Investigation of Design Specification Calculations for Concrete Filled Steel Tube Columns

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Ektifaa Salih KHUDHUR

ABSTRACT

INVESTIGATION OF DESIGN SPECIFICATION CALCULATIONS FOR CONCRETE FILLED STEEL TUBE COLUMNS

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M.Sc. in Civil Engineering

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Steel and concrete are frequently used materials in civil engineering structures. While the concrete members benefit from high compressive strength and stiffness, the steel members possess benefits of high tensile strength and ductility. For this reason, composite member combines both steel and concrete to produce a member with beneficial properties for both materials. The (CFST) column is the most common kind of composite column. Using of CFST columns is becoming an important solution in construction due to their advantage especially under the loads of seismic. The concrete is confined by the steel, and this will offer under compression a highly ductile reaction. The presence of infill concrete improves the local buckling of steel tube. Circular CFST column have advantages in comparing with other sections, where provides much more confinement to the concrete because of the improvement in the composite actions between concrete and steel. Depending on the material and geometrical properties, capacity calculations of the design specification lead to discrepancies. To project the differences between the most popular design specifications a computational study is done by employing AISC 360 – 10, the Eurocode 4, AS 5100.6, and AIJ 2001. This study goals to investigate the differences between these design codes for circular CFST columns under axial load and to evaluate how they model the real column behavior through a series of statistical comparisons. Also, the parameters which are used in design specification calculations steps are assessed.

Keywords: CFST column, compression capacity, design specifications, experiments.

ÖZET

İÇİ BETON DOLDURULMUŞ ÇELİK TÜP KOLONLAR İÇİN TASARIM ŞARTNAMELERİNİN İNCELENMESİ

KHUDHUR, Ektifaa Salih

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Çelik ve beton, inşaat mühendisliği yapılarında sıkça kullanılan malzemelerdir. Beton elemanlar yüksek basınç mukavemeti ve rijitlikten faydalanırken, çelik elemanlar yüksek mukavemeti ve süneklik avantajlarına sahiptir. Bu nedenle, kompozit elemanlar, her iki malzemenin de yararlı özelliklere sahip bir eleman üretmek için hem çelik hem de betonu birleştirir. İçi Beton Doldurulmuş Çelik Tüp (CFST) kolonlar, en yaygın kompozit kolon türüdür. Özellikle sismik yükler altında avantajları nedeniyle CFST kolonlarının kullanımı inşaatta önemli bir çözüm haline gelmektedir. Çelik boru beton çekirdeği basınç altında oldukça sünek bir tepki verir. Dolgu betonunun varlığı, çelik tüpün yerel çökmesini iyileştirir. Dairesel kolonda çelik ve beton arasındaki kompozit hareketin iyileşmesi nedeniyle beton daha fazla sargı etkisine maruz kalır ve diğer durumlarla karşılaştırıldığında avantajları vardır. Malzeme ve geometrik özelliklere bağlı olarak, tasarım şartnamesinin kapasite hesaplamalarında tutarsızlıklara neden olmaktadır. En çok kullanilan tasarım şartnameleri arasındaki farklılıkları yansıtmak için, AISC 360 - 10, Eurocode 4, AS 5100.6, ve AIJ 2001 kullanılarak değişik kolon geometrileri ve malzeme özellikleri çalışılmalıdır. Bu çalışma, eksenel yük altında dairesel CFST kolonlar için tasarım şastnamelerı arasındaki farkları araştirip bir dizi istatistiksel karşılaştırma ile gerçek kolon davranışını nasıl modellediğini değerlendirmektedir. Ayrıca, tasarım özellikleri ve hesaplama adımlarında kullanılan parametreler irdelenmektedir.

Anahtar kelimeler: içi beton doldurolmeş çelik tüp kolon, basınç kapasitesi, tasarım şartnameleri, deneyler.

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LIST OF SYMBOLS

SYMBOLS

CFST	Concrete filled steel tube.
AISC	American institute of steel construction.
EC4	The European code for composite structures design.
AS	Australian Standard.
AIJ	Architectural Institute of Japan.
NSC, HSC, UHSC	Normal, high, and ultra-high strength concrete, respectively.
MSS, HSS	Mild and high strength steel, respectively.
t	Steel tube thickness (mm).
l, L	Buckling length, and system length of the composite column, respectively (mm).
D	External diameter of the steel tube (mm).
f_c', f_c, f_{kc}	Concrete compressive strength (MPa).
f_y	Yield strength of steel (MPa).
A_c, A_s	Cross-sectional areas of concrete and steel (mm^2), respectively.
ξ	Confinement index.
SI	Strength index.
P_u	The capacity of a CFST column (N).
P_{uo}	The sectional capacity or squash load (N).
I_c, I_s	Moment of inertia of concrete and steel (mm^4), respectively.
E_c, E_s	Modulus of elasticity of concrete and steel (MPa), respectively.
$\lambda, \lambda_p, \lambda_r$	Slenderness ratios.

P_n	Ultimate load capacity of the column (N).
P_{no}	The nominal axial strength of the cross section (N).
P_e	Euler critical buckling load (N).
P_p	Plastic strength.
P_y	Yield strength.
C_2	0.95 for circular sections.
C_3	Coefficient of effective rigidity of filled composite compression member.
F_{cr}	Critical buckling stress.
EI_{eff}	The effective stiffness of the composite section.
K	Effective length factor.
δ	Steel contribution ratio.
e	The eccentricity of the imposed load.
γ_c, γ_a	Partial safety factors for concrete and steel, respectively.
$N_{Pl,Rd}$	Plastic resistance to compression.
N_{cr}	Elastic critical load.
η_c	The concrete enhancement factor due to confinement effect.
η_a	The steel reduction factor.
χ	Reduction coefficient for buckling.
α	An imperfection parameter.
$\bar{\lambda}$	Relative slenderness ratio.
ϕ	Capacity factor.
N_{us}	Nominal section capacity.
${}_c r_u$	Reduction factor for concrete strength.
N_{cu1},N_{cu2}, N_{cu3}	Ultimate strengths of a CFST column.
CCR	The contribution of concrete.

CHAPTER 1

INTRODUCTION

The most commonly used constructional materials in civil engineering applications are concrete and steel. Composite members consist of a mixture of concrete and steel incorporating the benefits of these two kinds since concrete members possess the property of high compressive strength and stiffness, while steel members possess the property of high tensile strength and ductility. Composite columns are the important part of the composite structures. There are two kinds of composite columns those commonly used in buildings; concrete encased composite column and concrete filled steel tube (CFST) column.

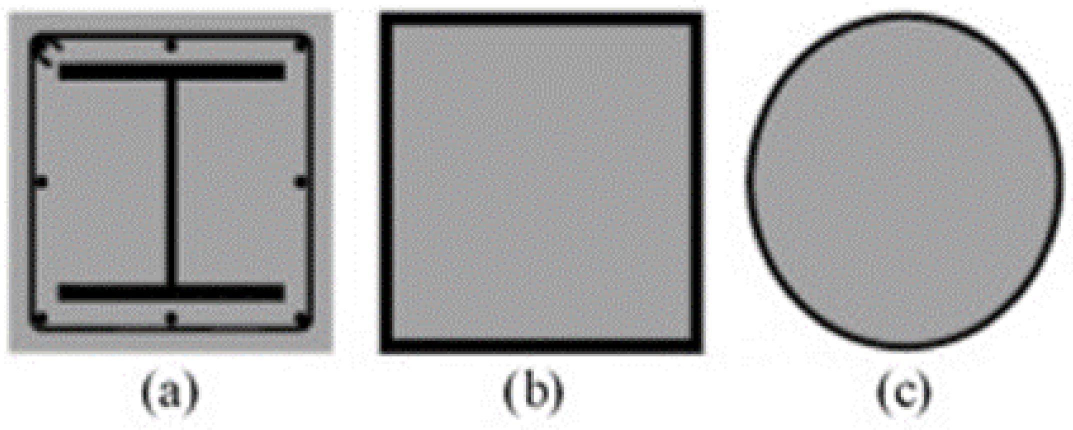


Figure 1.1 Composite Column: (a) concrete encased composite column, (b) rectangular concrete-filled steel tube column, and (c) circular concrete-filled steel tube column [1].

Concrete-filled steel tube (CFST) column has become a vital choice in practical applications and considered a good substitute type for high-performance structural; Consisting of hollow steel tube surrounding concrete core. Combining structural steel forms, pipes or with reinforced or non-reinforced concrete to provide adequate load carrying capacity to sustain either axial compressive loading alone or a combination of axial loads and bending moments. Composite columns subjected mainly to axial compressive forces and end moments. The general term composite column refers to any compression member in which the steel and concrete elements perform together so they share in the strength. The interactive and integral behavior of concrete and the structural steel elements makes the composite column a very cost effective and structural efficient member among the wide range of structural elements in building, bridge constructions, electric transmission line and offshore structures. Two cross-sections of steel ubes are commonly used in the construction of CFST: circular and rectangular.

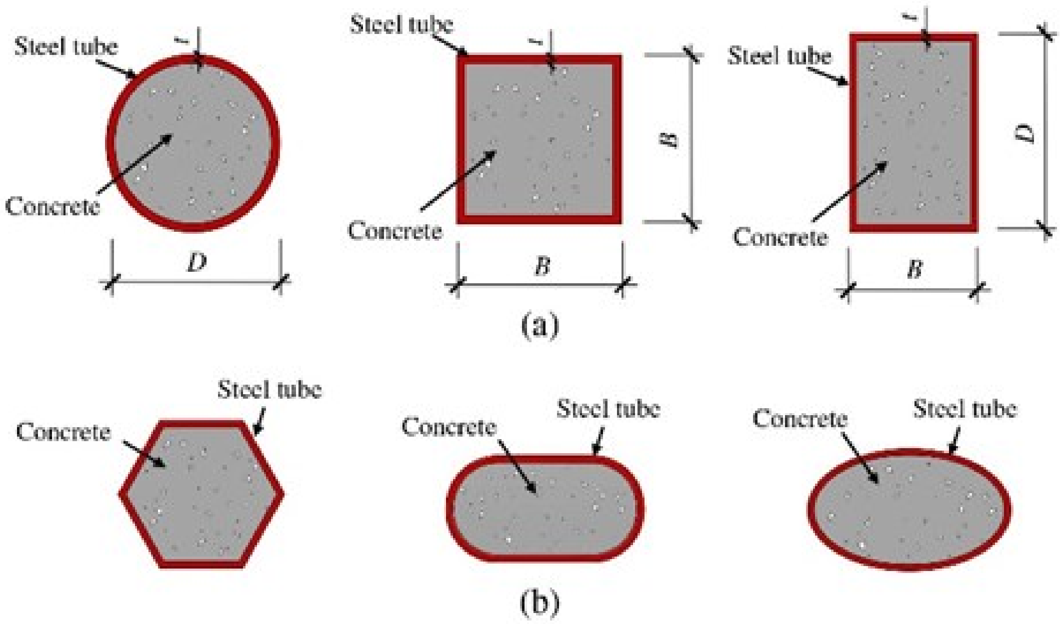


Figure 1.2 Standard cross-section of CFST columns [2]

While the steel lies at the outer circumference and extends to the bend resistance, which is resistant to axial tensile and compression, the concrete also forms a perfect core to help with the compression load. The composite member itself cost less than steel and approximately equals to traditional reinforced concrete on a strength per dollar for low up to medium strength concrete [3].

Nowadays, concrete filled steel tube is gaining more acceptance in construction area. It is component with good performance resulting from the confinement effect of steel with concrete and design flexibility. CFSTs are becoming increasingly important in various engineering constructions with basic cross-sectional shapes, and therefore the study on the behavior and characteristics of CFST columns is prerequisite [4].

The first research on CFST members began at the beginning of the 20th century, focusing mainly on strength capacity. In addition, recent researches have emphasized on the use of HSC to increase strength of the member. The ultimate capacity of the CFST columns relies on the compressive strength of the concrete and on the tensile strength of steel tube. Many factors affect Strength and behavior of materials such as the band between concrete and steel, strength of material, the confinement of concrete, and fire resistance. The technique of CFST columns adds structural features to the composite action when the steel tube is strengthened by concrete core, preventing the tube from buckling into the interior [5].

Increasing in the use of CFST columns is largely due to the mechanical and economic benefits offered by concrete filled tubes over hollow tubes, in addition to their artistic demands. The reduction in column size is mainly useful when the floor space is very required, particularly in parking lots and office blocks [6].

1.1 The Advantages of CFST Columns

CFST columns have abundant structural benefits compared with the structural steel and traditional reinforced concrete columns.

- The composite columns possess numerous advantages, like rising capacity, inherent ductility and toughness, capacity of energy absorption and high fire resistance besides fast construction without increasing the section size. Additionally, CFST columns with high strength involve a reduced section in order to carry the load and that is valued by building engineers and architects [5].
- The combined capacity of the steel and concrete significantly increases the stiffness and ultimate strength of CFST columns, making them particularly suitable for columns and other compression members.

- For a specified load, higher strength-to-weight ratio and smaller cross-section provided by the CFST column than a traditional reinforced concrete column [3].
- The local and global buckling of steel tube is restrained by the concrete core which significantly rising resistance and inelastic deformation capacity. Recent analytical and experimental studies have shown that for the same weight and diameter; CFST elements can develop more lateral resistance and greater inelastic deformation capacity while less fall of resistance comparing with reinforced concrete elements [2].Also, bending and inward local buckling of the steel tube is prevented by the concrete core effectively, so that the steel tube walls could just buckle locally outward.
- The steel tube works as lateral and longitudinal reinforcement for the concrete core and acts as lasting and integral formwork and can give a considerable amount of construction and permanent loads before pushing wet concrete, and that lead to construction rapidly[5] and noteworthy additional saving in construction cost. Besides, steel tube prevents the concrete from spalling [7], and it offers the concrete core by confining pressure, which sets the concrete under a stress triaxial state.
- The 60% saving of steel totally by application of CFST concept in comparison with the use of structural steel system and this concept took a wide range of admissions in the civil engineering fields. Due to an efficient use of construction materials, these members are ideal for all aspects of application.
- The potential economic advantages of CFST columns because of the use of these members, which decreases the required cross-section size leading to large cost saving through increasing area of tenancyable floors and reducing of steel usage in the compare with pure steel columns. In capitals wherever the rate of provide spaces are extraordinary, the design of tall building requires reducing the size of steel cross-section and this is very essential. These are chiefly important for the tall buildings in the lower storey where stub by columns usually be present [8].
- For a given cross-sectional area, circular tubular columns over all other section; have a benefit when used in constructions, because of huge flexural stiffness they have from all directions.

- (CFST) concrete filled steel tubes are being used usually in real structural projects for their exceptional static and dynamic (earthquake-resistant) characteristics [9], and their behavior in moment bending, shear, compression and fatigue resistance bellow cyclic capacity of seismic are also greater than reinforced elements, therefore these structures are used frequently in earthquake regions.
- For CFST columns, the filled concrete can significantly increase the fire resistance. Because the heat is absorbed by the concrete core, the temperature in the steel tube increases much slower than the bare hollow steel tubes. Moreover, the external tube offers a protection by confining the core concrete during fire exposure, the small fragment of the core concrete thus can be prevented, while the fire resistance of unprotected hollow steel tubular columns in high buildings is normally found to be in very short time [2].

These advantages directed CFST compression members to find use in a wide range of engineering constructions. High buildings, bridges, groundwork piles, towers, and tunnels are some examples in which CFST members have the possibility to work under high compression states [10].

The main disadvantages of the CFST columns are their low fire and corrosion resistance caused by using an uncovered steel tube compared with conventional reinforced concrete [11].To overcome corrosion associated matters, some researches have proposed replacing an aluminum tube or a cold-formed stainless steel with the carbon steel tube for higher corrosion resistance and appealing exterior [12]. Subsequently the effect of confinement in circular tubes is superior, in general they are chosen in these structures which involve high act. Tubes with square and rectangular sections offer small confinement in these tubes because the walls must resist the concrete pushing by plate bending, while in circular sections there is generated hoop stress [13].

Among the uses of concrete-filled steel tubular CFST columns, the construction of bridge piers and high buildings are two of the most usual applications and, in these cases, these members are subjected to significantly high axial loads. The performance of CFST columns could be further enhanced if materials with high-strength are used. High-strength steel and concrete are found to be valuable alternatives to the normal-strength steel and concrete for multi-storey and high construction. The employ of high

strength concrete HSC and even ultra-high strength concrete UHSC in this type of composite columns is becoming familiar given their ability of enhancement the load-bearing capacity of the member with a reduced economic cost. However, because of the reduction of the needed clear cross-sectional area of the composite column, so the use of high strength materials will cause an increment in the slenderness of the column [14, 15].
High-strength CFST columns are becoming achievable technically and economically with the using of high-strength steel and the production of high-strength concrete via traditional materials with specific quality control. Still, due to the lack of understanding of their structural performance and the required design recommendations; they are barely adopted in the construction fields [14].

1.2 Design Specifications Employed in the Thesis

Many design specifications had been recommended to evaluate the axial capacity of CFST columns, and these design codes which used in this study:-

- AISC 360-10 Code [16] (The specification for steel structures in the United States).
- Eurocode-4(EC4) Code [17] (The European code for composite structure design).
- AS5100.6-2004 Code [18] (Australian Standard for steel and composite constructions).
- AIJ 2001[19] (The Japanese guide for steel reinforced concrete structures).

1.3 Purpose of the Thesis

This study aimed to confirm the applicability and prediction of each code for circular CFST columns under axial loading and compare the results to evaluate how they model the real column behavior through a series of statistical comparisons. Also, the parameters which are used in specification calculations steps are assessed.
The important parameters in calculations would also be specified to underline the best method in the design field. Since many of experimental and analytical studies conducted on CFST columns with various cross-sections to understand their performance under axial loading, showed that the behavior and axial load capacity of these columns are influenced by the geometrical and physical parameters.

Therefore, this study investigated the behavior of such composite columns with different materials and geometric properties within and beyond the design codes limitations to fill this knowledge gap and tried to fix the lack of information and experience that Engineers have with CFST structural systems.

CHAPTER 2

LITERATURE REVIEW

2.1 General

CFST members consist of rectangular or circular steel tubes filled with concrete. The presence of the concrete infill improves the strength, stiffness, and fire resistance of these composite members as compared to hollow steel tube members. As an advanced and effective structural component, CFST members have been used around the world commonly in various types of structures including composite bridges, composite moment or braced frames, piles, and transmission towers, etc. The experimental studies indicate that the chief parameters affecting the behavior and capacity of CFST columns are the geometric parameters; D/t ratio, L/D ratio, and materials parameters; concrete compressive strength (f_c), and steel yield strength (f_y) .

2.2 Behavior of Circular CFST Columns

The act of CFST column is studied by many researches which have revealed that the performance of a column with rectangular section is not as respectable compare to the column with circular section. The reason for this is the fact that steel tubes with square and rectangular section may offer less restrictive pressure to concrete core; since the walls in the rectangular tubes have to resist the pushing of concrete by bending the plate instead of producing circumferential stress like in circular sections [13] and noted that local buckling is more likely happened. Permissible width-to-thickness ratio for the steel tube with rectangle cross-section is more restricted than that for circular one.

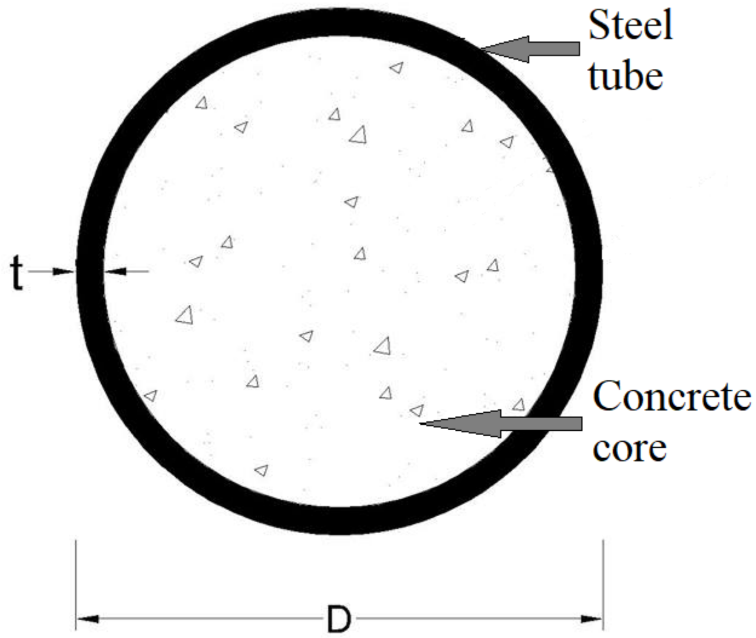


Figure 2.1 Cross section of circular CFST column.

Accordingly, an adequate stiffening measure for rectangle CFST is highly necessary. Such stiffening measure will make rectangle or square CFST an economical construction material, too, and consequently the barrier to promote construction using rectangle CFST could be overcome.

The variance between Poisson's ratios for concrete and steel has been affected the behavior of CFST column considerably. In the initial stage of loading, the poison's ratio for the concrete is lower than that of steel, therefore a separation between steel tube wall and concrete core occurs. Thus, there is no confining influence from steel towards the concrete. As the load increases, the longitudinal strain reaches a certain critical strain, and the lateral expansion or deformation of the concrete gradually becomes greater than expansion of steel tube.

When the load increases further, at this stage, a tensile hoop stress is developed in the steel tube becoming biaxially stressed, and the concrete core is subjected to triaxial compression. This phenomenon results in an increase of load capacity. The steel tube in a biaxial state of stress cannot bear the yield stress which causes a load to be transferred from the steel to the concrete. At the first loading phase, the steel tube keeps most of the burden till the yielding shown in the figure 2.2. At the point (a) there is a load transferring from steel to the concrete. The steel tube reveals steady decreases in load sharing till the concrete reaches its maximum compression stress (a to b). After this loading stage (point b), there is a load redistribution from concrete to the steel. At this point (b) the steel shows a behavior of hardening with almost the same slope as in uniaxial relationship of stress-strain hardening.

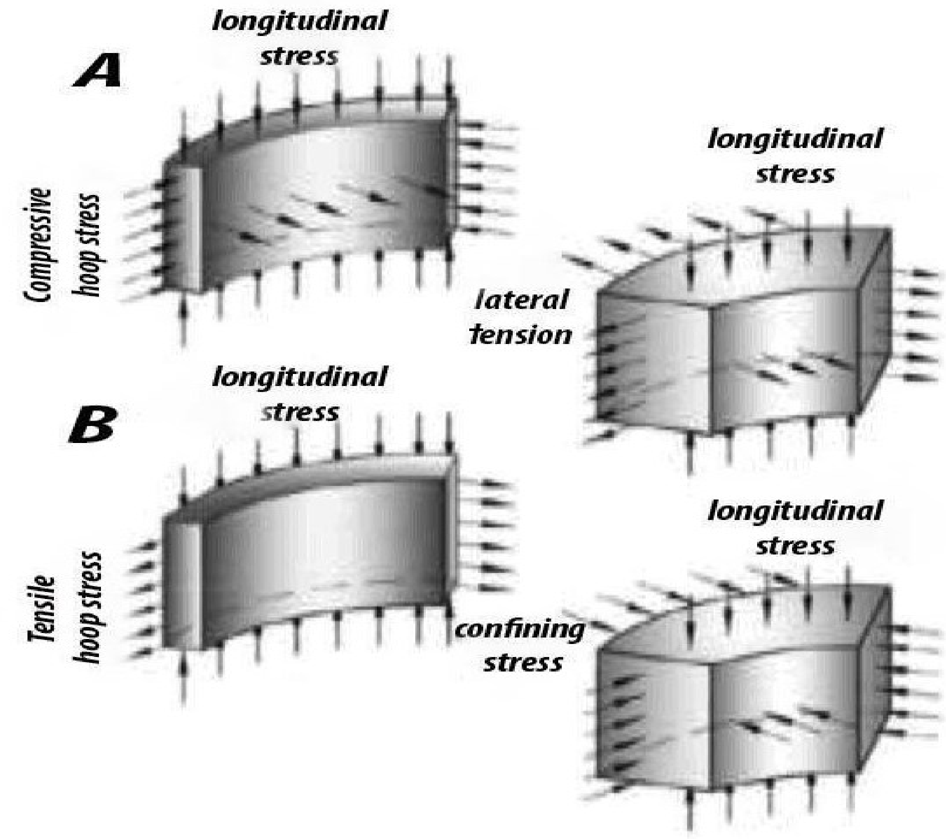


Figure 2.2 Stress condition in steel and concrete during different loading stages [20].

2.3 Effect of the Composite Action

In order to comprehend the "composite action" state between concrete and steel, for these composite sections under axial compression, much research works have been completed. Since the increase in strength, stiffness, and ductility of CFST columns are providing by the confining effect of steel tube into concrete core, it is necessary to pay attention to the effect of this mechanism, therefore, most codes and specifications acknowledge the effect of the "composite action," for the design of member under axial compression, especially for circular cross section members [2]. In concrete-filled rectangular hollow sections the confinement effect does not exist, excluding the places of corners, when the arising peripheral stresses along the side walls is not fixed .It can be established that the circular steel tubes can offer much more effective confinement to their core concrete than other types of tubular sections. When the compressive strength in the concrete core increased an additional strength occurs as a result and it would be bounded laterally by the surrounding steel member.

The concrete strength increment prevails over the steel yield strength reduction in compression due to bound concrete core by the confinement tension. In the CFST sections, as bending moments are applied; the influence of containment is reduced. The reason for this is the mean of concrete compressive strain (and the accompanying side extension) has been reduced. Beside, with increasing the column slenderness, it weakens; since the lateral deflection before failure increases the bending moment which reduces the mean of compressive strain in concrete. In general this is correct for columns. When the plate limit is little, the confinement will happen when the concrete is began to crush before the buckling occurs in steel [10, 21]. It is supposed that the confinement effect is related to the properties of the cross-section and the material properties. The factor of confinement is a useful parameter employed by the researchers to examine the ability of columns confinement approximately. This index could be used for circular, square, and rectangular CFST columns [13].

$$\xi = \frac{A_s f_y}{A_c f_c} \tag{2.1}$$

Where, A_s and A_c areas cross-section for steel tube and concrete, respectively; f_y is steel yield strength; f_c is concrete compressive strength. The an essential parameter in EC4 code, steel contribution ratio (δ) is mainly influenced by the values of these parameters (A_s,A_c, f_y and f_c). The steel contribution ratio with these parameters permit examining columns within and behind the EC4 limits of application. It was found that the increasing of the confinement index (ξ) is coming from the increment in confining concrete supplied by the steel tube. It is also noted that this factor can describe the confinement effect qualitatively for different kinds of cross-sectional shapes, such as circular, square or rectangular [2].

In the member design under axial compression, most codes and specifications acknowledge the effect of the "composite action," especially for members with a circular cross section; therefore the strength of the composite member is enhanced. Strength index (*SI*) used to analyze how the column acts. According to composite actions and confinement valuations in CFST column it is a very beneficial value and has been used in researches. (*SI*) is defined as follows:

$$SI = \frac{P_u}{P_{uo}} \tag{2.2}$$

Where, P_u is the axial capacity a CFST column predicted by codes or specifications. And P_{uo} is the sectional capacity or squash load:

$$P_{uo} = A_s f_y + 0.85 A_c f_c \tag{2.3}$$

in which A_s and A_c are the cross-sectional area of steel and concrete, respectively; f_y and f_c are the yield strength for steel and the compressive strength for concrete, respectively. The factor, 0.85, used to recompense concrete uncertainty itself. Also, the sectional capacity formulation used in ACI and AS3600-2009 specifications to compute the composite sections strength [2, 13].

2.4 Failure Modes

For circular CFST columns, the mode of failure usually delivers both the steel tube and concrete infill to their individual material strengths (yielding and crushing). However, the longitudinal (axial) stress capacity of the materials is influenced by the transverse interaction between the steel pipe and concrete filling, and that leads to the peripheral stresses (transverse tensile) in the steel tube wall and restriction to the concrete filling (Lai and Varma 2016) [22].Short composite columns show a failure mechanization known from a crushing in concrete and compressive steel yielding, while column with medium length has an inelastic behave and steel yielding failed partially, concrete cracking in tension and crushing in compression showing a sign of interaction between global and local modes of failure. For long composite columns, Global failure mode is more governing and the overall variability failure mode happens by cracking of concrete and partial compressive yielding of steel. Columns with high ratio of D/t, local buckling happens as a failure mode. Throughout the entire column length, stiffness changes because of an uncracked concrete at the column edges while near the center an increasing frequency of cracking occurs [21, 13, 9].

In very slender circular CFST column, the mode of failure is instable elastically, and the flexural rigidity control the ultimate strength essentially. Also, the concrete filler has two basic functions in the very slender columns, the first function is to increase the ultimate strength and improve the stiffness of the column, and the second one is participating in preventing the tube local buckling [23].

2.5 Applications of High Strength Materials

With an aim towards more sustainable construction, high strength materials may reduce the use of construction materials, thus reducing the use of water, energy and manpower in handling such materials. Request for the use of high strength materials for high-rise buildings began in the 1970s, primarily in the U.S.A. Nowadays; high strength construction materials are mostly used for high rise structural construction in Asia region in countries which have seismic activities such as Japan, Korea and China. Since this study suggested to cover practical cases within and beyond the limitations of design codes; and since these specifications have not given any suggestions when using high strength concrete in composite columns [24, 37]. It would be focused on the high strength materials to fill the gap and lack of information in this field and encourage the structural engineers to use these materials in a safe and efficient manner in design and construction.

2.5.1 Concrete

American Concrete Institute (ACI, 2010) states concrete with high strength as concrete with a compressive strength greater than 50 N/mm^2. The definition of high strength concrete is implicitly reflected in EN 1992-1-2 (2004) for fire resistant design of structural concrete. For high strength concrete, distinctive care are required for production and testing and at which special structural design requirements may be needed. As technology progresses and the use of concrete with even higher compressive strength develops, the definition of high strength concrete is likely to be revised. Concrete with high strength is made possible by decreasing inhomogeneity, porosity, and micro-cracks in the hydrated cement paste and in the interfacial transition zone between cement paste and aggregates. The employment of fine pozzolanic materials causes a minimizing the size of the crystalline mixtures in concrete with high strength, particularly, calcium hydroxide. Subsequently, the thickness of the interfacial

transition zone will be decreased. The tightening of the interfacial transition zone allows an efficient load transfer between the cement mortar and the coarse aggregate, contributing to the strength of the concrete. For high strength concrete with extremely dense matrix, a weak aggregate may be the weak bond for the strength of concrete.
It is noteworthy that high strength concrete HSC and high performance concrete HPC are not identical. HSC exhibits higher strength and secant modulus whereas HPC has their ingredients and mix proportions specifically prepared as to own particular properties for the probable use of the structure such as higher strength, better ductility and lower permeability. The concrete strength classes, as shown in Table 2.1 and Table 2.2, for normal strength concrete NSC and HSC could be used in the design of CFST columns. Concrete class higher than C90/105 can only be used with further examination and advices from specialists using performance based design approach. Concrete with the cylinder compressive strength more than 90 N/mm^2 could be defined as ultra-high strength concrete UHSC. In the concrete material, stiffness increases with its distinctive strength.

Table 2.1 Classes of normal strength concrete according to EC4-code

Class of strength	C12/15	C16/20	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
Cylinder strength (f_{ck}, N/mm2)	12	16	20	25	30	35	40	45	50
Cube strength (f_{ck},cube, N/mm2)	15	20	25	30	37	45	50	55	60
Modulus of elasticity (E_{cm},GP)	27	29	30	31	33	34	35	36	37

Table 2.2 Classes of high strength concrete according to EC4-code

Class of strength	C55/67	C60/75	C70/85	C80/95	C90/105
Cylinder strength (f_{ck}, N/mm2)	55	60	70	80	90
Cube strength (f_{ck},cube, N/mm2)	67	75	85	95	105
Modulus of elasticity (E_{cm},GPa)	38	39	41	42	44

The higher-strength Concrete possesses an active primer elastic modulus that approximately rises proportional with the compressive strength and density to the second or third root. Joyce and Rangan [25], Rangan and O'Brien [26] conveyed that in tests on slender steel tube columns loaded eccentrically and have been occupied with high-strength concrete, the results rise up to (115) MPa. In the same tests and at both column ends, the axial compressive load applied with the same eccentricity and single curvature bending exposed to the members. Most of samples had failed at mid-height point because of the concrete crushing in compression region. For all columns, the yield strain of steel had not reached by the extreme fiber tensile strains at failure.

The ultimate loads considered by Rangan and Joyce were found to undervalue the maximum difference of 68% with the experimental results. Similar experiments were conducted in order to assess the behavior of thin-walled circular steel pipes occupied with concrete with ultra-higher strength (115 Mpa). The specimens in the test have an effective ratio of D/t ranged from (60) up to (200) and a value 3.5 for L/D ratio. The samples have been examined under eccentric axial load. It had been displayed for concrete with high-strength and unloading response, was quick and might show strain axially reversing, the procedure of snap-back.

Additional experiments had done to estimate the effect of high-strength concrete confinement on the potential improvement in strength and possible enhancement in ductility. The high performance concrete HPC had less coarse aggregate and more powder materials than conservative concrete with air-entrained. Though reaching a higher ultimate strength, unlike in columns filled with the ordinary concrete; it loses its stiffness sharply. After the peak, the deformation rate is just high, though the applied load at a specific loading level, is continuous to be constant; the stiffness rapidly reduced consequently in CFST [22, 24].With using of high-strength concrete, CFST columns are often stronger per square foot than conservative reinforced concrete columns. Also, the smaller and lighter framework places less stress on the base [3, 27].

2.5.2 Steel

High strength steel possesses many benefits in its applications in high constructions. The research activities in this region have been enhanced due to the raise in high-strength steel ductility. Low weight and high strength both are valuable in the seismic design besides the seismic reactions which reduced through a low weight of a structure. Steel with yield strength lower and equal to 460 N/mm^2 is defined as mild steel, whereas those with yield strength higher than 460 $N/mm^2$are defined as high tensile steel.

Two types of high tensile steel should be notable which are mostly used in building and offshore structures. One type is called QT steel which is manufactured based on quenching and tempering processes after rolling. Quenching imparts a high degree of hardness, while leading to a high strength. However, the quenched steel is too brittle to be used. It is returned to the furnace for tempering which will reduce the hardness and then achieve better ductility. The technical delivery conditions for flat products of QT steels are documented by EN 10025-6 (2004).

The abbreviation form of delivery form is Q. Another type of high tensile steel is TMCP steel which is manufactured based on thermo-mechanically controlled process during rolling, in which case the rolling process is accompanied by a quenching procedure. The technical delivery conditions of TMCP plates are included in EN 10149-2 (1996). The abbreviation form of delivery condition is M. The strength classes as shown in Table 2.3 for mild steel and high tensile steel could be used for the design of CFST columns, with hot-rolled, cold-formed or welded steel sections.

Table 2.3 Strength classes of mild and high tensile steel according to EC4-code

Grade	Nominal values of yield strength f_y(N/mm2) with thickness (mm) less than or equal to					
	16	40	63	80	100	150
S235	235	225	215	215	215	195
S275	275	265	255	245	235	225
S355	355	345	335	325	315	295
S420	420	400	390	370	360	340
S460	460	440	430	410	400	380
S500	500	500	480	480	480	440
S550	550	550	530	530	530	490
S620	620	620	580	580	580	560
S690	690	690	650	650	650	630

For steel, the modulus of elasticity is taken as 210 GPa. Patil and Uy [28] offered a survey upon the behavior of CFS with high-strength manufactured columns. A steel column with high-strength filled by concrete, steel as usual might not have gotten yield before concrete getting its ultimate strain that influenced by steel yield strain. Many of the benefits pushed to the using of HSS in the construction of reinforced CFS box column have been debated. A cost investigations display that a concrete padded box column construction can save up to 50% in costs compared to traditional steel box column and also by reduction in column size, it can provide more useful floor space [21, 24].

2.6 Past Researches

O'Shea and Bridge [8, 29] tested behavior of the circular thin-walled steel pipes which had ratio of D/t alternating from 55upto 200. Tests contained; hollow steel tubes, tubes with steel section only loaded and unbounded concrete, tubes with steel filled with concrete had loaded at the same time, and just the concrete infill tubes loaded. A comparison was made between the test strengths and the strength of models in standards and design specifications, and the results indicated that the concrete infill in the thin-walled circular steel tubes affects slightly on the local buckling tubes improvement, while (in 1997) they examined the buckling effect upon performance of short CFST column, two probable modes of failure of the steel tube had been recognized, yield failure and local buckling of the steel tube. These were considered to be independent of the D/t ratio. While the fail was restricted from the linking between steel and concrete filler. A planned method of design had been submitted; on the basis of the roles in Euro code 4.

Kilpatrick et al. [30, 31] tested the Euro code 4 pertinence for the design of CFST columns that use concrete with high-strength and compared Euro code 4 with others columns from six different investigations. Concrete strength for columns varied between (23) up to (103) MPa. Results showed that Euro code predicted the failure load in (73%) of studied members.

Schneider [32] studied experimentally and analytically the behavior of short concrete filled steel tube columns loaded concentrically in compression up to failure. The test included 14 specimens which were examined to investigate the effect of the shape and thickness of the steel tube on the ultimate strength of the column. Confinement of the concrete core provided by the tube shape was also addressed. *D/t* ratios between $17 < D/_t < 50$, and the L/D ratios of $4 < L/_D < 5$ were examined. The results of ultimate strength were compared with specifications that governing the using design of concrete filled steel tube columns. The experimental results were used to develop and verify the nonlinear finite-element models. Further, the analytical models were used to investigate how the design specifications are adequate. Testing results suggest that circular pipes offer much more post-yield axial ductility and stiffness, does not available in most square or rectangular cross sections columns. Also, by these results was observed that design specifications were satisfactory to expect the yield load under most conditions, for a variety of structural shapes.

O'Shea and Bridge [8, 33] estimated the CFST strength under different conditions of loading with small eccentricities. All the specimens were short with L/D ratio of 3.5 and a D/t ratio ranged from (60) up to (220). The internal concrete had three different values for compressive strength. Bridge and O'Shea determined from those experiments; that the confinement degree had offered to the internal concrete by a circular steel tube is relied upon condition of loading.

By having an adequate bond between the steel and the concrete, the local buckling of the tube does not occur in thin-walled steel tubes loaded axially. For concrete strengths up to 80 MPa, with no reduction for local buckling, Eurocode 4 can be used. For strengths of concrete in excess of (80) MPa, Eurocode 4 could be used but with no decreased in steel strength from local buckling and biaxial effects from confinement and with no enhancement of confinement in the concrete.

Georgios et al. [20] investigated the behavior of circular concrete filled steel pipes CFST with different concrete strengths axially loaded. In the study, the thickness of steel tube effects, the bond strength between the steel tube and concrete, and the concrete confinement are observed. The strengths of studied column are imposed to compare with the European, Australian, and American Codes values. The values showed by these codes were lower than values taken through the tests. Authors proposed that ACI and AS code to take the confinement effect in to account also give a value also show the different of load displacement behavior in greased and non-greased inner surface. Author indicated that Euro code 4 gives a good prediction of the axial strength of CFST columns, the largest difference was 17% among the experimental and predicted readings on the capacity. Exceptional prediction was carried out for high strength CFST, with 'n' number of tests/NEC4 ratio around unity. The axial strengths predicted by ACI and AS were lower than the results obtained from experiments by 35%. A coefficient is proposed for the ACI/AS equations to take the effect of concrete confinement into account on the axial load capacity of CFST column. The bond between concrete and steel tube effect are extra critical for concrete with high strength. Reduction in axial capacity because of losing connection among steel and concrete was omitted, for normal strength concrete. The best estimation was given by Euro code 4 for both CFST with NSC and HSC.

Sakino et al. [34] tested experimentally loaded short CFST columns. The test objectives were to explain the interaction between steel and filled concrete, also to derive methods to describe the relationship of the load–deformation of CFST columns.

Parameters employed in the tests are as follows: steel tube shape, steel tube tensile strength, D/t ratio, and concrete strength. In determining the variety of parameters, the emphasis was placed on obtaining test data with wide range for founding an appropriate design method of CFST column systems, generally. Design formulations are offered to estimate the ultimate axial compressive load capacities for CFST columns for both circular and square sections established by tests.

Zeghiche and Chaoui [35] studied the influence of slenderness of column and strength of concrete on the behavior of CFST columns and its capacity. An increase in the CFST columns capacity occurs as a result on increment in concrete strength, however when the tube filled with NSC, the confinement was found to be more effective, because of the advanced capacity of deformation in comparison when concrete is high strength. For shorter column, the enhancement in the column loading capacity is more noteworthy. In slender columns the failure caused by overall instability while in shorter columns failure happened by concrete crushing and steel yielding. For higher concrete strength, the increase in the effective length of the columns considerably affected much higher the column loading capacity with a rate of load decreasing.

Han et al. [36] tested experimentally the behavior of self-consolidating concrete (SCC) filled hollow structural steel (HSS) short columns subjected to an axial load. In the test fifteen samples were investigated. The chief factors were: (1) type of section. (2) the grade of steel; and (3) the D/t ratio variance. Where a comprehensive theory of the effect of the confinement factor to measure composite action between steel and concert was presented. Also theoretic models were taken to study the impact of parameters on the ultimate strength of CFST column. Also making comparisons between results taken experimentally and the existing codes.

de Oliviera et al. [37] experimentally studied the effects of the confinement in composite columns concerning two different parameters: slenderness of columns, compressive strength of concrete. Under axial loading, (16) CFST columns with circular sections were investigated. The studied member was filled with concrete of various compressive strength, and different L/D ratios. The columns' ultimate load's experimental values were imposed to compare with the predictions of four codes: Brazilian Code, European code, American code and Canadian code. The results showed that for the capacity of column; increase with the rising of concrete strength and decrease with the increasing of L/D ratio. Generally, the codes were very precise in predicting the capacity. The Brazilian specifications were very conventional, however Euro code 4 offered the closest to the real results.

Portolés et al. [38] verified the advisability of the use of high strength concretes as opposed to that of normal strength concretes by comparing three performance indices: strength index, ductility index, and concrete contribution ratio. The test parameters were the nominal strength of concrete (30, 70 and 90 MPa), the diameter to thickness ratio D/t, the eccentricity ratio e/D and the column slenderness L/D. The results show for the limited cases examined that the using of high strength concrete for slender composite columns is interesting since this achieves ductile behavior despite the increase in load-carrying capacity is not significantly enhanced.

Dundu [39] investigated the behavior and CFST columns capacity of various diameters, lengths, various concrete strength. Concluded that large slenderness ratio caused overall buckling failure for columns. An assessment of the tests results with predicted loads by the South African code and European code.

An and Han [23] studied a slender (CFST) column performance under compression. Parametric studies were carried out and the ultimate strengths from tested results and design codes were compared and discussed. The results showed that concrete core in a slender CFST column had two functions, the first one is improving the stiffness and increasing the ultimate capacity of the column, the second function is participating to avoid the buckling of the tube inwardly. When the slenderness ratio (λ) increases the ultimate capacity reduced, while the capacity increases with the increase of steel contribution ratio (α) and concrete strength. The impact of steel yield strength (f_y) on the ultimate capacity of a slender CFST column was unimportant.

Abed et al. [40] investigated experimentally the compressive behavior of CFST column exposed to net axial loading .The impacts of two main parameters are taken into study, namely; concrete compressive strength ($f_c{}'$) and D/t ratio. The effect of D/t on the samples was found to be higher than that of other indexes. It is also noted that ductility decreases with the increased concrete strength of higher D/t ratio, but for the lower D/t ratios the reverse was true. The increase in ratio D/t reduces stiffness of the CFST member and its axial force because of the decrease in confinement.

Uy et al. [41] reported that the use of various high-strength materials is currently limited by international codes of practice and further research is required to inform and update these standards. (AISC) limits the maximum steel yield stress up to 525 MPa and the concrete compressive strengths up to 70 N/mm2 (American Institute of Steel

Constructions 2010). The Eurocode 4 document limits the maximum yield stress of steel to 460 N/mm2 and compressive strengths of concrete to 60 N/mm2 (Eurocode 4 2004). The Chinese Standards also limit the maximum yield stress of steel to 420 N/mm2 and compressive strength to 80 N/mm2 (National Standards of the People's Republic of China 2002, 2003).

Aslani et al. [6, 42] investigated the suitability of the several codes to predict the capacity of steel tube column filled with HSC under the axial load. According to the statistical results, formulas were advanced and simplified to predict the cross section capacity and buckling capacity of NSC and HSC filled short and long CFST columns with different shapes, under compression.

W.-H. Kang et al. [43] re-evaluated the capacity reduction factor in AS-5100.6, the Australian Bridge Standard for concrete-filled steel tube columns, estimated the dependability of present equation, and investigated the effect of these features. These results indicate that the interaction between the concrete and steel should be measured for the current reduction factor in AS-5100,and showed that these current capacity factor values in AS 5100 for CFST columns are adequate with regards to safety and can be maintained to meet the target reliability suggested in the current design codes.

Tao et al [44, 45] utilized a wide range of experimental data from available test databases (2194 test results altogether) to evaluate the applicability of AS 5100 in calculating the strength of CFST members, also compared the test results with many international design specifications and concluded that after ignoring all code limitations and taking material partial safety factors as unity, there are considerable differences among different code predictions. The predicted results using AS 5100 are quite close to those from EC4.

Ekmekyapar and AL-Eliwi [13] tested the capacity and the confinement of circular concrete filled steel tube CFST column. Three L/D ratios, two D/t ratios, two steel grades and three concrete classes were used to examine the influence of parameters and confinement. The results showed that the L/D ratio is very essential parameter which has direct impact on column capacity, and D/t and confinement index does not have a direct impact on the performance of CFST column.

Güneyisi et al. [9] prepared a data set for training and examining the default model which gotten by the use of gene expression programming (GEP). The factors were taken as column diameter (D), tube thickness (t), column length (L), concrete compressive strength (f_c), and steel yield strength (f_y). The offered model had compared with other one has been existed in the design specifications (American, Australian Standards, Japanese, and European) codes. The results obtained by using "GEP" program compared to the formulations exist in the codes were favored with a small rate of error.

Yadav and Chen [7] investigated the ultimate loading of CFST column by varying D/t ratio, steel grades, and concrete classes. The results display that the loading capacity of CFST column could be considerably raised with a smaller D/t ratio in the design for the section. Also, the steel yield strength does not depend on the early stage of flexural stiffness for column. Yet, the ultimate capacity of column is proportion with steel yield strength. Finally, the primer stiffness for columns raises a little bit because of using HSC. Moreover, HSC using leads to increase in the load carrying.

2.7 Applications of CFST Columns

The concrete-filled steel tubes are used as main compressive components or key members under various loading conditions in buildings, bridges and other structures. Several examples are presented as follows.

2.7.1 Buildings

In the 1980s, the CFST column was used in buildings to avoid having a very large size column and several buildings with CFST columns were built in China. Since the 1990s, more buildings were built in various cities in China. The CFST column typically served as the member resisting compressive load, and is usually connected to steel or reinforced RC core tubes or steel shear walls. The concrete filled steel tubular columns used by frame; integrates the high stiffness and ductility, also acts fine with the core pipes or shear walls in mixture structural constructions Fig. 2.3 shows the SEG Plaza in Shenzhen, China which was one of the earliest applications of CFST columns in super high buildings. The main structure about 291.6 m, and CFST columns with circular cross- section were used. The profile of the steel section is Φ 1600 mm × 28 mm, and Q345 steel and C60 concrete were used. When compared with the column using hollow steel section, the steel usage for the CFST column was only a half, and the use of very thick steel plate was stopped.

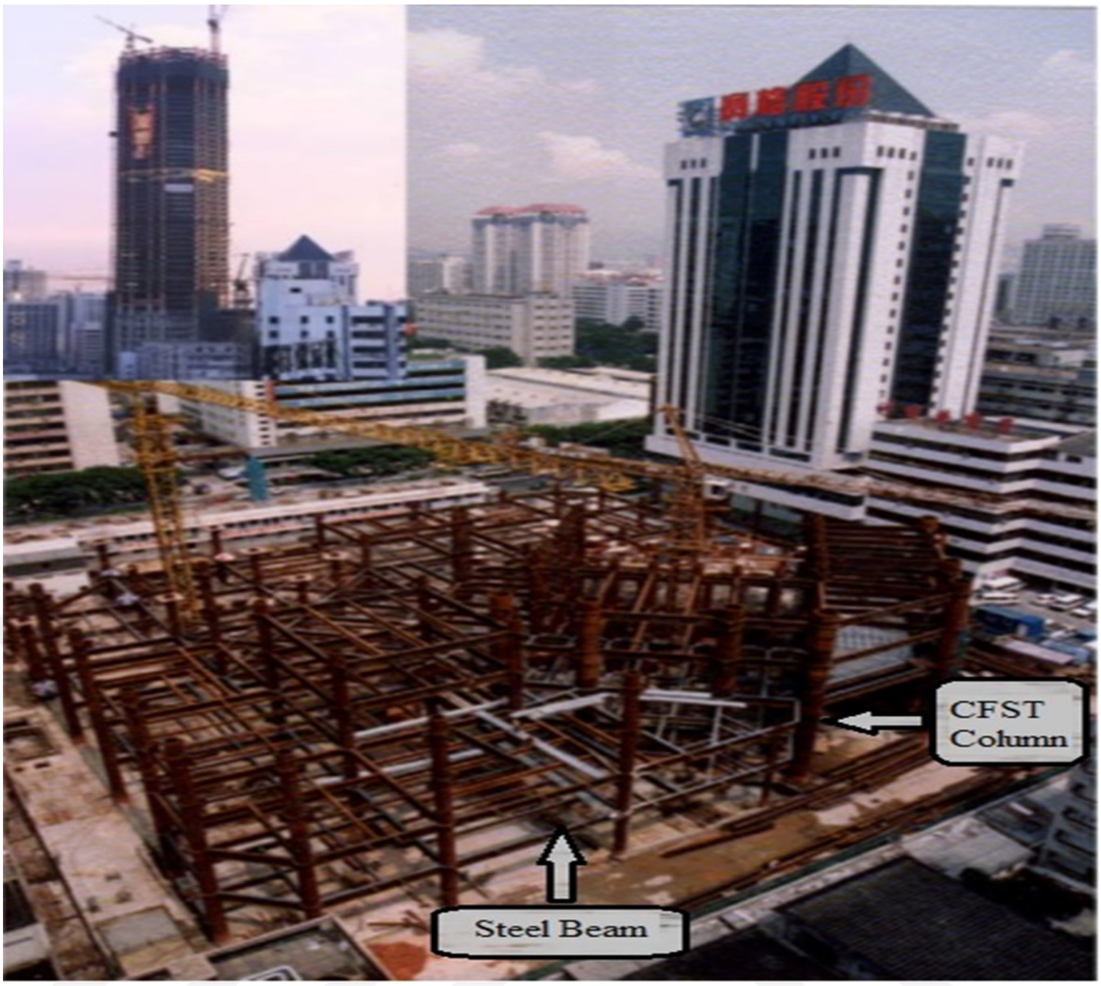


Figure 2.3 SEG plaza in Shenzhen, China [2].

2.7.2 Bridges

CFST members have been applied in numerous types of bridges, such as arch bridges; cable stayed bridges, hanging bridges, and truss bridges. CFST members can serve as piers, bridge towers and arches, and they can also be used in the bridge deck system. Fig.2.4 shows one of the earliest CFST arch bridges in China, the Wangcang East River Bridge, which was built in 1992 . The cross section of the main arch is in dumbbell shape, and the total depth is 2 meters. Steel tubes with a diameter of

Figure 2.4 Wangcang East River Bridge, China [2].

800 mm and a thickness of 10 mm are used for upper and lower chords, and the hollow sections are filled with C30 concrete. The main span of this bridge is 115 meters. The use of CFST members in arch bridges effectively utilizes the advantages of that kind of construction.

A significant benefit of using CFST members in an arch bridge is that, during the different stages of erection, the hollow steel tubes can serve as the formwork for casting the concrete, which significantly minimizes the cost of construction and saving the time during the construction period.

Furthermore, the composite arch can be erected without the aid of a temporary bridging due to the inherent stability of tubular structure. The bare steel pipes can be occupied with concrete to transform the system into a composite structure. Since the weight of the bare steel tubes is relatively small, moderately simple construction technology can be used for the erection. The most common methods that including cantilever launching approaches, and either horizontal or vertical “swing” methods, whereby each half arch can be alternated horizontally into location [2].

CHAPTER 3

DESIGN SPECIFICATIONS

Many design specifications have been employed to predicate the axial loading capacity of CFST columns; AISC 360 – 2010 (The steel structures specification in the United States), EC4 (the European code for composite structure design), AS5100.6 – 2004 (Australian Standard for steel and composite constructions), and AIJ 2001 (The Japanese guide for steel reinforced concrete structures).

3.1 AISC 360-2010 Code

3.1.1 General requirements

Composite sections design needs paying attention for the behavior of both concrete and steel. Those requirements were developed to minimize both, the struggles between existing concrete and steel design and providing an accurate confirmation of the advantages of composite design, determined for the elaboration of the specimens.
The designer should calculate the imposed loads from the steel section by itself while evaluating the load effect in the period of constructions. Also, the designer should pay attention to the deformation during structure lifetime and for these deformations, choose the suitable section. By consider these final boundary conditions, the allowance must be given for additional long term variations in stresses and deformations, depending on the concrete creep and shrinkage.

3.1.2 The nominal strength for composite section

Typically, nominal strength is defined as the resistance resulting from load effects, like applied force axially, shear or torque, bending moment, while in other cases it can be defined as a stress. Composite sections strength should be calculated depending on any of the two methods in the current code. The strain compatibility method is the first one that offers a calculation method in general. The plastic stress distributions method is the second one, which a subdivision of the first method. The plastic stress distribution technique offers an appropriate and simple evaluation approach for the familiar design cases, and for this reason it would be treated first. With the plastic stress distributions approach it has been assumed that steel have gotten F_y of stress in compression or tension and concrete in compression caused by axial force have gotten $0.85f_c'$of stress. For circular hollow steel section filled with concrete, a stress of $(0.95f_c')$ allowed to be employed for concrete in the compression because of the axial force to be accounted considering influence of confinement the concrete.

3.1.2.1 Method of Plastic Stress Distribution

The plastic stress distributions approach basing upon a supposition of linear strain and elasto plastic behavior across the cross section. It adopts that the concrete at a strain of 0.003, has reached its crushing strength in compression status and an identical stress, typically $(0.85f_c')$ on a rectangular block of stress, where (F_y /E_s); for steel had passed over its yield strain.

Those assumptions supposed that; for various groups of bending moment and axial force, the strength of cross section might be estimated for composite compression typical member cross section. For compression members, plastic stress approach adopts sliding does not happen between steel and concrete, and the desired (*D*/t) ratios stop the happening of local buckling till steel yielding and concrete crushing occurred.

3.1.2.2 Local Buckling Classified Filled Composite Sections

Filled composite section is categorized as compact, non-compact or slender, for compression, To qualify a section by means of compact, the maximum ratio (D/t) of the compressive steel elements should not go beyond the limit ratio (D/t), (λp), in Table 3.1 If the maximum ratio (D/t) for steel compression members overtakes λp, and not passing λr in Table 3.1, the filled section is non-compact. If the optimum ratio (D/t) for any steel member overtakes λr, then that section is slender.

Table 3.1 Composite section classifications

Compact	Non-compact	Slender
D/t < $\left(\lambda_p = 0.15\frac{E}{F_y}\right)$	$\left(\lambda_p = 0.15\frac{E}{F_y}\right)$<D/t<$\left(\lambda_r = 0.19\frac{E}{F_y}\right)$	D/t > $\left(\lambda_r = 0.19\frac{E}{F_y}\right)$

Basically, the filled composite members' behavior is unlike that for the bare steel members'. The concrete infill had an important impact on the composite members regarding stiffness, ductility, and strength. Concrete contribution takes more noteworthy role, when the area of steel section decreases. By the existence of concrete infill, elastic buckling of steel is affected considerably. The buckling type of steel tube is changed by concrete infill existing (for local and global buckling), through restraining it from buckling inwardly.

Bradford et al. [46] showed that for filled round sections, the elastic local buckling stress was (1.73) times for HSS. In axial compression, the non-compact/slender limit, λr, for round filled steel section; recognized by (0.19*E/Fy*), which was (1.73) more than (0.11E/*Fy*) for HSS. That relied upon the conclusions of Bradford et al. (2002) mentioned previously, likens with readings taken experimentally. The optimum allowable value of (D/t) equals (0.31*E/Fy*) which relied upon the absence of experimental readings and the possible influences of the concrete position in enormously slender filled with HSS section.

3.1.3 Limitations of CFST members

- The steel cross-sectional area (steel contribution ratio) should cover (1%) of the gross compounded cross section, for column.
- Filled composite section can be categorized as compact, non-compact or slender reliant on the tube slenderness, D/t, with addition to Table 3.1 limitations.

For the limitation of material properties, a compressive strength, f_c', for normal weight concrete should be between (21) MPa up to (70) MPa while for lightweight concrete (21) MPa up to (42) MPa. The identified minimum steel yield stress of structural and reinforcing bars used in calculating the strength of composite members shall not exceed (525MPa). The physical properties available from tests reflected by these physical limitations. For design of reinforced concrete, a limit of (70) MPa is given for strength calculations. The value of (21) MPa is identified for normal and lightweight concrete and the value of (42) MPa is identified for lightweight concrete in order to promote using good quality. In the calculations of modulus of elasticity, a higher capacities can be used. If an approval of suitable tests and analyzes is obtained, the extension of the given limits for capacity could be done.

3.1.4 Compressive strength

The nominal compressive strength for filled composite members loaded axially doubly symmetric, the limit state of flexural buckling should be determined with respect to the following adjustments:

$$\text{for} \quad \frac{P_{no}}{P_e} \leq 2.25 \qquad P_n = P_{no}\left[0.658^{\frac{P_{no}}{P_e}}\right] \tag{3.1}$$

$$\text{for} \quad \frac{P_{no}}{P_e} > 2.25 \qquad P_n = 0.877P_e \tag{3.2}$$

- For compact sections:-An appropriate thickness the compact hollow section structure HSS possesses to improve the steel yield stress longitudinally for HSS, supplying the concrete inside it with confinement which leads to improve its compression stress ($0.95f_c'$), also in compression, the full plastic strength, P_P, could be improved by compact section:

$$P_{no} = P_p \tag{3.3}$$

where,

$$P_p = A_s F_y + C_2 f'_c A_c \tag{3.4}$$

$C2$ = 0.95 for circular sections.

- For non-compact sections:- An adequate tube thickness for non-compact section so that the steel yield stress is produced longitudinally, while the concrete cannot be sufficiently confined, after reaching $0.70f_c'$ compressive strength and begins passing volumetric expansion with noteworthy inelasticity, pushing the HSS of steel , from a quadratic interpolation between yield strength, P_y and plastic strength, P_P, , nominal strength, P_{no}, for noncompact sections could be specified, for the tube slenderness. The reason for quadratic interpolation is due to the confine of concrete by the tube, which undergoes a state of elasticity and volumetric expansion decreases rapidly with the slenderness of the hollow steel section.

$$P_{no} = P_p - \frac{P_p - P_y}{(\lambda_r - \lambda_p)^2}(\lambda - \lambda_p)^2 \tag{3.5}$$

Where,

λ, λp and λr are ratios of slenderness given in Tab. (3.1)

Pp is determined from Equation (3.4)

$$P_y = A_s F_y + 0.7 f'_c A_c \tag{3.6}$$

- For slender sections:- In the case of the slender section, steel yield stress cannot be developed in the steel HSS longitudinally, even the concrete core cannot be confined after it reaches f_c'compressive strength passing inelastic strains and noteworthy volumetric expansion pushing the HSS, the slender sections are restricted towards improving the $0.7f_c'$ of the concrete core and critical buckling stress,F_{cr}, for HSS.

$$P_{no} = A_s F_{cr} + 0.7 f_c' A_c \quad (3.7)$$

Where,

For circular filled sections

$$F_{cr} = \frac{0.72 F_y}{\left[\left(\frac{D}{t}\right)\frac{F_y}{E_s}\right]^{0.2}} \quad (3.8)$$

For all sections, the composite section effective stiffness, EI_{eff}, should be as follows:

$$EI_{eff} = E_s I_s + C_3 E_c I_c \quad (3.9)$$

Where,

C_3 = factor for determination the effective rigidity.

$$= 0.6 + 2\left[\frac{A_s}{A_c + A_s}\right] \leq 0.9 \quad (3.10)$$

P_e = elastic critical buckling load, (N)

$$P_e = \frac{\pi^2 \, (EI_{eff})}{(kL)^2} \quad (3.11)$$

A_c = concrete area, (mm2).

A_s = steel section area, (mm2).

E_c = elastic modulus of concrete.

= $(0.043 \, w^{1.5}\sqrt{f_c'}$, Mpa).

E_s = elastic modulus of steel.

F_y = minimum steel yield stress, (MPa).

I_C = second moment of inertia for concrete section (mm^4).

I_S = second moment of inertia for steel section (mm^4).

K = factor of effective length.

L = member length unbraced laterally, (mm).

f_c' = concrete compressive strength, (MPa).

w_c = concrete weight ($1500 \le w_c \le 2500$ kg/m3).

For the effect of length of composite compression members, the nominal axial strength, P_{no}, might be determined by the use of eqs. (3.1) and (3.2) [16].

3.2 EN 1994-1-1 Eurocode 4 (EC4)

3.2.1 Range of Eurocode 4

- Eurocode 4 used to the design of composite structures and members for buildings and civil engineering works. It complies with the principles and requirements for the safety and serviceability of structures, the basis of their design and verification that are given in EN 1990 – Basis of structural design.

- Eurocode 4 is concerned only with requirements for resistance, serviceability, durability and fire resistance of composite structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.

3.2.2 The basic of design method

An isolated column in EC4, is well known as compression member, an integral part of a stable or non-sway frame, however it is treated as an isolated in calculations of design. The classifications for non-sway structure in EC2, are as follow:

- For structure or structural element, with or with no bracing element, the impact of the connection displacement upon designed moment and force might be ignored, it is known as non-sway. Else, it is known as sway.
- For braced structural construction, the large shear walls or core structures offering bracing, might be considered as non-sway.
- For Frame which might be known as non-sway when the connection first order displacement not increase the actions influences determined and not allowing this displacement to be more than (10%). Generally, the relevant bending moments is appropriate to be studied only because of these effects of the second-order.

A parallel classification for non-sway frame within EN 1993:1992 (EC3) which taken as a reference also:

- A frame might be defined as non sway if the reaction to horizontal forces in the plane is enough stiff to be accurate well to ignore any moments or internal forces due to horizontal displacements of the contract.

There are two approaches to design isolated braced composite members or, non-sway frame have been treated in EC4:

3.2.2.1 The general design approach

It deals with non-symmetrical or nonuniform cross-section of composite columns along the member length. Also, when application limits are not met for a simplified design approach, this approach used for composite columns with a double symmetric, and uniform cross-section along the column height. For this case, some of important design aspects to consider when using this approach, are as follows:

- Nonlinear material and geometrical.
- Effects of second order upon the slender column.
- The concrete shrinkage and creep, under conditions of long-term loading.
- The tensile strength supporting between cracks of concrete.
- The calculations imperfection of the internal moments and forces about the two axes.
- The internal force and moment distribution between steel and concrete by an obviously premium load path.
- Shear stress transfer longitudinally at the interference between steel and concrete beneath crosswise shear.
- Mechanical shear connection with friction and chemical bond together when there is a necessary.

  It is required to use advanced computer software in order to allow these design considerations, which work with both geometric and non-linear material. Generally, this approach is not favorite to apply in practical design because of the design efforts are substantial, and will not be used in the current publication.

3.2.2.2 The simplified design approach

In this method, the composite columns of the cross-section are symmetrical and uniform along the column height, also it based on particular assumption related with the geometric formations of the cross- sections for composite members, and also the Euro buckling curves of the steel column as a basis for the design of the buckling. This method is also included in the following topic with the limits of application in EC4. In case the application limits for this method are incomplete then refer to the general method mentioned above. It is worth mentioning that in this approach accounts are handy when the design in practice. In this publication, this method was introduced in detail with calculation procedure was put in six portions as bellow:

- To check the simplified design approach limits if they are fulfilled.
- To evaluate the cross-section properties.
- To evaluate the column buckling resistance.
- To estimate that the effects of second order should be taken.
- To evaluate the interaction influence between bending moment and axial load.
- To evaluate the shear longitudinally and transversely.

3.2.2.3 The Simplified Design Approach Restrictions

This approach has various boundaries, as bellow:

- The composite column is symmetrical and has a constant cross section along the column height.
- The contribution ratio of steel(δ) should meet the following qualifications:

$$0.2 \leq \delta \leq 0.9$$

When the contribution ratio greater than 0.9, the concrete must be neglected in the design process, and the column would be considered a bare steel structure. When the contribution ratio less than 0.2, the column is considered as in EC2. The steel contribution ratio (δ) expressed as:

$$\delta = \frac{A_s f_{yd}}{N_{pl,Rd}} \quad (3.12)$$

where:

$N_{Pl,Rd}$ is the plastic resistance to compression defined in (3.15).

- For the ratio of non-dimensional slenderness the maximum limit of the composite section ($\bar{\lambda}$) restricted up to 2.0.

3.2.3 Limitations of CFST members

In addition to the above restrictions, limitations of material properties for columns and compression members in this code would be for steel grades S235 up to S460 and normal weight concrete of strength classes C20/25 to C50/60.Also, the early local buckling to be prevented, the following limits must be satisfied by the (D/t) ratio of steel section in compressive:

Concrete filled circular hollow steel $\frac{d}{t} \leq 90\ \varepsilon^2$

where:

t thickness of CHS (mm).

d outer diameter of CHS (mm).

$$\varepsilon = \sqrt{\frac{235}{f_y}}\ \bar{\lambda} \tag{3.13}$$

f_y steel yield strength (N/mm²)

3.2.4 Properties of CFST cross-section

3.2.4.1 Sectional capacity (squash load), $N_{Pl,Rd}$

The plastic compression resistance of the cross section signifies the optimum load could be cast on the short column. It is worth noting that the CHS sections filled concrete show better resistance due to the effect of tri-axial containment. While RHS sections filled with concrete do not show such improvement in the resistance.

Circular Hollow Sections CHS Filled with Concrete

The concrete increased resistance as a result of the confining effect coming from the round hollow steel section might be considered for the composite columns, CHS filled with concrete. The increased in concrete resistance leads to limitation in transverse strain in a three dimensional confinement as result. Meanwhile, also tensile hoop stresses in the CHS arises, that decreases the axial capacity.

Generally, the CHS filled with concrete resistance to compression might be increased by 15% when taking the tri-axial confinement into account. In circular column with a non dimensional slenderness of ($\bar{\lambda} > 0.5$), the influence of confinement should be ignored and the plastic resistance evaluated like in rectangular hollow sections RHS.

Linear updating further of any active load is necessary to be taken into account eccentricities, in additionally. Yet, the eccentricity, *e,* for imposed load might not passed over (d/10) value

where

d outer circular hollow section diameter.

Eccentricity, e ,would be expressed as bellow:

$$e = \frac{M_{Sd}}{N_{Sd}} \tag{3.14}$$

where:

M_{Sd} optimum designed moment (when second order effects neglected).

N_{Sd} designed exposed load.

For a concrete filled CHS, the plastic resistance might be found as follows:

$$N_{pl,Rd}=\eta_2 A_a f_{yd}+A_c f_{cd}\left[1+\eta_1 \frac{t\, f_y}{d f_{ck}}\right] \qquad (3.15)$$

where:

t thickness of the wall for HS sections in (mm).

When $0 < e < (\frac{d}{10})$

$$\eta_2 = \eta_{20}+(1-\eta_{20})\frac{10e}{d} \qquad (3.16)$$

$$\eta_1=\eta_{10}\left[1-\frac{10e}{d}\right] \qquad (3.17)$$

When $e > (\frac{d}{10})$

$$\eta_1 = 0 \qquad (3.18)$$

$$\eta_2 = 1.0 \qquad (3.19)$$

The η_{10}, η_{20} values which relied on the ratio of non-dimensional slenderness, $\bar{\lambda}$, and are expressed as bellow:

$$\eta_{10} = 4.9 - 18.5\,\bar{\lambda} + 17\,\bar{\lambda}^2 \qquad \text{but } \eta_{10} \geq 0 \qquad (3.20)$$

$$\eta_{20} = 0.25(3 + 2\bar{\lambda}) \qquad \text{but } \eta_{20} \leq 1.0 \qquad (3.21)$$

If the eccentricity e exceeds the value $d/10$, or if the non-dimensional slenderness ratio $\bar{\lambda}$ exceeds the value 0.5, then η_{10} = 0 and η_{20} = 1.0. Table 3.2 gives the basic values η_{10} and η_{20} for different values of $\bar{\lambda}$.

Table 3.2 The η10, η20 values, when e = 0

Non-dimensional slenderness ratio $\bar{\lambda}$	**0**	**0.1**	**0.2**	**0.3**	**0.4**	$\geq$ **0.5**
η_{10}	4.90	3.22	1.88	0.88	0.22	0
η_{20}	0.75	0.80	0.85	0.90	0.95	1.0

3.2.4.2 The effective flexural stiffness

Short-term loading

By collecting the flexural stiffness for each separate component for the composite sections, the effective flexural stiffness for composite column $(EI)_e$ obtained as bellow:

$$(EI)_e = E_a I_a + 0.6 E_{cm} I_c \tag{3.22}$$

where:

I_a , I_c second moment of inertia for steel and concrete.

E_a modulus of elasticity of steel.

$0.6E_{cm}I_c$ component effective stiffness (Factor 0.6 is the experimental multiplier, which is determined by the calibration process).

E_c modulus of elasticity of steel.

Long-term loading

For composite column exposed to long-term loading; the creep and shrinkage of concrete leads to a decrease in the$(EI)_e$, thus decreasing the buckling resistance. Yet, just for slender column this impact is substantial. If the buckling length to depth ratio of a column exceeds 15; the impact of long-term condition should be taken. If the eccentricity of axial load is more than twice of cross-section; the influence of the applied moment resulting from the increased deflection of creep and shrinkage of the concrete; would be too small. As a result, the condition of long-term loading might be omitted and become unnecessary.

3.2.5 Column buckling resistance

$N_{pl,Rd}$ is the maximum value of loading that could be exposed to a short composite column, also could be named as the plastic compression resistance. Yet, Global buckling observance might be more important, with low elastic critical load, for slender columns.

For Fig. 3.2.1(b), the proportion χ which is the buckling resistance of a column of the plastic resistance to compression N_{Rd}, thus the vertical axis with compare to Fig. 3.2.1(a) non dimensionialising. In Fig. 3.2.1(b) the horizontal axis might be non dimensionalised in similar way with using critical buckling load N_{cr}.

The effects of residual stresses would be incorporated with geometric imperfection, so the Euro buckling curves might be sketched as illustrated in Fig. 3.2.1(c). For both steel and composite columns, these curves on the basis of column buckling are designed.

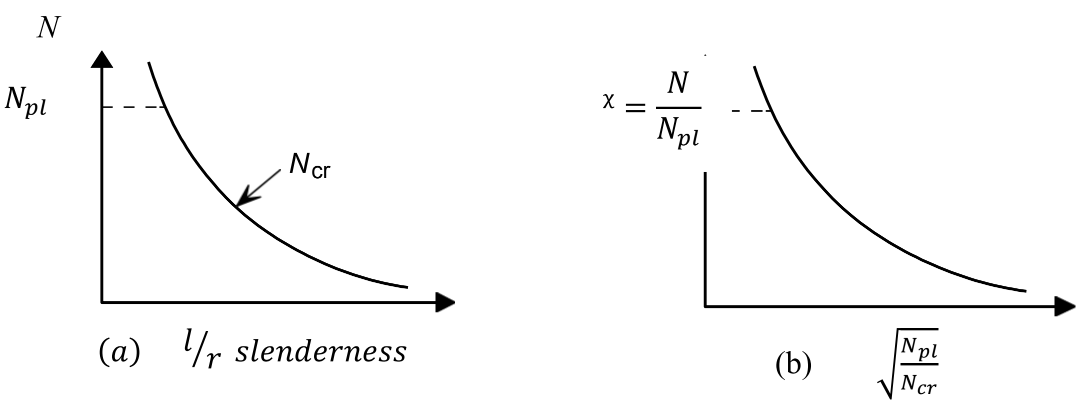
The resistance of buckling would be evaluated from plastic resistance and the Euler critical loading from the EC3 buckling curve 'a'. Critical loading is determined as bellow:

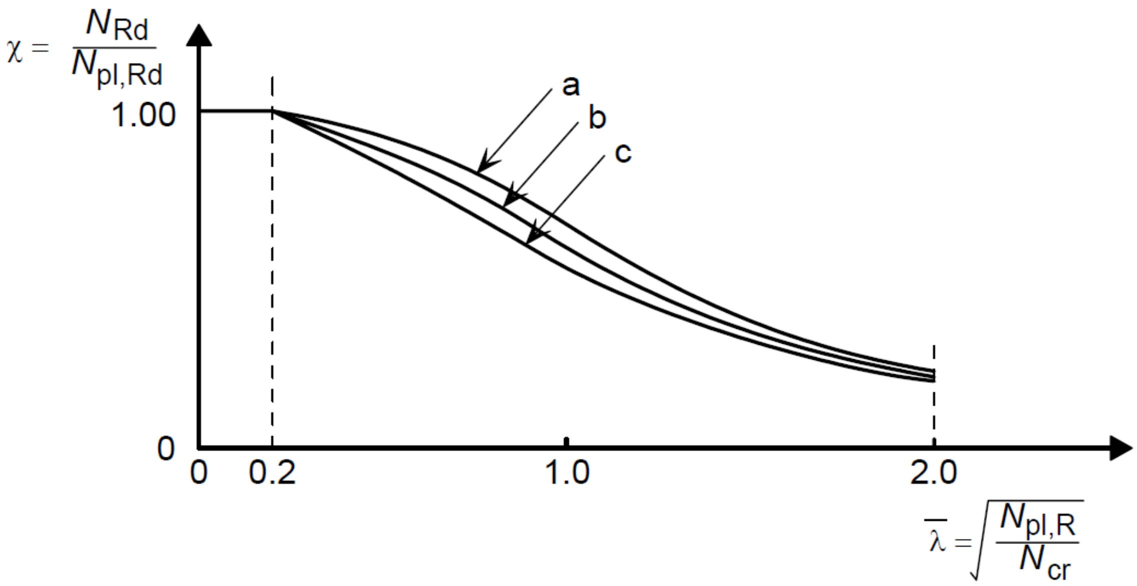
$$N_{cr} = \frac{\pi^2 (EI)_e}{l^2} \quad (3.23)$$

where:

$(EI)_e$ effective elastic flexural stiffness.

l length of column buckling.





(c)

Figure 3.1 (a) Idealized curve of buckling column, (b) Non dimensionalised curve of buckling column, (c) The Euro buckling curves in EC3.

EC4-1-1 advises about the length of buckling *l* for an isolated non sway column might be the same length as the column prior to the buckling (*L*). Buckling length of might be calculated from EC3, alternatively. The ratio of non dimensional slenderness expressed by:

$$\bar{\lambda} = \sqrt{\frac{N_{pl,Rd}}{N_{cr}}} \tag{3.24}$$

where:

$N_{pl,R}$ the composite section plastic resistance to compression, with $\gamma_a = \gamma_c = 1.0$.

The buckling load or the compression resistance, calculated by:

$$N_{Rd} = \chi . N_{pl,R} \tag{3.25}$$

χ the factor of reduction buckling calculated from curve 'a' in (EC3), which dependent upon the ratio of non dimensional slenderness $\bar{\lambda}$.

Calculate reduction factor might be gotten by:

$$\chi = \frac{1}{\phi + [\phi^2 - \bar{\lambda}^2]^{0.5}} \qquad \text{but} \leq 1.0 \tag{3.26}$$

where:

$$\phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right] \tag{3.27}$$

α a parameter of an imperfection dependent upon the curve of buckling [17].

3.2 AS 5100.6-2004 Code

3.3.1 General scope

The Australian Standard AS 5100 was organized by the Standards Australia Committee BD-090, Bridge Design, to supersede HB 77.6-1996, Australian Bridge Design Code. Seven parts are included in AS 5100 with objectives to offer nationally acceptable requirements for the design of rail, road, pedestrian and bicycle-path bridges; the specific application of concrete, steel and composite construction; and the valuation of the capacity of bridges. Part 6 of standard AS 5100 is associated with the design of steel and composite construction, in which procedures are provided for the design of CFS members that take account of the composite action among the various components forming the cross-section. In the design of composite steel and members, the general requirements of AS5100.5 concerning the design of concrete will apply besides the requests of the code.

3.3.2 Limitations of CFST members

The steel tube should be symmetrical, fabricated from steel with a maximum yield strength of 350 MPa. The elastic modulus of steel *E* is given as 200,000 MPa. The selection of wall thickness (*t*) should guarantee that the plate slenderness (λ_e) is lower than the yield slenderness limit (λ_{ey}). For circular hollow sections CHS, the slenderness (λ_e) is given as $(\frac{d_o}{t})\,(\frac{f_y}{235})$, where d_o is the outside diameter of a CHS. The yield slenderness limit (λ_{ey}) for CHSs is equal to 82, Concrete with normal density and strength is mentioned in AS 5100 to fill the steel tubes. The maximum aggregate size is 20 mm. As far as the concrete modulus of elasticity (E_c) is concerned, a formula presented AS 5100 as follows:

$$E_c = \rho^{1.5} 0.043 \sqrt{f_c'} \tag{3.28}$$

where ρ is the concrete density taken as no more than 2400 kg/m3 for normal weight concrete. A steel contribution factor α_s is indicated in AS 5100 with an allowed range from 0.2 to 0.9, where α_s is defined as the ratio of the contribution of the steel section ($\phi\, A_s f_y$) to the total axial capacity (N_{us})

$$\alpha_s = \frac{\phi A_s f_y}{N_{us}} \tag{3.29}$$

ϕ = capacity factor (given in table 3.1).
A_s = cross –sectional area of steel section.
N_{us}= nominal section capacity for a concentrically loaded composite member determined in the following section.

Table 3.3 Capacity reduction factor (ϕ)

Design capacity for	Capacity reduction factor (ϕ)	
Composite compression members :-	Steel	Concrete
– section capacity	0.9	0.6
– relative stiffness	1.0	1.0

3.3.3 Members subjected to axial compression

3.3.3.1 Ultimate section capacity

To calculate the section capacity under axial compression, an assumption was used that the steel yields before the concrete reaches its ultimate stress state. Thus, the ultimate section capacity (N_{us}) for a circular CFST member, the increase in concrete strength benefits caused by confinement of the concrete core might be taken into account if the relative slenderness of the member (λr) is not exceed 0.5. N_{us} May be calculated as follows:

$$N_{us} = \phi A_s \eta_2 f_y + \phi_c A_c f_c' \left[1 + \frac{\eta_1 t f_y}{d_\circ f_c'}\right] \quad (3.30)$$

in which, η_1 and η_2 are coefficients used to reflect the confinement benefit that are dependent on the relative slenderness (λ_r) and. The coefficient of η_2 is used to account for the strength reduction of the steel because of the circumferential tensile strains in the steel because of confining the concrete, while the coefficient of η_1 is used for the concrete to reflect the strength increase from the tube confinement. The values of η1 and η2 when eccentricity of loading e = 0, i.e., η10, η20, might be evaluated by these eqs.:

$$\eta_{10} = 4.9\text{-}18.5\lambda + 17\,(\lambda_r)^2 \quad (\text{but} \geq 0) \quad (3.31)$$

$$\eta_{20} = 0.25(3+2\lambda_r) \quad (\text{but} \leq 1) \quad (3.32)$$

The calculation formulae for η_{10} and η_{20} of a concentrically loaded composite compression member are given in the table below:

Table 3.4 η10 and η20 values, when e = 0

λ_r	0.0	0.1	0.2	0.3	0.4	≥ 0.5
η_{10}	4.90	3.22	1.88	0.88	0.22	0.00
η_{20}	0.75	0.80	0.85	0.90	0.95	1.00

From the above, it can be seen that the procedure for calculating the ultimate section capacities is virtually the same as those suggested in EC4 because they have the same design philosophy, except different values have been used for the capacity factors.

3.3.3.2 Ultimate member capacity

Like many other codes, a slenderness reduction factor *αc* is presented in AS 5100 to replicate the basic relationship between strength and stability for an axially loaded column, as follows:

$$\alpha_c = \xi \left[1 - \sqrt{1 - \left[\frac{90}{\xi \lambda}\right]^2}\right] \quad (3.33)$$

where ξ and λ are coefficients related to the relative slenderness (λ_r). λ_r is defined as $\sqrt{\frac{N_s}{N_{cr}}}$, in which N_s is determined according to equation (3.3-c), with ϕ and ϕ_c taken as 1.0, and N_{cr} is the elastic critical load. The expression for N_{cr} is given as equation (3.34), and $(EI)e$ is the effective flexural stiffness determined in relation to equation (3.35), and l_e is the effective length of a composite compression member.

$$N_{cr}=\frac{\pi^2(EI)_e}{l_e^2}$$

$$(EI)_e=\emptyset E_s I_s+\emptyset_c E_c I_c \qquad (3.35)$$

In equation (3.35), I_s and $I_c$are the second moment of areas of the steel section and the uncracked concrete section, respectively; and ϕ and ϕ_c are also taken as 1.0. After the slenderness reduction factor α_c is determined from equation (3.33), the member capacity of a composite column can be stated as [18]:

$$N_{uc} = \alpha_c N_{us} \leq N_{us} \qquad 3.36)$$

3.4 AIJ (2001)

3.4.1 Design of CFST column system

3.4.1.1 CFST design recommendations

In 1997, CFST Recommendations were available by AIJ, basing upon current researching developments according to these topics:

- The braces and truss members as special forms of CFST structures. Beside, beams, columns, and connections as compression members.
- The formulations to calculate the capacity of deformation of CFST composite columns and frames.
- In fire conditions structural features.
- The steel and concrete manufacturing.

- CFST composite column behavior and frame analysis.
- The formulations of capacity used universally [47, 48].

3.4.1.2 Design formulas

General description about the design formulations for CFST members in the 2001 edition shown for the AIJ SRC Standard, is as bellow:-

- The allowable stress design method is the employed method for designing in the current standard. In seismic design, the ultimate lateral capacity of the allowable stress designed constructions should be larger than the desired value to bear acute seismic.

3.4.2 Limitations of CFST members

The steel yield stress ranges from 235MPa to 355MPa regarding some steel grades which containing HSS SM520.

- limiting values of (D/t) ratio for circular sections are as follows:

Circular $$\frac{D}{{}_{s}t} \leq 1.5\frac{23500}{F} \tag{3.37}$$

where

D: a circular tube diameter.

${}_{s}t$: steel tube wall thickness.

F: standard stress to evaluate the allowable steel stresses (MPa).

Those values are minimized to (1.5) times that for bare steels based upon the study of the restraining influence of the concrete infill on the local buckling.

- The allowable stress long-term between concrete and the insider of steel tube is (0.15) MPa for a circular sections. The bond stress is independent from the concrete strength. For the short-term stress condition, the values are 1.5 times for the long-term condition.
- For the long-term, the allowable compressive stress of concrete ${}_{c}f_{c}$ is equal to $F_c/3$ stress condition, and for the short-term one is$2F_c/3$, where F_c is the design standard compressive strength of concrete.
- The CFST member maximum effective length l_k is limited to:

for compression member $\frac{l_k}{D} \leq 50$ (3.38)

Where:

l_k: a member effective buckling length.

D: a cross section minimum depth.

3.4.3 Compressive strength

3.4.3.1 CFST column ultimate compressive strength

A CFST column ultimate compressive strength is calculated by eqs. (3.39) through (3.42).

$$\frac{l_k}{D} \leq 4; \qquad N_{cu1} = {}_cN_{cu} + (1+\eta)\, {}_sN_{cu} \tag{3.39}$$

$$= A_c\, {}_cr_u f_c + (1+\eta) A_s f_y \tag{3.40}$$

$$4 < \frac{l_k}{D} \leq 12; \qquad N_{cu2} = N_{cu1} - 0.125 \left\{ N_{cu1} - N_{cu3} \left(\frac{l_k}{D} = 12 \right) \right\} \left[\frac{l_k}{D} - 4 \right] \tag{3.41}$$

$$12 < \frac{l_k}{D}; \qquad N_{cu3} = {}_cN_{cr} + {}_sN_{cr} \tag{3.42}$$

where

l_k : a CFST column effective length.

D: a steel tube section diameter.

A_c: cross-sectional area of a concrete.

${}_cr_u = 0.85$: concrete strength reduction factor.

η = 0.27: confinement factor for a circular CFST column.

A_s: cross-sectional area of a steel tube.

N_{cu1}, N_{cu2}, N_{cu3}: CFST column ultimate capacities.

${}_{c}N_{cu}$: concrete ultimate capacity.

${}_{s}N_{cu}$: steel ultimate capacity.

${}_{c}N_{cr}$: concrete buckling stress.

${}_{s}N_{cr}$: steel buckling stress.

N_{cu1} in eq. (3.39) shows the capacity of a CFST cross-section, in which capacity of confined concrete is taken for circular CFST sections.

buckling stress ${}_{c}N_{cr}$ for a concrete column is evaluated as follows:

$$ {}_{c}N_{cr} = A_c \ {}_{c}\sigma_{cr} \tag{3.43}$$

For ${}_{c}\lambda_1 \leq$ 1.0;
$$ {}_{c}\sigma_{cr} = \frac{2}{1+\sqrt{{}_{c}\lambda_1^{\ 4}+1}} \ {}_{c}r_u f_c \tag{3.44}$$

For ${}_{c}\lambda_1 > 1.0$;
$$ {}_{c}\sigma_{cr} = 0.83exp\{C_c(1 - {}_{c}\lambda_1)\} r_u f_c \tag{3.45}$$

Where,

$$ {}_{c}\lambda_1 = \frac{{}_{c}\lambda}{\pi}\sqrt{{}_{c}\varepsilon_u} \tag{3.46}$$

$$ {}_{c}\varepsilon_u = 0.93({}_{c}r_u f_c)^{\frac{1}{4}} * 10^{-3} \tag{3.47}$$

$$ C_c = 0.568 + 0.00612 f_c \tag{3.48}$$

${}_{c}\lambda$: a concrete ratio of the slenderness column.

Buckling stress ${}_{s}N_{cr}$ for a steel tube is calculated as follows:-

for ${}_{s}\lambda_1 < 0.3$
$$ {}_{s}N_{cr} = A_s f_y \tag{3.49}$$

for $0.3 \leq {}_{s}\lambda_1 < 1.3$
$$ {}_{s}N_{cr} = \{1 - 0.545({}_{s}\lambda_1 - 0.3)\} A_s f_y \tag{3.50}$$

for ${}_s\lambda_1 \geq 1.3$ $${}_sN_{cr} = \frac{{}_sN_E}{1.3} \quad (3.51)$$

where,

$${}_s\lambda_1 = \frac{{}_s\lambda}{\pi}\sqrt{\frac{f_y}{E_s}} \quad (3.52)$$

$${}_sN_E = \frac{\pi^2 E_s I_s}{{l_k}^2} \quad (3.53)$$

where

${}_s\lambda$: a steel tube column ratio of slenderness.

E_s: a steel modulus of elasticity.

I_s: moment of inertia of a cross-section steel tube column [19].

Finally, these codes have some limitations on geometrical properties of the steel tube, and materials properties of steel and concrete and these restrictions are different according to the code. Table 3.5 shows the limitations of these design specifications. Elastic modulus of the concrete, E_c is calculated in each specification as presented in Table 3.6.

Table 3. 5 Limitations of design specifications

Parameter	AISC 360 – 2010	EC4	AS5100.6 - 2004	AIJ 2001
f_y(MPa)	$f_y \leq 525$	$235 \leq f_y \leq 460$	$f_y \leq 350$	$235 \leq f_y \leq 355$
f_c of NW (MPa)	$21 \leq f_c \leq 70$	$20 \leq f_c \leq 50$	$25 \leq f_c \leq 65$	$f_c \leq 60$
$\frac{D}{t}$	$\leq 0.31\left(\frac{E_s}{f_y}\right)$	$\leq 90\left(\frac{235}{f_y}\right)$	$\leq 82\left(\frac{250}{f_y}\right)$	$\leq 1.5\left(\frac{23500}{f_y}\right)$
Steel amount	≥1% of gross area	$0.2 \leq \delta \leq 0.9$	$0.2 \leq \delta \leq 0.9$	-
Slenderness	$\frac{KL}{r} \leq 200$	$\lambda \leq 2$	$\lambda \leq 0.5$	-

Table 3.6 Elastic modulus of the concrete, E_c

Specification	E_c (MPa)	Details
AISC 360-10	$0.043{w_c}^{1.5}\sqrt{f_c}$	w_c: concrete density (1500≤ $w_c \leq 2500\frac{kg}{m^3}$)
EC4	$22000\left(\frac{(f_c+8)}{10}\right)^{0.3}$	Normal strength concrete
AS5100.6	$0.0043\sqrt{f_c}{w_c}^{1.5}$	$f_c \leq 40Mpa$
	${w_c}^{1.5}(0.024\sqrt{f_c}+0.12)$	$f_c > 40Mpa,$ (2100≤ $w_c \leq 2500\frac{kg}{m^3}$
AIJ 2001	$3.35\times10^4\left(\frac{w_c}{2400}\right)\left(\frac{f_c}{60}\right)^{\frac{1}{3}}$	Normal strength concrete

Table (3.7) shows the variation of the compressive strength formulations for CFST columns which given within these design specifications.

Table 3.7 Compression strength formulations of the design specifications

Specification	Compression strength of CFST column
AISC 360-10	$P_{AISC} = P_{no}\left[0.658^{\frac{P_{no}}{P_e}}\right]$ for $\frac{P_{no}}{P_e} \leq 2.25$
	P_{AISC}=$0.877P_e$ for $\frac{P_{no}}{P_e} > 2.25$
EC4	$N_{EC4} = \eta_a A_s f_y + A_c f_c\left[1+\eta_c \frac{t f_y}{d f_c}\right]$
AS5100.6-2004	$N_{AS5100.6} = A_s \eta_a f_y + A_c f_c'\left[1+\frac{\eta_c t f_y}{d_\circ f_c'}\right]$
AIJ 2001	$N_{AIJ-1} = {}_cN_{cu} + (1+\eta)\ {}_sN_{cu}$ for $\frac{l_k}{D} \leq 4$ $N_{AIJ\text{-}2}=N_{AIJ\text{-}1}\text{-}0.125\left\{N_{AIJ-1}\text{-}N_{AIJ\text{-}3}\left(\frac{l_k}{D}=12\right)\right\}\left[\frac{l_k}{D}\text{-}4\right]$ for $4 < \frac{l_k}{D} \leq 12$ $N_{AIJ-3} = {}_cN_{cr} + {}_sN_{cr}$ for $\frac{l_k}{D} > 12$

In Table 3.7 for the AISC 360-16 calculations, P_{no}, the nominal compression strength, and P_e is the critical loading. The nominal compression strength of a column is evaluated on the basis of the type of section which might be compact, non-compact or slender. In EC4 and AS5100.6-2004 calculations, both codes have the identical basic design philosophy and the difference in their results is given by the different values of η_a and η_c due to the confinement effect when the relative slenderness of the column $(\bar{\lambda}) \leq 0.5$ where, η_a is the reduction factor of steel tube and η_c is the factor of concrete enhancement provided by the steel tube confinement. These parameters depend on $(\bar{\lambda})$ and the confinement effect is ignored beyond a relative slenderness value of 0.5.

For the AIJ 2001 l_k is the CFST columns effective length, η is the confinement factor for circular section only and equal to 0.27. This parameter is independent of the material properties and dimensions of the column and $r_u$which is equal to 0.85 is the reduction factor for concrete strength. ${}_cN_{cr}$ and ${}_sN_{cr}$are concrete buckling strength and steel buckling strength, respectively.

CHAPTER 4

PARAMETRIC STUDY

In order to investigate the suitability of AISC 360-10, EC4, AS5100.6-2004, and AIJ 2001 of predicting the capacity of a circular hollow steel tube filled with plain concrete (CFST) column under axial loading, where the data will be within and beyond the limitations of these codes and analyze the results to clarify the effect of such increase on the carrying capacity and behavior of CFST columns. Because of different assumptions, the results from each code are to some extent different. The variation of geometrical and material properties covered in this study and four different parameters are considered to check which parameter has the significant effect on the capacity of CFST column.

A total of 2401 virtual samples were included to simulate the behavior of the real column and the results have been assessed through a series of statistical comparisons, by using different structural parameters which varied to investigate their combined effect: concrete compressive strength (f_c) taken as (20, 35, 50, 65, 80, 95, and 110) MPa to cover normal, high, and ultra-high strength concrete, steel tube yield strength (f_y) taken as (235, 315, 395, 475, 555,635, and 715) MPa to cover mild and high tensile steel, while *D/t* ratio taken as (10, 20, 30, 40, 50, 60, and 70) to examine effect of varying this ratio on the confinement of concrete. *L/D* ratio taken as (2, 5, 8, 11, 14, 17, and 20) to cover short, medium and long columns.

Table 4.1 Levels of parameters considered for analysis

Parameter	Level 1	Level 2	Level 3	Level 4	Level 5	Level 6	Level 7
f_c	20	35	50	65	80	95	110
f_y	235	315	395	475	555	635	715
D/t	10	20	30	40	50	60	70
L/D	2	5	8	11	14	17	20

In definition of short and long CFST columns the AISC 360 – 10, EC4 and AS5100.6 – 2004 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. While in AIJ 2001 the columns are classified as short ($L/D \leq 4$), medium ($4 \leq L/D \leq 12$), and long ($L/D \geq 12$); in this study the classification of column done according to AIJ 2001 classification.

In this thesis work; seven different models are created by using different concrete compressive strengths to study effect of class of concrete on the CFST column axial capacity with stabilizing other parameters, other seven different models are created by using different steel yield strengths to study the influence of steel grade on the CFST column capacity also with stabilizing other parameters, other seven different models are included by using different thickness of steel tube to study the influence of increasing thickness of steel tube on the capacity of CFST column with stabilizing the other parameters, and other seven different models for the effect of L/D ratio on the capacity with using different lengths of column to study effect of slenderness on behavior of CFST column. This study also assumed keeping the diameter (D = 400 mm) and the effective length factor (K = 1) constants.

4.1 Results and Discussion

Numerical analysis on CFST column have been done by using computational tools. There are physical and geometric parameters which affect on the capacity of column, (1) concrete compressive strength f_c, (2) steel tube yield strength f_y, (3) D/t ratio, and (4) L/D ratio. The influence of these different parameters on CFST column capacity are illustrated as follows:

4.1.1 Effect of concrete compressive strength (f_c)

In order to stimulate the actual behavior of CFST column, different seven levels of concrete compressive strength is taking to show the impact of concrete class on the capacity of CFST column. The main effect of concrete compressive strength (f_c) is shown in Figure 4.1 for all codes, where the capacity increases with (f_c) increasing even when exceeding the limitations of all codes and using normal-strength concrete NSC, high- strength concrete HSC, and ultra-high strength concrete UHSC. It is plotted by considering the means of the responses at each level of the parameter. This increasing in axial capacity of the CFST columns is because of the confining effects with the increasing of infill concrete compressive strength (f_c). The maximum capacity of CFST column gives when (f_c) = 110 MPa.

Concrete strength determined the stiffness of composite column. Stiffness increases with increasing in concrete core strength though column fails because of the concrete crushing showing brittle behavior when filled with HSC, despite that, it is true when concrete compressive strength increased; the capacity increases regardless of the ratio D/t or L/D.

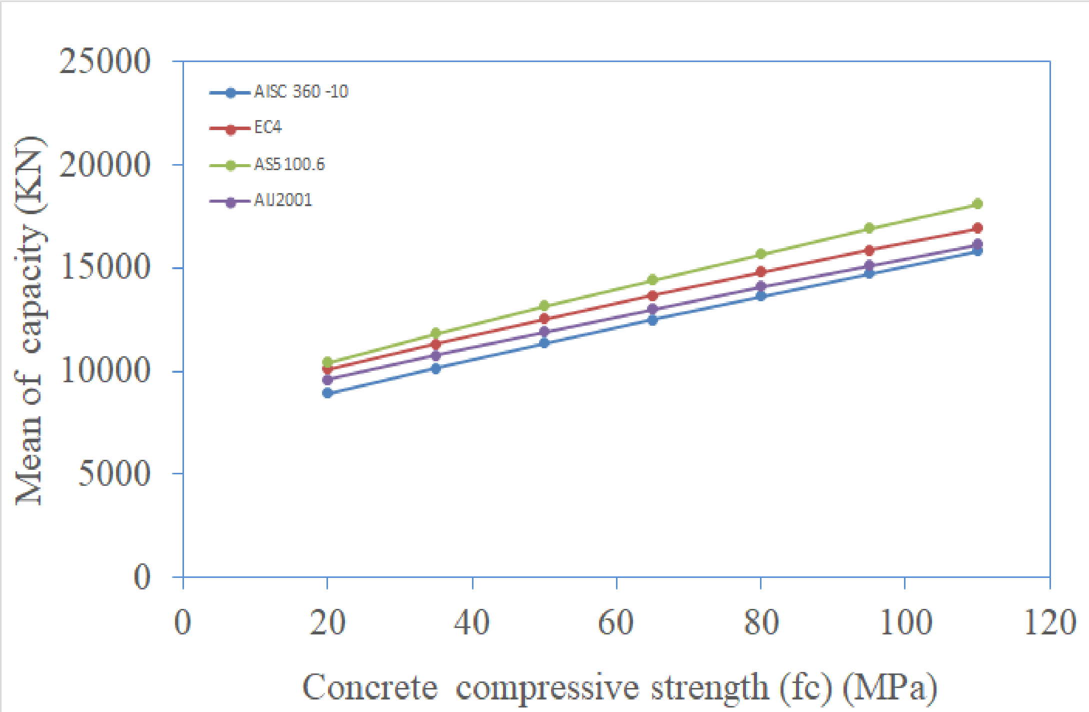


Figure 4.1 Main effects plot of (f_c) on capacity of CFST columns

The early flexural stiffness of CFST columns has a slight increasing due to the use of higher strength concrete. The results indicated that using high-strength concrete will not lead to significant increasing in flexural stiffness [7].

4.1.2 Effect of steel yield strength (f_y)

Seven different levels of steel yield strength were subjected to study the influence of variation of steel grade on capacity of the composite column. Steel yield strength is deciding the capacity of composite columns. Directly, the capacity of composite columns is linearly increased with increasing steel grade [7] as shown in figure 4.2, It was found that with the increase in steel yield strength (f_y) , the capacity also increases considerably even when exceeding the limitations of all codes and using mild steel and high tensile steel. The maximum capacity gives when (f_y) = 715 MPa. With respect to stiffness of CFST column, it is independent of the steel yield strength [7].

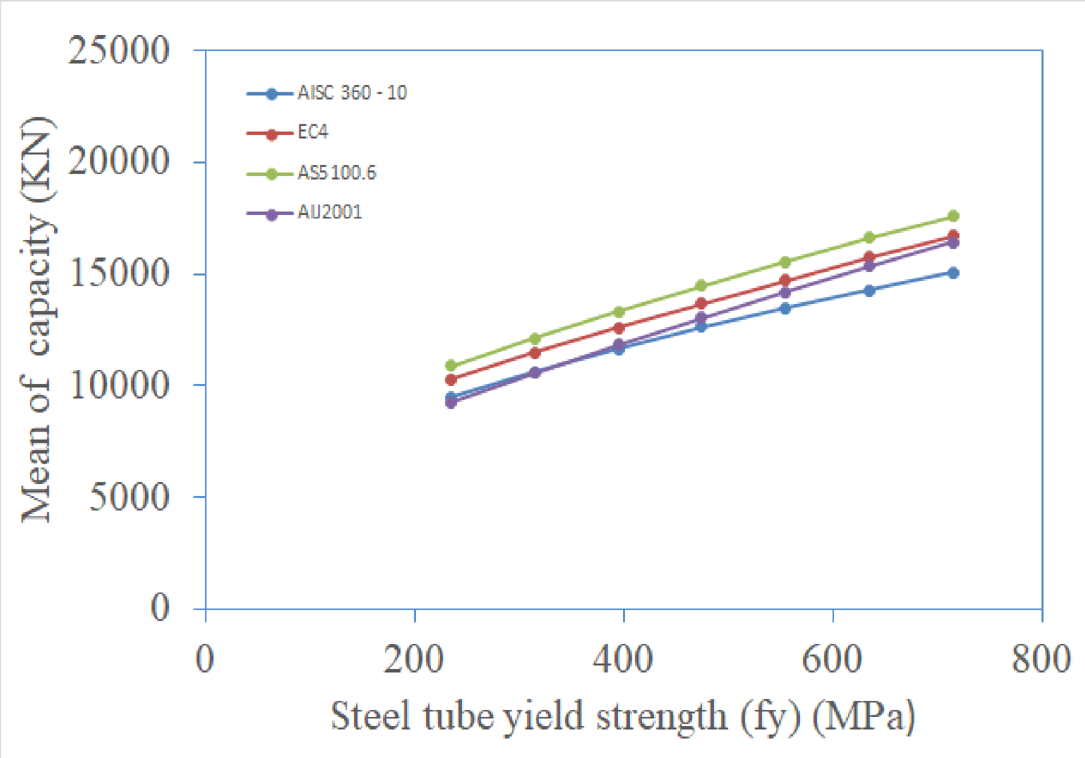


Figure 4.2 Main effects plot of (f_y) on capacity of CFST columns

4.1.3 Effect of diameter - to – thickness D/t ratio

To avoid the local buckling of cross – section of the CFST column, D/t ratio or (section slenderness ratio) must not increase the limits of each codes for local buckling, however, for AISC 360 – 10 the local buckling accounted according to the classification of cross-section as compact, noncompact and slender. While, for the other codes the local buckling occurs when this ratio passed the maximum value. Furthermore, this parameter affects the confinement of CFST column [37]. A comparison is conducted on seven different levels of D/t ratios ranged from 10 to 70 to examine the thickness variation effect on CFST column axial capacity since the diameter was assumed to be constant. For all codes the capacity of the CFST column is found to be decreased with the increase of D/t ratio and that is because the reduction in confinement provided by small thickness. The results showed that the capacity could be increased considerably by adopting a smaller D/t ratio for the section of CFST column in the design. Figure 4.3 is showing the main effect of D/t ratios on the CFST column capacity. However, the D/t = 10 gives the maximum effect.

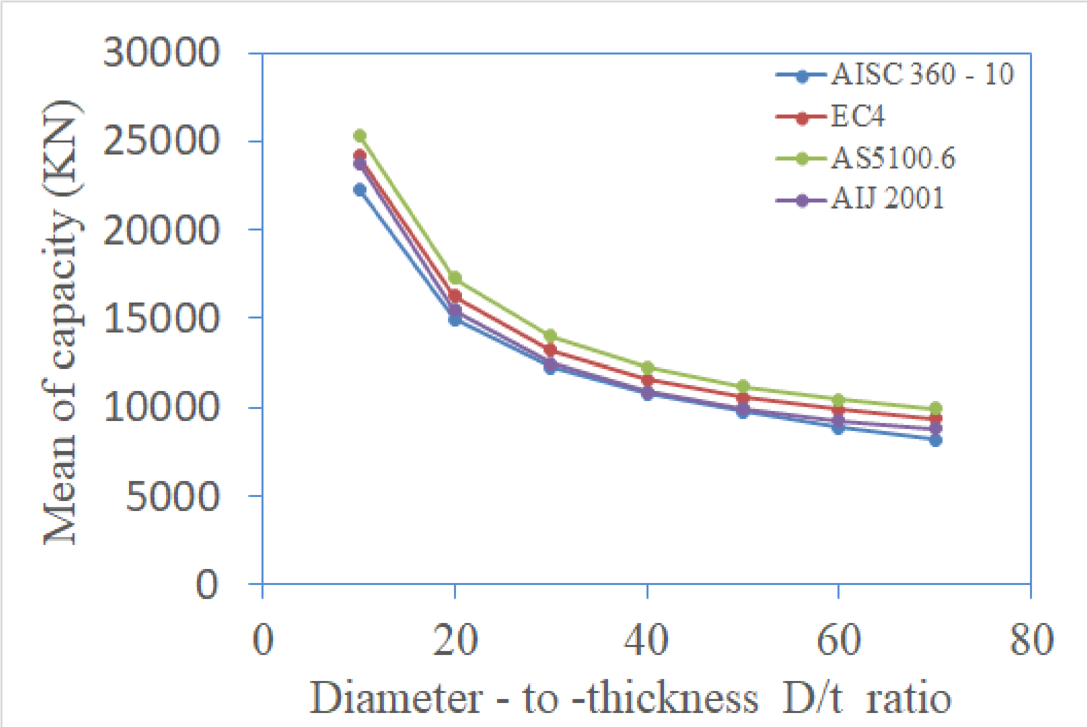


Figure 4.3 Main effects plot of D/t ratio on capacity of CFST columns

4.1.4 Effect of length – to – diameter L/D ratio

The slenderness of the column is represented by L/D ratio. The failure mode of the CFST column is characterized by steel yielding followed by concrete crushing. The experimental results show the effect of slenderness ratio on the capacity and the confinement effect of the CFST columns, where, both decrease when L/D ratio increased. Columns with larger L/D ratio will fail by global buckling [13, 37]. In this study L/D ratio is varying from 2 up to 20 to clarify the effect of the slenderness. Figure 4.4 shows the main effect of L/D ratio on the axial capacity of the CFST columns for all four codes. Where the capacity of short columns L/D = 2 gives the maximum value, greater than medium and long columns.

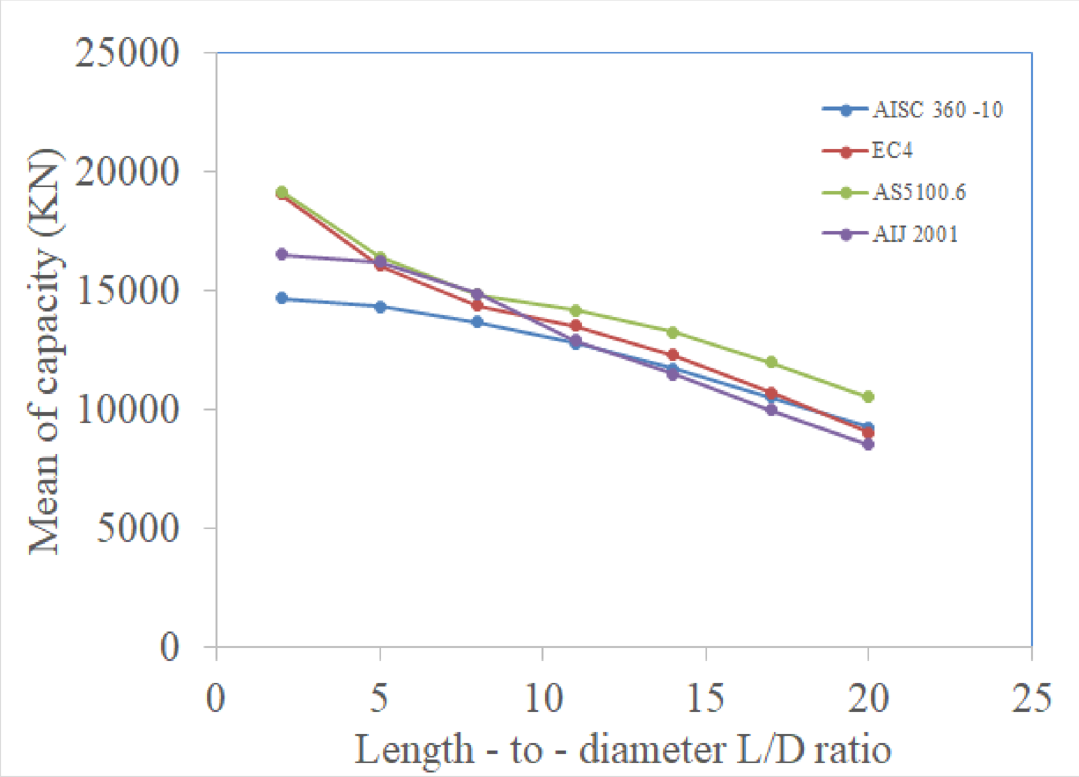


Figure 4.4 Main effects plot of $\mathrm{L/D}$ ratio on capacity of CFST columns

Further, with the increase of L/D ratio the capacity doesn't reach its maximum value and the column fails duo to the overall buckling.

4.1.5 The strength index (*SI*) and the confinement index (ξ)

Strength index (*SI*) and the confinement index (ξ) are useful measures for composite action and assessments of confinement in composite columns. Where, the confinement index (ξ), the strength index (*SI*) are defined in equations (2.1), (2.2) mentioned before. The following Figures 4.5 to 4.8 show the effect of physical and geometric parameters in this parametric study on the strength index. For AISC 360 – 10 (*SI*) ranges from 0.49 to 1.10, for EC4 ranges from 0.46 to 1.48, for AS5100.6-2004 ranges from 0.57 to 1.49, and for AIJ 2001 ranges from 0.48 to 1.26, these differences are due to the effect of the confinement calculations in each code. In this study, the Australian code gives the highest prediction, while the AISC 360 gives the most conservative one.

Figures 4.5 and 4.6 show that (*SI*) for normal strength concrete ($f_c < 50MPa$) greater than high strength concrete ($50MPa < f_c < 90MPa$) and ultra - high strength concrete ($f_c > 90MPa$) while for mild steel strength ($f_y \leq 460MPa$) *SI* is greater than high tensile steel ($f_y > 460MPa$) because the squash load of the CFST column depends on cross – section and the materials properties f_c and f_y.

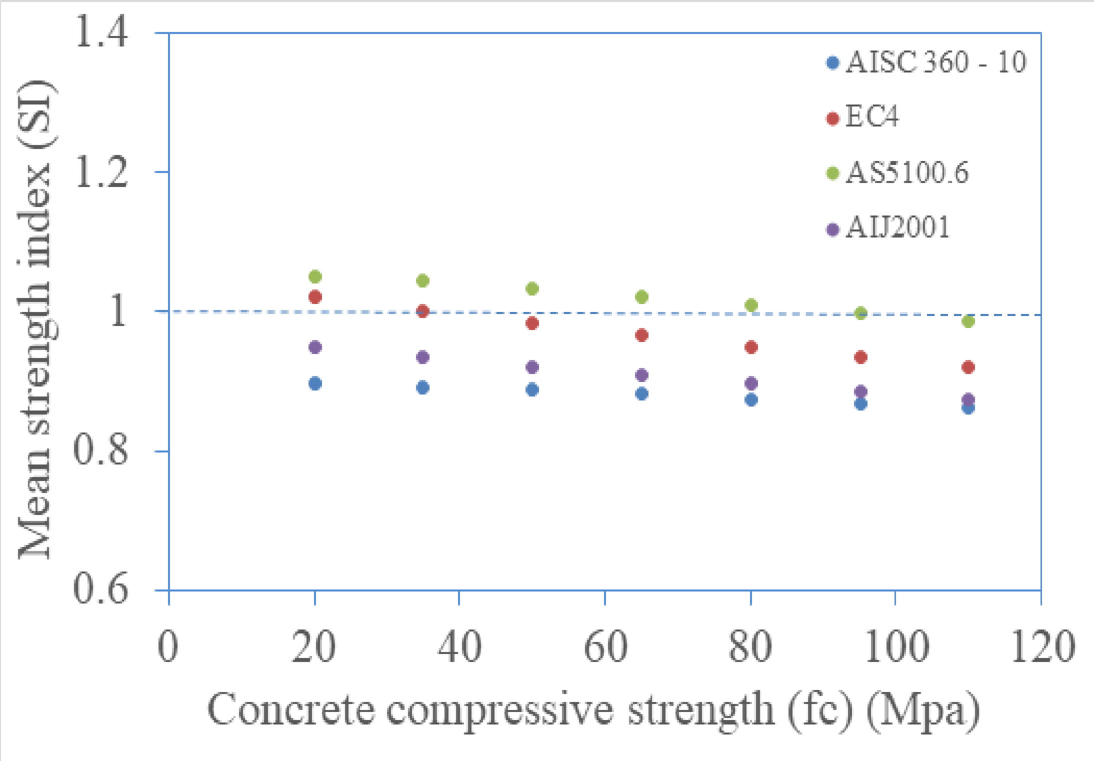


Figure 4.5 Effects of (f_c) on the strength index (*SI*)

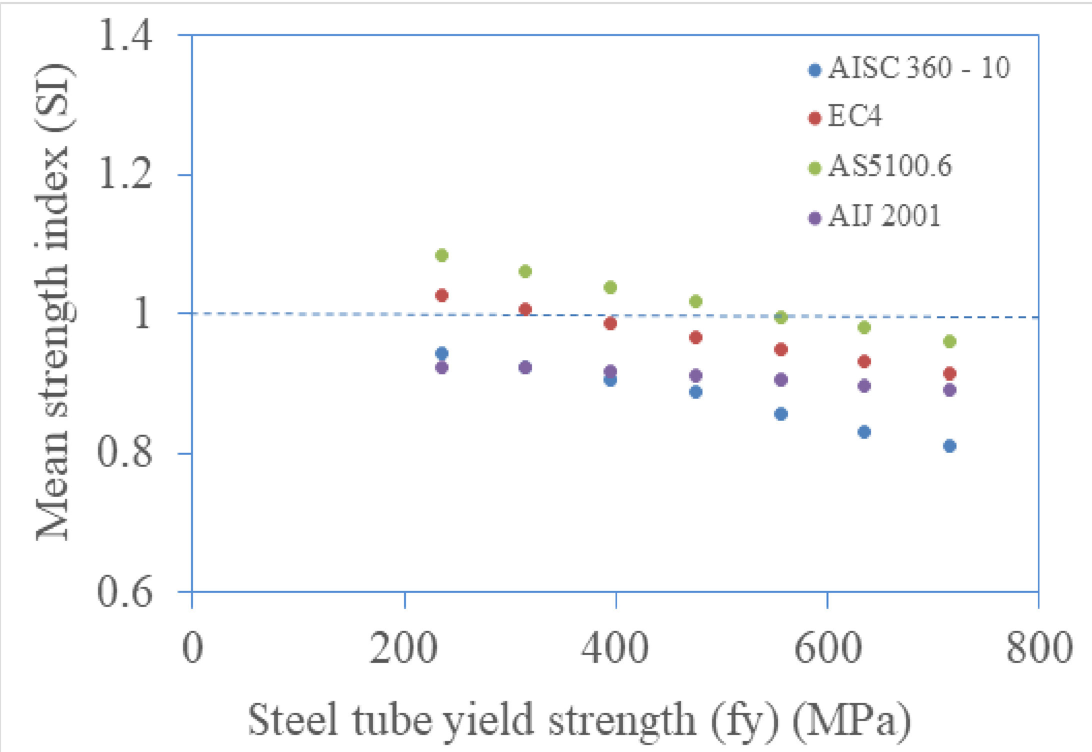


Figure 4.6 Effects of (f_y) on the strength index (*SI*)

Figure 4.7 shows the effect of D/t ratio on the strength index for the four codes. For the Australian, European, and American codes, the strength index begins to increase slightly with the increase of D/t ratio but after D/t =30 it decreased with the increase of D/t ratio while for the Japanese code, the strength index decreased slightly when the D/t ratio increases this means the thicker tube provides improvement in cross section of the column and confined the concrete core more than the thinner tube and the confinement factor for the CFST columns decreases with the increase of D/t ratio, i.e. the composite action of steel tube and concrete core becomes smaller. The decrease in D/t ratio with increased thickness hence produces greater section capacity. Where, the ductility of column decreases with the increase of concrete compressive strength for higher D/t ratios, while for smaller D/t ratios the opposite is true [36, 40].

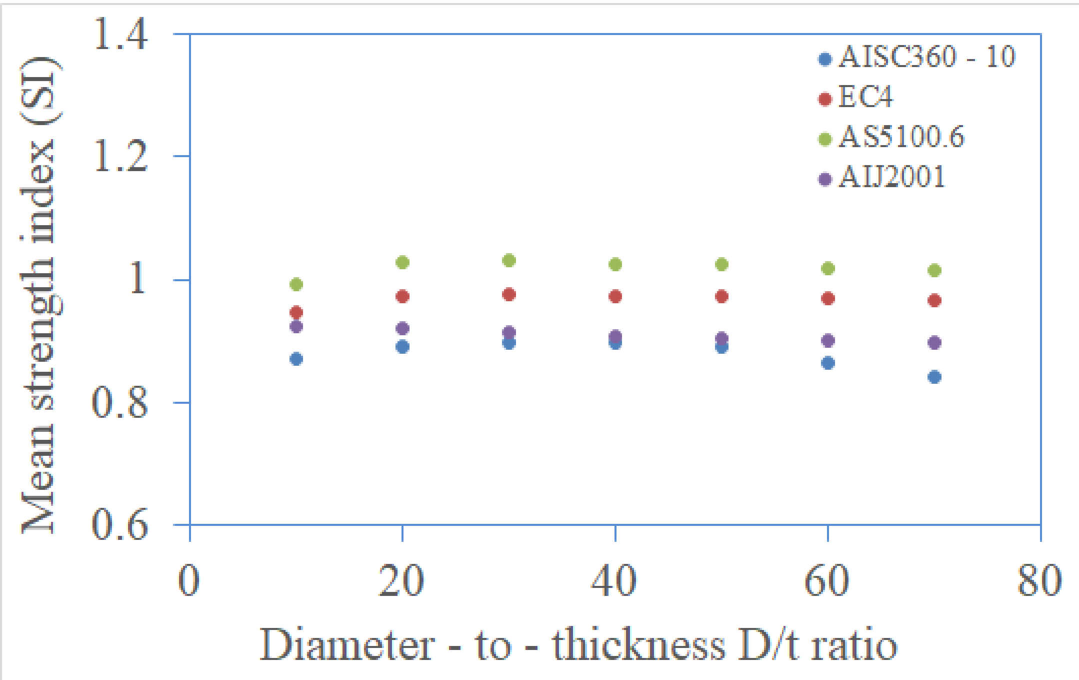


Figure 4.7 Effects of D/t ratio on the strength index (*SI*)

The L/D ratio has a significant effect on the capacity of the CFST column as shown in Figure 4.4. The short columns have a capacity greater than medium and long columns for all codes. This also clear in Figure 4.8 where the short columns have strength index more than unity for all codes, i.e. the section strength is acceptable and the performance of CFST column has been improved. For a column with small L/D ratio, the failure is recognized by material yielding while for high L/D ratio the failure mode is featured by global instability with small deformation before facing the confinement. Therefore, from the results obtained; it can be said that reducing the L/D ratio leads to increase the capacity of the CFST column [7, 37].

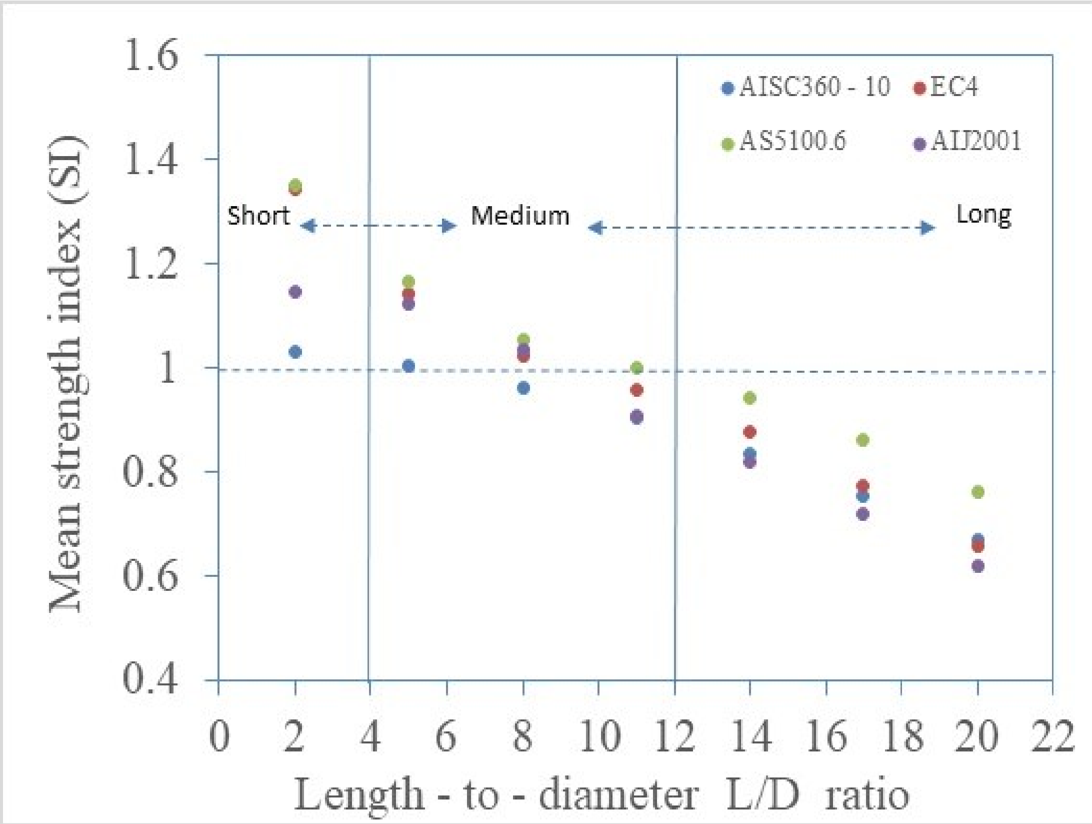


Figure 4.8 Effects of L/D ratio on the strength index (SI)

The confinement index (ξ) is a function of D/t ratio, in addition to the material properties f_c and f_y, for this parametric study the confinement index ranges from 0.128 to 20.109. Table 4.3 shows the ANOVA results to investigate the effect of parameters on the confinement index (ξ). Where the(f_c), D/t and (f_y) have a significant effect on (ξ), respectively, while the L/D ratio has constant effect on this index. Figure 4.9 shows the main effect of the parameters on the mean of the confinement index (ξ).

Table 4.2 ANOVA results of the confinement index (ξ)

Source	Contribution (%)
D/t	44.27
L/D	0.00
f_c(MPa)	21.90
f_y (MPa)	6.04
Error	27.79
Total	100

The confinement index (ξ) increases when (f_c) and D/t ratio decreases, and increases when (f_y) increases. It could be said that steel tube with high steel yield strength and wall with heavier thickness has large impact on concrete confinement [13]. The concrete contribution on the capacity of composite columns is a significant parameter, where the confinement of concrete by steel tube improves the capacity and prevents the inward local buckling of steel tube.

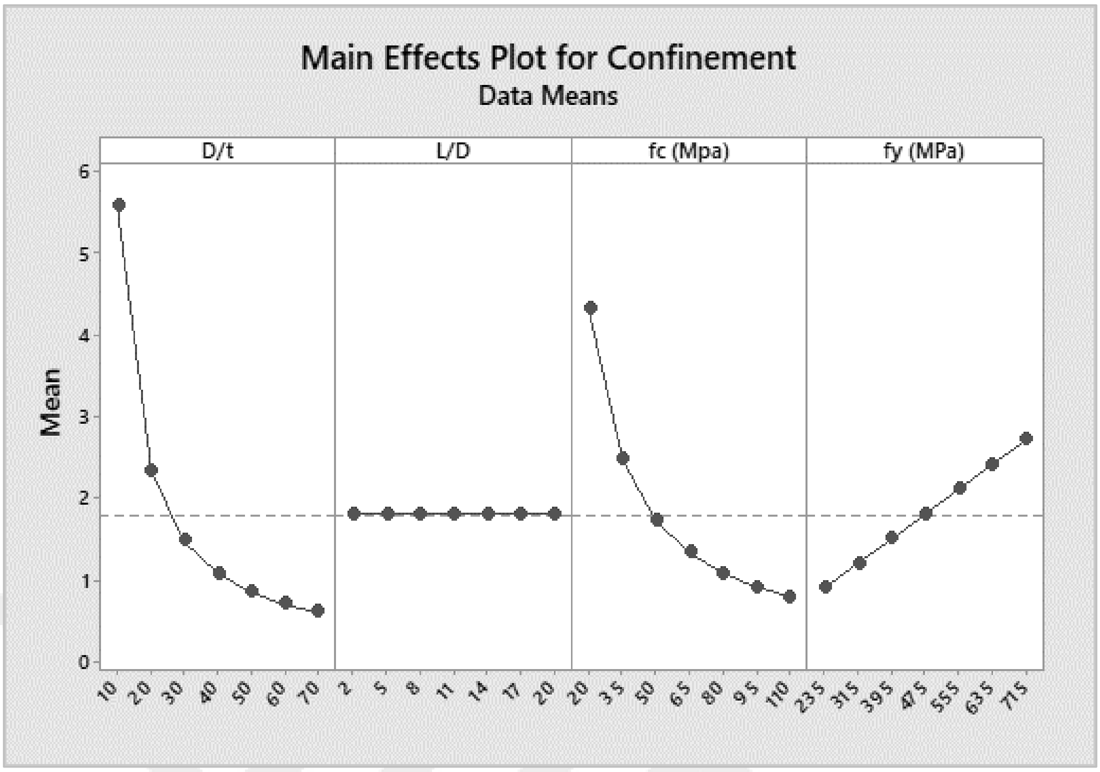


Figure 4.9 Main effect of parameters on mean of the confinement index (ξ)

However, the contribution of concrete CCR is defined as the ratio between the maximum numerical load (P_u) of the composite column and the empty hollow steel member, ($P_{u.hollow\ tube}$) [38] :

$$CCR = \frac{P_u}{P_{u.hollow\ tube}} \quad (4.3)$$

Where, the $P_{u.\ hollow\ tube}$ is obtained from Eurocode 3 [49].

$$P_{u.\ hollow\ tube} = A_S f_y \quad (4.4)$$

Figure 4.10 presents the relationship between the contribution of concrete *CCR* and the confinement index (ξ). Where, the CCR decreased when the confinement increases. However, the term $A_S f_y$ is the denominator in *CCR* calculations while is the numerator in ξ calculations.

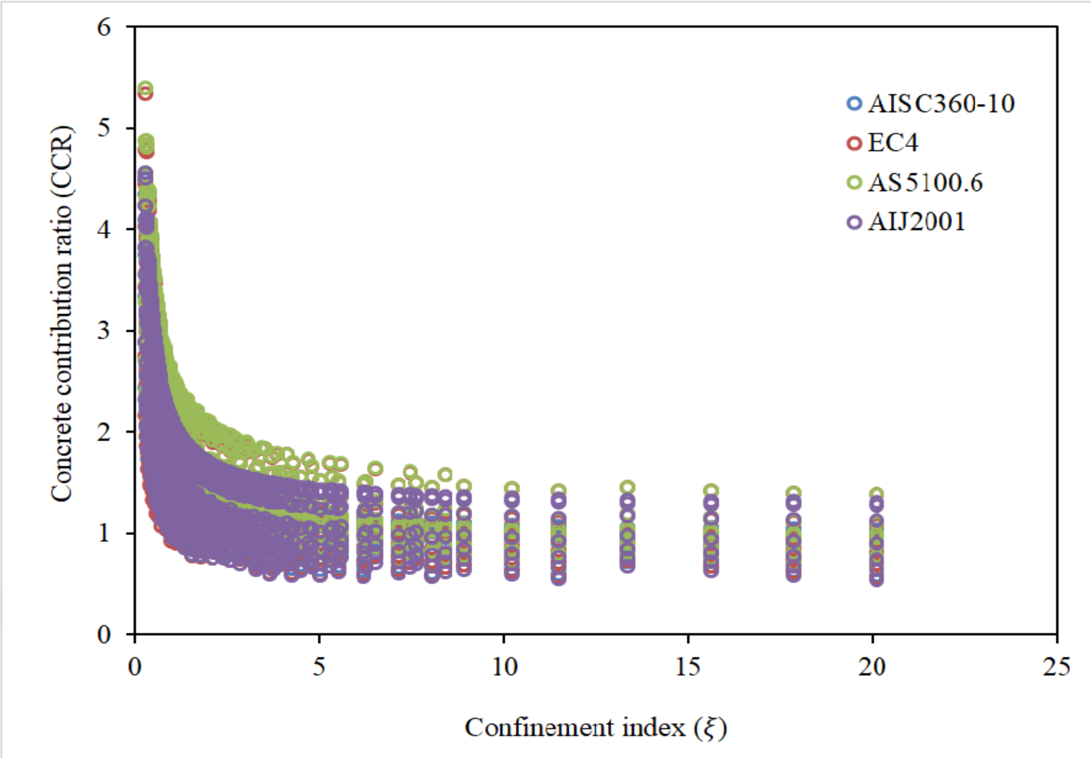


Figure 4.10 The confinement index (ξ) versus the contribution of concrete *CCR*

Table 4.3 shows the contribution of the four parameters by using Minitab software. Results of the Analysis Of Variance (ANOVA), show the parameters effect on the capacity of composite columns predicted by the four codes as shown below, D/*t* ratio has the most significant effect on the capacity of CFST columns for all codes with different percentages; this difference is due to the difference in theories of each code. The results show that the CFST column of (D/t = 10, L/D = 2, f_c = 110 MPa, and f_y= 715 MPa) gives the maximum capacity of 40,487.84 KN, 51,327.46 KN, 51,538.04 KN, and 48,618.49 KN for AISC 360 – 16, EC4, AS5100.6 – 2004, and AIJ 2001, respectively.

Table 4.3 ANOVA results for AISC 360, EC4, AS 5100.6, and AIJ 2001 codes

Source	Contribution (%)			
	AISC 360 -10	EC4	AS5100.6 - 2004	AIJ 2001
D/t	56.14	48.59	51.71	48.61
L/D	9.47	20.05	14.23	16.68
f_c (MPa)	14.26	10.74	13.18	9.68
f_y (MPa)	9.36	9.51	10.20	11.58
Error	10.78	11.12	10.69	13.44
Total	100	100	100	100

This difference is due to the effect of confinement where, the AISC 360 – 10 do not show the clear confinement while the EC4 and AS5100.6 – 2004 take the confinement in their calculations and AIJ2001 gives constant confinement of 0.27.While for (*D/t* =70, *L/D* =20, f_c =20 MPa, and f_y =235 MPa) the minimum capacity was as follows : 3,080.62 KN, 3,339.29 KN, 3,605.73 KN, 2,889.61 KN for AISC 360 – 16, EC4, AS5100.6 – 2004, and AIJ 2001, respectively.

Table 4.4 shows the summary of this parametric study, columns classified according to length (short, medium, and long), concrete compressive strength (normal strength concrete NSC, high strength concrete HSC, and ultra-high strength concrete UHSC), and steel tube strength (mild steel strength MSS and high steel strength HSS).

Table 4.4 Summary of the parametric study

	Mean of P_{AISC} (KN)	Mean of P_{EC4} (KN)	Mean of $P_{AS5100.6}$(KN)	Mean of $P_{AIJ2001}$(KN)
Total col.	12447.811	13595.548	14356.826	12948.4
Short col.	14695.726	19072.793	19203.223	16557.8
Med. col.	13618.781	14676.078	15155.864	14680
Long col.	10527.536	10689.269	11942.317	10013.6
NSC	10147.947	11311.785	11796.620	10752.2
HSC	13062.993	14228.098	15050.053	13547.2
UHSC	15282.423	16388.641	17503.899	15643.7
MSS	10569.668	11444.898	12099.119	10545.7
HSS	13856.417	15208.535	16050.102	14750.4

Short columns give the higher capacity in comparing with the medium and long columns for all codes, this difference between codes is due to the confinement effect and the variation between the limitations of codes for materials and geometrical properties. Also, for normal strength concrete (NSC), high strength concrete (HSC), and ultra-high strength concrete (UHSC); the variation is clear for prediction of capacity between codes and between NSC, HSC, and UHSC.

Table 4.4 Continued...

	Mean of SI_{AISC}	**Mean of SI_{EC4}**	**Mean of $SI_{AS5100.6}$**	**Mean of $SI_{AIJ2001}$**
Total col.	0.880	0.968	1.020	0.910
Short col.	1.030	1.342	1.352	1.146
Med. col.	0.959	1.042	1.074	1.023
Long col.	0.752	0.771	0.856	0.719
NSC	0.892	1.002	1.042	0.936
HSC	0.878	0.958	1.014	0.903
UHSC	0.865	0.928	0.992	0.880
MSS	0.925	1.006	1.062	0.921
HSS	0.847	0.940	0.989	0.902

Generally, the confinement is more effective when the infill concrete is NSC due to its higher deformation capacity compared to HSC [37], this result is obvious in mean of SI_{EC4}, $SI_{AS5100.6}$, and $SI_{AIJ2001}$ between NSC, HSC, and UHSC due to confinement effect while it is slight difference in SI_{AISC}.

Table 4.4 Continued...

	Mean of (ξ)
Total col.	1.796
NSC	2.827
HSC	1.20
UHSC	0.844
MSS	1.191
HSS	2.249

The same conclusion observed when comparisons between the mean of the confinement index (ξ) and between NSC, HSC and UHSC. While there is no effect of length on the mean of (ξ) because the equation 2.1 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 (HSS) ($f_y > 460$ MPa).

CHAPTER 5

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

The present study is an attempt to compare between the prediction of AISC 360 – 10, EC4, AS5100.6 – 2004, and AIJ 2001 of composite columns, with the rapid growth of research and application of concrete-filled steel tube in the world, the circular concrete-filled steel tube column under axial loading is considered as a parametric study. On the basis of this parametric study, the following conclusions can be drawn:

- This parametric study suggested to cover the practical cases in this field.
- These specimens were within and behind the limitations of these codes.
- All codes depend on different functions for the assessment of the compression capacity of CFST column. Therefore, there are differences in their results.
- The EC4 and AS5100.6 take the confinement effect by the terms η_a and $(1 + \eta_C \; t/D \, f_y/f_c)$; η_a: represents the steel strength reduction factor and η_C: represents the concrete strength increase factor. while for AISC 360 – 10 and AIJ 2001, the confinement is constant.
- Even though the EC4 and AS5100.6 have the same formulation to compute the compression capacity, the results show that the capacity predicted by AS5100.6 is higher than EC4 and this due to the difference in the effect of confinement determined by the two parameters η_a, and η_C.
- The parameters of geometrical and material properties have effects on the assessments of codes with different percentages.
- The analysis of variance showed that (f_c) and D/t ratio have the most significant effect.

- The column of (D/t = 10, L/D = 2, f_c= 110 MPa, and f_y = 715 MPa) gives the maximum capacity for all codes. While the CFST column of (D/t = 70, L/D = 20, f_c = 20 MPa, and f_y = 235 MPa) gives the minimum capacity of all codes.
- The capacity increased when (f_c) and (f_y) increase, while decreased when D/t ratio and L/D ratio increase.
- Strength index (*SI*) is a useful parameter to measure composite action in CFST column. (*SI*) for NSC and short column is greater than HSC, UHSC, and long column, while the HSS gives confinement index more than MSS.
- The concrete contribution on the CFST column capacity is a significant parameter, while the confinement index has the ability to assess the confinement of CFST columns. The confinement index (ξ) increases when the (f_c) and D/t ratio decreases, and increases when (f_y) increases.

5.2 Recommendations

- It is necessary to increase this kind of parametric studies to expand searching field and fill the gap of information and application to promote the use of composite construction.
- Including other international and local codes in the investigation to achieve the design requirements in the related countries.

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