ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

A DATABASE ON CONCRETE FILLED STEEL TUBE COLUMNS

A Thesis in STRUCTURAL ENGINEERING

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November 4, 2016

Addis Ababa

A Thesis

Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science

The undersigned have examined the thesis entitled '**A DATABASE ON CONCRETE FILLED STEEL TUBE COLUMNS** presented by **DANIEL BELETE**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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UNDERTAKING

I certify that research work titled "A DATABASE ON CONCRETE FILLED STEEL TUBE (CFT) COLUMNS" is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

Signature of Student

DANIEL BELETE

ACKNOWLEDGEMENTS

Firstly, I would like to extend my appreciation to my graduate advisor Dr. Girma Zerayohannes. I am very thankful for the unique opportunity and the privilege of working on this thesis that has become one of the best experiences in my career and my personal life.

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LIST OF SYMBOLS

A_c = area of concrete, mm^2

A_s = area of the steel tube, mm^2

β_1, β_2 = moment amplification factors

B/t = ratio of longer dimension-to-thickness for rectangular section.

D = diameter of the steel tube

D/t = diameter-to-thickness ratio for circular steel tubes

e = eccentricity of applied load, mm.

e/D = eccentricity-to-diameter ratio for circular CFT

E_c = elastic modulus of concrete, MPa.

EI_{eff} = effective bending stiffness of composite section, N-mm^2

E_s = elastic modulus of steel shape or tube, MPa.

f_{ck} = cylindrical compressive strength of concrete, MPa.

f_y = yield stress of steel shape or tube, MPa.

h_n = distance from centroidal axis to neutral axis, mm

I_c = moment of inertia of concrete, mm^4.

I_s = moment of inertia of steel shape or tube, mm^4

k = effective length factor

K_e = a correction factor that should be taken as 0.6

M_c = available flexural strength, kNm.

M_{max} = maximum internal moment, kNm.

M_n = nominal moment resistance, kNm.

$M_{pl,Rd}$ = the design plastic moment resistance of the section, kNm.

M_u = ultimate moment, kNm.

$N_{pm,Rd}$ = axial force resistance of concrete portion of cross-section, kN.

N_{cr} = critical buckling load, kN

N_{exp} = experiment ultimate axial load, kN.

$N_{pl,Rd}$ = the design plastic axial resistance of composite cross-section, kN.

t = thickness of steel tube, mm.

α = slenderness parameter, imperfection factor

η_1 = reduction factor for steel yield stress due to confinement

η_2 = factor for increasing concrete compressive strength due to confinement

η_{10} , η_{20} = intermediate values for confinement calculations

λ = slenderness parameter.

μ = percentage of plastic moment available for resisting applied loads.

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ABSTRACT

A significant amount of research has been conducted worldwide on the behavior of concrete filled steel tube columns. this thesis synthesize available experimental data on concrete-filled steel tube (CFT) specimens and summarizes the data from 1100 tests on concrete-filled steel tube columns (294 CCFT and 289 RCFT columns without moment, 286 CCFT and 231 RCFT columns with moment) and compares their failure load with the prediction of Eurocode 4 and AISC-2010. Information on material and geometric properties on each specimen are summarized. The comparison between Eurocode 4 and AISC-2010 show that Eurocode 4 can predict very well for columns subjected to axial load with no moment and for circular CFT beam-columns while the AISC-2010 gives good prediction for rectangular beam columns (columns with moment). Additionally, comparison made based on material strength, both Eurocode 4 and AISC-2010 predict on a safe side for steel tubes filled with high strength concrete.

CHAPTER 1

INTRODUCTION

1.1. BACKGROUND

The use of concrete filled steel tube (CFT) columns in many structural systems is increasing worldwide due to the intrinsic synergy between concrete and structural steel when designed and detailed properly together.

The CFT structural member has a number of distinct advantages over an equivalent steel, reinforced concrete, or steel-reinforced concrete member.

The orientation of the steel and concrete in the cross section optimizes the strength and stiffness of the section. The steel lies at the outer perimeter where it performs most effectively in tension and in resisting bending moment. Also, the stiffness of the CFT is greatly enhanced because the steel, which has a much greater modulus of elasticity than the concrete, is situated farthest from the centroid, where it makes the greatest contribution to the moment of inertia. The concrete withstands the compressive loading in typical applications, and it delays and often prevents local buckling of the steel, particularly in rectangular CFTs. Additionally, it has been shown that the steel tube confines the concrete core, which increases the compressive strength for circular CFTs, and the ductility for rectangular CFTs. Therefore, it is most advantageous to use CFTs for the columns subjected to the large compressive loading. In contrast to reinforced concrete columns with transverse reinforcement, the steel tube also prevents spalling of the concrete and minimizes congestion of reinforcement in the connection region, particularly for seismic design.

A number of additional economic benefits stem from the use of CFTs. The tube serves as formwork in construction, which decreases labor and material costs. In moderate- to high-rise construction, the building can ascend more quickly than a comparable reinforced concrete structure since the steelwork can precede the concrete by several stories.

A CFT member contains two materials with different stress strain curves and distinctly different behavior. The interaction of the two materials poses a difficult problem in the determination of combined properties such as moment of inertia and modulus of elasticity. The failure mechanism depends largely on the shape, length, diameter, steel tube thickness, and concrete and steel strengths. Parameters such as bond, concrete confinement, residual stresses, creep, shrinkage, and type of loading also have an effect on the CFT's behavior. Axially loaded columns and CFT beam-columns have been studied worldwide and to some extent many of the aforementioned issues have been reconciled for these types of members. The scope of this thesis covering member behavior is limited to CFTs that are filled completely with concrete, and CFTs that make no use of reinforcing bars.

1.2. SCOPES & OBJECTIVES

- To compare failure load prediction of EC-4 and AISC-2010 for CFT columns.
- To compare the test result of CFT columns with material and geometric properties which are outside EC-4's limitation with the resistance predicted by EC-4 and AISC-2010.
- To summarize key results of published studies on experimental setup and properties.
- To encompass the design procedures and limitations of EC-4 and AISC-2010 specification.

CHAPTER 2

LITERATURE REVIEW

Two versions of the composite column design specifications for design of concrete filled steel tube (CFT) columns are discussed and compared in this work. The AISC-2010 (the American institute of steel construction) Specifications and the Eurocode (ENV 1994).

2.1 CFT COLUMN DESIGN ACCORDING TO Eurocode 4

Eurocode 4 applies to columns with steel grades S235 to S460 and normal weight concrete of strength classes C20/25 to C50/60. The steel contribution ratio δ should fulfill the condition of

0.2 ≤δ ≤0.9. $$\delta = \frac{A_a f_{yd}}{N_{pl,Rd}} \qquad (2.1)$$

The influence of local buckling of the steel section on the resistance shall be considered with the following condition.

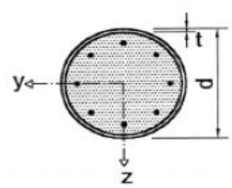
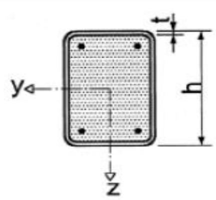
Cross-section		max (d/t), max (h/t) and max (b/t)
Circular hollow steel sections		$\max\ (d/t) = 90\frac{235}{f_y}$
Rectangular hollow steel sections		$\max\ (h/t) = 52\sqrt{\frac{235}{f_y}}$

Table 2-1: Maximum values (d/t) and (h/t) with f_y in N/mm^2

Second-order effects shall be considered in any direction in which failure might occur, if they affect the structural stability significantly. For simplification, instead of the effect of residual stresses and geometrical imperfections, equivalent initial bow imperfections (member imperfections) is used. The simplified method of EC-4 is limited to members of doubly symmetrical and uniform cross-section over the member length with rolled, cold-formed or welded steel sections.

The relative slenderness should fulfill $\lambda \leq 2.0$.

The ratio of the depth to the width of the composite cross-section should be within the limits 0.2 and 5.0 for rectangular sections.

a) Resistance of sections and members in axial compression

The plastic resistance to compression $N_{pl\ Rd}$ of a composite cross-section should be calculated by adding the plastic resistances of its components:

$$N_{pl,Rd} = A_a f_{yd} + A_c f_{cd} \tag{2.2}$$

For members in axial compression, the design value of the normal force N_{Ed} should satisfy:

$$\frac{N_{Ed}}{\chi N_{pl,Rd}} \leq 1.0 \tag{2.3}$$

Where: $N_{pl,Rd}$ is the plastic resistance of the composite section.

χ is the reduction factor for the relevant buckling mode.

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda}^2}} \quad \text{but} \quad \chi \leq 1$$

$\Phi = 0.5(1 + \alpha\ (\lambda - 0.2) + \lambda^2)$

For concrete filled tubes of circular cross-section, account is taken of increase in strength of concrete caused by confinement provided that the relative slenderness $\overline{\lambda}$ does not exceed 0.5 and $e/d < 0.1$, where e is the eccentricity of loading given by M_{Ed}/ N_{Ed} and d is the external diameter of the column. The plastic resistance to compression is calculated from the following expression:

$$N_{pl,Rd} = \eta_a A_a f_{yd} + A_c f_{cd}\left(1 + \eta_c \frac{t}{d} \frac{f_y}{f_{ck}}\right) \tag{2.4}$$

Where: t is the wall thickness of the steel tube, for members with e = 0 the values $\eta_a = \eta_{ao}$ and $\eta_c = \eta_{co}$ are given by the following expressions:

$\eta_{ao} = 0.25\ (3 + 2\lambda)$ (but ≤ 1.0)

$\eta_{co} = 4.9 - 18.5\lambda^2 + 17\lambda^2$ (but ≥ 0)

For members in combined compression and bending with $0 < e/d \leq 0.1$

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

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

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

For high strength concrete with f_{ck} > 50 MPa, the effective compressive strength of concrete in accordance with EC2 (EN 1992-1-1, 2004) is determined by multiplying the characteristic strength by a reduction factor η as given below.

$\eta = 1.0 - (f_{ck} - 50)/200$

The relative slenderness λ for the plane of bending being considered is given by:

$$\lambda = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}} \tag{2.5}$$

N_{cr} is the elastic critical normal force for the relevant buckling mode, calculated with the effective flexural stiffness $(EI)_{eff}$

$$(EI)_{eff} = E_a I_a + K_e E_{cm} I_c \tag{2.6}$$

Where: K_e is a correction factor that should be taken as 0.6.

I_a, and I_c are the second moments of area of the structural steel section, the un-cracked concrete section for the bending plane being considered.

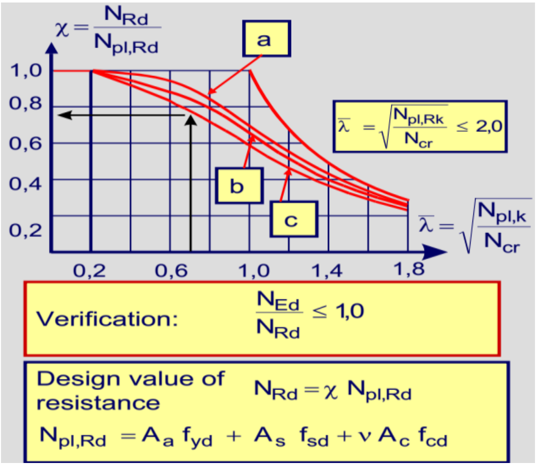


Figure 2.1: buckling curve

b) Resistance of sections and members in combined compression and uniaxial bending

For second-order linear elastic analysis. $(EI)_{eff,II}$ should be determined as:

$$(EI)_{eff,II} = K_o (E_a I_a + K_{e,II} E_{cm} I_c) \tag{2.7}$$

Where: $K_{e,II}$ is a correction factor which is taken as 0.5.

K_o is a calibration factor which should be taken as 0.9.

The elastic critical load is determined with the flexural stiffness $(EI)_{eff,II}$.

Within the column length, second-order effects may be allowed for by multiplying the greatest first-order design bending moment MEd by a factor k given by:

$$k = \frac{\beta}{1 - N_{\mathrm{Ed}} / N_{\mathrm{cr,eff}}} \geq 1.0 \qquad (2.8)$$

Where: $N_{cr,eff}$ is the critical normal force for the relevant axis and corresponding to the effective flexural stiffness with the effective length taken as the column length.

β is an equivalent moment factor.

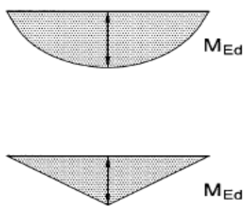
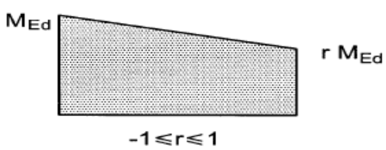
Moment distribution	Moment factors β	Comment
	First-order bending moments from member imperfection or lateral load: $\beta = 1.0$	M_{Ed} is the maximum bending moment within the column length ignoring second-order effects
	End moments: $\beta = 0.66 + 0{,}44r$ but $\beta \geq 0.44$	M_{Ed} and $r\ M_{\mathrm{Ed}}$ are the end moments from first-order or second-order global analysis

Table 2.2: factors β for the determination of moments to second order theory

The resistance of a cross-section to combined compression and moments may be calculated based on interaction curve assuming rectangular stress blocks. The plastic stress distributions of a CFST cross section for the points A, B, C, D and E are also shown in Figure 2.2.

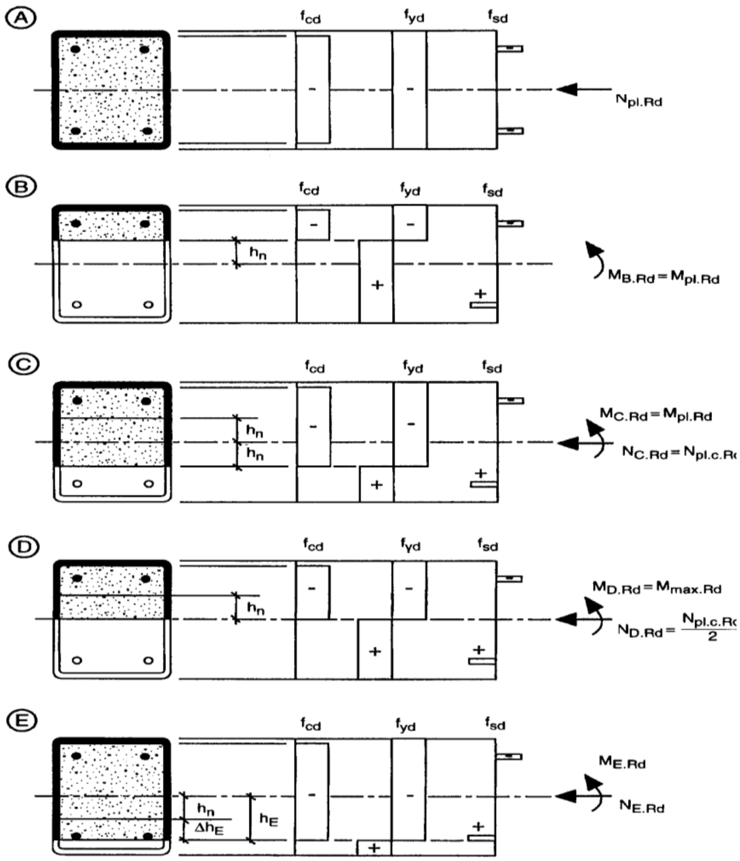
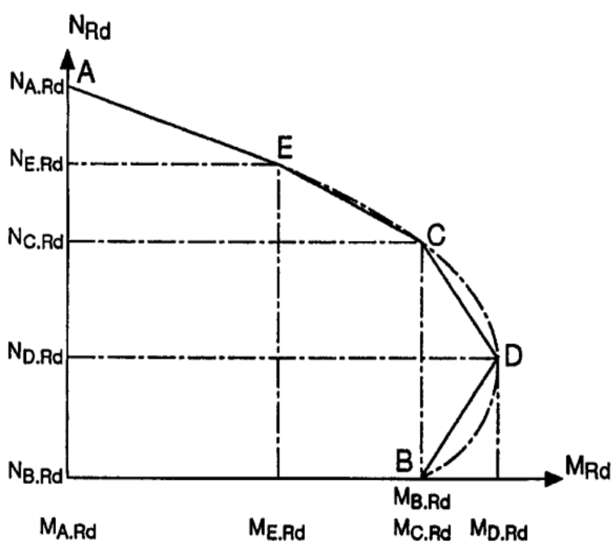


Figure 2.2: plastic stress distribution from point A to E



- Interaction curve approached by a polygonal connection of the points A to E

Figure 2.3: interaction curve by a polygonal path from point A to E

The following expression based on the interaction curve should be satisfied for members subjected to combined compression and uniaxial bending:

$$\frac{M_{Ed}}{M_{pl,N,Rd}} = \frac{M_{Ed}}{\mu M_{pl,Rd}} \leq \alpha_m \qquad (2.9)$$

M_{Ed} is the greatest of the end moments and the maximum bending moment within the column length, calculated including imperfections and second order effects if necessary. $M_{pl,N,Rd}$ is the plastic bending resistance taking into account the normal force N_{Ed}, given by μ_d $M_{pl,Rd}$.

$M_{pl,Rd}$ is the plastic bending resistance, for steel grades between S235 and S355 inclusive, the coefficient αm should be taken as 0.9 and for steel grades S420 and S460 as 0.8.

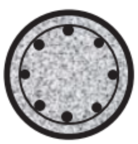
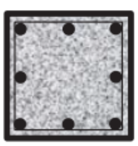
Cross section	Limits	Axis of buckling	Buckling curve	Member imperfection
	$\rho_s \leq 3\%$	any	a	$L/300$
	$\rho_s \leq 3\%$	any	b	$L/200$

Table 2.3 Buckling curves and member imperfections for composite columns

Buckling curve	a_0	a	b	c	d
Imperfection factor	0.13	0.21	0.34	0.49	0.76

Table 2.4 imperfection factors for buckling curves

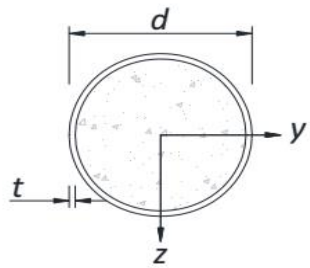
Point	Defining equations
	
A	$N_{pl,Rd} = \eta_a A_a f_{yd} + A_c f_{cd}\left(1 + \eta_c \frac{t}{d}\frac{f_y}{f_{ck}}\right)$
B	$h_n = \frac{A_c f_{cd}}{2df_{cd} + 4t(2f_{yd} - f_{cd})}$
	$W_{pc} = \frac{(d-2t)^3}{6}$
	$W_{pc,n} = (d-2t)\,h_n^2$
	$W_{pa} = \frac{d^3}{6} - W_{pc}$
	$W_{pa,n} = dh_n^2 - W_{pc,n}$
	$M_{pl,Rd} = (W_{pa} - W_{pa,n})\,f_{yd} + 0.5\,(W_{pc} - W_{pc,n})\,f_{cd}$
C	$N_{pm,Rd} = A_c f_{cd}\left(1 + \eta_c \frac{t}{d}\frac{f_y}{f_{ck}}\right)$
D	$M_{pl,Rd} = W_{pa} f_{yd} + 0.5 W_{pc} f_{cd}$

Table 2.5 Section analysis of circular CFST column

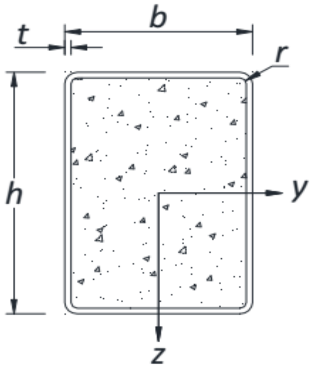
Point	Defining equations
	
A	$N_{pl,Rd} = A_a f_{yd} + A_c f_{cd}$
B	$h_n = \frac{A_c f_{cd}}{2bf_{cd}+4t\left(2f_{yd}-f_{cd}\right)}$
	$W_{pc} = \frac{(b-2t)(h-2t)^2}{4} - \frac{2}{3}r^3 - r^2\left(4-\pi\right)\left(\frac{h}{2}-t-r\right)$
	$W_{pc,n} = \left(b-2t\right)h_n^2$
	$W_{pa} = \frac{bh^2}{4} - \frac{2}{3}\left(r+t\right)^3 - \left(r+t\right)^2\left(4-\pi\right)\left(\frac{h}{2}-t-r\right) - W_{pc}$
	$W_{pa,n} = bh_n^2 - W_{pc,n}$
	$M_{pl,Rd} = \left(W_{pa} - W_{pa,n}\right)f_{yd} + 0.5\left(W_{pc} - W_{pc,n}\right)f_{cd}$
C	$N_{pm,Rd} = A_c f_{cd}$
D	$M_{pl,Rd} = W_{pa} f_{yd} + 0.5 W_{pc} f_{cd}$

Table 2.6 Section analysis of rectangular CFST column

2.2 CFT COLUMN DESIGN ACCORDING TO AISC-2010(LRFD)

a) Nominal Strength of Composite Sections

For the plastic stress distribution method, the nominal strength shall be computed assuming that steel components have reached a stress of F_y in either tension or compression and concrete components in compression due to axial force and/or flexure have reached a stress of $0.85f'_c$. For round HSS filled with concrete, a stress of $0.95f'_c$ is permitted to be used for concrete components in compression due to axial force and/or flexure to account for the effects of concrete confinement. the following material limitations shall be met, unless justified by testing or analysis: (1) For the determination of the available strength, concrete shall have a compressive strength, f'_c, of not less than 3 ksi (21 MPa) nor more than 10 ksi (70MPa), for normal weight concrete and not less than 3 ksi (21 MPa) nor more than 6 ksi (42 MPa) for lightweight concrete. (2) The *specified minimum yield stress* of structural steel and reinforcing bars used in calculating the strength of composite members shall not exceed 75 ksi (525 MPa).

For compression and flexure, filled composite sections are classified as compact, non-compact or slender.

Description of Element	Width-to-Thickness Ratio	λ_p Compact/ Noncompact	λ_r Noncompact/ Slender	Maximum Permitted
Walls of Rectangular HSS and Boxes of Uniform Thickness	b/t	$2.26\sqrt{\frac{E}{F_y}}$	$3.00\sqrt{\frac{E}{F_y}}$	$5.00\sqrt{\frac{E}{F_y}}$
Round HSS	D/t	$\frac{0.15E}{F_y}$	$\frac{0.19E}{F_y}$	$\frac{0.31E}{F_y}$

Table 2.7 Limiting width to thickness ratio for compression steel elements in composite members subjected to axial compression

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

Table 2.8 Limiting width to thickness ratio for compression steel elements in composite members subjected to flexure.

For filled composite members, the cross-sectional area of the steel section shall comprise at least 1% of the total composite cross section.

b) Compressive Strength

The design compressive strength, $\phi_c P_n$ of doubly symmetric axially loaded concrete filled composite members shall be determined for the limit state of flexural buckling based on member slenderness as follows:

ϕ_C = 0.75 (LRFD)

When $p_{no}/p_e \leq 2.25$, $\quad p_n = p_{no}\left[0.658^{\frac{p_{no}}{pe}}\right]$ (2.10)

When $p_{no}/p_e > 2.25$, $\quad p_n = 0.877[p_e]$ (2.11)

For *compact sections*

$\boldsymbol{P_{no} = P_p}$ where

$$P_p = F_y A_s + C_2 f'c\left(A_s + A_{sr}\frac{E_s}{E_c}\right) \tag{2.12}$$

C_2 = 0.85 for rectangular section and 0.95 for circular section.

For *non-compact sections*

$$P_{no} = P_p - \frac{P_p - P_y}{(\lambda r - \lambda_p)^2}(\lambda - \lambda_p)^2 \tag{2.13}$$

Where λ, λ_r, λ_p are slenderness ratios and P_p is determined from above.

For *slender sections*

$$P_{no} = F_{cr}A_s + 0.7f'c(Ac + A_{sr}\frac{Es}{Ec}) \quad (2.14)$$

Where, i) for rectangular filled sections

$$F_{cr} = \frac{9E_s}{\left[\frac{b}{t}\right]^2}$$

ii) For round filled sections

$$Fcr = \frac{0.72F_y}{\left[\left|\frac{D}{t}\right|\left[\frac{F_y}{E_s}\right]\right]^2}$$

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

$$EI_{eff} = E_sI_s + E_sI_{sr} + C_3E_cI_c \quad (2.15)$$

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

$$C_3 = 06 + 2\left[\frac{As}{As+Ac}\right] \leq 0.9 \quad (2.16)$$

The available compressive strength need not be less than specified for the bare steel member.

$$p_e = \frac{\pi^2 EI_{eff}}{(KL)^2} \quad (2.17)$$

(Elastic critical buckling load, kips or N)

$A_c =$ Area of concrete, in^2 (mm^2)

$A_s =$ Area of the steel section, in^2 (mm^2)

$E_c =$ Elasticity modulus of concrete. $= W_c^{1.5}\sqrt{f_c'}$, Ksi $\left(0.043W_c^{1.5}\sqrt{f_c'}\right)$, Mpa.

EI_{eff} = effective *stiffness* of composite section, kip-in.^{2} (N-mm^2)

Es = modulus of elasticity of steel. (= 29,000 ksi (200,000 MPa))

F_y = *specified minimum yield stress* of steel section, ksi (MPa)

F_{ysr} = specified minimum yield stress of reinforcing bars, ksi (MPa)

I_c = moment of inertia of the concrete section about the elastic neutral axis of the composite section, in.^{4} (mm^4).

I_s = moment of inertia of steel shape about the elastic neutral axis of the composite section, in.^{4} (mm^4).

I_{sr} = moment of inertia of reinforcing bars about the elastic neutral axis of the composite section, in.^{4} (mm^4)

K = *effective length factor*

L = laterally *unbraced length* of the member, in. (mm)

f_c' = specified compressive strength of concrete, ksi (MPa)

w_c = weight of concrete per unit volume ($90 \leq w_c \leq 155$ lbs/ft^3 or $1500 \leq w_c \leq 2500$ kg/m^3)

c) Flexural Strength

The *available flexural strength* of *filled composite members* shall be determined as follows:

$\varphi_b = 0.90$ (LRFD)

The nominal flexural strength, M_n, shall be determined as follows:

For compact sections

$$M_n = M_p$$

Where, M_p = moment corresponding to plastic *stress* distribution over the composite cross section, kip-in. (N-mm)

For non-compact sections

$$M_n = M_p - \left[M_p - M_y\right]\left[\frac{\lambda - \lambda p}{\lambda r - \lambda p}\right] \tag{2.18}$$

Where, λ, λ_p and λ_r are slenderness ratios determined from above table given.

M_y = *yield moment* corresponding to yielding of the tension flange and first yield of the compression flange, kip-in. (N-mm). The capacity at first yield shall be calculated assuming a linear elastic stress distribution with the maximum concrete compressive stress limited to $0.7fc'$ and the maximum steel stress limited to F_y.

For slender sections:-M_n, shall be determined as the first yield moment. The compression flange stress shall be limited to the local buckling stress, F_{cr}, determined. The concrete stress distribution shall be linear elastic with the maximum compressive stress limited to $0.70f'_c$.

d) Combined compression and flexure

The interaction of flexure and compression in doubly symmetric members and singly symmetric members for which $0.1 \leq I_{yc}/I_y \leq 0.9$, constrained to bend about a *geometric axis* (x and/or y) shall be limited by equations described below, where I_{yc} is the moment of inertia of the compression flange about the y-axis, in.^{4} (mm^4).

a) When $p_r/p_c \geq 0.2$,

$$\left[\frac{p_r}{p_c}\right] + \frac{8}{9}\left[\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right] \leq 1.0 \qquad (2.19)$$

b) When $p_r/p_c < 0.2$,

$$\left[\frac{p_r}{2p_c}\right] + \left[\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right] \leq 1.0 \qquad (2.20)$$

Where,

P_r = *required axial strength* using LRFD load combinations, kips (N)

P_c = φcPn = *design axial strength* ,kips (N)

M_r = required flexural strength using LRFD load combinations, kip-in. (N-mm)

M_c = φbMn = *design flexural strength* determined, kip-in. (N-mm)

φ_c = *resistance factor* for compression (= 0.90)

φ_b = *resistance factor for flexure (= 0.90)*

x = subscript relating symbol to strong axis bending

y = subscript relating symbol to weak axis bending

e) Approximate second order analysis

an approximate second order analysis provides, as an alternative to a rigorous second-order analysis, a procedure to account for second-order effects in structures by amplifying the required strengths indicated by a first-order analysis.

The *required second-order flexural strength, M_r,* and axial strength, P_r, of all members shall be determined as follows:

$M_r = \beta_1 M_{nt} + \beta_2 M_{lt}$ (2.21)

$P_r = P_{nt} + \beta_2 P_{lt}$ (2.22)

where β_1 = multiplier to account for *P-δ effects*, determined for each member subject to compression and flexure, and each direction of bending of the member, and β_1 shall be taken as 1.0 for members not subject to compression.

β_2 = multiplier to account for *P-Δ effects*, determined for each direction of lateral translation of the story.

M_{lt} = first-order moment using LRFD load combinations, due to lateral translation of the structure only, kip-in. (N-mm)

M_{nt} = first-order moment using LRFD load combinations, with the structure restrained against lateral translation, kip-in. (N-mm)

M_r = required second-order flexural strength using *LRFD load combinations*, kip-in. (N-mm)

P_{lt} = first-order axial force using LRFD load combinations, due to lateral translation of the structure only, kips (N)

P_{nt} = first-order axial force using LRFD load combinations, with the structure restrained against lateral translation, kips (N)

P_r = required second-order axial strength using LRFD load combinations, kips (N)

Multiplier β_1 for P-δ Effects: - The β_1 multiplier for each member subject to compression and each direction of bending of the member is calculated as follows:

$$\beta_1 = \frac{C_m}{1-\alpha\frac{P_r}{P_{e1}}} \geq 1 \qquad (2.23)$$

Where, α = 1.00,

C_m = coefficient assuming no lateral translation of the frame determined as follows:

For *beam-columns* not subject to transverse loading between supports in the plane of bending

$C_m = 0.6 - 0.4(M_1/M_2)$

where M_1 and M_2, calculated from a *first-order analysis*, are the smaller and larger moments, respectively, at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1/M_2 is positive when the member is bent in reverse curvature, negative when bent in single curvature for beam-columns subject to transverse loading between supports, the value of C_m shall be determined either by analysis or conservatively taken as 1.0 for all cases.

P_{e1} = elastic critical buckling strength of the member in the plane of bending, calculated based on the assumption of no lateral translation at the member ends, kips (N)

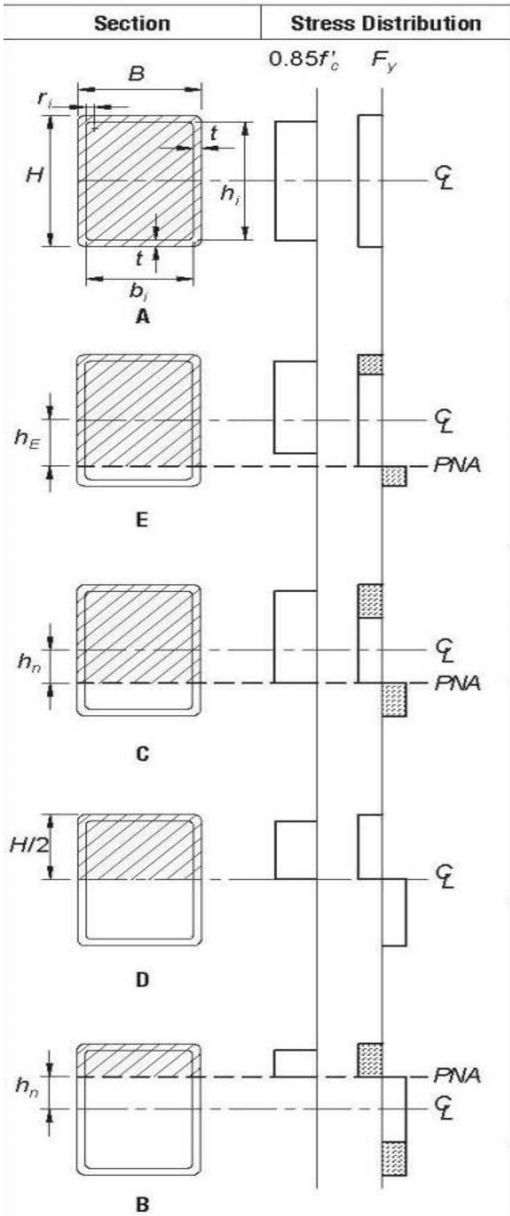
$$P_{e1} = \frac{\pi^2 EI^*}{(k_1 l)^2} \qquad (2.24)$$

Where, EI* = flexural rigidity required to be used in the analysis (= 0.8τbEI when used in the direct analysis method, and = EI for the effective length and first-order analysis methods). E = modulus of elasticity of steel = 29,000 ksi (200,000 MPa).

I = moment of inertia in the plane of bending, in.^{4} (mm^4)

L = length of member, in. (mm)

K_1 = *effective length factor* in the plane of bending, calculated based on the assumption of no lateral translation at the member ends, it is equal to 1.0 unless analysis justifies a smaller value. It is permitted to use the first-order estimate of P_r (i.e., $P_r = P_{nt} + P_{lt}$) in finding β_1.



Pt.	Defining Equations
A	$P_A = F_y A_s + 0.85 f'_c A_c$ $M_A = 0$ A_s = area of steel shape $A_c = b_i h_i - 0.858 r_i^2$ $b_i = B - 2t$ $h_i = H - 2t$ $r_i = t$
E	$P_E = \frac{0.85 f'_c A_c}{2} + 0.85 f'_c b_i h_E + 4 F_y t h_E$ $M_E = M_D - F_y Z_{sE} - \frac{0.85 f'_c Z_{cE}}{2}$ $Z_{cE} = b_i h_E^2$ $Z_{sE} = 2 t h_E^2$ $h_E = \frac{h_n}{2} + \frac{H}{4}$
C	$P_C = 0.85 f'_c A_c$ $M_C = M_B$
D	$P_D = \frac{0.85 f'_c A_c}{2}$ $M_D = F_y Z_s + \frac{0.85 f'_c Z_c}{2}$ Z_s = full *x*-axis plastic section modulus of HSS $Z_c = \frac{b_i h_i^2}{4} - 0.192 r_i^3$
B	$P_B = 0$ $M_B = M_D - F_y Z_{sn} - \frac{0.85 f'_c Z_{cn}}{2}$ $Z_{sn} = 2 t h_n^2$ $Z_{cn} = b_i h_n^2$ $h_n = \frac{0.85 f'_c A_c}{2\left[0.85 f'_c b_i + 4 t F_y\right]} \le \frac{h_i}{2}$
	Note: Equations in this table are equally applicable to bending about the shape's *X-X* axis (when $H \ge B$) and to bending about the shape's *Y-Y* axis (when $B > H$).

Table 2.9 plastic capacities for rectangular CFT column section bent about any axis

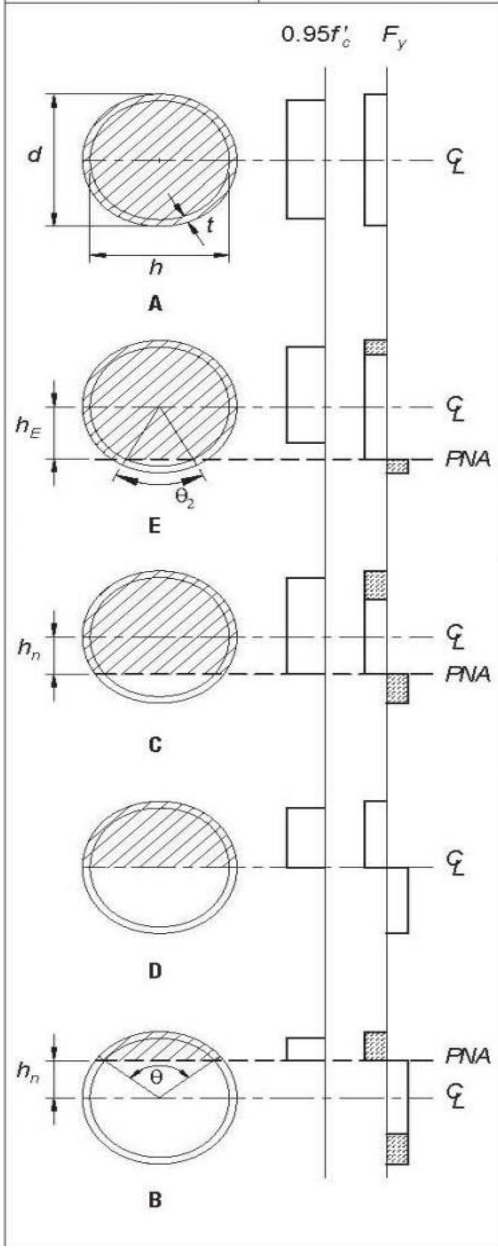
Section / Stress Distribution	Pt.	Defining Equations
	A	$P_A = F_y A_s + 0.95 f'_c A_c$ $M_A = 0$ $A_s = \pi\left(dt - t^2\right)$ $A_c = \frac{\pi h^2}{4}$
	E	$P_E = P_A - \frac{1}{4}\left[F_y\left(d^2 - h^2\right) + \frac{0.95 f'_c}{2} h^2\right]\left(\theta_2 - \sin\theta_2\right)$ $M_E = F_y Z_{sE} + \frac{0.95 f'_c Z_{cE}}{2}$ $Z_{cE} = \frac{h^3}{6}\sin^3\left(\frac{\theta_2}{2}\right)$ $Z_{sE} = \frac{\left(d^3 - h^3\right)}{6}\sin\left(\frac{\theta_2}{2}\right)$ $h_E = \frac{h_n}{2} + \frac{h}{4}$ $\theta_2 = \pi - 2\arcsin\left(\frac{2h_E}{h}\right)$
	C	$P_C = 0.95 f'_c A_c$ $M_C = M_B$
	D	$P_D = \frac{0.95 f'_c A_c}{2}$ $M_D = F_y Z_s + \frac{0.95 f'_c Z_c}{2}$ Z_s = plastic section modulus of steel shape $= \frac{d^3}{6} - Z_c$ $Z_c = \frac{h^3}{6}$
	B	$P_B = 0$ $M_B = F_y Z_{sB} + \frac{0.95 f'_c Z_{cB}}{2}$ $Z_{sB} = \frac{\left(d^3 - h^3\right)}{6}\sin\left(\frac{\theta}{2}\right)$ $Z_{cB} = \frac{h^3 \sin^3\left(\frac{\theta}{2}\right)}{6}$ $\theta = \frac{0.0260K_c - 2K_s}{0.0848K_c} + \frac{\sqrt{\left(0.0260K_c + 2K_s\right)^2 + 0.857K_c K_s}}{0.0848K_c}$ (rad) $K_c = f'_c h^2$ $K_s = F_y\left(\frac{d-t}{2}\right)t$ ("thin" HSS wall assumed) $h_n = \frac{h}{2}\sin\left(\frac{\pi - \theta}{2}\right) \le \frac{h}{2}$

Table 2.10 plastic capacities for circular CFT column section bent about any axis

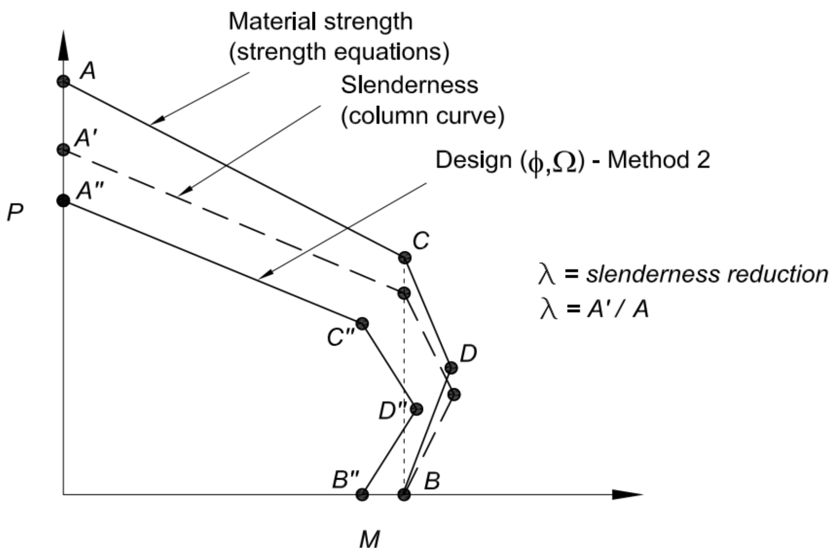


Figure 2.4 *interaction diagram for beam column from plastic stress distribution.*

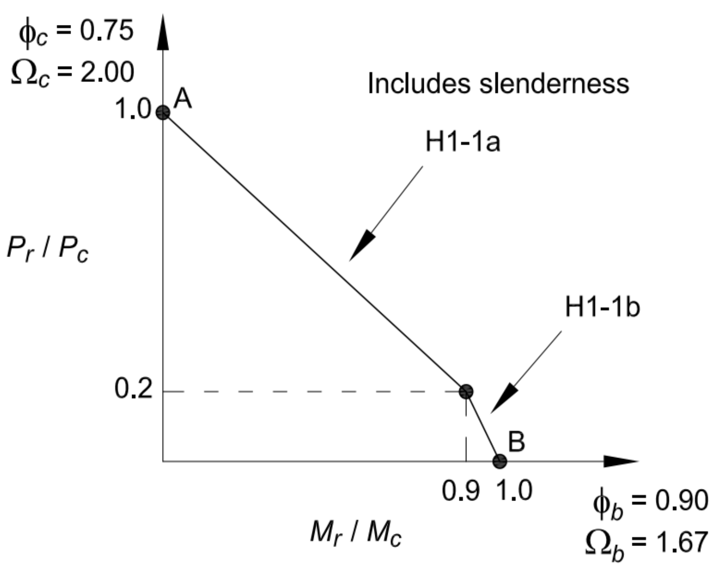


Figure 2.5 *interaction diagram for composite beam column of section H-1*

CHAPTER 3

DATABASE DEVELOPMENT

3.1 GENERAL

In the database, materials and geometric properties of the test specimens are concerned. The data is divided into circular section and rectangular section columns, with pure axially loaded and with beam column section. The columns are short (L/D or L/b $\leq$ 4) or long (L/D or L/b > 4). The results of average test/code comparisons are summarized for each type of column and the standard deviation of this ratio for each set. The principal limitations and conditions specified in EC4 and AISC-2010, as far as CFST columns are concerned Tests which were outside these limits have not been excluded from the comparison. In total, four databases are reported: - CCFT, RCFT columns and CCFT, RCFT beam columns. Specimens those tested with monotonic loading and single curvature were used in the comparisons.

The material properties collected consisted of the concrete compressive strength, the yield stress of the steel section and the modulus of elasticity of the steel and concrete. Sectional properties included the area, moments of inertia, elastic and plastic section modulus, and radius of gyration of the steel section, and concrete. In addition both the effective length and the structural steel ratio were recorded. For circular concrete filled tube columns, the diameter, thickness and diameter to-thickness ratio were added to the database. Similarly, for rectangular concrete filled tube columns, the depth, thickness, and depth-to-thickness ratio were added. Finally, the experimental value of ultimate axial capacity is included for pure axially loaded columns, while experimental load eccentricity and ultimate moment were included for beam-columns.

After the main properties had been established, the axial strength for each specimen, as calculated by the AISC and Eurocode methods, are computed. Local buckling checks are also carried out. For beam-columns, with a constant eccentricity the axial load capacity of the column is predicted from their respective interaction equations. Finally, the ratio of experimental-to-predicted axial load is calculated.

The Eurocode data included the axial load plastic resistance $N_{pl,Rd}$, the critcal buckling load N_{cr}, the buckling reduction factor χ and the moment of inertia of the composite section are calculated. For beam-columns, the predicted load and moment was calculated by interpolation between the simplified points in the interaction diagram. The interaction curve was calculated by full plastic theory and approximated by a polygonal path (Figure 3-2). Point A is the pure axial strength, Point B is determined as the flexural strength. Point C corresponds to a plastic neutral axis location that results in the same flexural strength as Point B, but including axial compression. Point D corresponds to an axial compressive strength of one half of that determined for Point C.

$$N_{Ed} \leq \frac{N_{pm,Rd}}{2}:\ \mu_d = 1 + \frac{2N_{Ed}}{N_{pm,Rd}}\left[\frac{M_{max,Rd}}{M_{pl,Rd}} - 1\right] \tag{3.1}$$

$$\frac{N_{pm,Rd}}{2} < N_{Ed} \leq N_{pm,Rd}:\ \mu_d = 1 + \frac{2[N_{pm,Rd} - N_{Ed}]}{N_{pm,Rd}}\left[\frac{M_{max,Rd}}{M_{pl,Rd}} - 1\right] \tag{3.2}$$

$$N_{Ed} > N_{pm,Rd}:\ \mu_d = \frac{[N_{pl,Rd} - N_{Ed}]}{N_{pm,Rd} - N_{pm,Rd}} \tag{3.3}$$

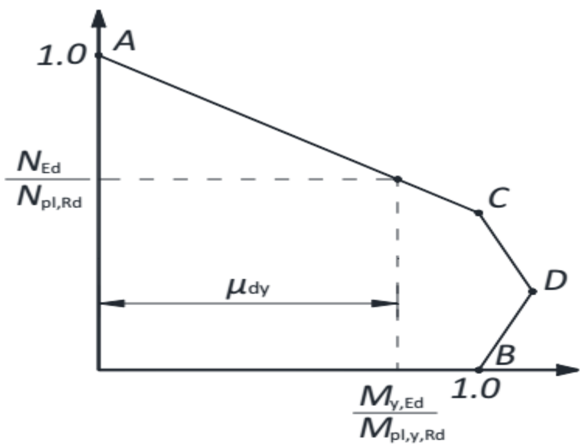


Figure 3.1 interaction diagram for axial resistance prediction

For the AISC 2010 specification, the axial load plastic resistance, P_{no}, the buckling load, P_n, and the effective rigidity (EI_{eff}) of the composite cross-section are computed. For beam-columns, a predicted load and moment are calculated from the interaction curve. This interaction diagram was drawn using a simplified method which utilizes key points from A to E (Figure 3-1). Once the nominal strength interaction surface is determined, length effects must be applied. The same slenderness reduction factor ($\lambda = P_n/P_{no}$) applies to points A, C, D and E. Finally, for comparison without applying the resistance factor, the axial capacity and flexural capacity is determined and the value of test-to-prediction is given as the ratio of OT to OX.

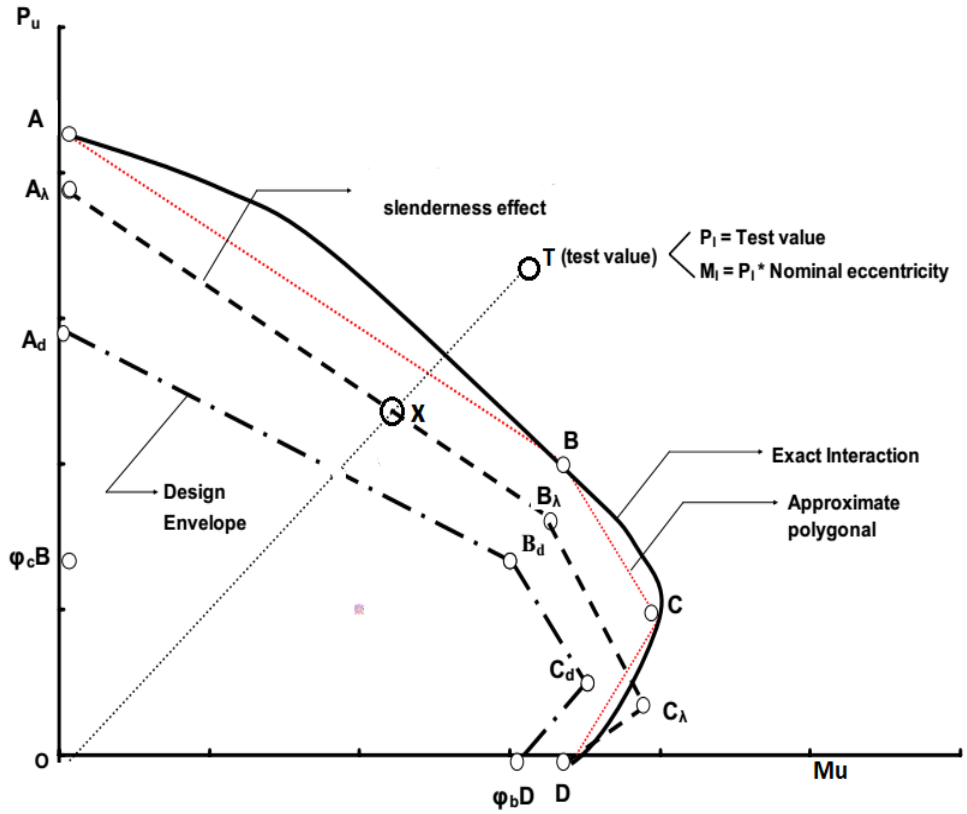


Figure 3.2 interaction diagram for axial resistance prediction

CHAPTER 4

REVIEW & SUMMARY OF CFT EXPERIMENTAL DATA

A summary of the test series in the database are given below. The number given to each tested specimens are as it was in the original reports. For each data set, there is a brief description that includes the following items:

4.1 CIRCULAR COLUMNS (CCFT)

G. Giakoumelis, D. Lam (2003)

Experimental synopsis. The behavior of circular concrete-filled steel tubes (CFT) with various concrete strengths under axial load was presented. The effects of steel tube thickness, the bond strength between the concrete and the steel tube, and the confinement of concrete were examined. As the concrete strength increases the effects of the bond of the concrete and the steel tube became more critical. For normal concrete strength, the reduction on the axial capacity of the column due to bonding was negligible. For high-strength concrete, the variation between Non Greased and Greased was 17%

Main test parameters. Concrete strength, wall thickness and the bond strength between the concrete and steel.

Number of tests. 15 specimens were tested.

Steel properties. Hot finished circular hollow section (CHS) were used for the tests.

- ***Tube size D (mm):-*** The diameter of the tubes was 114 mm.

- ***Wall thickness t (mm):-*** Wall thickness were 3.6 mm and 5.0 mm.

- ***Yield strength of steel f_y (MPa):-*** The yield strength of the steel was 343 and 365 MPa.

Concrete properties.

- ***Compressive strength of concrete (MPa):-*** Cylindrical compressive strengths were 94.7 and 110.3 MPa.

Effective length. All specimens were 300 mm in length to reduce end effects and to ensure that the specimens would be stub columns with little effect from column slenderness.

End condition. The specimens were capped on both ends with rigid steel caps to distribute the applied load uniformly over the concrete and steel.

Loading method. Concentrically compressive axial load was carried out using a 3000 kN capacity testing machine.

M.D., O'shea, R.Q., Bridge (1997)

Experimental synopsis. The influence of local buckling on the behavior of short circular thin-walled concrete filled steel tubes has been examined. Two possible failure modes of the steel tube have been identified, local buckling and yield failure. These were found to be independent of the diameter to wall thickness ratio. Instead, bond (or lack of it) between the steel and concrete infill determined the failure mode.

Main test parameters. Confining action of thin-walled circular steel tubes, D/t ratio and concrete strength.

Number of tests. 10 specimens were tested.

Steel properties. Two series of tests were performed on the steel tubes. In the first series the specimens were axially loaded with no internal restraint. In the other series, the specimens were filled with un-bonded concrete to restrain the possible formation of internal buckles. Consequently the internal concrete was found to have no influence on the buckling mode of the axially loaded steel tube.

- ***Tube size D (mm):-*** tubes size was 165, 190 mm.

- ***Wall thickness t (mm):-*** wall thickness were 0.86, 1.13, 1.52, 1.94, and 2.82.

- ***Yield strength of steel f_y (MPa):-*** steel tubes were cold formed rolling of hot-rolled steel sheet. Yield strength varies from 185.7 - 363.3 MPa.

- ***Residual stresses: -*** the maximum residual stress occurs at the vicinity of the weld. ***- Out of straightness:*** - out of straightness of the tubes were measured 0.4mm.

Concrete properties. Cylindrical compressive strength of the concrete were 50

- ***Compressive strength of concrete f_{ck} (MPa):-*** cylindrical compressive strength were 50 and 80.

End condition. Pinned-pinned support condition and the specimen's ends were ground square and flat to ensure that steel and concrete were loaded together.

Loading method. Axially loaded.

Sakino, K., Nakahara, H., Morino, S., Nishiyama (2004)

Experimental synopsis. The confining effect of steel tubes on concrete strength and the restraining effect of the concrete fill on local buckling of the steel tube wall has been investigated. Based on the experimental data it was concluded that the difference between the ultimate strength and the nominal squash load of circular CFT columns, which was provided by confining the concrete, could be estimated as a linear function of the tube yield strength. The maximum axial load was greater than the nominal squash load in almost of all the circular CFT columns due to the confinement effect.

Main test parameters. Tube shapes, tube tensile strength, D/t ratio and concrete strength.

Number of tests. 36 circular concrete filled steel tubes (CCFT).

Steel properties. Tubes were cold formed from a flat plate by press bending and seam welding. The material properties of the steel tubes were obtained from tensile tests of coupons taken from each steel plate before manufacturing.

- ***Tube size D (mm):-*** tubes size were from 108 to 450.
- ***Wall thickness t (mm):-*** wall thickness were from 2.96 to 6.47.
- ***Yield stress of steel f_y (MPa):-*** Yield stress were from 279 to 853.

Concrete properties Cylindrical Compressive strength of filled is estimated by multiplying the compressive strength of 10 cm by 20 cm cylinder by the reduction factor introduced to take scale effect into consideration.

- ***Compressive strength of concrete f_{ck} (MPa):-*** cylindrical compressive strength were from 25.4 to 85.1 MPa.

End condition. End plates butt welded on top and bottom of the test specimen.

Loading method. Axially loaded.

J. Zeghiche, K. Chaoui (2004)

Experimental synopsis. The test results demonstrate the influence of the column slenderness, the load eccentricity with single or double curvature bending and the compressive strength of the concrete core on the strength and behavior of concrete-filled steel tubular columns. A comparison of experimental with the predicted failure loads in accordance with the method described in Eurocode 4 showed good agreement for axially and eccentrically loaded columns with single curvature bending whereas for columns with double curvature bending the Eurocode loads were higher and on the unsafe side.

Main test parameters. The test parameters were the column slenderness, the load eccentricity covering axially and eccentrically loaded columns with single or double curvature bending and the compressive strength of the concrete core.

Number of tests. 15 specimens were tested.

Steel properties. Strips of the steel tubes were tested in tension for each tested column, in accordance with the European standards. From these tests the modulus of elasticity was 212 × 103 MPa. -

Tube size D (mm):- tubes size was 160 mm.

- ***Wall thickness t (mm):-*** wall thickness was 5 mm.

- ***Yield strength of steel f_y (MPa):-*** the average tensile yield strength was found to be 275. MPa.

Concrete properties. Progressive vibration was employed in order to eliminate air pockets in the concrete and give a homogeneous mix. Three concrete mixes were used in this investigation with a maximum size (river gravel) of 10 mm aggregate. -

Compressive strength of concrete f_{ck} (MPa):- Cylindrical compressive strength were 40, 70, 100 MPa.

End condition. A set of adapter endplates equipped with half-spherical bearings were fixed to both ends of each column to form a simply supported column

Loading method All columns were tested in an Amsler compressive testing machine with a maximum load capacity of 10 000 kN. Each increments of load was about 10% of the predicted failure load.

J.C.M. Ho, M.H. Lai (2013)

Experimental synopsis. Confinement in the form of tie bars was proposed in this study to restrict the elastic lateral dilation of concrete. 30 CFST columns of various dimensions cast with normal- (NSC) or high-strength concrete (HSC) and installed with tie bars were tested under uni-axial compression. From the results, it was evident that, tie-confined CFST columns had uni-axial strength larger than those of unconfined CFST columns, it can increase slightly the elastic stiffness and restrict effectively the lateral dilation of CFST columns. The failure modes of tie-confined NSCFST and HSCFST columns were the local buckling at the ends and formation of longitudinal steel cracks initiated at bolt holes.

Main test parameters. The test parameters were the concrete strength, thickness of the tubes, spacing of the stiffeners.

Number of tests. 30 specimens were tested.

Steel properties. -

Tube size D (mm):- tubes size was 168.3 mm.

- ***Wall thickness t (mm):-*** wall thickness was 5 and 8 mm. -

Yield strength of steel f_y (MPa):- the average tensile yield strength was found to be 365. MPa.

Concrete properties. -

Compressive strength of concrete f_{ck} (MPa):- Cylindrical compressive strength were 29.1, 35.2, 34.1, 83.1, 75.2, 114.3 MPa.

Effective length. Length of the specimens were 248 and 330 mm.

End condition. The end of the specimens were fixed to platens.

Loading method. All the hollow steel tubes and CFST columns were tested under displacement control. The initial loading rate was 0.3 mm/min or 0.2 mm/min for specimens of 330 mm or 248 mm height. The loading rate increased at a rate of 0.05 mm/min for every 2 mm increase in the axial displacement after the specimens had yielded.

J.M. Portoles, E.Serra, M.L. Romero (2013)

Experimental synopsis. Tests were conducted on slender circular tubular columns filled with normal, high, and ultra-high strength concrete to analyze the influence of each type of infill and establish the best option for practical application.). For the limited cases analyzed the results showed that the addition of high or ultra-high strength infill is more useful for concentric loaded cases. The experimental ultimate load of each test was compared with the design loads from Eurocode 4.

Main test parameters. The test parameters are nominal strength of concrete (30, 90 and 130 MPa).

Number of tests. 20 specimens were tested.

Steel properties. -

Tube size D (mm):- tubes size was between 108 and 133 mm.

- ***Wall thickness t (mm):-*** wall thickness was 1 and 7 mm. -

Yield strength of steel f_y (MPa):- the average tensile yield strength was found to be between 232 and 429 MPa.

Concrete properties. Chinese ordinary Portland cement corresponds to ACI type I cement was used as binder, natural sand with a fineness modulus of 2.7 as fine aggregate, and crushed limestone with a maximum nominal size of 20 mm as coarse aggregate, blast furnace slag and fly ash used in mix. -

Compressive strength of concrete f_{ck} (MPa):- Cylindrical compressive strength were 39.4, 61.3, 77.4, 84.7 MPa.

Effective length. Length of the specimens were 378 and 438 mm.

End condition. The end of the specimens were pinned-pinned support.

Loading method. Specimens were tested under concentrically loaded axial compression. 15 increments of load were applied to each specimen.

L.H. Han, G.H. Yao (2004)

Experimental synopsis. The influence of concrete compaction methods on the member capacities of the composite columns was investigated. The specimens were tested with different type of conditions likely to arise in the manufacture of concrete: cured, well compacted with a poker vibrator, well compacted by hand, and self-consolidating without any vibration.

Main test parameters. The main parameters varied in the tests were, tube diameter (or depth) to thickness ratio; and load eccentricity ratio.

Number of tests. 12 stub and 6 slender columns were tested.

Steel properties. The tubes were all manufactured from a mild steel sheet, tack welded into circular or square shape and then welded with a single bevel butt weld. The modulus of elasticity was about 206,500 MPa.

- ***Tube size D (mm):-*** tubes size was between 100 and 200 mm.

- ***Wall thickness t (mm):-*** wall thickness was 3 mm.

- ***Yield strength of steel f_y (MPa):-*** Three coupons were taken and the average yield strength of the tube was found to be 303.5 MPa.

Concrete properties. The modulus of elasticity (Ec) of the concrete was measured, the average value being 37,420 MPa. -

Compressive strength of concrete (MPa):- A kind of SCC mix was designed for compressive cube strength at 28 days of approximately 58.5 MPa.

Effective length. Length of the stub columns were 300 and 600 mm, for the slender columns 2000 mm.

End condition. The loading ram is a solid steel plate, which acts like an end stiffener.

Loading method. All the tests were performed on a 5000 kN capacity testing machine. The specimens were placed into the testing machine and loads were applied on the specimens directly.

M. Dundu (2006)

Experimental synopsis. The behavior of circular concrete-filled steel tube columns, loaded concentrically in compression to failure was investigated. Variables in the tests include the length, diameter, strength of the steel tubes and the strength of the concrete. The slender columns failed by overall flexural buckling while the stockier columns failed by crushing of the concrete and yielding of the steel tube.

Main test parameters. The main parameters varied in the tests were column section type, circular and square; tube diameter (or depth) to thickness ratio; and load eccentricity ratio.

Number of tests. 24 columns were tested.

Steel properties. The tensile coupons were prepared and tested according to the guidelines provided by the British Standard BS.

- ***Tube size D (mm):-*** tubes size was between 114.85 and 193.7mm.
- ***Wall thickness t (mm):-*** wall thickness was 3.0 and 3.50 mm.
- ***Yield strength of steel f_y (MPa):-*** The average yield strength of the tube was found to be between 345.2 and 488.2 MPa.

Concrete properties. The concrete mix design consisted of cement, 19 mm stone (coarse aggregates) and river sand (fine aggregate). The cubes were filled in two or three layers and compacted using steel rod.

- ***Compressive strength of concrete (MPa):-*** compressive cylindrical strengths were 32.1 and 25.6 MPa.

Effective length. 1000 mm for stocky columns. 2000 mm and 2500 mm for slender column.

End condition. Each circular hollow section (CHS) was filled with concrete in 4 layers.

Loading method. A Morh and Federhaff compression testing machine was applied which has a load capacity of 9000 kN and maximum loading rate of 30 mm/min.

O'Shea M.D., Bridge R.Q. (1994)

Experimental synopsis. The effects of local buckling on concrete filled tubes with a diameter to wall thickness ratio of 165 filled with 120 MPa concrete were tested mainly in axial compression. Measurements of geometric properties including imperfections, material properties of the concrete and steel including residual stresses were taken. The significance of local buckling and concrete confinement is discussed.

Main test parameters. The main parameters varied in the tests were tube diameter (or depth) to thickness ratio; and concrete compressive strength.

Number of tests. 7 specimens were tested.

Steel properties. The tubes were fabricated using cold-formed rolling of hot-rolled steel sheet.

- ***Tube size D (mm):-*** tubes size was 190 mm.

- ***Wall thickness t (mm):-*** wall thickness was 1.15 mm. -

Yield strength of steel f_y (MPa):- average yield strength of the steel 330 MPa.

Concrete properties. The coarse aggregate was basalt with a maximum size of 10 mm, the fine aggregate was with a maximum size of 3 mm, fly ash was used to provide additional fines, and the super-plasticizer.

- ***Compressive strength of concrete (MPa):-*** Cylindrical compressive strengths were 94.7 and 110.3 MPa.

Effective length. Length of the specimens was 665 mm.

End condition. The ends of the specimens was connected to stiff platens.

Loading method. A set of load, strain and displacement readings were taken at load increments varying from 100 kN at the start of the test to 20 kN near ultimate load.

Z.W. Wu, F.X. Ding, C.S. Cai (2006)

Experimental synopsis. The behavior of circular, concrete-filled steel tube (CFT) stub columns with self-compacting concrete (SCC) and normal concrete (NC) concentrically loaded in compression to failure were studied. Four measurement methods on the axial deformation of specimens were compared. The effects of concrete strength, notched holes or slots, and different loading conditions on the ultimate capacity and the load–deformation behavior of the columns have been studied. By using higher strength concrete, the specimens with the entire section loaded experienced a significant increase in the ultimate capacity, but their residual capacity after failure was almost constant. The axial compressive stiffness of specimens was reduced for the steel tube that was notched. Eurocode 4 predicted a reasonable capacity for the un-notched CFT stub columns with both SCC and NC if the entire section of the specimen is loaded.

Main test parameters. The main parameters of the test were concrete strength, notched holes or slots, and different loading condition.

Number of tests. A total of 17 test specimens were tested.

Steel properties. From these tests modulus of elasticity (E_s) was 2.13 × 105 MPa.

- ***Tube size D (mm):-*** tubes size was 190 mm.

- ***Wall thickness t (mm):-*** wall thickness was between 2.72 and 4.78 mm.

- ***Yield strength of steel f_y (MPa):-*** three tension coupons were cut from, with dimensions in accordance with the Chinese standard GB/T228-2002., the average yield strength of all types of steel tubes was 350 MPa.

Concrete properties. Self-compacting concrete (SCC) and normal concrete were used for the test.

- ***Compressive strength of concrete (MPa):-*** the cube compressive strengths were between 42.6 and 77.2 MPa.

Effective length. Length of the specimens were 500, 510 and 650 mm.

End condition. Each tube was welded to a square, steel base plate of 5 mm thick at the bottom.

Loading method. Under concentric axial compression loads all of the columns were tested with a universal testing machine with a 5000 kN capacity. The load was applied in small increments at a very slow rate (about 50–100 kN/min).

4.2 RECTANGULAR COLUMNS (RCFT)

Han, L.-H, Yao, G.H (2003)

Experimental synopsis the influence of concrete compaction on member capacity was investigated. Tests on 35 concrete-filled steel RHS columns under different concrete compaction methods was reported The main parameters varied in the tests are: (1) column section depth-to-width ratio, from 1.0–2.0, (2) tube depth to thickness ratio from 34–136, (3) load eccentricity (e) from 0 to 31 mm and (4) column slenderness (λ) from 21 to 62.

Experimental study results and discussion. The average yield strength (f_{sy}) of the tube in accordance with the Chinese standard GB2975 (1982) was found to be 340.1 MPa and the modulus of elasticity was about 207,000 MPa. The concrete mix was designed for compressive cube strength (f_{cu}) at 28 days of approximately 22 MPa. The modulus of elasticity (E_s) was found to be 25,306 MPa. The ends of the steel tubes were cut and machined to the required length. For the specimens compacted by hand, local buckling effects become increasingly important because of the lack of compaction of the concrete and local buckling occur earlier. The experimental results clearly show that good concrete compaction result in larger critical buckling loads. It was found that the ultimate strengths of the short columns (with slenderness ratio λ = 21) compacted by poker vibrator were 3–30% higher than those of columns compacted by hand. The ultimate strengths of the long columns (with slenderness ratio λ = 62) with concrete compacted by poker vibrator were 20 to 27% higher than those of columns with concrete compacted by hand.

Analysis of test results. The strength loss index (*SLI*) as following was employed to quantify member strength loss of the composite columns because of the un-compaction of its core concrete. The higher the slenderness ratio and the load eccentricity ratio (*e*/*r*), the bigger is the strength loss index (*SLI*). The bigger is the sectional dimension, the higher is the strength loss index (*SLI*).

Sakino, K., Nakahara, H., Morino, S., Nishiyama (2004)

Experimental synopsis. The confining effect of steel tubes on concrete strength and the restraining effect of the concrete fill on local buckling of the steel tube wall was investigated. Based on the experimental data it is concluded that The capacity reduction factor due to local buckling of the square steel tube wall was empirically derived based on the test results of hollow square steel tube columns with a thin wall, and then modified applicable to the steel tube in square CFT columns by considering the restraining effect of filled concrete on local buckling of the steel tube wall.

Main test parameters. Tube shapes, tube tensile strength, B/t ratio and concrete strength.

Number of tests. 48 axial square concrete filled steel tubes (SCFT).

Steel properties. The square steel tubes were fabricated by welding together two pieces of channel section, which were cold formed from a flat plate. The material properties of the steel tubes were obtained from tensile tests of coupons taken from each steel plate before manufacturing. -

Tube size B and D (mm):- tubes size were from 119 to 324.

- ***Wall thickness t (mm):-*** wall thickness was from 4.38 to 9.45.

- ***Yield strength of steel f_y (MPa):-*** Yield stress were from 262 to 835.

Concrete properties. Cylindrical Compressive strength of filled is estimated by multiplying the compressive strength of 10 cm by 20 cm cylinder by the reduction factor introduced to take scale effect into consideration.

- ***Compressive strength of concrete f_{ck} (MPa):-*** cylindrical compressive strength were from 25.4 to 91.1.

<u>Effective length (mm)</u>.

<u>End condition</u>. End plates butt welded on top and bottom of the test specimen.

<u>Loading method</u>. Axially loaded.

B. Uy (1998)

<u>Experimental synopsis.</u> The study of the local and post-local buckling of steel plates in concrete filled box columns were carried out. Two sets of specimens were constructed for considering axial strength only. The test specimens considered were both of concrete filled steel columns, with one set designed to load the steel only. The second set of tests used was designed to load the composite section uniformly. a method were presented for determining the required slenderness limits for inelastic local buckling in a concrete filled thin walled box column and it was compared with International Standards for steel structures.

<u>Main test parameters</u>. B/t ratio and concrete strength.

<u>Number of tests.</u> 5 square concrete filled steel tubes (RCFT).

<u>Steel properties</u>. The box columns were assembled from four equal width plates. These were initially tack welded along the length of the column and then longitudinally fillet welded.

- ***Tube size B and D (mm):-*** tubes size were from 40, 50, 60, 80, and 100 mm.

- ***Wall thickness t (mm):-*** wall thickness were 3 mm.

- ***Yield strength of steel f_y (MPa):-*** The average yield stress from ten coupon tests was 300 MPa and the average ultimate stress from these tests was 410 MPa.

- ***Residual stresses: -*** Residual strain measurements were taken across the width of the steel box using a non-destructive technique involving a combination of electrical and mechanical (DEMEC) strain gauges. And residual stress σ_r at discrete points were 45 to 55MPa.

<u>Concrete properties</u>. The concrete cylinder compressive strength was determined at regular time intervals after curing to assess the strength of the concrete at the time of testing of the particular specimen. -

Compressive strength of concrete f_{ck} (MPa):- cylindrical compressive strength were 38 and 40 MPa.

<u>Effective length (mm)</u>.

<u>End condition</u>. Each column had a series of stiffeners welded to the ends so that the failure would be initiated in the center of the column and would not be influenced by any significant end effects.

Loading method. All columns were tested under displacement control so that the full displacement to failure and the resultant ductility could be observed.

C.S. Huang; Y.K. Yeh; G.-Y Liu; H.-T. Hu; K.C. Tsai (2002)

Experimental synopsis. 14 specimens were tested to examine the effects of cross-sectional shapes, width-to-thickness ratios, and stiffening arrangements on the ultimate strength, stiffness, and ductility of CFT columns. A new stiffening scheme was proposed as an alternative for enhancing the behavior of square CFT columns in terms of strength and ductility. The proposed stiffening scheme involves welding a set of four steel tie bars at regular spacing along the tube axis. The stiffened specimen appears to have a smaller wavelength of the appearance for a local buckling than the unstiffened one. An appropriate arrangement of stiffeners in the proposed stiffening scheme can improve strength degradation after reaching the ultimate load, and can even alter strain softening characteristics to achieve elastic–perfectly plastic behavior.

Main test parameters. The primary parameters considered in this test program include B/t, the spacing of the tie bars (*Ls*), and the diameters of the tie bars.

Number of tests. 14 stiffened square concrete filled steel tubes (RCFT).

Steel properties. All steel tubes were cold-formed carbon steel. The yield strength was determined from tests on tensile coupons. The size of coupons and the test procedures follow American Society for Testing and Materials, ASTM E10-84 (ASTM 1991) requirements. The square tubes were constructed by seam welding two U-shaped cold-formed steel plates.

- ***Tube size B and D (mm):-*** tubes size were between 200 and 300 mm.
- ***Wall thickness t (mm):-*** wall thickness were between 2 and 5 mm.
- ***Yield strength of steel f_y (MPa):-*** a yield strength were between 265 and 342 MPa.
- ***Residual stresses: -*** to investigate the behavior of CFT columns influenced by the residual stresses from the welding, one specimen was constructed by annealing its stiffened steel tube before pouring concrete into the tube.

Concrete properties. Portland cement, sand, and a maximum aggregate size of 2 cm were mixed to obtain an 18 cm slump.

- ***Compressive strength of concrete f_{ck} (MPa):-*** cylindrical compressive strength were 27.47 MPa.

Effective length (mm).

End condition. Pinned-pinned under concentric compression.

Loading method. The compression tests were conducted in a 4900 kN universal testing machine.

C. Petrus; H.A. Hamid; A. Ibrahim (2010)

Experimental synopsis. An experimental investigation into the structural behavior of concrete filled, thin walled, steel tubular stub column with tab stiffeners was presented. The stiffening was attained by welding together four pieces of lipped angle, whereupon two parts of the lips were notched and folded vertically in order to form the tab stiffeners. Tab stiffeners were found increasing the ultimate capacity, improved the ductility and overcome the shortcoming of weak concrete confinement at the center of the sidewalls of the rectangular steel tubes. The push out test and compression test were conducted to determine the bond strength between the steel and concrete interface and the axial load capacity.

Main test parameters. The primary parameters studied were the tab stiffeners spacing and the behavior of different types of stiffeners.

Number of tests. 10 specimens, including 6 specimens with tab stiffeners and 4 specimens with longitudinal stiffeners were prepared for the compression test.

Steel properties. Steel tubes were made from mild steel sheeting by cutting and pressing to form a lipped equal angle of 100 mm×100 mm width×600 mm long and 25 mm lips height. Finally, a square tube with two longitudinal stiffeners and two tab stiffeners was produced by seam welding together two pieces of lipped channel with tab stiffeners. Steel tubes with four longitudinal stiffeners were also fabricated by seam welding four pieces of lipped angle section for comparison purposes.

- ***Tube size B and D (mm):-*** tubes size were 200 mm x 200 mm.
- ***Wall thickness t (mm):-*** wall thickness was 2 mm.
- ***Yield strength of steel f_y (MPa):-*** a yield strength were 236 and 300 MPa.

Concrete properties. Normal strength concrete with a water–cement ratio of 0.5 was used for the infill. The compressive strength of the concrete was determined from three 150 mm cubes taken from each batch of concrete.

- ***Compressive strength of concrete (MPa):-*** The average concrete cube strengths (f_{cu}) at 28 days of curing age were 40 MPa and 36 MPa.

Effective length (mm). The effective length of the specimens were 550 mm.

End condition. Pinned-pinned under concentric compression. Prior to testing, the top and bottom surfaces of the concrete filled columns were cut and ground smooth. A 50 mm thick steel loading pad was placed at the top end between the specimen and the loading surface of the testing machine.

Loading method. A 2000 kN compression capacity universal testing machine was used with Load increment of less than one tenth of the estimated load capacity.

D. Liu; W.-M. Gho; J. Yuan (2003)

Experimental synopsis. The ultimate capacity of high-strength rectangular concrete-filled steel hollow section (CFSHS) stub columns was experimentally investigated with different cross sectional aspect ratios under axial concentric loading. It is noted that the strength of specimens decreases with the increase of cross-sectional aspect ratios. It is shown that the codes (EC4, AISC and ACI) have underestimated the ultimate capacity of test specimens. EC4 predicts closely with a difference of 6% while AISC and ACI underestimated the critical loads by 16 and 14%, respectively. The primary reason for the discrepancy of results is that the strength enhancement of CFSHS columns due to the confinement of core concrete by steel hollow section is considered in EC4 but not in AISC and ACI.

Main test parameters. The primary parameters studied were cross-sectional aspect ratios B/H, compressive strength of concrete, yield strength of steel, volumetric steel-to-concrete ratio.

Number of tests. 22 rectangular CFSHS specimens

Steel properties. The steel hollow sections were fabricated by welding the four steel flat plates together. The specimens were considered to be compact during loading and strength reduction due to local buckling of the steel hollow section was not expected.

- ***Tube size B and D (mm):-*** width of the tubes were between 100.3 and 202.2 mm, while depth of the tubes were between 80.1 and 180.4 mm.

- ***Wall thickness t (mm):-*** wall thickness was 4.18 mm. - -----

- Yield strength of steel f_y (MPa):- the steel plate were tested under the requirement of Chinese testing standard GB2975. The yield strength were found to be 550 MPa.

Concrete properties. Two different mix designs of high-strength concrete were used for the test specimens. -

Compressive strength of concrete f_{ck} (MPa):- The cylindrical strengths were determined as being 60.8 and 72.1 MPa.

Effective length (mm). The effective length of the specimens were 300, 360, 480, 540 and 600 mm.

End condition. The platens of the test rig were firmly attached at both ends of the specimen. The welds at the four corners of the specimen at the bottom remained intact before the critical load.

Loading method. All the specimens were tested to failure using the 5000-kN capacity test rig. The concentric compressive load was applied centrally to the specimens.

D. Liu; W.-M. Gho (2005)

Experimental synopsis. Axial load behavior of rectangular concrete-filled steel tubular (RCFT) stub columns were investigated. A comparison of axial load capacity between the tests and the design codes was shown and EC4 overestimates the ultimate capacity of the specimens fabricated from mild steel and high-strength concrete. On the other hand ACI and AISC give safe estimation by 7 and 8%, respectively.

Main test parameters. The test parameters were material strengths, volumetric steel-to-concrete ratio (A_s/A_c) and cross-sectional aspect ratio (H/B).

Number of tests. A total of 26 RCFT specimens

Steel properties. The steel tube was fabricated by welding four component plates together. Two grades (Grades A and B) of steel plates were used.

- ***Tube size B and D (mm):-*** width of the tubes were between 100 and 200 mm, while depth of the tubes were between 120 and 220 mm.
- ***Wall thickness t (mm):-*** The tubes wall thickness were 4.0 and 5.8 mm.
- ***Yield strength of steel f_y (MPa):-*** Three coupons were cut from each grade and machined according to the requirements in ASTM. They were tested at a loading rate of 0.5 mm/ min before the yield load and 2.0 mm/min after the yield load. The average yield stress, tensile strength and modulus of elasticity for Grade A were determined as 495, 581 and 206,000 MPa, respectively. For steel plate of Grade B, they were 300, 474 and 203,000 MPa, respectively.

Concrete properties. The concrete was mixed in three batches. The concrete cubes and cylinders were tested at a loading rate of 330 and 265 kN/min, respectively. Each layer of concrete was compacted using a poker vibrator. The concrete was cured inside the steel tube for 21 days. As the concrete shrank over time, a layer of high-strength cement mortar was filled to flush the concrete core with the steel tube.

- ***Compressive strength of concrete f_{ck} (MPa):-*** The cylindrical strengths were determined as being 55, 83 and 106 MPa.

Effective length (mm). The effective length of the specimens were between 360 and 660 mm.

End condition. The platens of the testing machine were firmly attached to both ends of the specimen.

Loading method. The axial concentric compression load was applied by a 5000-kN capacity Instron testing machine at a loading rate of 0.3 mm/min.

Ge, H.; Usami, T. (1992)

Experimental synopsis. An experimental study on the strength and deformation of concrete filled square box stub-columns is presented. Six specimens of concrete-filled composite columns were tested under cyclic compressive loads. For comparison, four specimens of steel columns were also loaded to failure. The ultimate strength, ductility, and collapse behavior of the two types of columns were compared. In the comparison, the effect of width thickness ratio and stiffener rigidity on the behavior of the columns was examined. The study showed that high strength and high ductility can be expected from the concrete-filled composite column.

Main test parameters. The test parameters were concrete strengths, width to thickness (B/t).

Number of tests. 6 RCFT specimens.

Steel properties. The steel used was mild steel.

- ***Tube size B and D (mm):-*** width of the tubes were between 197 and 329 mm.

- ***Wall thickness t (mm):-*** The tubes wall thickness was 4.5 mm.

- ***Yield strength of steel f_y (MPa):-*** The mechanical properties were determined by using tensile-coupon tests. Yield stress of the plate 266 MPa.

- ***Initial out of straightness: -*** The initial plate deflections of two flange surfaces of each specimen were measured using a dial gage with an accuracy of 0.002 mm

Concrete properties. The concrete used had a water/cement ratio of 55.2% (by weight) for ordinary concrete, made' with ordinary port-land cement; and 31.6% (by weight) for high-performance concrete made with low-heat cement. The aggregate was well graded with a maximum size of 20 mm. the modulus of elasticity was determined to be between 27.9 and 31.5 GPa.

- ***Compressive strength of concrete f_{ck} (MPa):-*** The cylindrical strengths were determined as being between 39.2 and 48.3 MPa.

Effective length (mm). The effective length of the specimens were between 592 and 988 mm.

End condition. The concrete surface at each end of the concrete-filled column was roughened by chipping and then capped with a thin layer of neat cement paste so that the steel and concrete could be loaded uniformly.

Loading method. Concentric compression.

L.-H, Han. (2001)

Experimental synopsis. The behavior of stub columns of concrete-filled rectangular hollow sections (RHS) subjected to axial load was investigated experimentally. The influence of constraining factor and width ratio on the behavior of stub concrete-filled RHS columns were analyzed. Accuracy of the recommendations of LRFD (AISC, 1994), (AIJ, 1997), (EC4, 1996) and GJB4142- 2000 predictions were compared to the experimental values.

Main test parameters. The main parameters varied in the tests are: constraining factor from 0.5 to 1.3 and tube width ratio from 1.0 to 1.75.

Number of tests. A total of 24 concrete filled RHS tubes specimens were studied.

Steel properties. The tubes were all manufactured from mild steel sheet, with four plates cut from the sheet, tack welded into a rectangular shape and then welded with a single bevel butt weld at the corners.

- ***Tube size B and D (mm):-*** width of the tubes was between 70 and 135 mm while the depth was between 90 and 150 mm.

- ***Wall thickness t (mm):-*** The tubes wall thickness was 2.89 and 7.6 mm.

- ***Yield strength of steel f_y (MPa):-*** The tension coupons were tested in tension with the Chinese standard GB2975 and from these tests the average yield strengths were 194 and 228 MPa.

Concrete properties. The modulus of elasticity of concrete measured in accordance with the Chinese standard GBJ81-85 was 29,200 MPa.

- ***Compressive strength of concrete (MPa):-*** The compressive cube strength was 59.3 MPa.

Effective length (mm). The effective length of the specimens were between 300 and 450 mm.

End condition. Each tube was welded to a rectangular steel base plate 25 mm thick. The top surfaces of the concrete-filled steel tubes were ground smooth and flat.

Loading method. Axial compressive tests were performed under a 2000 kN capacity testing machine at loading interval of less than one-tenth of the estimated load capacity for about 2–3 min.

Guo, L.H., Zhang, S., Kim, W.J., Ranzi, G. (2007)

Experimental synopsis. The effect of local buckling in bare steel and concrete-filled tubes and the effect of depth-to-thickness ratios on the response of the steel component was studied. The presence of the concrete has been observed to affect the exhibited buckling mode and to significantly increase the buckling bearing capacity of the concrete-filled steel tubes.

Main test parameters. The main parameters varied in the tests are: constraining factor from 0.5 to 1.3 and tube width ratio from 1.0 to 1.75.

Number of tests. A total of 24 concrete filled RHS tubes specimens were studied.

Steel properties. The tubes were all manufactured from mild steel sheet, with four plates cut from the sheet, tack welded into a rectangular shape and then welded with a single bevel butt weld at the corners.

- ***Tube size B and D (mm):-*** width of the tubes was between 70 and 135 mm while the depth was between 90 and 150 mm.

- ***Wall thickness t (mm):-*** The tubes wall thickness was 2.89 and 7.6 mm.

- ***Yield strength of steel f_y (MPa):-*** The tension coupons were tested in tension with the Chinese standard GB2975 and from these tests the average yield strengths were 194 and 228 MPa.

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Effective length (mm). The effective length of the specimens were between 300 and 450 mm.

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Loading method. Axial compressive tests were performed under a 2000 kN capacity testing machine at loading interval of less than one-tenth of the estimated load capacity for about 2–3 min.

Han, L.H., Liu, W., Yang, Y.F (2007)

Experimental synopsis. The behavior of concrete-filled steel tubular (CFST) stub columns subjected to axially local compression was experimentally investigated. The influences of sectional type, local compression area ratio, thickness of the top endplate on the behavior of locally-loaded CFST specimens have been studied.

Main test parameters. Sectional type, local compression area ratio, thickness of the top endplate.

Number of tests. 16 specimens were tested.

Steel properties. The tubes were all manufactured from mild steel sheet, with the plate being cut from the sheet, tack welded into a square shape and then welded with a single bevel butt weld. The Poisson's ratio (μ_s) of the steel was 0.274. Modulus of elasticity (E_s) were 214,000 MPa.

- ***Tube size B and D (mm):-*** width and depth of the tubes was 177 mm.

- ***Wall thickness t (mm):-*** The tubes wall thickness was 2.83 mm.

- ***Yield strength of steel f_y (MPa):-*** the average yield strength (f_y) and tensile strength (f_u) were 362.9 MPa and 449.8 MPa respectively.

Concrete properties. A kind of self-consolidating concrete (SCC) mix, with the measured modulus of elasticity (E_c) of concrete was measured, and the average value was 35,300 MPa.

- ***Compressive strength of concrete (MPa):-*** compressive cube strength (f_{cu}) was approximately 74.3 MPa.

Effective length (mm). The effective length of the specimens was 531 mm.

End condition. The size of the bearing plate was varied to obtain different local compression area ratio (β).

Loading method. All the tests were performed on a 5000 kN capacity testing machine. The concentric loads were applied on the specimens through the loading ram of the machine (for fully-loaded specimens) or the steel-bearing plate (for locally- loaded specimens). Each load interval was maintained for about 2–3 min.

Mursi, M., Uy, B. (2004)

Experimental synopsis. Thin walled steel sections utilizing high strength steel of a thin walled nature and filled with normal strength concrete was studied. The behavior of concrete filled steel slender columns (with compact, non-compact and slender sections) affected by elastic or inelastic local buckling was investigated and compared with relevant experimental results. The load deflection response was recorded.

Main test parameters. B/t ratio, length of the specimen.

Number of tests. 8 specimens were tested.

Steel properties. All composite and hollow columns were formed by welding steel plates together. A single run of fillet weld was laid along each fillet for each column. All composite and hollow columns were formed by welding steel plates together.

- ***Tube size B and D (mm):-*** width and depth of the tubes was 177 mm.
- ***Wall thickness t (mm):-*** The tubes wall thickness was 2.83 mm.
- ***Yield strength of steel f_y (MPa):-*** The design of the material property tests was carried out according to the specifications of ASTM A 370-97 and AS 1391 for the tensile properties of steel. In the conducted tests, the yield stress (f_y) and the ultimate stress (f_u) were determined. The mean yield stress of the steel in tension was 761 N/mm2.

Concrete properties. Concrete was placed manually into the columns and vibrated simultaneously. -

Compressive strength of concrete (MPa):- Concrete with a nominal compressive strength of 20 N/mm^2 was used.

Effective length (mm). The effective length of the specimens was between 430 and 3020 mm.

End condition. The short columns were tested under concentric load and fixed supports at both ends. For the slender column tests, cylinder seats were used at either end to ensure the rotations in one direction.

Loading method. Each of the tests were conducted in a self-straining rig with a 5000 kN capacity loading jack.

Song, Kwon, J.Y, Bong, Y (1997)

Experimental synopsis. An experimental study on the behavior of concrete-filled steel box stub columns was performed. The result of the test showed composite box columns had high ductility as well as high strength due to mutual confinement between concrete and steel plate.

Main test parameters. B/t ratio, length of the specimen.

Number of tests. 8 specimens were tested.

Steel properties..

- ***Tube size B and D (mm):-*** width and depth of the tubes was between 130 and 220 mm.
- ***Wall thickness t (mm):-*** The tubes wall thickness was 3.2 and 3.0 mm.
- ***Yield strength of steel f_y (MPa):-*** The tensile coupon test was performed to determine the mechanical properties of the steel used (yield stress f_y = 313.82 MPa).

Concrete properties. The cylinders were made with a water/cement ratio of 50% from ordinary Portland cement and well graded aggregate (maximum size 19mm) and were cured for 28 days until the column specimens were tested.

- ***Compressive strength of concrete (MPa):-*** Concrete with a nominal compressive strength of 25.4 MPa was used.

Effective length (mm). The effective length of the specimens was between 390 and 660 mm.

End condition. To assure uniform compression and prevent the eccentricity, very thick loading plates (t=40 mm) were attached at each end (top and bottom) of test specimens.

Loading method. The loading process was paused at every step of 5 tons for a minute.

B. Uy, M. Khan, Z. Tao, F. Mashiri (2013)

Experimental synopsis. The behavior and design of high strength steel-concrete composite columns used in major infrastructure engineering systems was considered and an experimental study of the use of high strength steel-concrete composite columns which evaluates the in-plane residual stresses was presented.

Main test parameters. B/t ratio, strength of concrete.

Number of tests. 10 specimens were tested.

Steel properties.

- ***Tube size B and D (mm):-*** width and depth of the tubes was between 75 and 200 mm.

- ***Wall thickness t (mm):-*** The tubes wall thickness were 5 mm.

- *Yield strength of steel f_y (MPa):-* The yield strength of the steel was 690 MPa.

Concrete properties. -

Compressive strength of concrete (MPa):- The cylindrical compressive strength of concrete was between 80 and 95 MPa.

Effective length (mm). The effective length of the specimens was between 262.5 mm and 700 mm.

End condition. The support condition was fixed end.

Loading method. The monotonic loading was applied to the specimens.

D.M; Lue, J-L; Liu, T, Yen (2006)

Experimental synopsis. The study was aimed to assess if the LRFD CFT column formulas are applicable to intermediate to long rectangular columns with higher concrete strengths. The design CFT strength (P_u) predicted by the AISC-LRFD formulas and the test results (P_{test}) were found to be in good agreement. The comparisons indicated that the 2005 AISC-LRFD was more conservative than the 1999 AISC-LRFD. The design strength of a CFT column with high-strength concrete determined by the 2004 EC 4 is more conservative than that from the AISC-LRFD (1999, 2005).

Main test parameters. Concrete strength.

Number of tests. 22 specimens were tested.

Steel properties.

- ***Tube size B and D (mm):-*** width the tubes was 150mm and depth of the tubes was 100 mm.

- ***Wall thickness t (mm):-*** The tubes wall thickness was 4.5 mm.

- *Yield strength of steel f_y (MPa):-* The mean yield strength f_y = 379.8 MPa.

Concrete properties. Concrete strengths and concrete elastic moduli were obtained based on ASTM-C39 and ASTM-C469 procedures.

- ***Compressive strength of concrete (MPa):-*** The sections in each group are filled with approximate concrete strength f_{ck} of 29, 63, 70, and 84 MPa, respectively

Effective length (mm). The effective length of the specimens was 1855mm.

End condition. Two bearing plates (340 × 340 × 20 mm) were welded at the top and bottom ends of each specimen with eight spot-welded stiffeners to provide the rigidity at the specimen ends and to make sure that the plane remains plane at the ends of the specimen when the rotation occurs at the onset of buckling.

Loading method. The concentric axial compression load were conducted through the use of the 600-metric ton MTS machine.

F.R; Mashiri, B. Uy, Z.Zhao (2015)

Experimental synopsis. The behavior of fabricated section stub columns made through welding facet plates but also incorporating small diameter circular hollow sections at the vertices was studied. The CHS-plate square stub columns failed through outward buckling of the facet plates and local buckling of the CHS tubes in the vicinity of the facet plate buckle. Eurocode 4 were found to be the most suitable predictors of peak experimental load for the fabricated CHS-plate square stub columns under axial compression.

Main test parameters. Concrete strength.

Number of tests. 6 specimens were tested.

Steel properties. CHS-plate stub columns are manufactured by welding mild steel facet plates to cold-formed tubes at the vertices The tubes used in the fabrication of the stub columns are cold-formed high strength steel with a and a nominal ultimate strength (funt) of 430MPa.

- ***Tube size B and D (mm):-*** width the tubes was between 90 and 150mm.
- ***Wall thickness t (mm):-*** The tubes wall thickness was 3.0 mm.
- ***Yield strength of steel f_y (MPa):-*** f_y is 350MPa.

Concrete properties. -

Compressive strength of concrete (MPa):- concrete strength f_{ck} of 32MPa was used.

Effective length (mm). The effective length of the specimens was between 300 and 500 mm.

End condition. The end plates are of grade 250, and 10mm in thickness. The end plates are welded to form the stub columns using the pulsed metal arc welding process.

Loading method. A 3000 kN capacity Instron testing machine was employed.

Cai, J., Long, Y-L (2007)

Experimental synopsis. Experimental study on the axial load behavior of rectangular concrete-filled steel tubular (R-CFT) stub columns with binding bars was presented. Experimental results indicate that the binding bars increase the confinement of the concrete core and delay local buckling of the tube, ductility of the tube than those without binding bars. A comparison of the ultimate strengths between tests and design codes shows that EC4 (1996), conservatively estimate the ultimate strength by 17.6%.

Main test parameters. Depth-to-thickness ratio D/t; horizontal spacing of binding bars; diameter of binding bars; cross-sectional aspect ratio D/B.

Number of tests. 8 specimens were tested.

Steel properties. The rectangular steel tubes were fabricated by welding together four pieces of steel plate. Holes in the steel plates were drilled to accommodate the binding bars.

- ***Tube size B and D (mm):-*** width of the tubes was 150 mm while the depth was 300 mm.
- ***Wall thickness t (mm):-*** The tubes wall thickness was 4.0, 6.0 and 10.0 mm.
- ***Yield strength of steel f_y (MPa):-*** f_y is 465 MPa.

Concrete properties.

- ***Compressive strength of concrete (MPa):-*** the cylindrical compressive concrete strength f_{ck} of 32.4 MPa was used.

Effective length (mm). The effective length of the specimens was 1200 mm.

End condition. The specimen had a stiffened footing and a stiffened cap at both column ends to ensure that the concrete core and the steel tube were loaded simultaneously.

Loading method. The tests were carried out in a universal testing machine with a capacity of 15,000 kN. Concentrically compressive axial loads were applied to the specimens at a rate of 100 kN/min in the initial elastic stage. After the load–displacement curve became nonlinear, automatic displacement–load computer control was adopted at a rate of 0.5 mm/min.

Cai, J., He, Z.Q. (2005)

Experimental synopsis. The axial load behavior of square concrete filled steel tubular (S-CFT) stub columns with binding bars provided to improve the mechanical behavior of S-CFT columns was studied. Ten specimens with binding bars and 5 specimens without binding bars were tested to examine the effects of width-to-thickness ratios and binding bars on ultimate strength, stiffness and ductility of S-CFT columns. A prediction by the methods of EC4(1996) scattered from the experimental results for square concrete filled steel tubular with binding bars.

Main test parameters. Width-to-thickness ratios and binding bars on ultimate strength, stiffness and ductility of S-CFT columns.

Number of tests. 15 specimens were tested.

Steel properties. The square steel tubes were fabricated by welding together four pieces of flat plates. Before assembling the tube, holes on the steel plates were made by drilling where the binding bars are arranged. Each binding bar joins together with the steel tube through two square stiffening steel plates on its two ends by welding.

- ***Tube size B and D (mm):-*** width and depth of the tubes was 300 mm.
- ***Wall thickness t (mm):-*** The tubes wall thickness was 6.0, 8.0 and 12.0 mm.
- ***Yield strength of steel f_y (MPa):-*** f_y was 292.48, 382.5 MPa.

Concrete properties.

- ***Compressive strength of concrete (MPa):-*** the cylindrical compressive concrete strength f_{ck} of 39.82 MPa was used.

Effective length (mm). The effective length of the specimens was 1500 mm.

End condition. The specimen had a stiffened cap at both column ends.

Loading method. The compression tests were carried out in a 15 000 kN universal testing machine.

Z., Tao, B., Uy, L.-H., Han

Experimental synopsis. The design of stiffened concrete filled thin walled steel tubular columns were studied. The limit of width-to-thickness ratio for the sub-panels and the rigidity requirement for the stiffeners was discussed. The feasibility using existing design codes to predict the load-carrying capacities of the stiffened composite columns was also dealt.

Main test parameters. Concrete strength.

Number of tests. 12 specimens were tested.

Steel properties. The steel type was mild steel sheets with an elastic modulus of Es=207 GPa,

- ***Tube size B and D (mm):-*** width and depth of the tubes was 300 mm.

- ***Wall thickness t (mm):-*** The tubes wall thickness was 2.5 mm.

- ***Yield strength of steel f_y (MPa):-*** Tension tests on three coupons were conducted. The average yield strength f_y was 338 MPa.

Concrete properties. Four different concrete mixes were used in the test program. They had a water/cement ratio of 0.48 and 0.7, respectively, made with ordinary Portland cement. The maximum size of coarse aggregate was 15 mm.

- ***Compressive strength of concrete (MPa):-*** the cube compressive strength f_{cu} were 25.5, 50.8, 51.2 MPa.

Effective length (mm). The effective length of the specimens was 1500 mm.

End condition. The specimen had a stiffened cap at both column ends.

Loading method. The compression tests were carried out in a 15 000 kN universal testing machine.

4.3 CIRCULAR BEAM-COLUMNS (CCFT)

Kilpatrick, A. and Rangan B. V., 1997 (bc; m)

Introduction. Tests were conducted on circular high strength CFTs under eccentrically applied axial load. In addition, a deformation control method of analysis was described to estimate the strength and load-deformation response of the test specimens. The analytical and experimental results were compared.

Experimental Study Results and Discussions. The main parameter of the experimental study was the amount of eccentricity in the applied axial load. The measured yield strength of the steel tubes was 59.47 ksi (410 Mpa). The specimens filled with high strength concrete having a measured compressive strength of 13.92 ksi (95.97 Mpa). The *L/D* and *D/t* ratios were 21.43 and 42.29, respectively. The columns were tested both in double curvature and single curvature. The eccentricity at the bottom and top of the specimens were different in most of the cases. The ends of the columns were clamped to hardened knife-edge assemblages. The required eccentricity was provided by moving the column ends laterally from the loading axis.

Analytical Study. The experimental results were compared to analytical results using a classic Newmark iteration scheme based on use of moment-curvature-thrust relations. In general, the calculated strength and force-deflection response of the specimens matched with the experimental results quite accurately. However, the response of the specimens under single curvature showed better correlation than the specimens tested under double curvature. Analytically generated strength envelopes for equal eccentricity at the top and bottom showed a greater rate of increase and a higher concentric axial load capacity in the case of double curvature.

The authors also investigated the effect of variation in eccentricity on the column strength. For this purpose, columns in double curvature having equal magnitude of eccentricities at the bottom and at the top were analyzed with ±0.0394 in. error. In the first case, the eccentricity at the top and bottom was increased by 0.0394 in.. In the second case, the eccentricity at the top decreased by 0.0394 in. while the eccentricity at the bottom was increased by 0.0394 in.. It was found that the strength decreased most severely in the latter condition, with the amount of reduction being up to 25%. This was attributed to the change in the deformed shape as it was not perfectly asymmetrical any more.

J. Zeghiche, K. Chaoui (2004)

Experimental synopsis. Circular concrete-filled steel tubes were tested in single and double curvature bending. The carrying capacity of eccentrically loaded columns in single curvature bending is affected by the increase of the load eccentricity. The steel yielding process started first in the compression zone for small eccentricities and reached the tension zone for columns with higher eccentricities. The test results demonstrated the influence of slenderness, eccentricity and strength of concrete on the strength and behavior of concrete-filled steel tubular columns.

Main test parameters. The test parameters were the column slenderness, the load eccentricity covering axially and eccentrically loaded columns with single or double curvature bending and the compressive strength of the concrete core.

Number of tests. 8 circular CFT in single curvature bending and 4 circular CFT in double curvature bending were tested.

Steel properties. The modulus of elasticity of steel was found to be 212 000 MPa.

- ***Tube size B and D (mm):-*** The diameter of the specimen was 160 mm.
- ***Wall thickness t (mm):-*** wall thickness was 5 mm.
- ***Yield stress of steel f_y (MPa):-*** The average tensile strength was 275 MPa.

Concrete properties. -

Compressive strength of concrete (MPa):- The average cylindrical strength of concrete were determined as 100 MPa.

Eccentricity (mm). The eccentricity of the load varied from 8 mm to 32 mm.

Effective length (mm). The effective length of the specimens were 2000 or 4000 mm.

End condition. A set of adapter endplates equipped with half-spherical bearings were manufactured and fixed to both ends of each column to form a simply supported column.

Loading method. Columns were subjected to uniaxial eccentric loading.

L.H. Han, G.H. Yao (2004)

Experimental synopsis behavior of thin walled circular hollow structural steel columns filled with self-consolidating concrete has been investigated.

Main test parameters. A main parameter varied in the tests was load eccentricity, D/t ratio.

Number of tests. 5 beam columns were tested.

Steel properties. The tubes were all manufactured from a mild steel sheet, with plates were cut from the sheet, tack welded into circular shape and then welded with a single bevel butt weld. The modulus of elasticity was about 206,500 MPa

- ***Tube diameter D (mm):-*** The diameter of the specimens was 200 mm.
- ***Wall thickness t (mm):-*** wall thickness was 3 mm.
- ***Yield stress of steel f_y (MPa):-*** the average yield strength of the tube was found to be 303.5 MPa.

Concrete properties. The modulus of elasticity (Ec) of the concrete was measured, the average value being 37,420 MPa. -

Compressive strength of concrete (MPa):- The average cube strength of concrete were determined as 58.5 MPa.

Eccentricity (mm). The eccentricity of the load was 60 mm.

Effective length (mm). The effective length of the specimens was 2000 mm.

End condition. The axial load was applied through a very stiff top platen with an offset triangle hinge, which also allowed specimen rotation to simulate pin-ended supports

Loading method. The desired eccentricity was achieved by accurately machining grooves 6 mm deep into the stiff endplate that was welded together with the steel tubes.

Y.C. Wang (1998)

Experimental synopsis behavior of thin walled circular hollow structural steel columns filled with self-consolidating concrete has been investigated.

Main test parameters. A main parameter varied in the tests was load eccentricity, D/t ratio.

Number of tests. 5 beam columns were tested.

Steel properties. The tubes were all manufactured from a mild steel sheet, with plates were cut from the sheet, tack welded into circular shape and then welded with a single bevel butt weld. The modulus of elasticity was about 206,500 MPa

- ***Tube diameter D (mm):-*** The diameter of the specimens was 200 mm.
- ***Wall thickness t (mm):-*** wall thickness was 3 mm.
- ***Yield stress of steel f_y (MPa):-*** the average yield strength of the tube was found to be 303.5 MPa.

Concrete properties. The modulus of elasticity (Ec) of the concrete was measured, the average value being 37,420 MPa. -

Compressive strength of concrete (MPa):- The average cube strength of concrete were determined as 58.5 MPa.

Eccentricity (mm). The eccentricity of the load was 60 mm.

Effective length (mm). The effective length of the specimens was 2000 mm.

End condition. The axial load was applied through a very stiff top platen with an offset triangle hinge, which also allowed specimen rotation to simulate pin-ended supports

Loading method. The desired eccentricity was achieved by accurately machining grooves 6 mm deep into the stiff endplate that was welded together with the steel tubes.

Y.F. Yang, L.H. Han (2011)

Experimental synopsis. The behavior of circular concrete filled steel tubular (CCFST) stub columns subjected to eccentric partial compression was investigated. The test results indicated that, similar to the corresponding fully loaded CFST stub columns under eccentric loading, CFST stub columns under eccentric partial compression have generally reasonable bearing capacity and favorable ductility.

Main test parameters. The main parameters in test program include, load eccentricity ratio (including uniaxial and biaxial loading and shape of the loading bearing plate, circular, square, strip and rectangular).

Number of tests. 3 circular beam columns were tested.

Steel properties. The modulus of elasticity and Poisson's ratio were 206,000 MPa and 0.281, respectively.

- ***Tube diameter D (mm):-*** The diameter of the specimens was 150 mm.
- ***Wall thickness t (mm):-*** wall thickness was 3.0 mm.
- ***Yield stress of steel f_y (MPa):-*** the average yield strength of the tube was found to be 324.4 MPa.

Concrete properties. The modulus of elasticity of concrete was measured by testing three 150x300 mm prisms, and the average value was 33,600 MPa.

- ***Compressive strength of concrete (MPa):-*** The average cylindrical strength of concrete were determined as 59.3 MPa.

Eccentricity (mm). The eccentricity of the load was 15 or 30 mm.

Effective length (mm). The effective length of the specimens was 450 mm.

End condition. The columns were simply supported at both ends about both axes.

Loading method. Specimens were loaded eccentrically on partial cross-sectional area.

T. Perea, R.T. Leon, J.F. Hajjar, M.D. Denavit (2014)

Experimental synopsis. The behavior of slender circular concrete-filled steel tubes (CCFTs) under combined axial compression and biaxial flexure has been investigated. The experimental determination of the maximum stable axial load–bending moment (P−M) interaction strength has been addressed. The experimental result showed that for very slender specimens, the bilinear interaction diagram proposed in the provisions of the AISC 2010 is somewhat un-conservative and conservative for most practical CFT column sizes and lengths.

Main test parameters. The main parameters considered were length of the columns, strength of concrete, eccentricity and D/t ratio.

Number of tests. 10 circular beam columns were tested.

Steel properties. The modulus of elasticity was 202,400 MPa.

- ***Tube diameter D (mm):-*** The cross-section of the specimens was 141.3 mm, 323.9 mm or 508 mm.

- ***Wall thickness t (mm):-*** wall thickness was 6.40 mm or 3.40 mm.

- *Yield strength of steel f_y (MPa):-* The yield strength of the tube range from 365 or 406 MPa.

Concrete properties. The modulus of elasticity of concrete was in a range of 27,600 MPa to 41,900 MPa.

- ***Compressive strength of concrete (MPa):-*** The average cylindrical strength of concrete was between 37.9 MPa and 87.6 MPa.

Effective length (mm). The effective length of the specimens was 5,550 mm or 7,960 mm.

End condition. The base was fully fixed and the top rotations were under load control.

Loading method. A complex loading protocol was used in the experimental program, including monotonic and cyclic loading that allowed detailed evaluation of the complete beam-column response.

K. Tsuda, C. Matsui, E. Mino

Experimental synopsis. Concrete filled circular tubular columns were tested. Test is composed of two Series. In Series I, columns are subjected to concentric and eccentric axial force at both ends. In Series II, columns are cantilever columns, and subjected to alternating horizontal load under constant vertical load. Strength and behavior were examined, and design methods for slender composite columns were investigated.

Main test parameters. As a main experimental parameter, buckling length and section depth ratio of a column was selected. .

Number of tests. 10 circular beam columns were tested.

Steel properties. -

Tube diameter D (mm):- The diameter of the specimens was 165.2 mm.

- ***Wall thickness t (mm):-*** wall thickness was 4.5 mm.

- *Yield strength of steel f_y (MPa):-* The yield strength of the tube was 353 MPa.

Concrete properties.

- ***Compressive strength of concrete (MPa):-*** The average cylindrical strength of concrete was 34.7

MPa or 41 MPa.

Effective length (mm). The effective lengths of the specimens were between 660 mm and 4,950 mm.

End condition. Exact pin-ended conditions are obtained because the specimens are loaded through hemispherical oil film bearing at each end.

Loading method. The assigned eccentricity e is given to the specimen by moving the bearing plates. For series I, Axial load in one direction is applied to a specimen. For series II, Specimens are tested under the cyclic loading. The vertical load N was applied to a specimen and kept constant value assigned in the test program during horizontal loading process. The horizontal load was applied to the top of a specimen by a hydraulic jack.

A.Elremaily, A. Azizinamini (2002)

Experimental synopsis. The behavior of concrete-filled tube columns under seismic loads was studied. The test columns showed high ductility and maintained their strength up to the end of the test. The test results also indicate that the column capacity was significantly improved due to the concrete strength gained from the confinement provided by the steel tube.

Main test parameters. The test parameters included the level of axial load, the diameter-to-thickness ratio of the steel tube, and the concrete compressive strength. .

Number of tests. 6 circular beam columns were tested

Steel properties. -

Tube diameter D (mm):- The diameter of the specimens was 324 mm.

- ***Wall thickness t (mm):-*** wall thickness was 6.4 mm or 9.5 mm.

- ***Yield strength of steel f_y (MPa):-*** The yield strength of the tube was 372 MPa.

Concrete properties.

- ***Compressive strength of concrete (MPa):-*** The average cylindrical strength of concrete ranged between 34.0 MPa or 103 MPa.

Effective length (mm). The effective lengths of the specimens were 914 mm.

End condition. The specimens were connected to rigid reaction frame at the top and by rigid stub made of concrete filled tube to simulate the effect of rigid floor system. .

Loading method. The columns were subjected to a constant axial load in addition to a cyclic lateral load.

G. Muciaccia, F. Giussani, G. Rosati, F. Mola (2010)

Experimental synopsis. The study of a Normal Vibrated Concrete, a Self-Compacting Concrete, and an expansive SCC for structural applications as composite elements was presented. The experimental and analytical results showed that the behavior of eccentrically loaded columns is governed by the bending moment–axial load interaction.

Main test parameters. The parameters were L/D ratio, D/t ratio.

Number of tests. 6 circular beam columns were tested

Steel properties. -

Tube diameter D (mm):- The diameter of the specimens was 139.6 mm.

- ***Wall thickness t (mm):-*** wall thickness was 4.0 mm.

- ***Yield strength of steel f_y (MPa):-*** The yield strength of the tube was 374 MPa.

Concrete properties.

- ***Compressive strength of concrete (MPa):-*** The average cylindrical strength of concrete was 62 MPa.

Effective length (mm). The effective lengths of the specimens ranged between 1,310 mm to 4,670 mm.

Eccentricity (mm). Fixed eccentricity of the applied load was equal to 25 mm.

End condition. The ends were connected to fixed platens under vertical displacement control. To reach a stable control after the peak load it was chosen to hinge the column at both end. .

Loading method. The columns are loaded in compression with a fixed nominal eccentricity and a nominal critical length.

J.M. Portoles, M.L. Romero, F.C. Filippou (2011)

Experimental synopsis. Tests on slender circular tubular columns filled with normal and high strength concrete subjected to eccentric axial load were described. The advisability of the use of high strength concretes as opposed to that of normal strength concretes by comparing concrete contribution ratio, strength index and ductility index, the test result with EC4 was established.

Main test parameters. The test parameters were the nominal strength of concrete, the diameter to thickness ratio D/t, the eccentricity ratio e/D and the column slenderness (L/D).

Number of tests. 37 circular beam columns were tested.

Steel properties. The cold formed and welded steel tubes were employed.

- ***Tube diameter D (mm):-*** The diameter of the specimens was 100 mm, 125 mm or 160 mm.
- ***Wall thickness t (mm):-*** wall thickness was 3.0, 5.0 or 6.0 mm.
- ***Yield strength of steel f_y (MPa):-*** The yield strength of the tube was 322 MPa.

Concrete properties.

- ***Compressive strength of concrete (MPa):-*** The average cylindrical strength of concrete ranged between 32.70 and 98.5 MPa.

Effective length (mm). The effective lengths of the specimens were 2,135 mm or 3,135 mm.

Eccentricity (mm). Eccentricity of the applied load was equal to 20 mm or 50 mm.

End condition. The support conditions were hinged.

Loading method. Depending on the length, some of the columns were tested in the vertical setup and others in the horizontal setup. The eccentricity of the applied compressive load was equal at both ends, so the columns were subjected to single curvatures.

J.M. Portoles, E. Serra, M.L. Romero (2013)

Experimental synopsis. Tests on slender circular tubular columns filled with normal, high, and ultra-high strength concrete for plain column were conducted. The influence of each type of infill was established and for the limited cases analyzed the results show that the addition of high or ultra-high strength infill is more useful for concentric loaded cases than for eccentric loaded ones.

Main test parameters. The test parameters were nominal strength of concrete, eccentricity.

Number of tests. 15 circular beam columns were tested.

Steel properties. The cold formed and welded steel tubes were employed.

- ***Tube diameter D (mm):-*** The diameter of the specimens was ranged between 365 mm and 493.82 mm. - ----
- ***Wall thickness t (mm):-*** wall thickness was 6.0 mm.
- ***Yield strength of steel f_y (MPa):-*** The yield strength of the tube was between 394 MPa and 487

MPa.

Concrete properties.

- ***Compressive strength of concrete (MPa):-*** The average cylindrical strength of concrete ranged between 35.1 MPa and 120.1 MPa.

Effective length (mm). The effective lengths of the specimens were 2,135 mm.

Eccentricity (mm). Eccentricity of the applied load was equal to 20 mm or 50 mm.

End condition. The support conditions were hinged ends.

Loading method. Columns were subjected to both concentric and eccentric axial load.

L.H. Han, G.H. Yao (2003)

Experimental synopsis. Concrete-filled HSS columns with the steel tubes subjected to pre-load have been tested. Prediction of the load–deformation behavior of concrete filled HSS columns with the steel tubes subjected to pre-load was proposed.

Main test parameters. The main varying parameters in the tests are pre-load ratio; load eccentricity; and column slenderness.

Number of tests. 13 circular beam columns were tested.

Steel properties. The cold formed and welded steel tubes were employed.

- ***Tube diameter D (mm):-*** The diameter of the specimens was ranged between 365 mm and 493.82 mm.

- ***Wall thickness t (mm):-*** wall thickness was 6.0 mm.

- ***Yield strength of steel f_y (MPa):-*** The yield strength of the tube was between 394 MPa and 487 MPa.

Concrete properties.

- ***Compressive strength of concrete (MPa):-*** The average cylindrical strength of concrete ranged between 35.1 MPa and 120.1 MPa.

Effective length (mm). The effective lengths of the specimens were 2,135 mm.

Eccentricity (mm). Eccentricity of the applied load was equal to 20 mm or 50 mm.

End condition. The support conditions were hinged ends.

Loading method. Columns were subjected to both concentric and eccentric axial load.

L.H. Han, G.H. Yao (2005)

Experimental synopsis. Self-consolidating concrete filled circular hollow structural steel (SCCHSS) column specimens were tested under constant axial load and cyclically increasing flexural loading. It was found that SCCHSS columns exhibit very high levels of energy dissipation and ductility, particularly when subjected to high axial loads. The features of SCCHSS columns under constant axial load and cyclically increasing flexural loading are similar to those of normal concrete-filled HSS columns.

Main test parameters. The test parameters include the sectional types, the strength of concrete and steel and the axial load level.

Number of tests. 10 circular beam columns were tested.

Steel properties. The cold formed and welded steel tubes were employed.

- ***Tube diameter D (mm):-*** The diameter of the specimens was 200 mm.
- ***Wall thickness t (mm):-*** wall thickness was 3.0 mm.
- ***Yield strength of steel f_y (MPa):-*** The yield strength of the tube was 303.5 MPa.

Concrete properties.

- ***Compressive strength of concrete (MPa):-*** The average cylindrical strength of concrete was 40 MPa.

Effective length (mm). The effective lengths of the specimens were 2,000 mm.

Eccentricity (mm). Eccentricity of the applied load was equal to 30 mm.

End condition. The support conditions were hinged ends.

Loading method. The specimens was subjected to eccentric loading and the desired eccentricity was achieved by accurately machining grooves 6 mm deep into the stiff endplate that was welded together with the steel tubes.

4.4 RECTANGULAR BEAM-COLUMNS (RCFT)

A.H. Varma, J.M. Ricles, R. Sause, L.W. Lu (2004)

Experimental synopsis. The behavior of high strength square concrete filled steel tube beam columns subjected to constant axial load and cyclically varying flexural loading was investigated experimentally. Crushing of concrete and cyclic local buckling in the steel tube affected the cyclic strength and stiffness of the CFT specimens. The elastic section flexural stiffness under cyclic loading decreases rapidly due to tension cracking of the concrete infill and local buckling of the steel tube.

Main test parameters. The primary parameters studied were width-to-thickness (b/t) ratio, the yield stress of the steel tube, and the level of axial load.

Number of tests. 8 cyclic beam-column specimens were tested.

Steel properties. The steel specimens were made from either conventional A500 Grade-B; or high strength A500 Grade-80.

- ***Tube size B and D (mm):-*** tubes size were 305 by 305-mm.
- ***Wall thickness t (mm):-*** wall thickness was between 5.8 and 8.9 mm.
- ***Yield strength of steel f_y (MPa):-*** nominal yield stress were 317 and 552 MPa.

Concrete properties. The concrete material properties were determined by conducting uniaxial compression tests according to ASTM-1999b standards. The average measured values of the modulus of elasticity was 542 GPa.

- ***Compressive strength of concrete (MPa):-*** The average measured values of compressive strength was 110 MPa,

Effective length (mm). The effective length of the specimens were 1500 mm.

End condition. Fixed at the base of specimen and free at top.

Loading method. Each cyclic beam-column specimen was fixed at the base and subjected to a constant axial load P and cyclically varying lateral loading H at the top. The axial load P was applied and maintained constant by an axial loading arrangement, which consisted of a 9000 kN capacity hollow core hydraulic. The lateral load H was applied by imposing cyclically varying displacements under displacement control at the top of the test length by a 1000 kN capacity hydraulic ram.

D. Liu (2005)

Experimental synopsis. The behavior of high-strength rectangular concrete-filled steel tubular (CFT) columns subjected to eccentric loading was studied. Favorable ductility performance was observed for all specimens during the tests. Experimental failure loads were employed to calibrate the specifications in the design codes EC4, ACI and AISC. Results showed that EC4 overestimates the failure loads of the specimens by 4%. ACI and AISC conservatively predict the failure loads by 14% and 24%, respectively.

Main test parameters. The test parameters were material strengths, cross-sectional aspect ratio, slenderness ratio and load eccentricity ratio.

Number of tests. 4 slender and 16 stub CFT columns were tested.

Steel properties. Welded steel tubes were used for constructing the rectangular CFT specimens. The modulus of elasticity was 206,000 MPa.

- ***Tube size B and D (mm):-*** tubes size were 120x120, 100x150, 90x180, 130x130, 100x150 mm.
- ***Wall thickness t (mm):-*** wall thickness was 4.0 mm.
- ***Yield strength of steel f_y (MPa):-*** The average tensile strength was 495 MPa.

Concrete properties. The concrete cubes and cylinders were tested in compression to obtain the actual material properties. Modulus of elasticity of the concrete was 39,000 MPa.

- ***Compressive strength of concrete (MPa):-*** The average cylinder strength, cube strength of concrete was determined as 60 or 73 MPa.

Eccentricity (mm).

Effective length (mm). The effective length of the specimens were from 360 to 2600 mm.

End condition. Knife edges were constructed which allowed the load from the testing machine to be applied at given eccentricities to the specimen and to simulate pin–pin boundary conditions.

Loading method. All specimens were tested under eccentric loading about the major axis in a 5000 kN capacity universal testing machine.

D. Liu (2004)

Experimental synopsis. The behavior of high strength rectangular concrete-filled steel hollow section columns subjected to eccentric loading was studied. The primary test parameters were the cross-sectional aspect ratio, slenderness and load eccentricity. Favorable ductility performance was observed for all specimens during the test. The experimental ultimate capacities of the specimens were compared with the design strengths predicted by the codes. Comparison of results showed that EC 4 overestimated the ultimate capacities of the columns with a difference of 3%. ACI and AISC, on the other hand, conservatively predicted the failure loads by 11% and 25%, respectively.

Main test parameters. The test parameters were material strengths, cross-sectional aspect ratio, slenderness ratio and load eccentricity ratio.

Number of tests. 12 columns were tested.

Steel properties. Welded steel tubes were used for constructing the rectangular CFT specimens.

- ***Tube size B and D (mm):-*** tubes size were 150x150, 180x120, 120x80, 200x100, 160x80 mm.

- ***Wall thickness t (mm):-*** wall thickness was 4.18 mm. -

Yield strength of steel f_y (MPa):- The average tensile strength was 550 MPa.

Concrete properties. The concrete cubes and cylinders were tested in compression to obtain the actual material properties. -

Compressive strength of concrete (MPa):- The average cube strength of concrete was determined as 70.8 and 82.1 MPa.

Eccentricity (mm). Varies from 20 mm to 70 mm

Effective length (mm). The effective length of the specimens were from 870 to 2310 mm.

End condition. Knife edges were employed at both the bottom and top of the specimen to simulate pin–pin boundary conditions.

Loading method. During the test, the out-plane deflection of the specimen was less than 1 mm, thus the specimen was confirmed to be under compression combined with uniaxial bending.

G. Li, Z. Yang, Y. Lang (2010)

Experimental synopsis. The behavior of square steel tube columns filled with high-strength concrete was investigated. High strength concrete-filled square steel tube (HCFST) columns subjected to bi-axial eccentric loading were constructed and tested. The test results demonstrate the influence of these parameters on the strength and behavior of concrete filled square steel tube columns. The bearing capacity of the specimens was influenced by the steel ratio and decreased with the increasing slenderness ratio. The specimens failed to work out of instability.

Main test parameters. The primary test parameters were the slenderness ratios， steel ratios, and eccentricity along major axis was 71mm.

Number of tests. 6 columns were tested.

Steel properties. The steel tube was cold-formed square tube.

- ***Tube size B and D (mm):-*** The cross-section of the specimen was 200mm×200mm.

- ***Wall thickness t (mm):-*** wall thickness was 4 or 6mm. -

Yield strength of steel f_y (MPa):- The average tensile strength was 340 or 306 MPa.

Concrete properties. The modulus of elasticity of concrete was 3.81×104.

- ***Compressive strength of concrete (MPa):-*** The average cube strength of concrete was determined as 65 MPa.

Eccentricity (mm). The eccentricity along major axis was 71mm, and eccentric angle was 45°.

Effective length (mm). The effective length of the specimens were from 800 to 1600 mm.

End condition. Knife edges were used on either end to ensure that pinned ends were achieved.

Loading method. Columns were subjected to bi-axial eccentric loading.

L.H. Guo, Y.Y. Wang, S.M. Zhang (2011)

Experimental synopsis. Concrete filled square and rectangular hollow structural section (HSS) columns subjected to axial load, uniaxial bending or biaxial bending were tested. The overall buckling appeared accompanying with local buckling near the mid height of column.

Main test parameters. The main parameters considered under the test were eccentricity, width to depth ratio, length of columns.

Number of tests. 9 columns were tested.

Steel properties. Four rectangular section were tack welded, single bevel complete joint penetration groove welds were used to form the HSS tube.

- ***Tube size B and D (mm):-*** The cross-section of the specimen was 150x150 mm or 100x200 mm.
- ***Wall thickness t (mm):-*** wall thickness was 3.6 mm.
- ***Yield strength of steel f_y (MPa):-*** The average yield strength was 283.6 MPa.

Concrete properties. -

Compressive strength of concrete (MPa):- The average cylindrical strength of concrete was determined as 42.3 MPa.

Eccentricity (mm). The eccentricity e_x varied from 12.5 to 32.5 mm, and e_y varied from 12.5 to 40 mm.

Effective length (mm). The effective length of the specimens were 1350 or 1800 mm.

End condition. Knife edges were used on either end to ensure that pinned ends were achieved.

Loading method. Columns were subjected to bi-axial eccentric, uniaxial eccentric, axial compressive loading method.

L.H. Han, G.H. Yao (2004)

Experimental synopsis behavior of thin walled rectangular hollow structural steel columns filled with self-consolidating concrete has been investigated.

Main test parameters. A main parameter varied in the tests was load eccentricity.

Number of tests. 10 beam columns were tested.

Steel properties. The tubes were all manufactured from a mild steel sheet, with plates were cut from the sheet, tack welded into square shape and then welded with a single bevel butt weld. The modulus of elasticity was about 206,500 MPa

- ***Tube size B and D (mm):-*** The cross-section of the specimen was 200x200 mm.
- ***Wall thickness t (mm):-*** wall thickness was 3 mm.
- ***Yield strength of steel f_y (MPa):-*** the average yield strength of the tube was found to be 303.5 MPa.

Concrete properties. The modulus of elasticity (Ec) of the concrete was measured, the average value being 37,420 MPa.

- ***Compressive strength of concrete (MPa):-*** The average cube strength of concrete was determined as 58.5 MPa.

Eccentricity (mm). The eccentricity of the load was 60 mm.

Effective length (mm). The effective length of the specimens was 2310 mm.

End condition. . The axial load was applied through a very stiff top platen with an offset triangle hinge, which also allowed specimen rotation to simulate pin-ended supports.

Loading method. The desired eccentricity was achieved by accurately machining grooves 6 mm deep into the stiff endplate that was welded together with the steel tubes.

Y.C. Wang (1998)

Experimental synopsis. Slender composite concrete filled rectangular hollow steel section columns both with end eccentricities producing moments other than single curvature bending has been tested. In all tests, normal strength concrete of grade and low strength steel of grade were specified. These tests were designed specifically for composite columns with relative slenderness $\lambda > 1.0$.

Main test parameters. A main parameter varied in the tests was load eccentricity.

Number of tests. 8 beam columns were tested.

Steel properties. The steel section was formed from hot-rolled. The modulus of elasticity of steel was found to be 205,000 MPa. -

Tube size B and D (mm):- The cross-section of the specimen was 200x200 mm.

- ***Wall thickness t (mm):-*** wall thickness was 6.3 mm.

- ***Yield strength of steel f_y (MPa):-*** the average yield strength of the tube was found to be 370 MPa.

Concrete properties. The modulus of elasticity (Ec) of the concrete was measured, the average value being 37,000 MPa.

- ***Compressive strength of concrete (MPa):-*** The average cylindrical strength of concrete was determined as 55 MPa.

Eccentricity (mm). The eccentricity of the load was 55 or 110 mm.

Effective length (mm). The effective length of the specimens was 4,000 mm.

End condition. The columns were intended to be simply supported at both ends about both axes.

Loading method. Eccentric loading was applied and a set of adapter end plates were fixed to both ends of each column to form a simply supported column under double curvature bending.

Y.F. Yang, L.H. Han (2011)

Experimental synopsis. The behavior of concrete filled steel tubular (CFST) stub columns subjected to eccentric partial compression was investigated. The test results indicated that, similar to the corresponding fully loaded CFST stub columns under eccentric loading, CFST stub columns under eccentric partial compression have generally reasonable bearing capacity and favorable ductility.

Main test parameters. The main parameters in test program include, load eccentricity ratio (including uniaxial and biaxial loading and shape of the loading bearing plate, circular, square, strip and rectangular).

Number of tests. 9 rectangular and square beam columns were tested.

Steel properties. The modulus of elasticity and Poisson's ratio were 206,000 MPa and 0.281, respectively.

- ***Tube size B and D (mm):-*** The cross-section of the specimens was 120x180 mm or 150x150 mm.
- ***Wall thickness t (mm):-*** wall thickness was 3.0 mm.
- ***Yield strength of steel f_y (MPa):-*** the average yield strength of the tube was found to be 324.4 MPa.

Concrete properties. The modulus of elasticity of concrete was measured by testing three 150x300 mm prisms, and the average value was 33,600 MPa.

- ***Compressive strength of concrete (MPa):-*** The average cylindrical strength of concrete was determined as 59.3 MPa.

Eccentricity (mm). The eccentricity of the load was 15 or 30 mm for square sections and 18 or 36 mm for rectangular sections.

Effective length (mm). The effective length of the specimens was 440 or 540 mm.

End condition. The columns were simply supported at both ends about both axes.

Loading method. Specimens were loaded eccentrically on partial cross-sectional area.

T. Perea, R.T. Leon, J.F. Hajjar, M.D. Denavit (2014)

Experimental synopsis. The behavior of slender rectangular concrete-filled steel tubes (RCFTs) under combined axial compression and biaxial flexure has been investigated. The experimental determination of the maximum stable axial load–bending moment (P−M) interaction strength has been addressed. The experimental result showed that for very slender specimens, the bilinear interaction diagram proposed in the provisions of the AISC 2010 is somewhat un-conservative and conservative for most practical CFT column sizes and lengths.

Main test parameters. The main parameters considered were length of the columns, strength of concrete, eccentricity.

Number of tests. 8 rectangular beam columns were tested.

Steel properties. The modulus of elasticity was 202,400 MPa.

- ***Tube size B and D (mm):-*** The cross-section of the specimens was 508x305 mm.

- ***Wall thickness t (mm):-*** wall thickness was 7.90 mm. - -----

- ***Yield strength of steel f_y (MPa):-*** The yield strength of the tube range from 365 or 406 MPa.

Concrete properties. The modulus of elasticity of concrete was in a range of 27,600 MPa to 41,900 MPa.

- ***Compressive strength of concrete (MPa):-*** The average cylindrical strength of concrete was between 37.9 MPa and 87.6 MPa.

Effective length (mm). The effective length of the specimens was 5,550 mm or 7,960 mm.

End condition. The base was fully fixed and the top rotations were under load control.

Loading method. A complex loading protocol was used in the experimental program, including monotonic and cyclic loading that allowed detailed evaluation of the complete beam-column response.

T. Fujinaga, H. Doi, Y.P. Sun (2008)

Experimental synopsis. The behavior of square CFT columns subjected to eccentric compression with double curvature deformation has been studied. Experimental results have indicated that the flexural strength of slender square CFT columns increases with the concrete strength and the moment gradient.

Main test parameters. The experimental variables among the tests are, the buckling length to depth ratio of the column, the concrete strength, the moment gradient which deforms the column into double curvature, and the eccentricity of the axial load.

Number of tests. 43 square beam columns were tested.

Steel properties. -

Tube size B and D (mm):- The cross-section of the specimens was 125 mm x 125 mm.

- ***Wall thickness t (mm):-*** wall thickness was 3.20 mm. -

Yield strength of steel f_y (MPa):- The yield strength of the tube was 358 MPa.

Concrete properties.

- ***Compressive strength of concrete (MPa):-*** The average cylindrical strength of concrete was 27 MPa or 60 MPa.

Effective length (mm). The effective length of the specimens was 1,250 mm or 2,500 mm.

End condition. The knife-edge connection at the end of the column.

Loading method. Each specimen was at first loaded concentrically in elastic then the knife-edge at the end of the column was slid to the targeted eccentricity, and then the eccentric loading was applied till large deformation.

CHAPTER 5

ANALSIS AND RESULTS

The database listed in the previous chapter had been assembled without the assessment of validity of the specimens for calibrating. This chapter analyze and compare CFT tested specimens which are subjected to axially loaded columns and beam-columns. Test specimens which are outside the scope of this work are eliminated from the database.

Tests in which the specimens were subjected to cyclic loading, tests containing lightweight concrete, tests subjected to unequal end moments were removed from the database. The properties (D (or h & b), t, f_y, E_s, f_{ck}, E_c, L, e) are arranged and analyzed for the remaining tests as well as calculations of ultimate load capacity, N_{EC4} and $P_{AISC-2010}$ for each test with the material partial safety factor as unity are carried out. The data is divided into circular section and rectangular section columns, with and without moment, and whether the columns are short ($L/D \leq 4$) or long ($L/D > 4$). Results for each type of column and the average ratio of (Test/code prediction) and the standard deviation of this ratio for each set are summarized.

Test Specimens analyzed in this database are categorized based on material and geometric limitation specified by EC-4. As a result, the database categorize the specimens in to two groups based on the criteria of structural steel's local buckling limitation. These are specimens which were outside the limit of local buckling limitation according to EC-4, and specimens which lie within local buckling limitation. Specimens which fall within local buckling limitation are further categorized into three types. These are, specimens which are within EC-4 limitations, specimens with high strength concrete ($f_{ck} > 50$MPa) and specimens with high steel strength ($f_y > 460$MPa).

Comparisons were made between the EC-4 and AISC-2010 on the slenderness parameters used by EC-4 and AISC-2010, the values and distributions of yield stress, concrete compressive strength, structural steel ratio, e/D ratio for beam-columns, and D/t ratio for circular concrete filled tubes and B/t ratio for rectangular concrete filled tubes are done.

Columns loaded axially in compression either in concentrically or eccentrically will behave in one of the two distinct ways.

1) Short columns (x-sectional strength): - Columns with a small kL/D ratio are governed by cross-section strength. These types of columns reach their ultimate capacity when both the steel and the concrete reach their strength limit point, yielding of the steel and crushing of the concrete.

2) Intermediate or long (slender) columns: - columns with a larger kL/D ratio are governed by stability and fail by either elastic or inelastic column buckling. A load applied eccentrically will tend to cause buckling to occur earlier than an equal load applied concentrically (concentric also implies that the column is perfectly straight).

5.1 CIRCULAR COLUMNS (CCFT)

The range of material strength of steel and concrete, length to depth ratio, steel ratio for short and long CCFT columns used in the analyses are summarized in the table shown.

Columns		short CCFT	long CCFT
No. of specimens for analysis		145	149
f_y (MPa)	max	853	626
	min	237	221
f_{ck} (MPa)	max	110.3	120.1
	min	12	21
KL/D	max	3.99	45.5
	min	1.07	4.2
δ	max	0.93	0.77
	min	0.053	0.23

Table 5.1

5.1.1 Short CCFT columns

145 axially loaded circular x-sections were used for the final analyses. The maximum and minimum yield stress of the steel were 853MPa and 237 MPa respectively. The compressive strength ranges from 21 MPa to 110.3 MPa. The structural steel ratio ranges from 0.1 to 0.93. Out of 145 test specimens 65 specimens fall within the range of EC4 limitations, 54 specimens had not met local buckling criteria specified by EC4**.** Figures 4-1 to 4-4 show scatter plots of the data for EC-4 and AISC-2010 analysis.

a) Frequency distributions

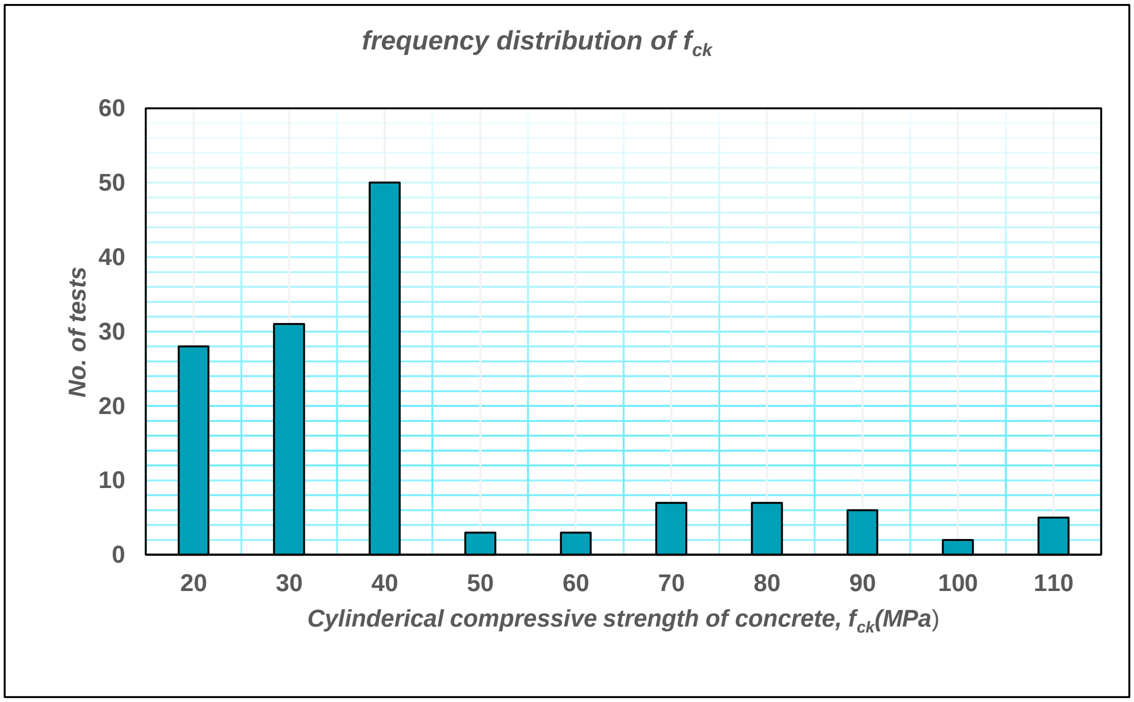

Figure 5.1

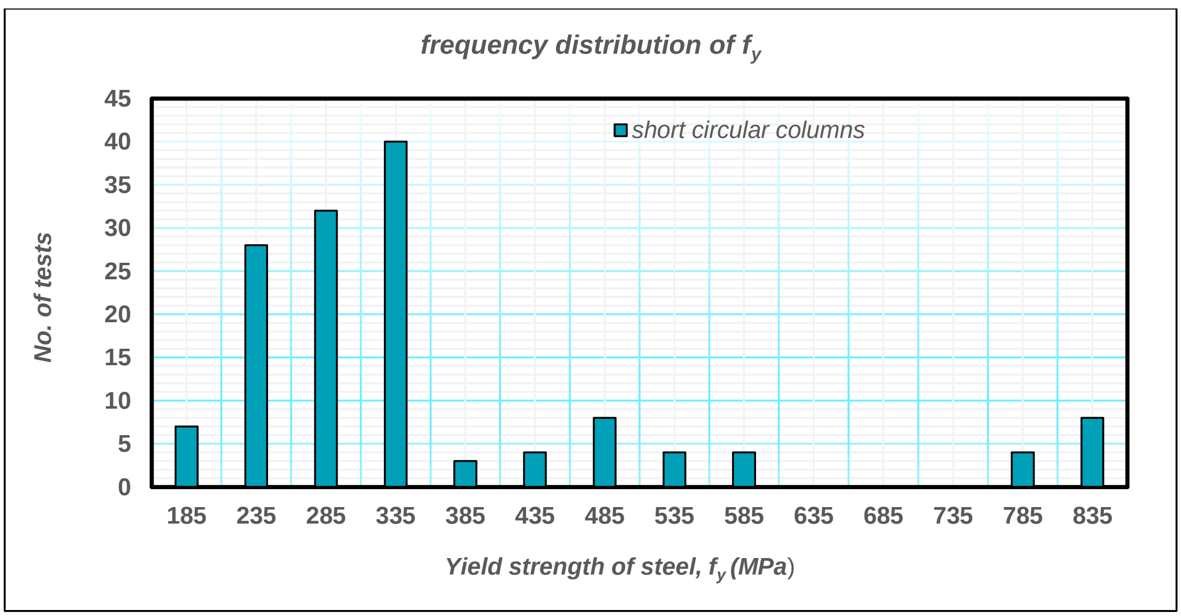

Figure 5.2

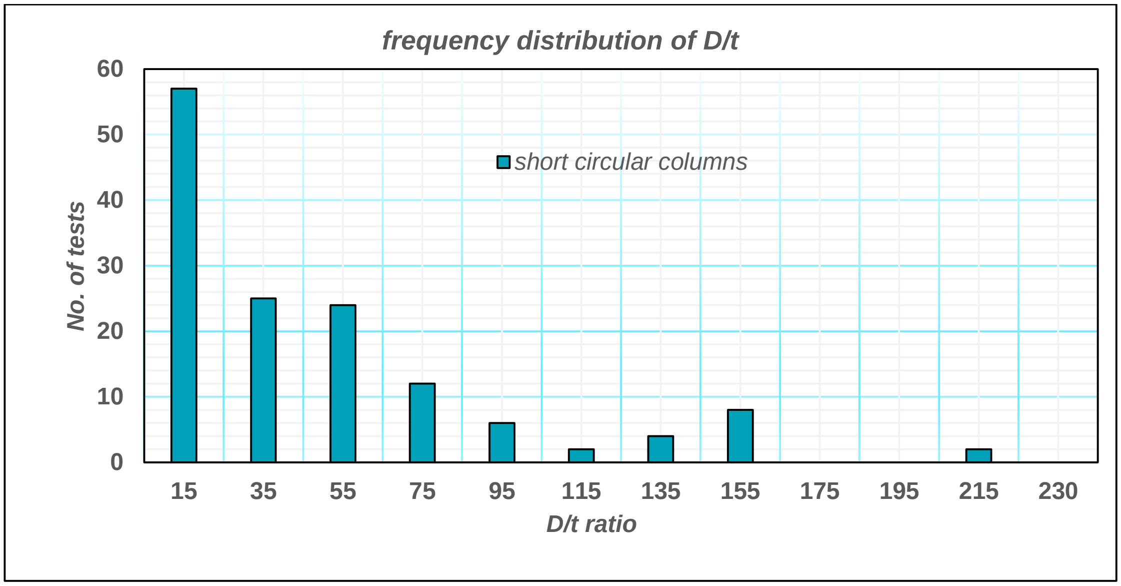

Figure 5.2

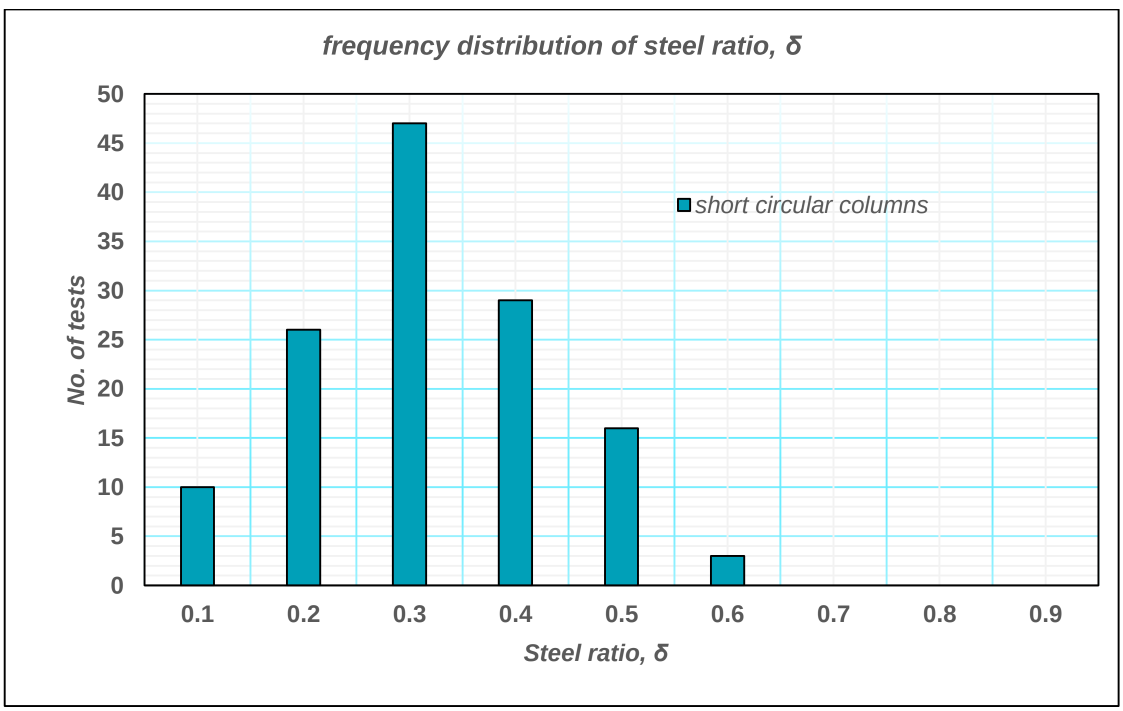

Figure 5.4

b) Analysis and results

-Specimens above local buckling limit: - 54 axially loaded short circular CFT columns specimens which failed local buckling criteria according to EC-4 are analyzed in the database. The mean value N_{test}/N_{Code} according to EC-4 was 1.04 and 59% of the specimens gave the value of N_{test}/N_{Code} greater than >1. The mean value N_{test}/N_{Code} according to AISC-2010 was 1.15 and 78% of the specimens gave the value of N_{test}/N_{Code} greater than >1.

-Specimens within EC-4 ranges: - 65 of the specimens were within EC-4 limitations and the average value of N_{test}/N_{Code} according to EC-4 gave 1.05 with a standard deviation of 0.15 and the AISC prediction for specimens which fall within EC-4 limitations was 1.39 with a standard deviation of 0.21, higher than EC-4 prediction. The AISC-2010 predicts load resistance conservatively than EC-4, but a wide scattering of results are observed.

		test/code	
short circular columns		*EC-4*	*AISC-2010*
specimens with (D/t) > 90*(235/f_y) of steel	*No. of tests*	54	
	avg	1.04	1.15
	st dev	0.21	0.17
	$N_{test}/N_{Code} \geq 1$	59%	78%
specimens with (D/t) < 90*(235/f_y) of steel	*No. of tests*	91	
	avg	1.09	1.35
	st dev	0.18	0.20
	$N_{test}/N_{Code} \geq 1$	67%	100%
specimens within EC-4 range	*No. of tests*	65	
	avg	1.05	1.39
	st dev	0.15	0.21
	$N_{test}/N_{Code} \geq 1$	65%	100%
$f_{ck} \geq 50$ MPa	*No. of tests*	16	
	avg	1.32	1.23
	st dev	0.10	0.09
	$N_{test}/N_{Code} \geq 1$	100%	100%
$f_y \geq 460$ Mpa	*No. of tests*	12	
	avg	1.00	1.27
	st dev	0.22	0.08
	$N_{test}/N_{Code} \geq 1$	32%	100%

Table 5.2

-*specimens with f_{ck} > 50MPa concrete strength*:- 16 specimens which have a value of compressive strength of concrete above 50MPa were analyzed separately as shown in table 4.1. The mean value according to EC-4 was 1.32 with a standard deviation of 0.10 and the AISC-2010 average prediction was 1.23 with a standard deviation of 0.09. This shows that there were not significantly more unsafe results when the concrete strength was outside the strength permitted by the codes.

-*specimens with f_y > 460MPa steel yield strength:* - it is observed that the average ratio of test/code prediction for columns with a high steel yield strength (f_y > 460MPa) is 1.0 according to EC-4 prediction and 1.27 with AISC-2010. This shows that EC-4 prediction gave a lower value than AISC-2010.

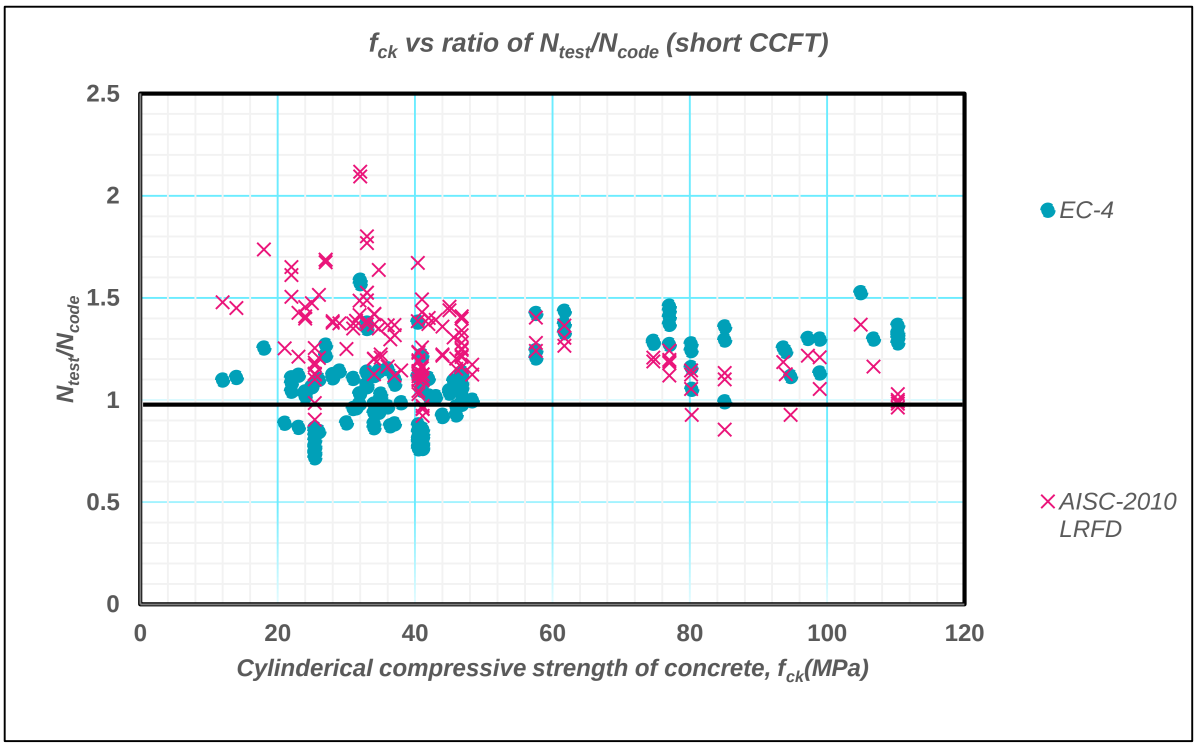

Figure 3.5

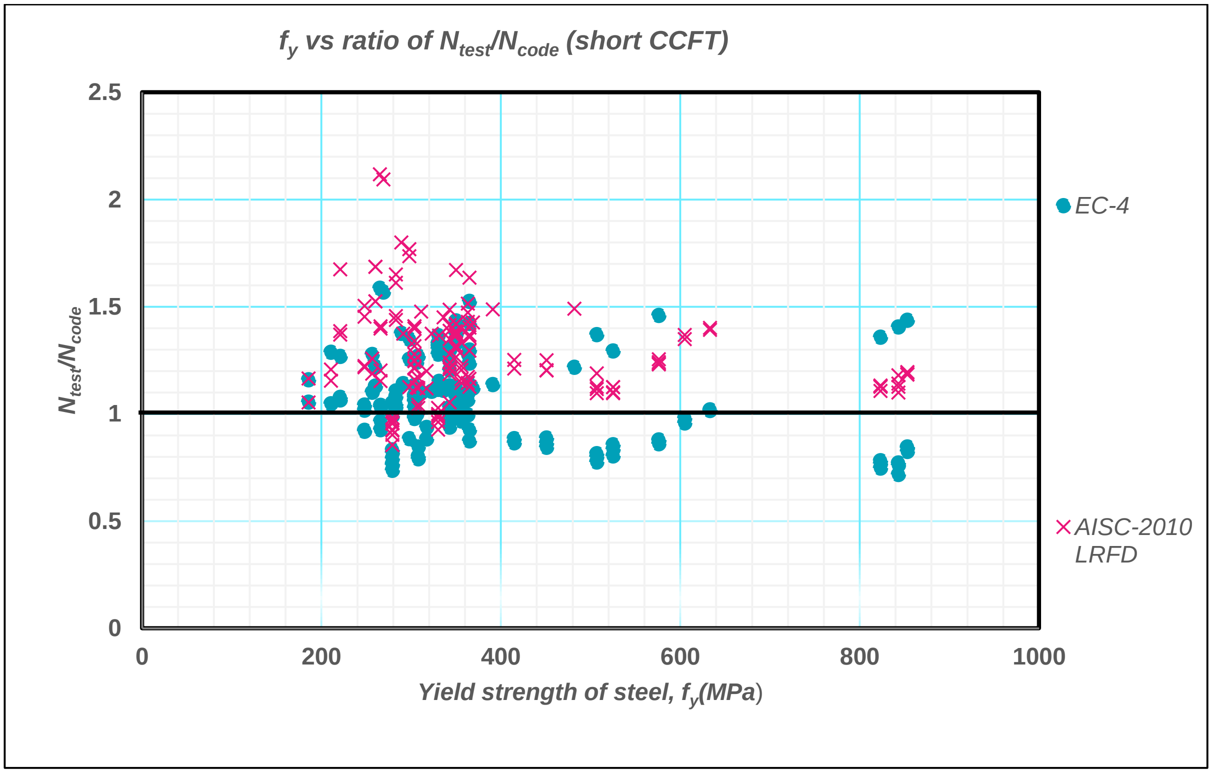

Figure 5.6

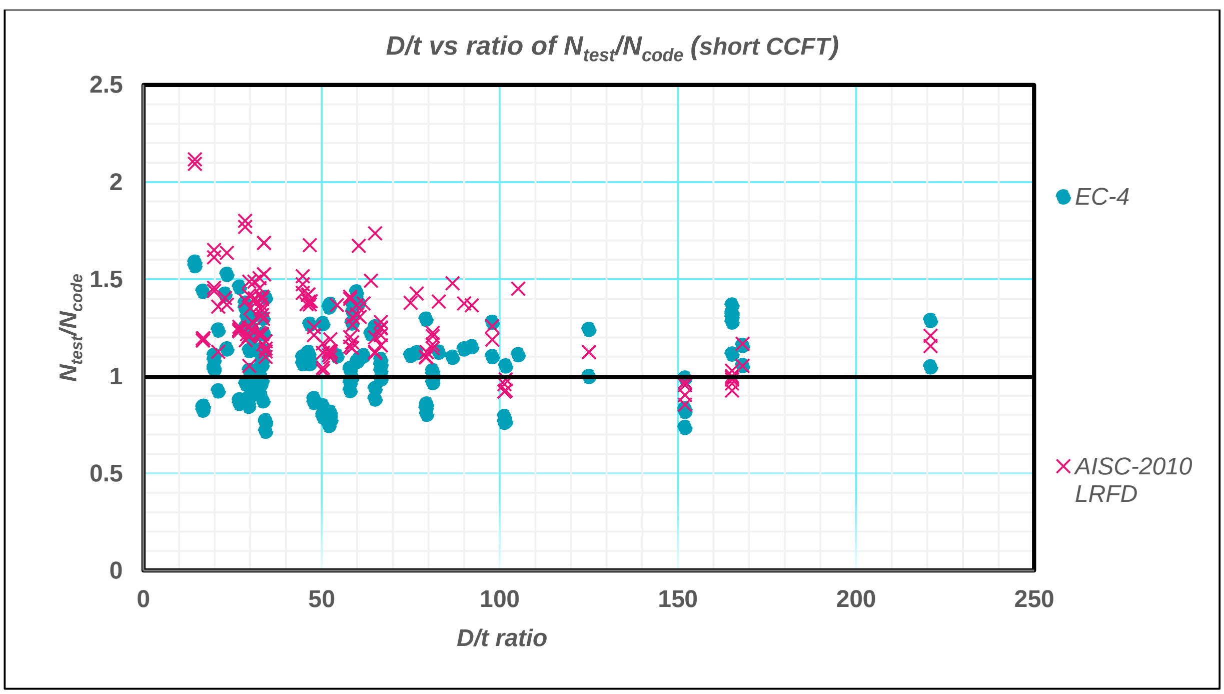

Figure 5.4

5.1.2 Long CCFT columns

149 axially loaded circular long columns were used for the final analyses. 95 tests fall within the range of EC4 limitations, 22 tests have cylindrical compressive concrete strength above 50 MPa and another 22 specimens were above EC4' local buckling limitation**.**

a) Frequency distributions

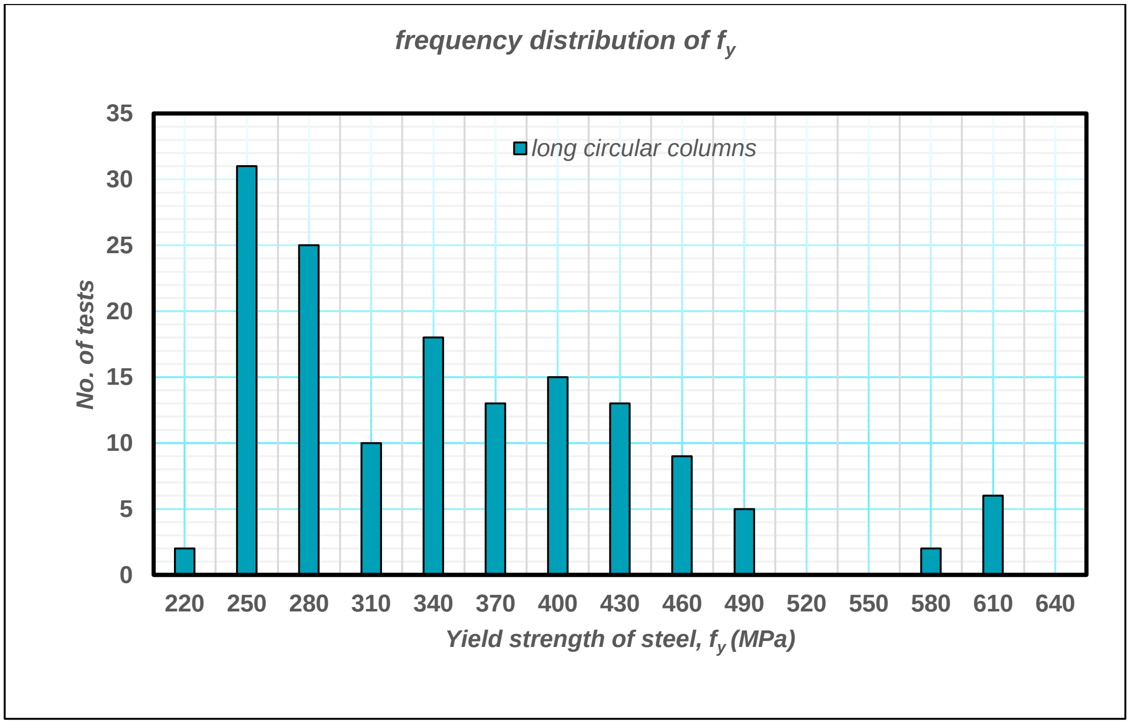

Figure 5.8

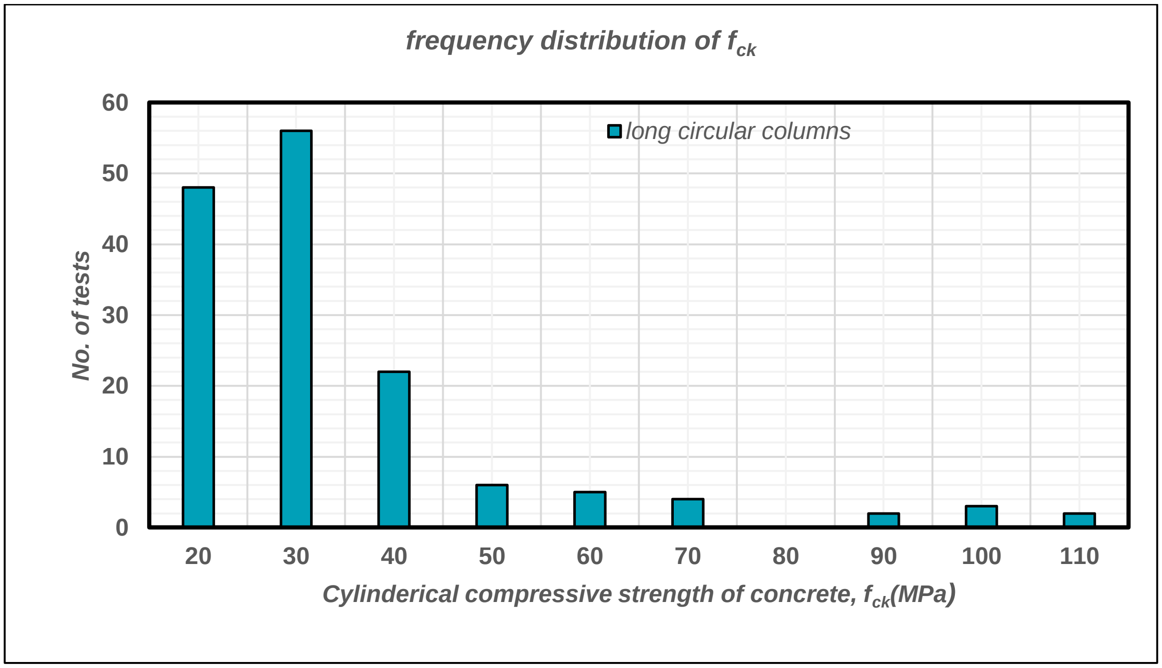

Figure 5.5

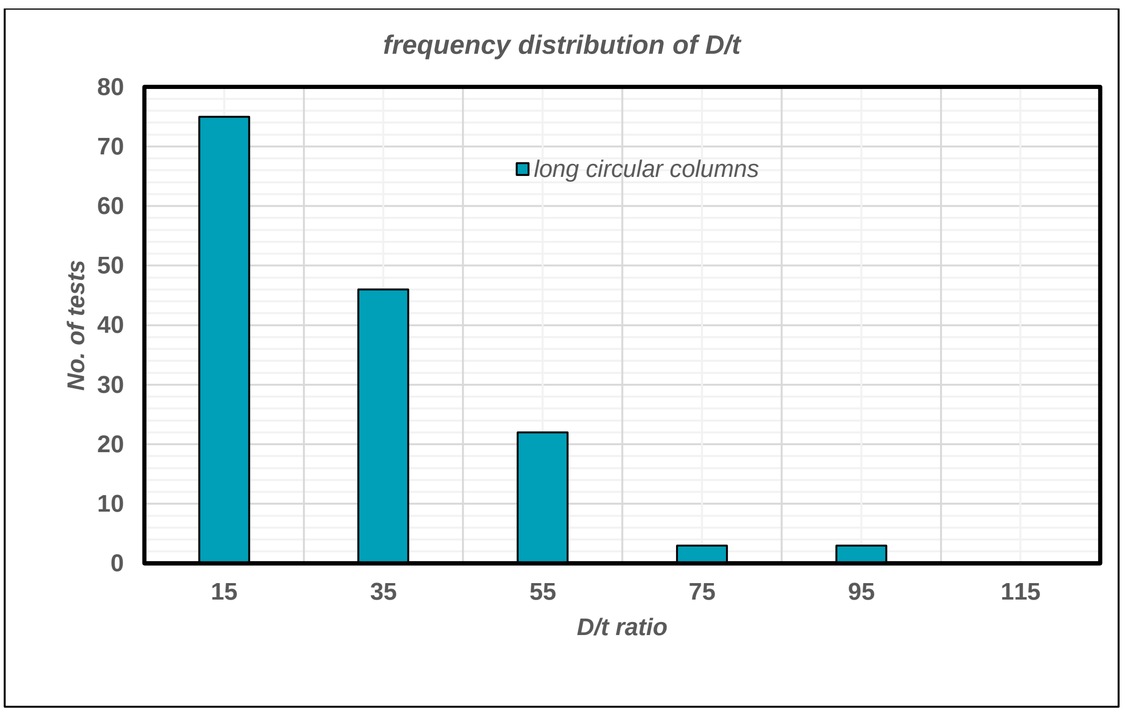

Figure 5.6

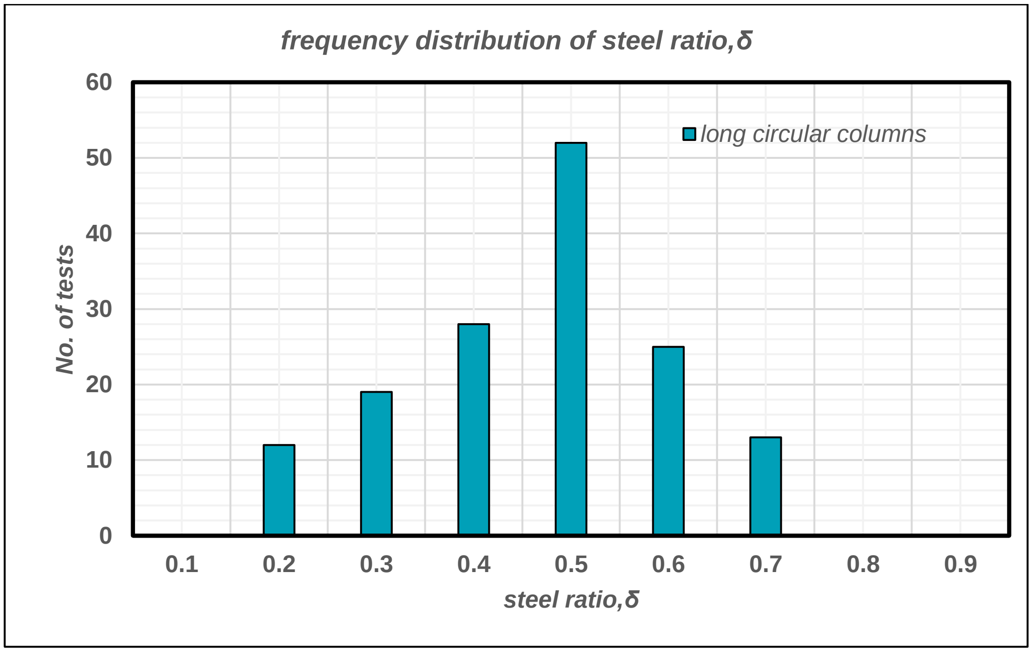

Figure 5.7

b) Analysis and results

-*Specimens above local buckling limit:* - 22 axially loaded long circular CFT columns specimens which failed local buckling criteria according to EC-4 are analyzed in the database and the mean value N_{test}/N_{Code} according to EC-4 was 1.15 and 90% of the specimens gave the value of N_{test}/N_{Code} greater than >1. The mean value N_{test}/N_{Code} according to AISC-2010 was 1.2 and 94% of the specimens gave the value of N_{test}/N_{Code} greater than >1.

-*Specimens within EC-4 ranges:* - 95 of the specimens were within EC-4 limitations and the average value of N_{test}/N_{Code} according to EC-4 gave 1.10 with a standard deviation of 0.13 and the AISC prediction for specimens which fall within EC-4 limitations was 1.16 with a standard deviation of 0.17, higher than EC-4 prediction as shown in the table.

		test/code	
long circular columns		*EC-4*	*AISC-2010*
specimens with (D/t) > 90*(235/f_y) of steel	*No. of tests*	22	
	avg	1.15	1.22
	st dev	0.18	0.19
	$N_{test}/N_{Code} \geq 1$	90%	94%
specimens with (D/t) < 90*(235/f_y) of steel	*No. of tests*	127	
	avg	1.11	1.14
	st dev	0.14	0.17
	$N_{test}/N_{Code} \geq 1$	82%	86%
specimens within EC-4 range	*No. of tests*	95	
	avg	1.10	1.16
	st dev	0.13	0.16
	$N_{test}/N_{Code} \geq 1$	83%	92%
$f_{ck} \geq 50$ MPa	*No. of tests*	20	
	avg	1.15	1.02
	st dev	0.17	0.08
	$N_{test}/N_{Code} \geq 1$	100%	67%
$f_y \geq 460$ MPa	*No. of tests*	18	
	avg	1.09	1.15
	st dev	0.11	0.20
	$N_{test}/N_{Code} \geq 1$	76%	73%

Table 5.3

- *Specimens with f_{ck} > 50MPa concrete strength*:- 20 specimens which have a value of compressive strength of concrete above 50MPa were analyzed separately as shown in the table. The mean value according to EC-4 was 1.15 with a standard deviation of 0.17 and the AISC-2010 average prediction was 1.02 with a standard deviation of 0.08. This shows that both codes can predict very well for high strength concrete columns.

- *Specimens with f_y > 460MPa steel yield strength:* -the average ratio of test/code prediction for columns with a high yield steel (f_y > 460MPa) is greater than unity in both cases.

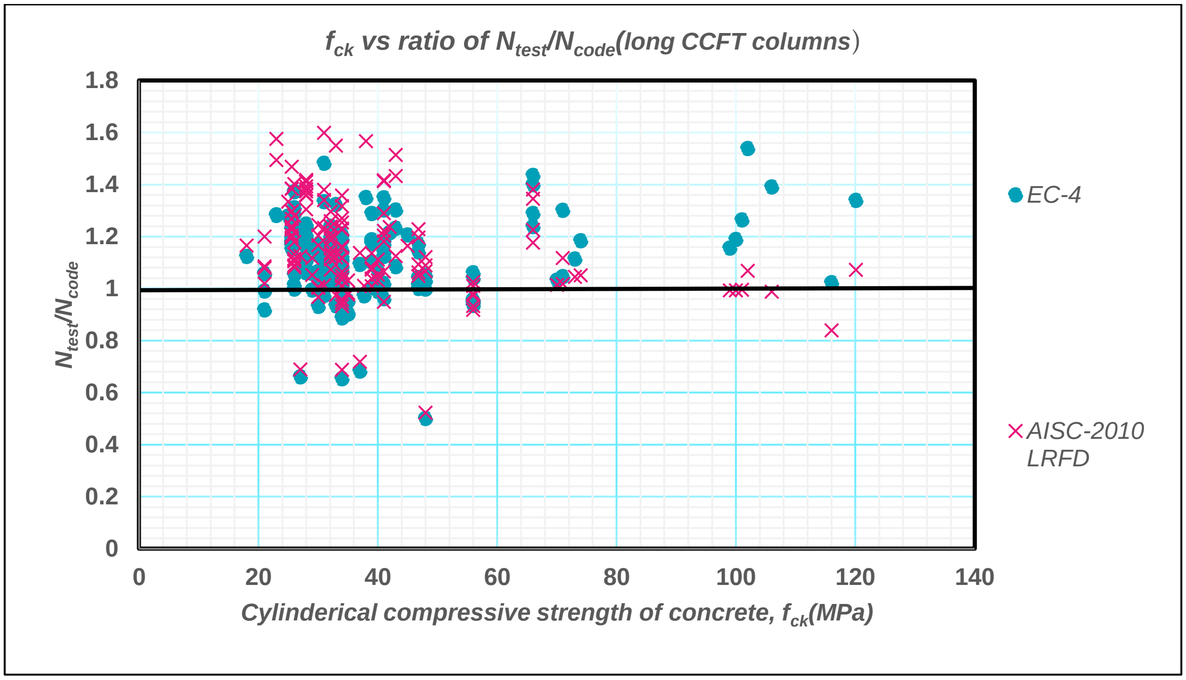

Figure 5.8

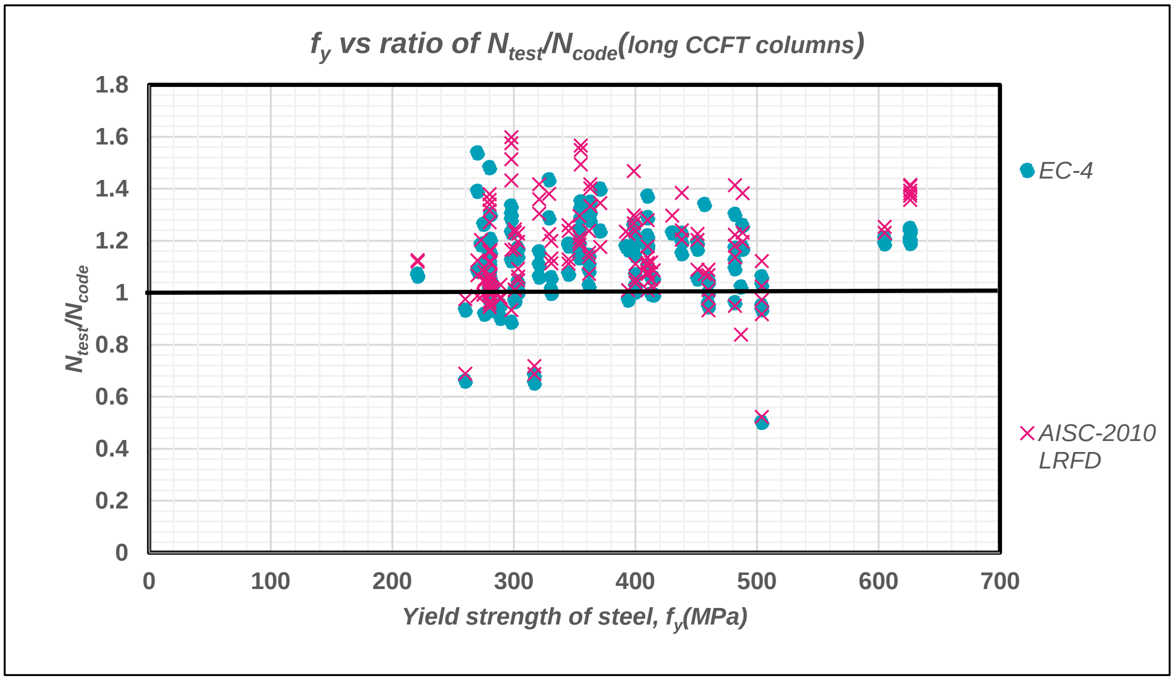

Figure 5.9

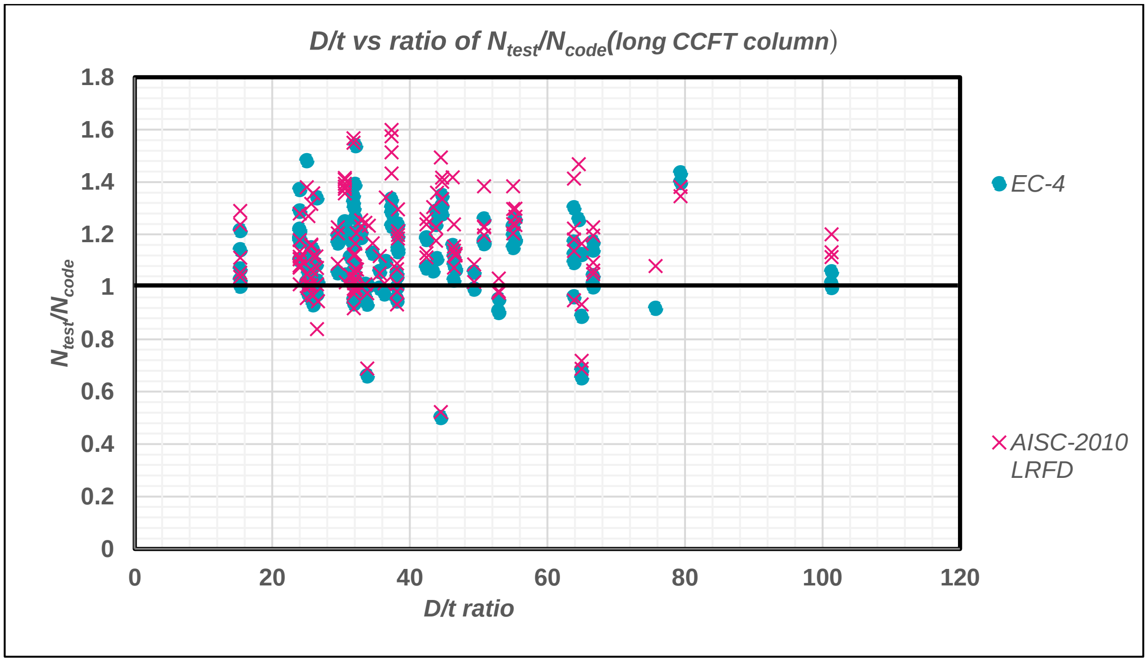

Figure 5.10

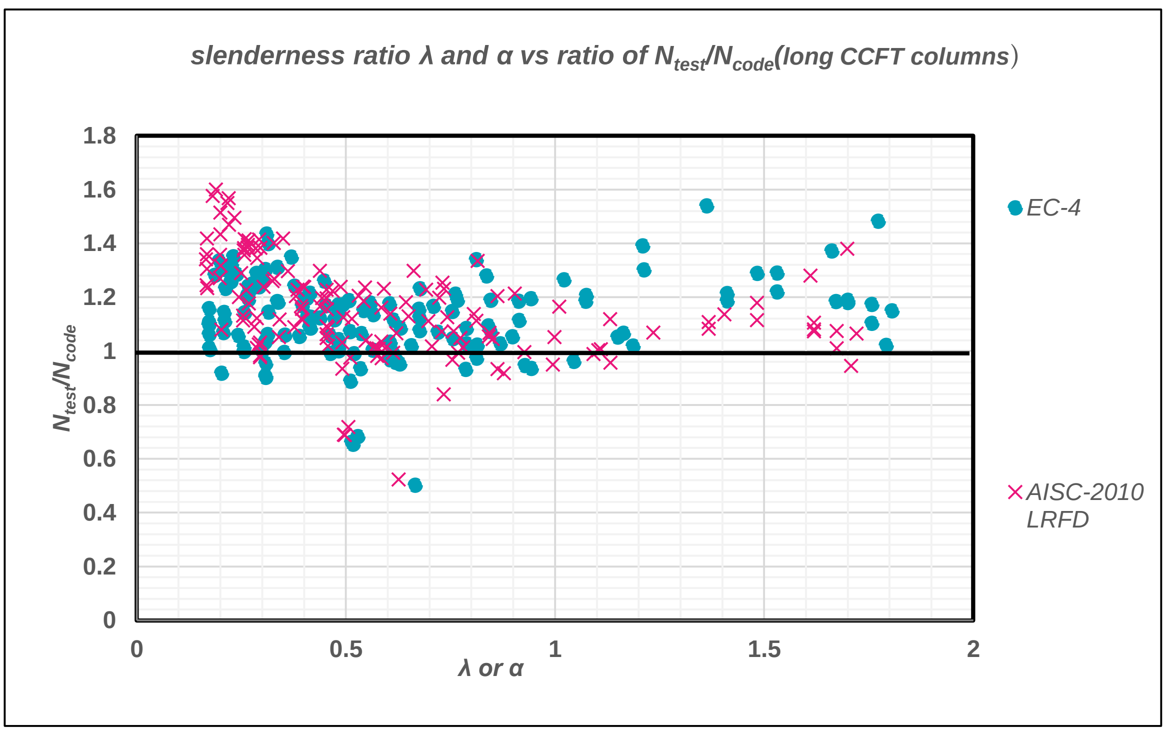

Figure 5.11

The axial capacity prediction is larger by EC-4 than by AISC-2010 for $\lambda < 0.5$, which shows that the additional confinement effect considered in the EC-4.

5.2 RECTANGULAR COLUMNS (RCFT)

The range of material strength of steel and concrete, length to depth ratio, steel ratio for short and long CCFT columns used in the analyses are summarized in the table shown.

columns		short RCFT	long RCFT
No. of specimens for analysis		210	82
f_y (MPa)	max	835	761
	min	239	246
f_{ck} (MPa)	max	106	96
	min	21	21
KL/D	max	3.99	27.3
	min	0.98	4.01
δ	max	0.89	0.88
	min	0.12	0.174

Table 5.4

5.2.1 Short RCFT columns

210 axially loaded rectangular x-section were used for the final analyses in the database. The maximum and minimum yield strength of the steel tubes were 835MPa and 239 MPa respectively. The compressive strength ranges from 21 MPa to 106 MPa. The structural steel ratio ranges from 0.12 to 0.89. Out of 210 tests, 52 tests fall within the range of EC4 limitations, 114 tests have cylindrical compressive concrete strength above 50 MPa and 78 specimens were out of local buckling criteria specified by EC4**.**

a) Frequency distributions

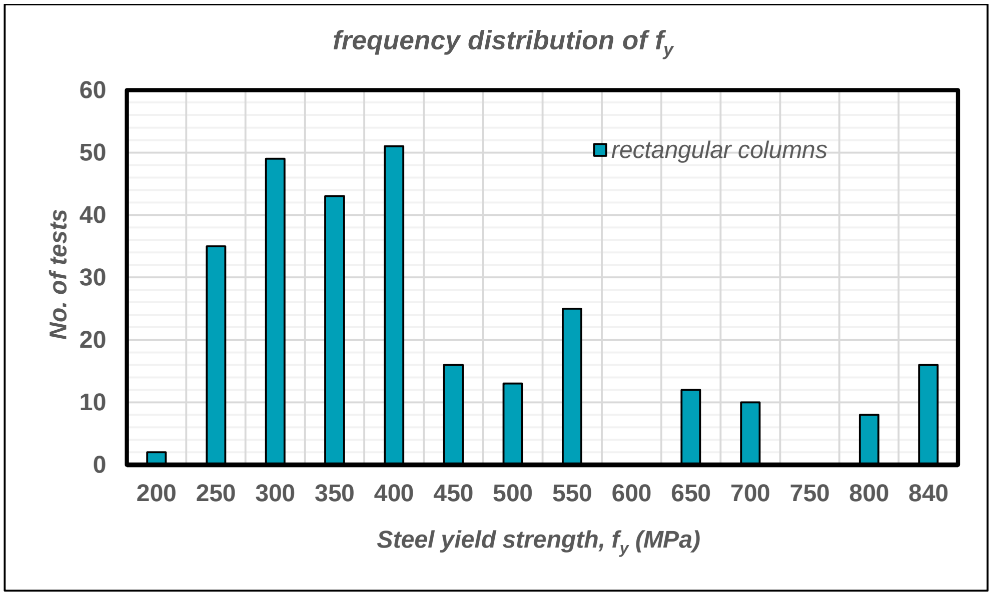

Figure 5.16

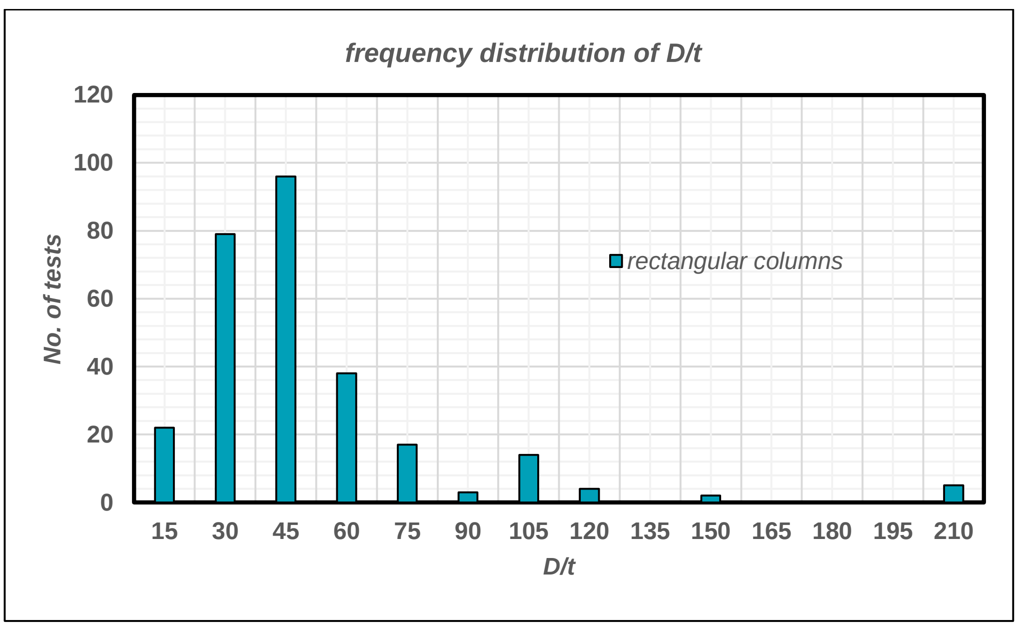

Figure 5.17

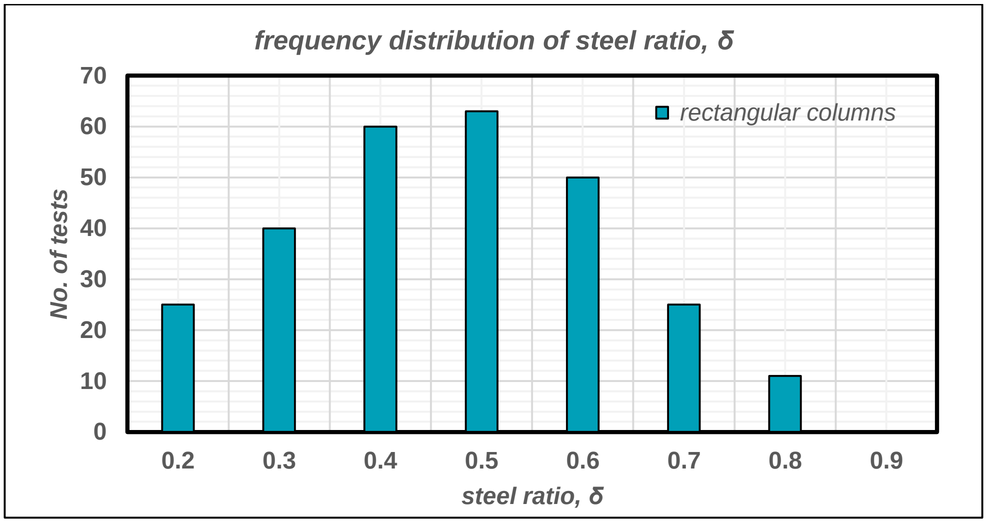

Figure 5.18

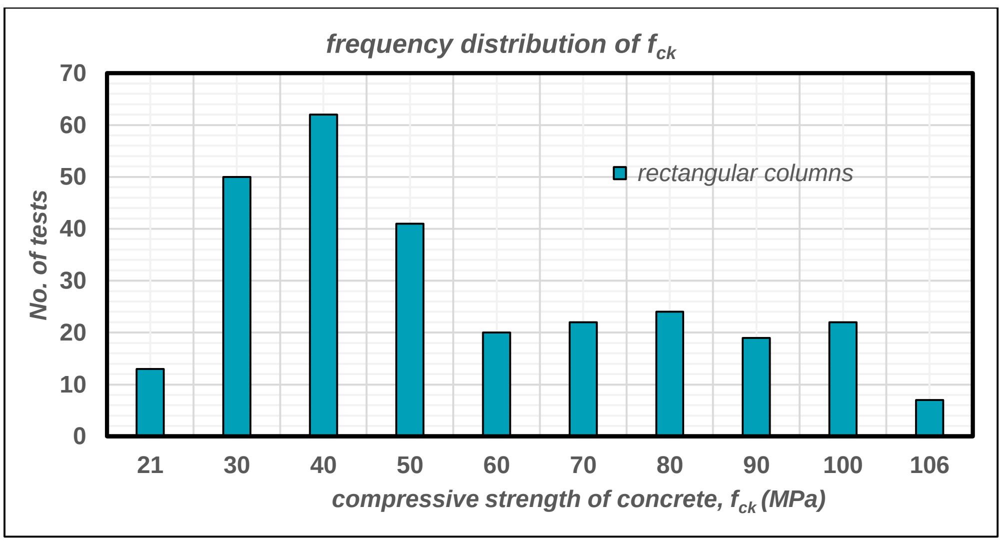

Figure 5.19

b) Analysis and results

- *Specimens above local buckling limit:* - 78 axially loaded short rectangular CFT columns specimens which failed local buckling criteria according to EC-4 are analyzed in the database and the mean value N_{test}/N_{Code} according to EC-4 was 0.95 and 32% of the specimens gave the value of N_{test}/N_{Code} greater than >1. The mean value N_{test}/N_{Code} according to AISC-2010 was 1.03 and 53% of the specimens gave the value of N_{test}/N_{Code} greater than >1.

-*Specimens within EC-4 ranges:* - 52 of the specimens were within EC-4 limitations and the average value of N_{test}/N_{Code} according to EC-4 gave 1.17 and the AISC prediction for specimens which fall within EC-4 limitations was 1.27.

		test/code	
short rectangular columns		*EC-4*	*AISC-2010*
specimens with (D/t) > 52*SQRT(235/f_y) of steel	*No. of tests*	78	
	avg	0.95	1.03
	st dev	0.19	0.21
	$N_{test}/N_{Code} \geq 1$	32%	53%
specimens with (D/t) < 52*SQRT(235/f_y) of steel	*No. of tests*	132	
	avg	1.08	1.12
	st dev	0.24	0.26
	$N_{test}/N_{Code} \geq 1$	80%	85%
specimens within EC-4 range	*No. of tests*	52	
	avg	1.17	1.27
	st dev	0.19	0.21
	$N_{test}/N_{Code} \geq 1$	84%	90%
$f_{ck} \geq 50$ MPa	*No. of tests*	92	
	avg	1.05	1.03
	st dev	0.26	0.25
	$N_{test}/N_{Code} \geq 1$	83%	80%
$f_y \geq 460$ MPa	*No. of tests*	37	
	avg	1.12	1.14
	st dev	0.12	0.12
	$N_{test}/N_{Code} \geq 1$	91%	99%

Table 5.5

- *Specimens with f_{ck} > 50MPa concrete strength*:- 92 specimens which have a value of compressive strength of concrete above 50MPa were analyzed separately. The mean value according to EC-4 was 1.05 with a standard deviation of 0.26 and the AISC-2010 average prediction was 1.03 with a standard deviation of 0.25.

- *Specimens with f_y > 460MPa steel yield strength:* -the average ratio of test/code prediction for columns with a high yield steel (f_y > 460MPa) is 1.12 by EC-4 prediction and 1.14 by AISC-2010 prediction.

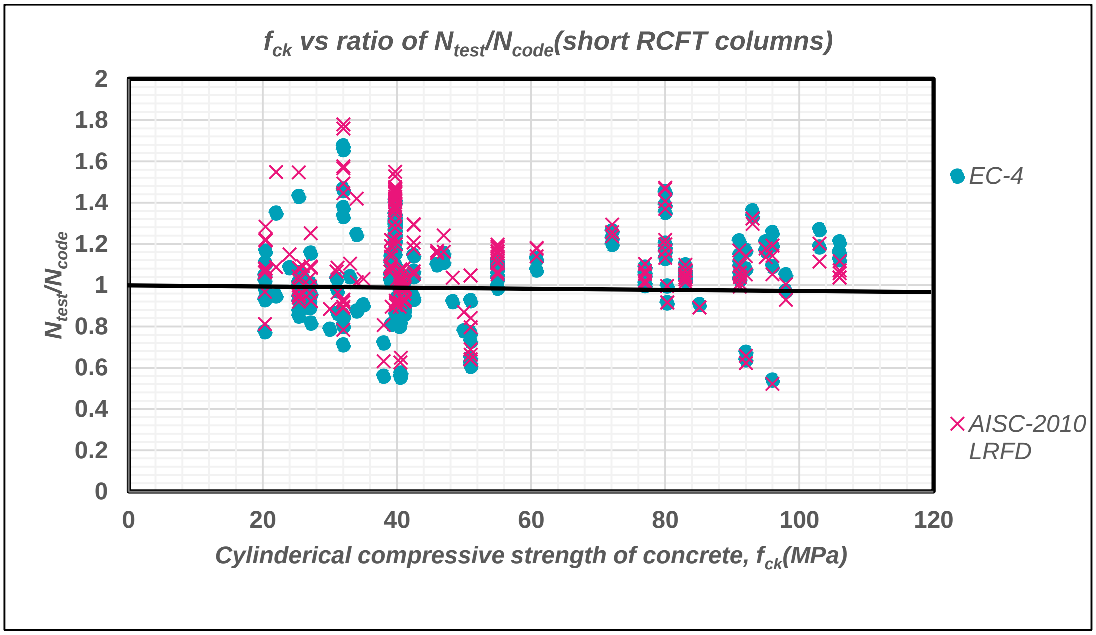

Figure 5.20

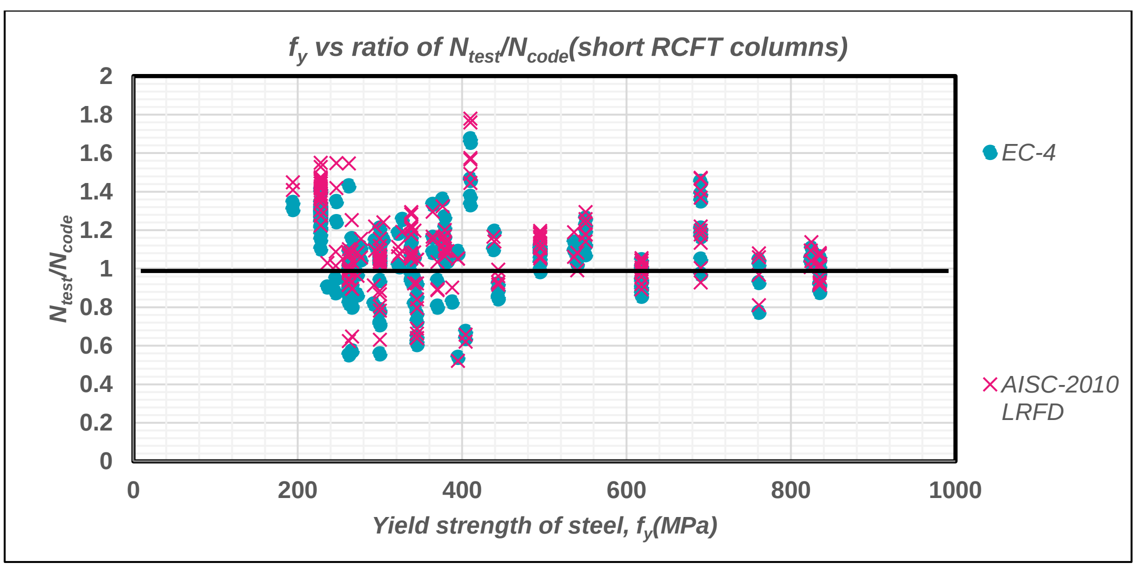

Figure 5.21

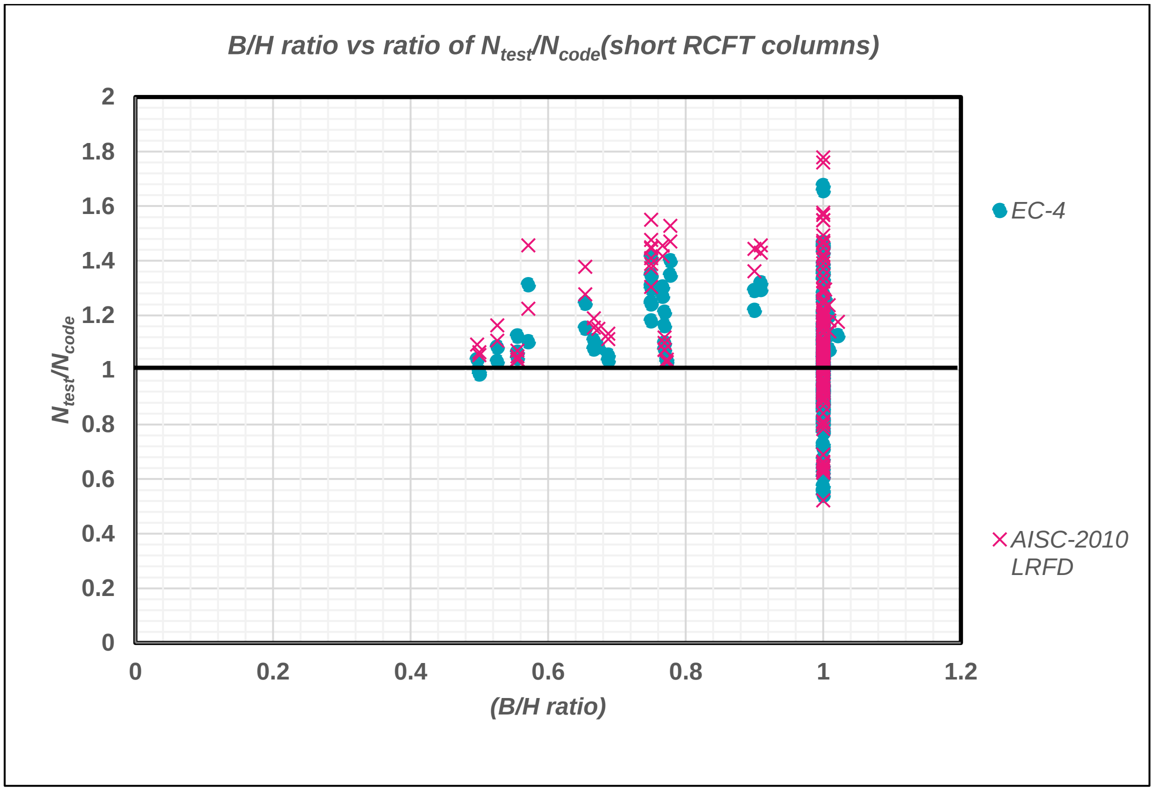

Figure 5.22

5.2.2 Long RCFT columns

82 axially loaded long rectangular columns were used for the final analyses. 32 tests fall within the range of EC4 limitations, 34 tests have cylindrical compressive concrete strength above 50 MPa and another 28 specimens were above EC4' local buckling limitations.

b) Analysis and results

- *Specimens above local buckling limit:* - 28 axially loaded short rectangular CFT columns specimens which failed local buckling criteria according to EC-4 are analyzed in the database and the mean value N_{test}/N_{Code} according to EC-4 was 1.04 and the AISC-2010 prediction was 1.13.

-*Specimens within EC-4 ranges:* - 32 of the specimens were within EC-4 limitations and the average value of N_{test}/N_{Code} according to EC-4 prediction was 1.03 and the AISC prediction for specimens which fall within EC-4 limitations was 1.09 as shown in the table

		test/code	
long rectangular columns		*EC-4*	*AISC-2010*
specimens with (D/t) > 52*sqrt(235/f_y) of steel	*No. of tests*	28	
	avg	1.04	1.13
	st dev	0.13	0.16
	$N_{test}/N_{Code} \geq 1$	79%	84%
specimens with (D/t) < 52*sqrt(235/f_y) of steel	*No. of tests*	54	
	avg	1.15	1.20
	st dev	0.27	0.27
	$N_{test}/N_{Code} \geq 1$	74%	85%
specimens within EC-4 range	*No. of tests*	32	
	avg	1.03	1.09
	st dev	0.18	0.19
	$N_{test}/N_{Code} \geq 1$	58%	79%
$f_{ck} \geq 50$ MPa	*No. of tests*	23	
	avg	1.36	1.40
	st dev	0.19	0.19
	$N_{test}/N_{Code} \geq 1$	97%	97%
$f_y \geq 460$ MPa	*No. of tests*	5	
	avg	1.10	1.15
	st dev	0.10	0.09
	$N_{test}/N_{Code} \geq 1$	91%	100%

Table 5.6

- *Specimens with f_{ck} > 50MPa concrete strength*:-. The N_{test}/N_{Code} mean value of 23 specimens which have f_{ck} > 50MPa was 1.36 with a standard deviation of 0.19 and the AISC-2010 average prediction was 1.4 with a standard deviation of 0.19.

- *Specimens with f_y > 460MPa steel yield strength:* -the average ratio of test/code prediction for columns with (f_y > 460MPa) was 1.10 by EC-4 and 1.15 by AISC-2010.

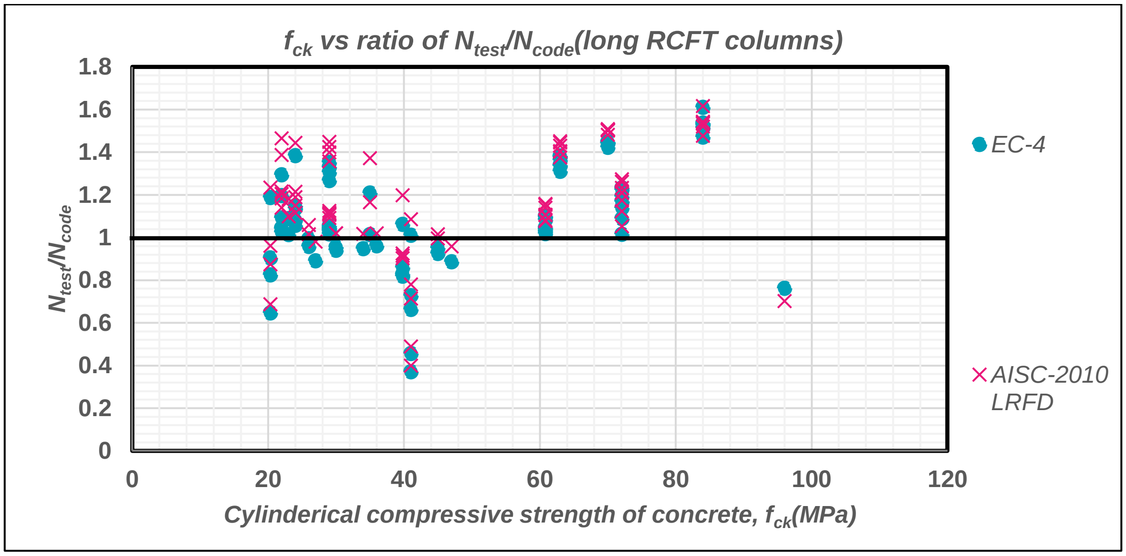

Figure 5.23

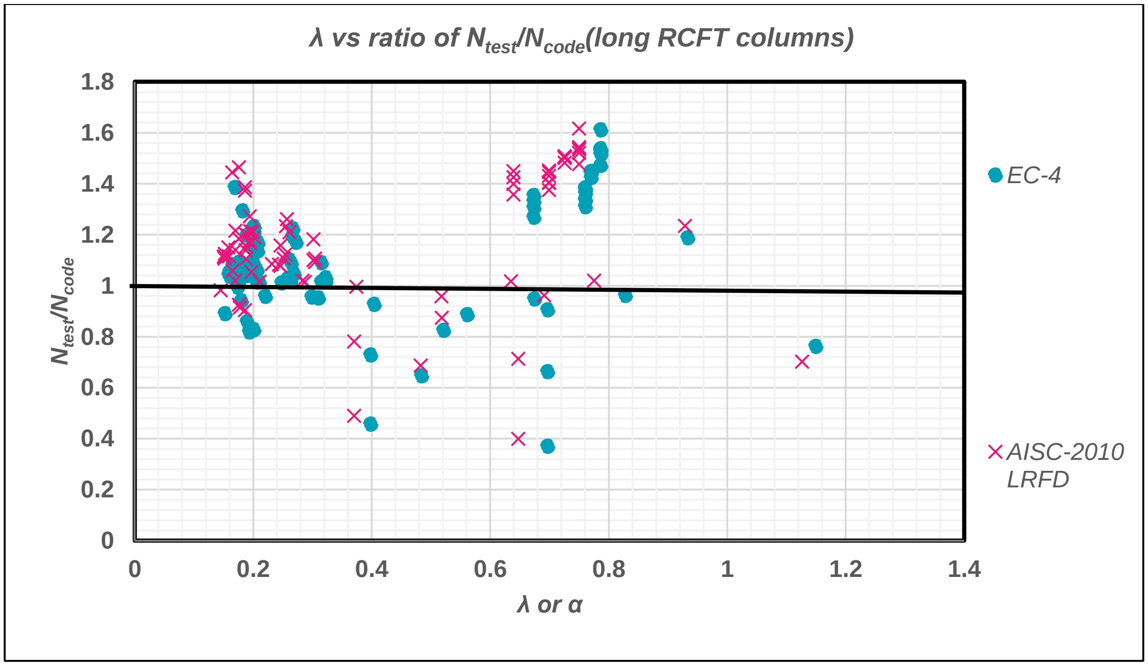

Figure 5.24

At a low slenderness ratio, EC-4 prediction gives a higher axial capacity.

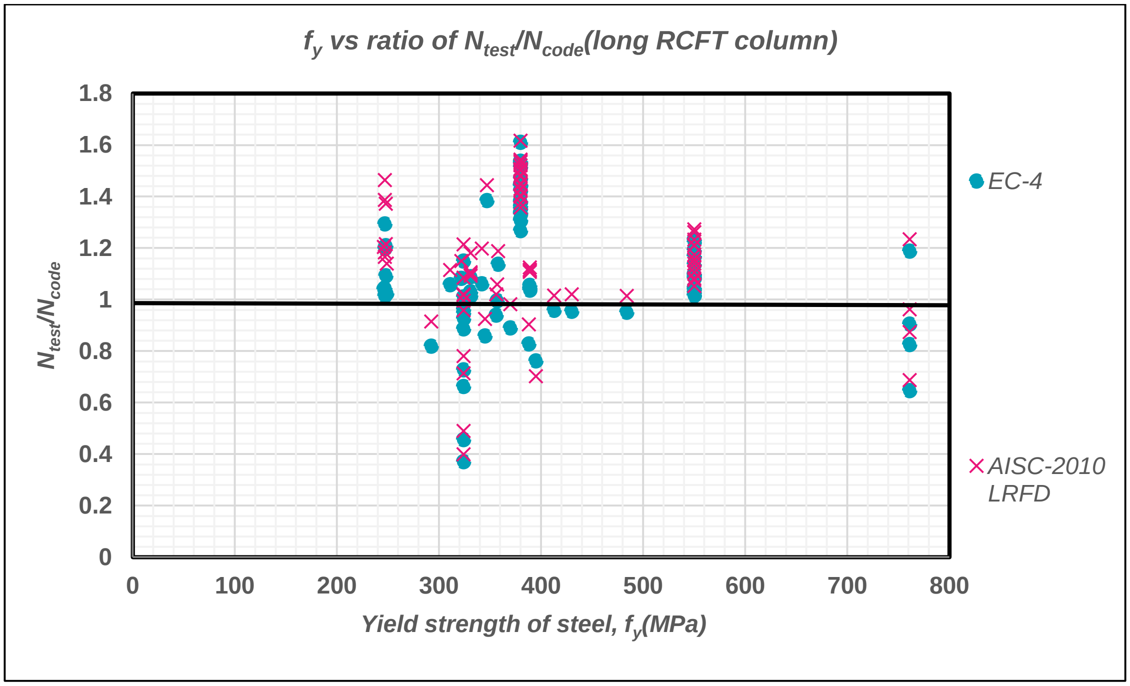

Figure 5.25

5.3 CIRCULAR BEAM-COLUMNS

266 circular CFT beam-columns were used for the final analysis in the database. 34 CCFT beam-columns were short and 232 CCFT beam-columns were slender or long. The range of material strength of steel and concrete, length to depth ratio, steel ratio for short and long CCFT columns used in the analyses are summarized in the table shown.

beam-columns		short CCFT	long CCFT
No. of specimens for analysis		34	232
f_y (Mpa)	max	363	517
	min	256	240
f_{ck} (Mpa)	max	113	131.2
	min	28.8	21
KL/D	max	4	33.4
	min	3	4.1
δ	max	0.67	0.92
	min	0.05	0.1

Table 5.7

5.3.1 Short Circular beam-columns

b) Analysis and results

- *Specimens above local buckling limit:* - 23 short circular beam columns specimens which failed local buckling criteria according to EC-4 are analyzed in the database and the mean value N_{test}/N_{Code} according to EC-4 was 1.14 and the AISC-2010 prediction was 1.12.

-*Specimens within EC-4 ranges:* - 11 of the specimens were within EC-4 limitations and the average value of N_{test}/N_{Code} according to EC-4 prediction was 1.5 and the AISC prediction for specimens which fall within EC-4 limitations was 1.43 as shown in the table.

		test/code	
short circular beam columns		*EC-4*	*AISC-2010*
specimens with (D/t) > 90*(235/f_y) of steel	*No. of tests*	23	
	avg	1.14	1.12
	st dev	0.12	0.13
	$N_{test}/N_{Code} \geq 1$	86%	85%
specimens with (D/t) < 90*(235/f_y) of steel	*No. of tests*	11	
	avg	1.50	1.43
	st dev	0.30	0.40
	$N_{test}/N_{Code} \geq 1$	100%	85%
specimens within EC-4 range	*No. of tests*	11	
	avg	1.50	1.43
	st dev	0.30	0.40
	$N_{test}/N_{Code} \geq 1$	100%	85%

Table 5.8

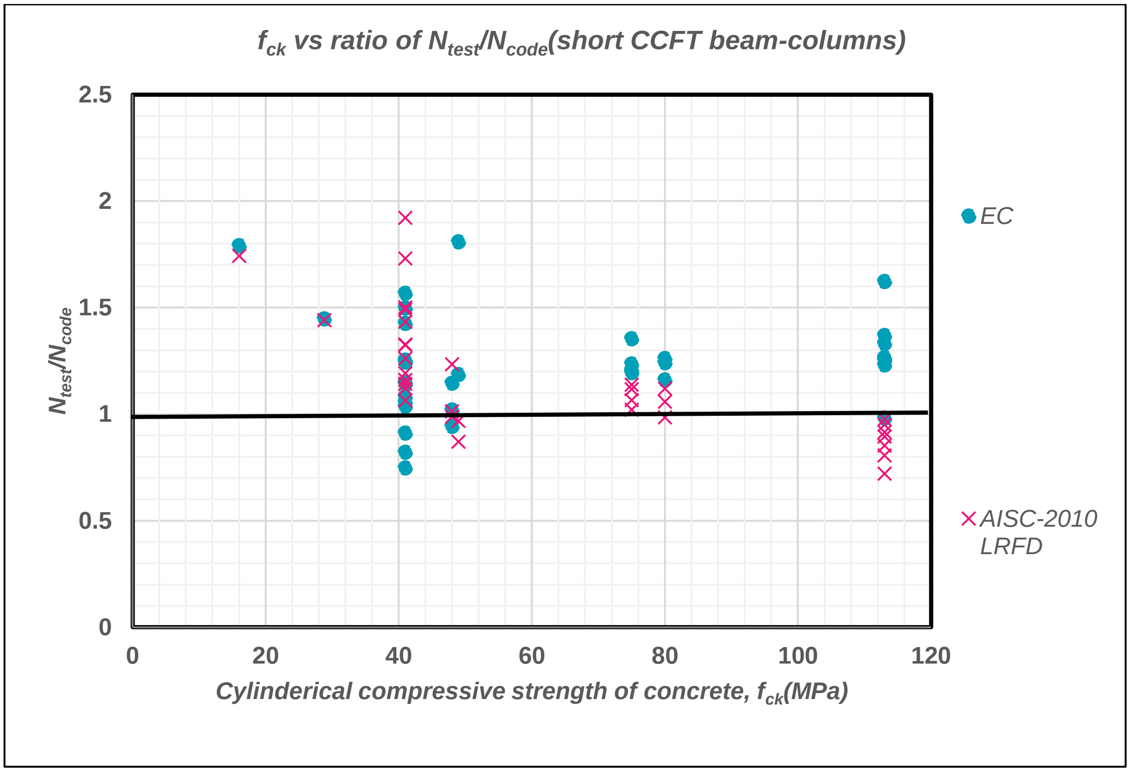

Figure 12

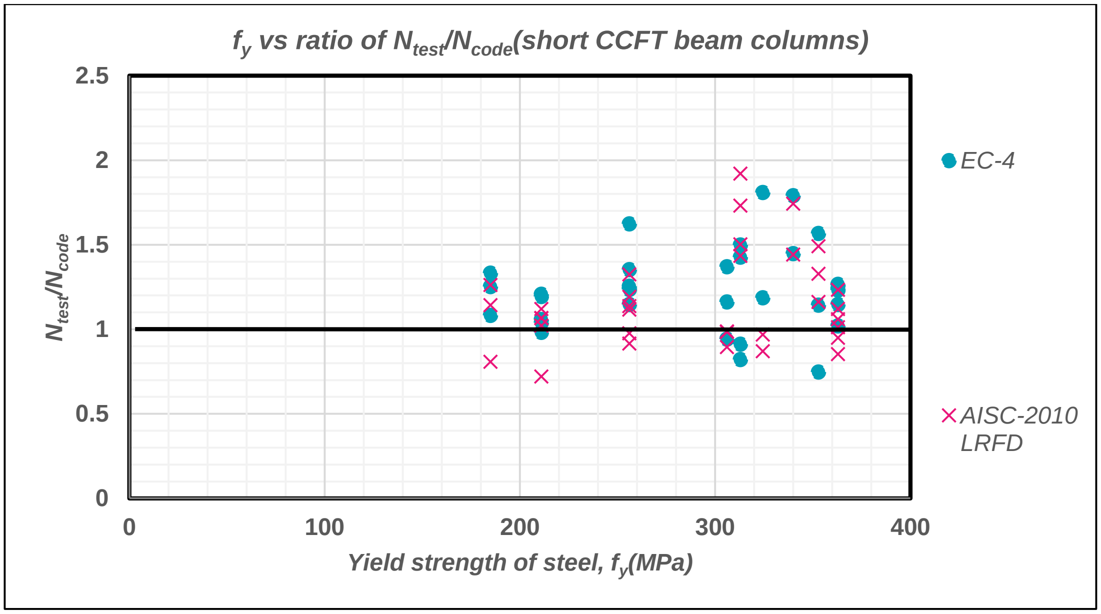

Figure 13

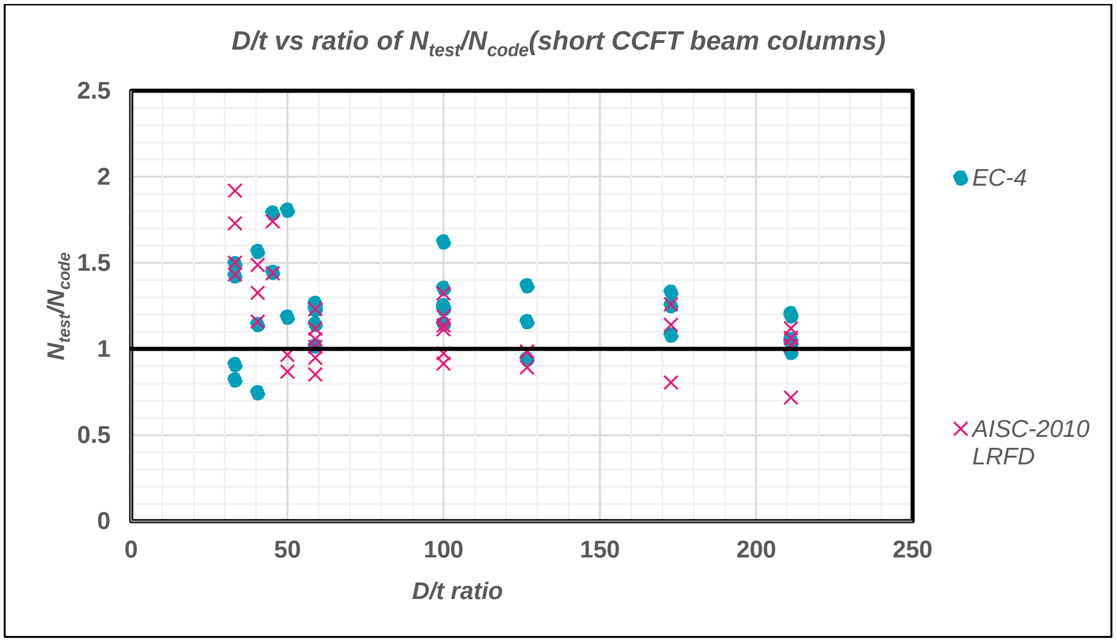

Figure 14

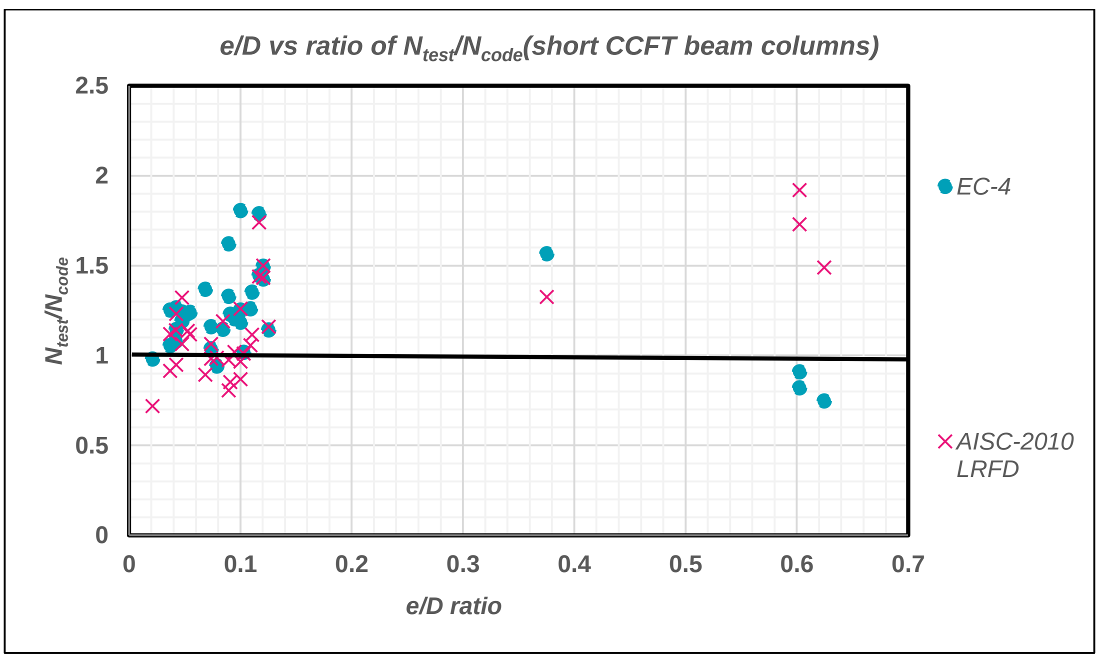

Figure 15

5.3.2 Long circular beam-columns

a) Frequency distributions

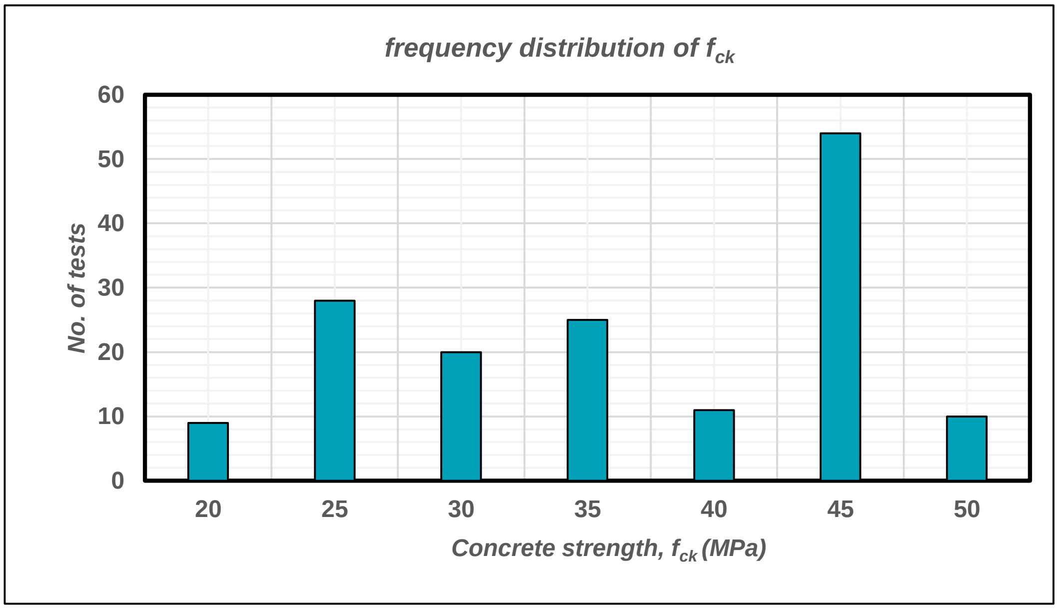

Figure 16

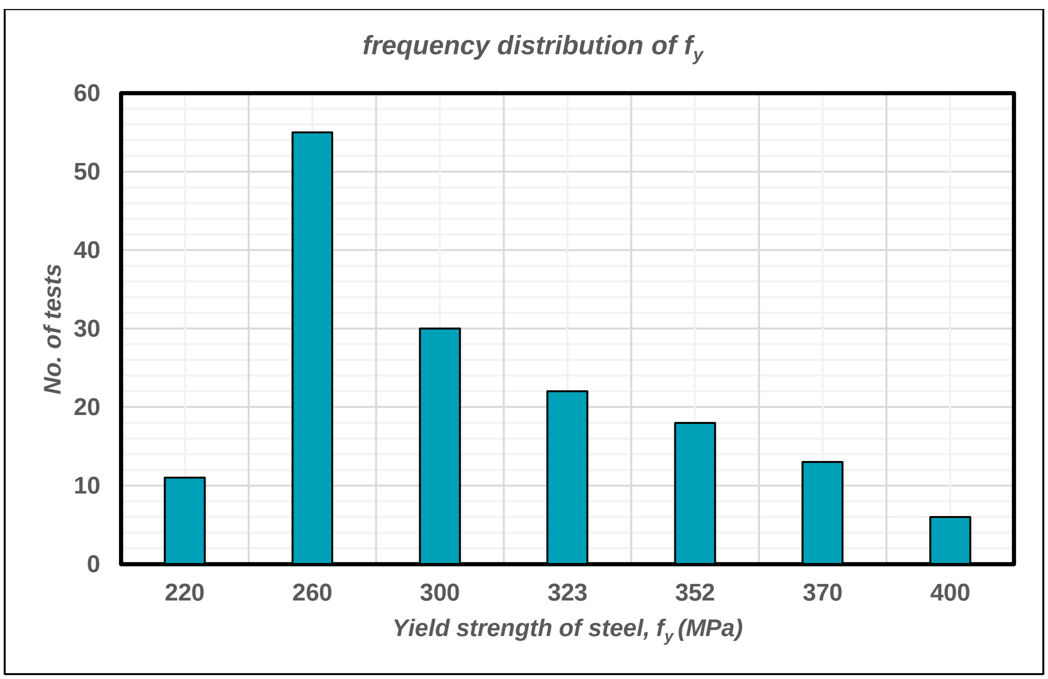

Figure 17

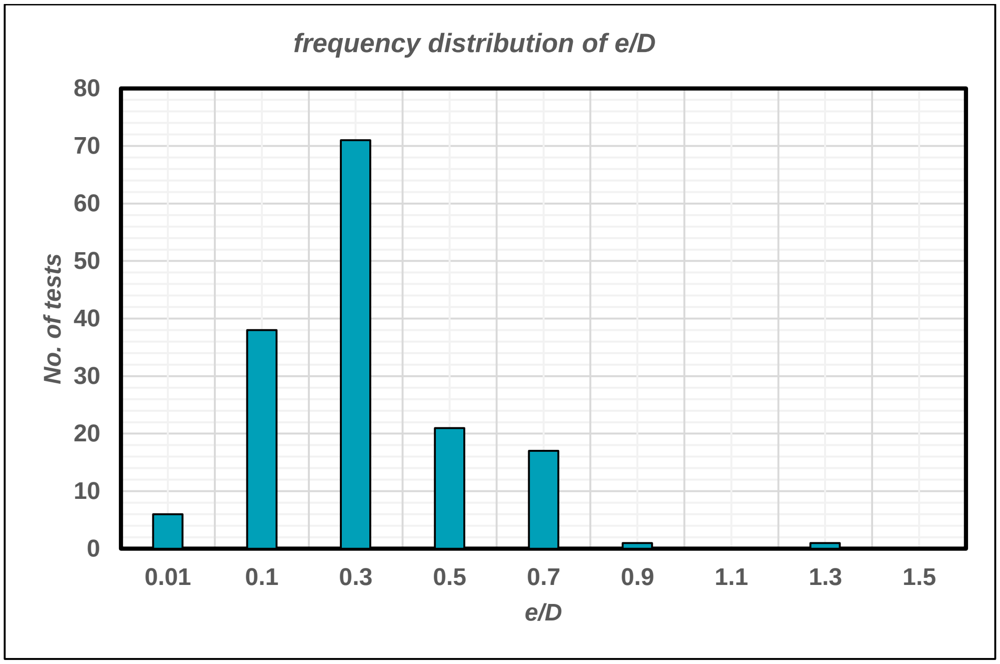

Figure 18

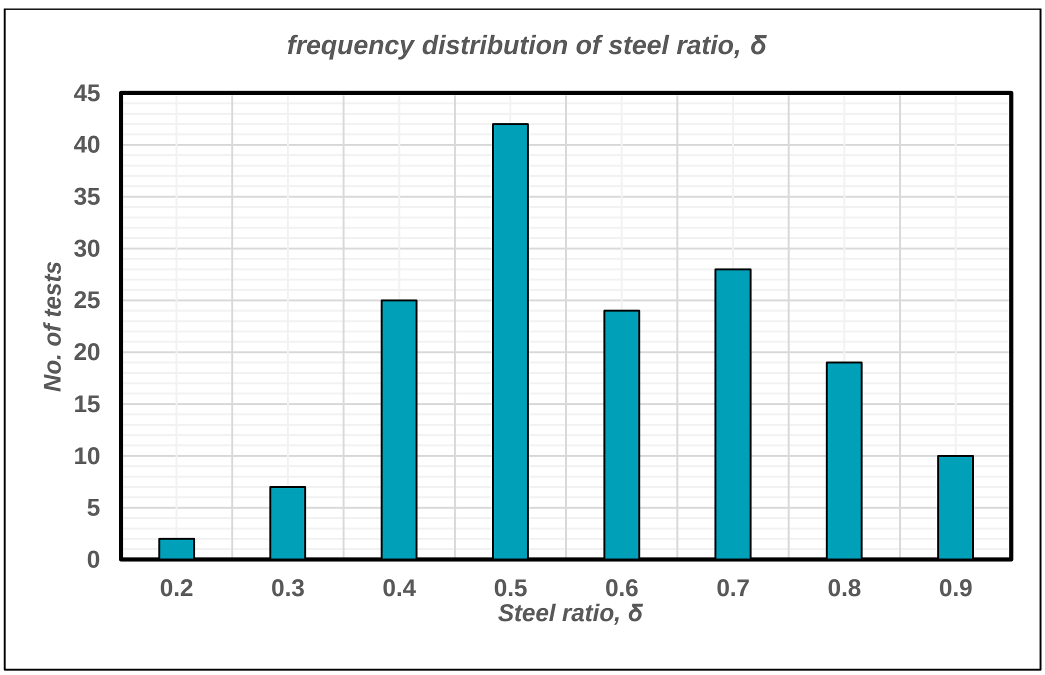

Figure 19

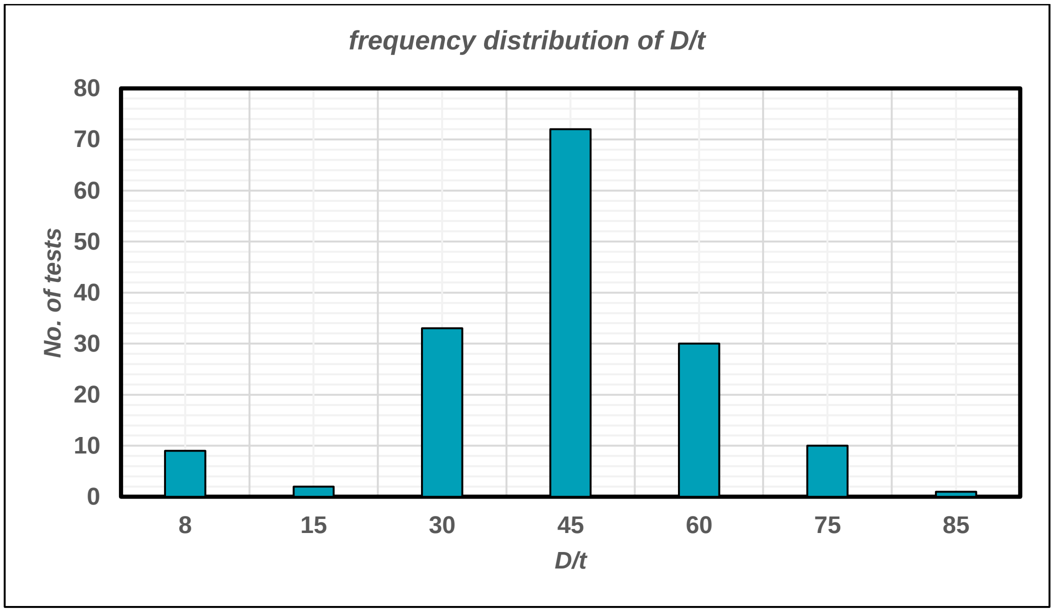
frequency distribution of D/t
No. of tests
80
70
60
50
40
30
20
10
0
8
15
30
45
60
75
85
D/t

Figure 20

b) Analysis and results

- *Specimens above local buckling limit:* - 17 long CCFT beam columns which failed local buckling criteria according to EC-4 are analyzed in the database and the mean value N_{test}/N_{Code} according to EC-4 was 0.84 and the AISC-2010 prediction was 0.91.

-*Specimens within EC-4 ranges:* - 133 of the specimens were within EC-4 limitations and the average value of N_{test}/N_{Code} according to EC-4 prediction was 0.99 and the AISC prediction for specimens which fall within EC-4 limitations was 1.17 as shown in the table.

		test/code	
long circular beam columns		*EC-4*	*AISC-2010*
specimens with (D/t) > 90*(235/f_y) of steel	*No. of tests*	17	
	avg	0.84	0.91
	st dev	0.33	0.38
	$N_{test}/N_{Code} \geq 1$	33%	46%
specimens with (D/t) < 90*(235/f_y) of steel	*No. of tests*	215	
	avg	0.92	1.07
	st dev	0.33	0.33
	$N_{test}/N_{Code} \geq 1$	35%	50%
specimens within EC-4 range	*No. of tests*	133	
	avg	0.99	1.17
	st dev	0.36	0.42
	$N_{test}/N_{Code} \geq 1$	50%	61%
$f_{ck} \geq 50$ MPa	*No. of tests*	80	
	avg	0.79	0.910
	st dev	0.19	0.290
	$N_{test}/N_{Code} \geq 1$	0%	33%
$f_y \geq 460$ MPa	*No. of tests*	5	
	avg	0.74	0.76
	st dev	0.11	0.12
	$N_{test}/N_{Code} \geq 1$	-	-

Table 5.9

- *Specimens with f_{ck} > 50MPa concrete strength*:-. The N_{test}/N_{Code} mean value of 80 specimens which have f_{ck} above 50MPa was 0.79 with a standard deviation of 0.19 and the AISC-2010 average prediction was 0.91 with a standard deviation of 0.29

- *Specimens with f_y > 460MPa steel yield strength:* -For columns with high yield strength (f_y > 460 MPa), EC-4 prediction was 0.74 and AISC-2010 prediction gave a value of 0.76.

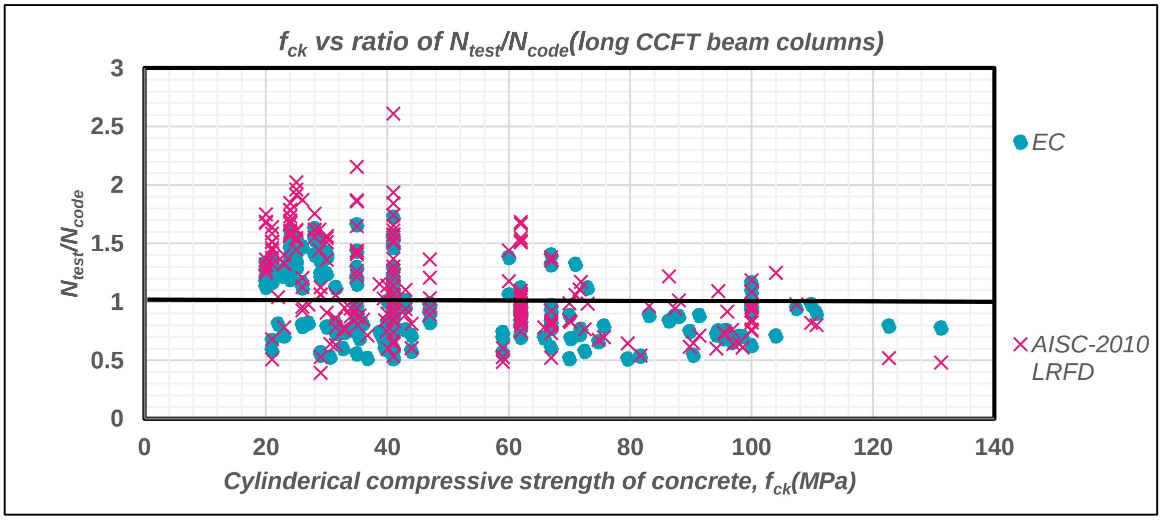

Figure 21

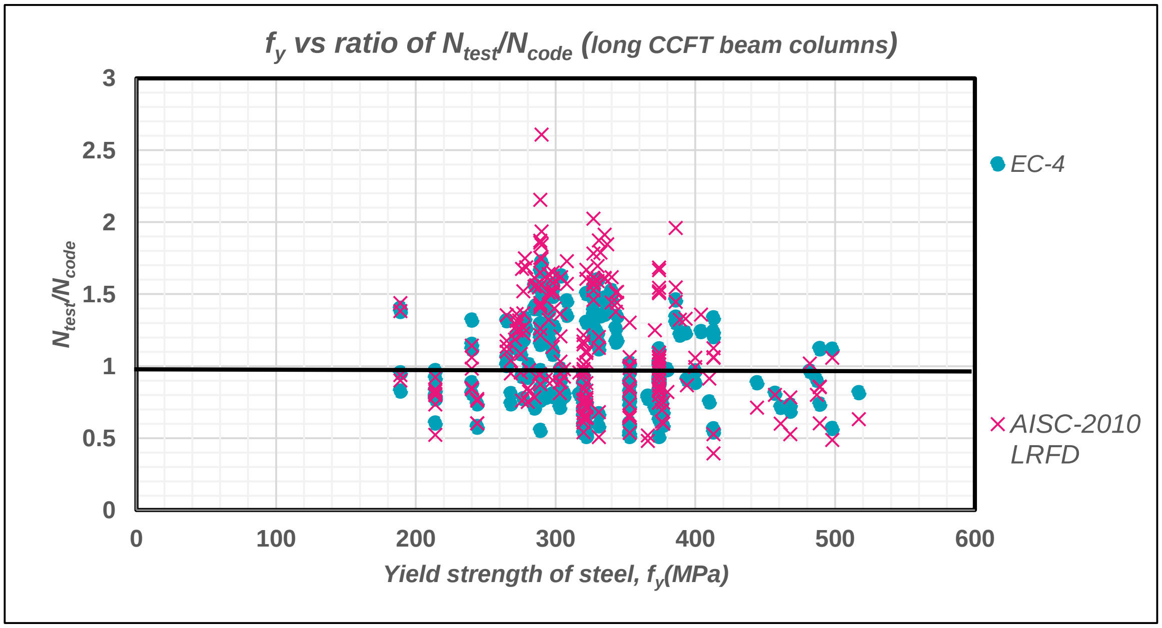

Figure 22

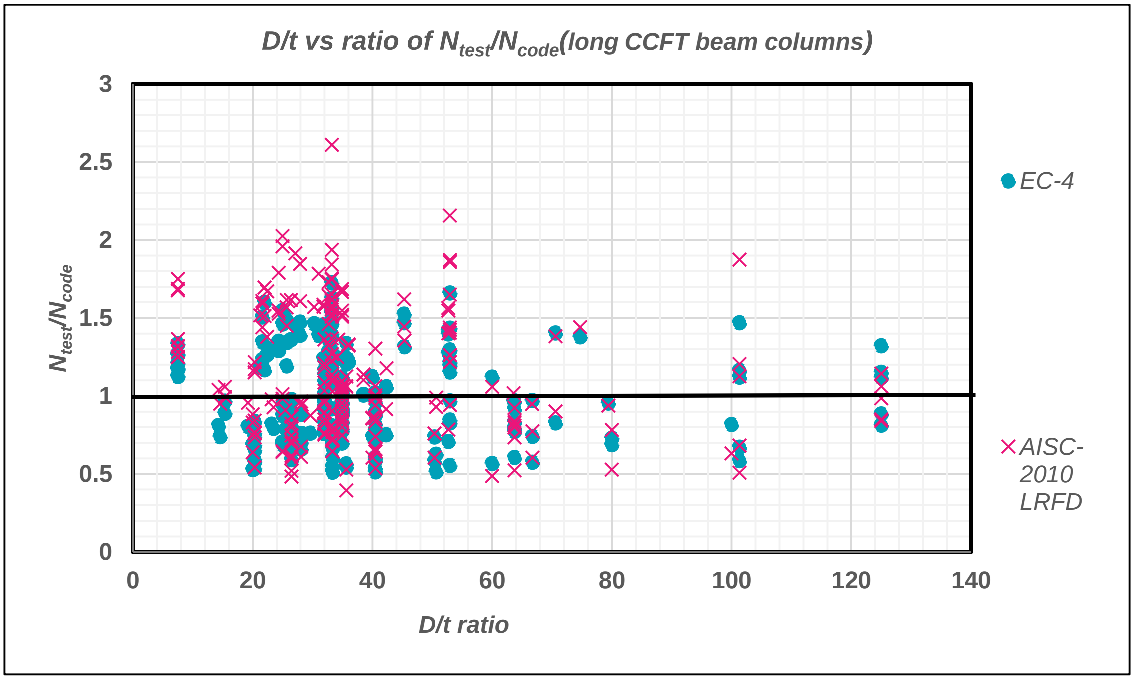

Figure 23

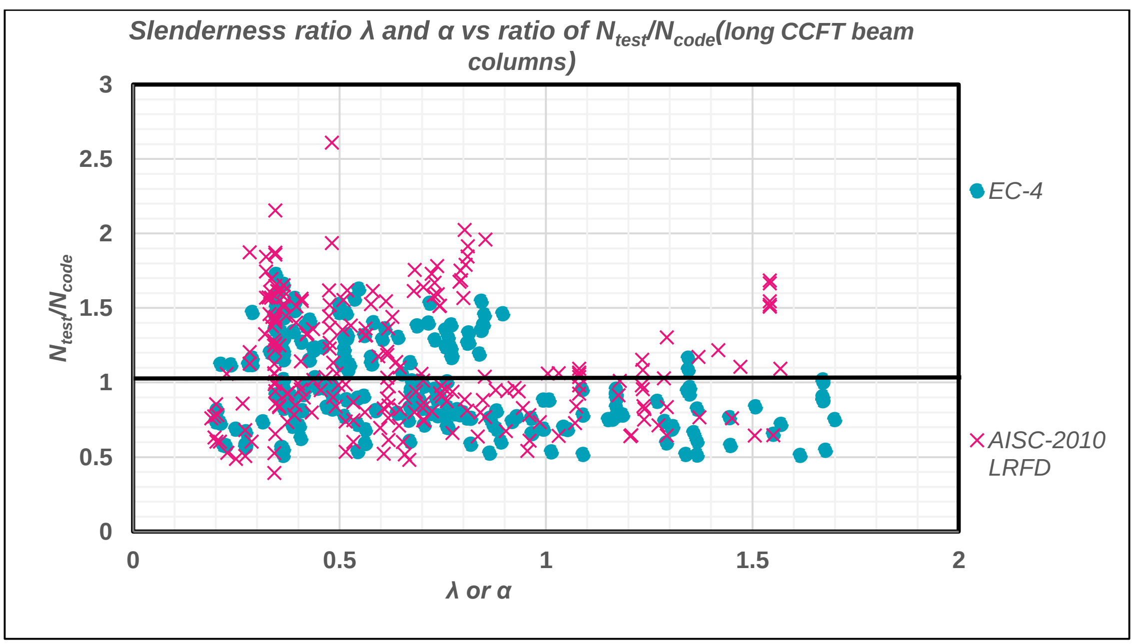

Figure 24

5.4 RECTANGULAR BEAM-COLUMNS

135 RCFT beam columns were used for the final analyses in the database. 66 RCFT beam columns were short and were slender beam column. The range of material strength of steel and concrete, length to depth ratio, steel ratio for short and long CCFT columns used in the analyses are summarized in the table shown.

beam-columns		short RCFT	long RCFT
No. of specimens for analysis		66	135
f_y (MPa)	max	835	750
	min	262	254
f_{ck} (MPa)	max	96	103
	min	25.4	23
KL/D	max	4	30
	min	2	5.8
δ	max	0.84	0.84
	min	0.25	0.28

Table 5.10

5.4.1 Short rectangular beam-columns

a) Frequency distributions

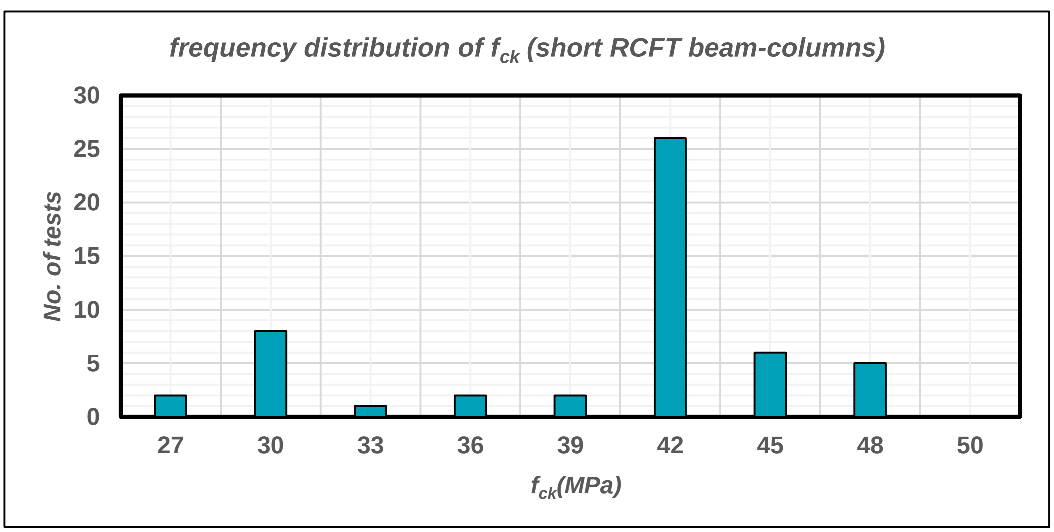

Figure 25

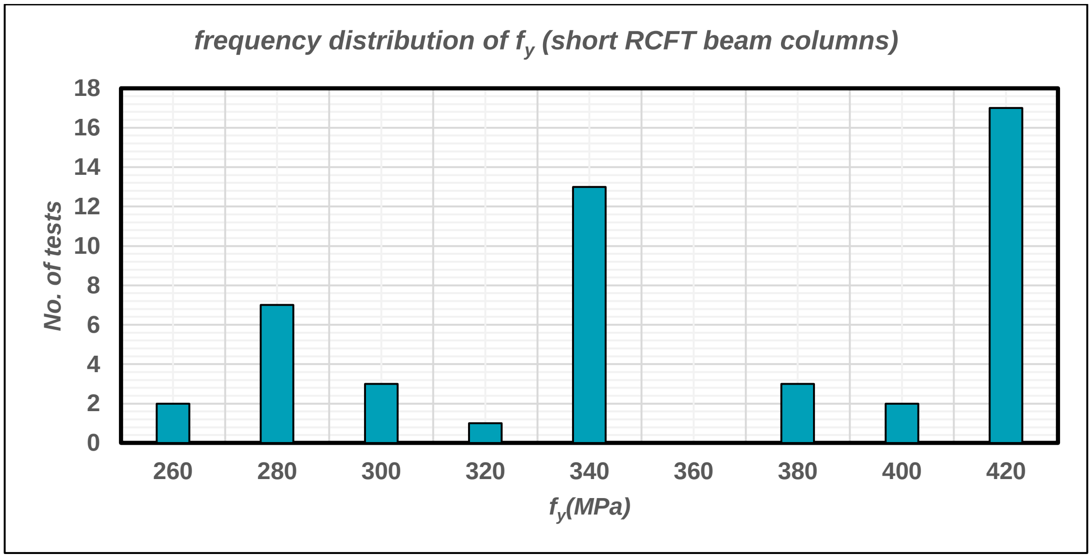

Figure 26

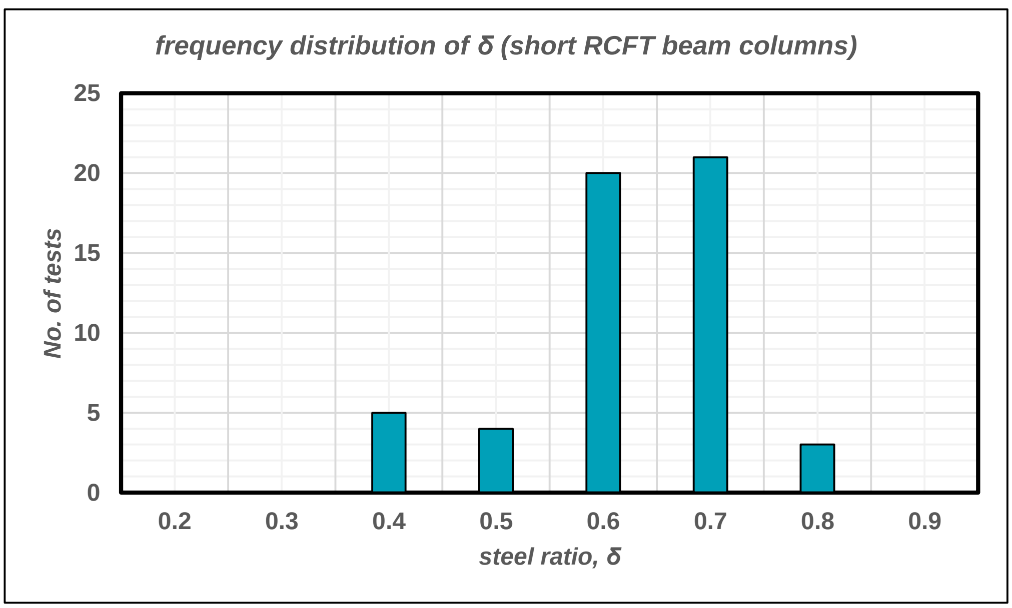

Figure 27

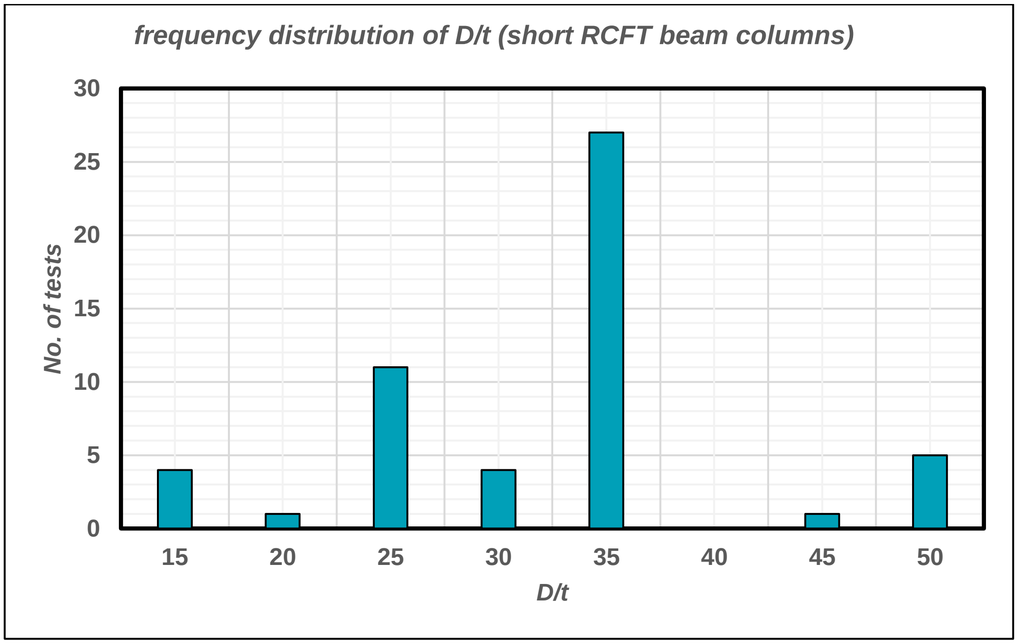

Figure 28

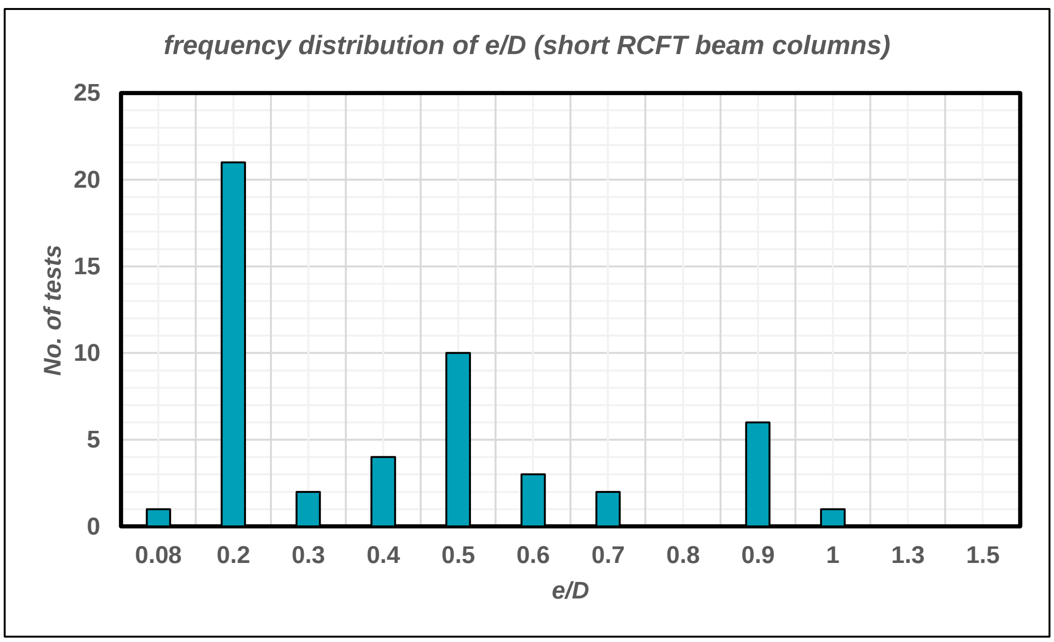

Figure 29

b) Analysis and results

- *Specimens above local buckling limit:* - 45 short rectangular beam columns specimens which failed local buckling criteria according to EC-4 are analyzed in the database and the mean value N_{test}/N_{Code} according to EC-4 was 0.9 and the AISC-2010 prediction was 0.89.

-*Specimens within EC-4 ranges:* - 5 of the specimens were within EC-4 limitations and the average value of N_{test}/N_{Code} according to EC-4 prediction was 0.79 and the AISC prediction for specimens which fall within EC-4 limitations was 0.86 as shown in the table.

		test/code	
short rectangular beam columns		*EC-4*	*AISC-2010*
specimens with (D/t) > 52*sqrt(235/f_y) of steel	*No. of tests*	45	
	avg	0.90	0.89
	st dev	0.33	0.32
	$N_{test}/N_{Code} \geq 1$	40%	39%
specimens with (D/t) < 52*sqrt(235/f_y) of steel	*No. of tests*	21	
	avg	1.16	1.03
	st dev	0.45	0.25
	$N_{test}/N_{Code} \geq 1$	63%	56%
specimens within EC-4 range	*No. of tests*	5	
	avg	0.79	0.86
	st dev	0.21	0.22
	$N_{test}/N_{Code} \geq 1$	-	-
$f_{ck} \geq 50$ MPa	*No. of tests*	12	
	avg	1.18	1.06
	st dev	0.27	0.18
	$N_{test}/N_{Code} \geq 1$	81%	74%
$f_y \geq 460$ MPa	*No. of tests*	14	
	avg	1.32	1.09
	st dev	0.45	0.22
	$N_{test}/N_{Code} \geq 1$	88%	78%

Table 5.11

- *Specimens with f_{ck} > 50MPa concrete strength*:-. The N_{test}/N_{Code} mean value of 12 specimens which have f_{ck} above 50MPa was 1.18 with a standard deviation of 0.27 and the AISC-2010 average prediction was 1.06 with a standard deviation of 0.18.

- *Specimens with f_y > 460MPa steel yield strength:* -For columns with high yield strength (f_y > 460 MPa), EC-4 prediction was 1.32 and AISC-2010 prediction gave a value of 1.09.

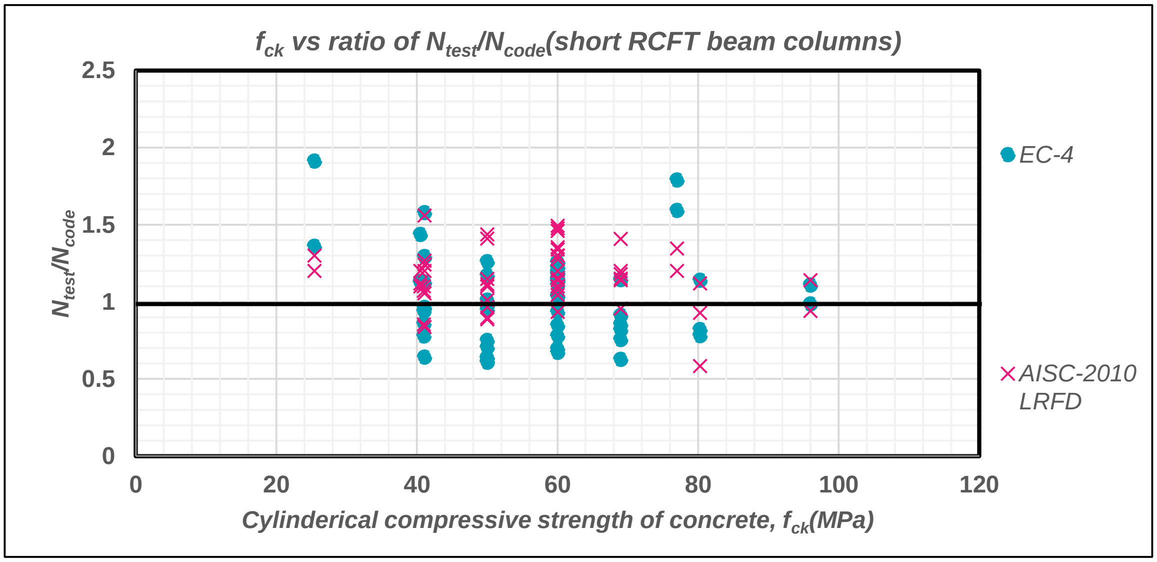

Figure 30

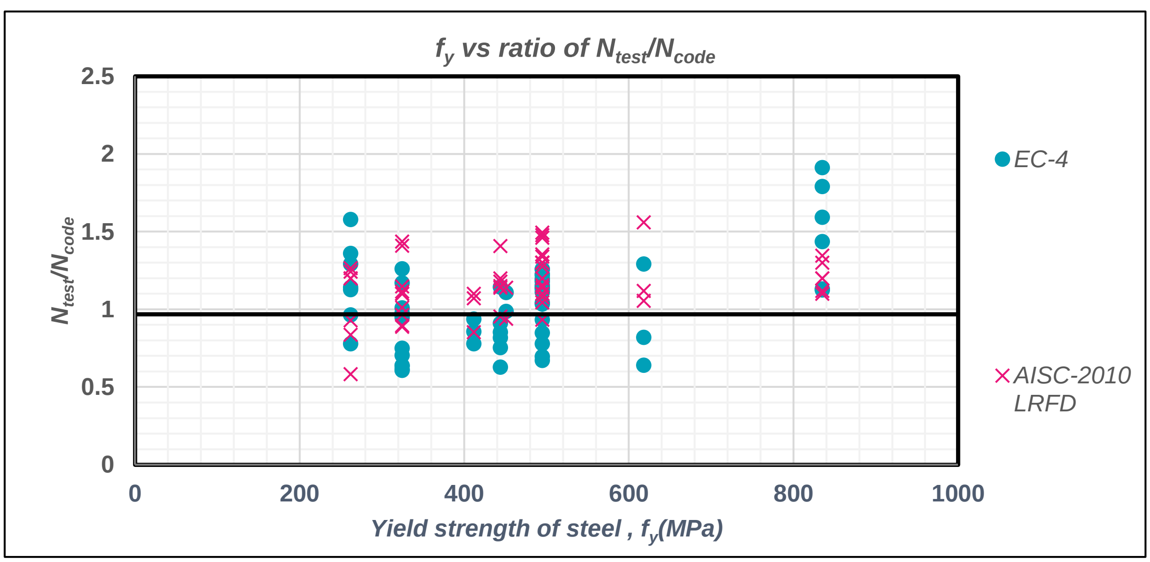

Figure 31

5.4.2 Long rectangular beam columns

135 long RCFT beam columns were analyzed for the final cases, out of which 84 specimens were above local buckling limit and 51 specimens were within local buckling limit.

b) Analysis and results

Specimens above local buckling limit: - 84 long rectangular beam columns specimens which failed local buckling criteria according to EC-4 are analyzed in the database and the mean value N_{test}/N_{Code} according to EC-4 was 0.8 and the AISC-2010 prediction was also 0.8.

-*Specimens within EC-4 ranges:* - 30 of the specimens were within EC-4 limitations and the average value of N_{test}/N_{Code} according to EC-4 prediction was 0.79 and the AISC prediction for specimens which fall within EC-4 limitations was 0.86 as shown in the table.

		test/code	
long rectangular beam columns		*EC-4*	*AISC-2010*
specimens with (D/t) > 52*sqrt(235/f_y) of steel	*No. of tests*	84	
	avg	0.80	0.80
	st dev	0.42	0.33
	$N_{test}/N_{Code} \geq 1$	14%	20%
specimens with (D/t) < 52*sqrt(235/f_y) of steel	*No. of tests*	51	
	avg	0.97	0.95
	st dev	0.37	0.35
	$N_{test}/N_{Code} \geq 1$	40%	22%
specimens within EC-4 range	*No. of tests*	30	
	avg	0.82	0.84
	st dev	0.22	0.23
	$N_{test}/N_{Code} \geq 1$	15%	13%
$f_{ck} \geq 50$ MPa	*No. of tests*	17	
	avg	1.15	1.060
	st dev	0.32	0.330
	$N_{test}/N_{Code} \geq 1$	81%	36%
$f_y \geq 460$ MPa	*No. of tests*	5	
	avg	1.16	1.04
	st dev	0.33	0.24
	$N_{test}/N_{Code} \geq 1$	67%	49%

Table 5.12

- *Specimens with f_{ck} > 50MPa concrete strength*:-. The N_{test}/N_{Code} mean value of 17 specimens which have f_{ck} above 50MPa was 1.15 with a standard deviation of 0.32 and the AISC-2010 average prediction was 1.06 with a standard deviation of 0.33.

- *Specimens with f_y > 460MPa steel yield strength:* -For columns with high yield strength (f_y > 460 MPa), EC-4 prediction was 1.16 and AISC-2010 prediction gave a value of 1.04.

CHAPTER 6

CONCLUSIONS

Published studies on experimental work of CFT columns are summarized with an emphasis on experimental properties and key results from the work. The data includes specimens which are subjected to concentric, eccentric, cyclic loading test. Experimental test specimens on CFT columns which are tested under monotonic loading and beam columns with a single curvatures are compiled and Comparisons are made between the EC-4 and the AISC-2010 predictions for these specimen’s failure load.

The database includes 1100 tested specimens which are 294 circular CFT columns, 289 rectangular CFT columns, 286 circular CFT beam-columns, and 232 rectangular CFT beam-columns. The range of material and geometric properties of CFT columns are summarized in the table shown below. Number of tested CFT column specimens within the range of EC-4 limitations and Comparisons made on prediction of CFT specimen’s failure load by EC-4 and AISC-2010 are also summarized in the table shown

6.1 COLUMNS WITHIN EC-4 RANGE

columns		short CCFT	long CCFT	short RCFT	long RCFT
No. of specimens for analysis		145	149	210	82
f_y (MPa)	max	853	626	835	761
	min	237	221	239	246
f_{ck} (MPa)	max	110.3	120.1	106	96
	min	12	21	21	21
KL/D	max	3.99	45.5	3.99	27.3
	min	1.07	4.2	0.98	4.01
δ	max	0.93	0.77	0.89	0.88
	min	0.053	0.23	0.12	0.174

Table 6.1

specimens within EC-4 range(columns)		test/code	
		EC-4	*AISC-2010*
short CCFT	*No. of tests*	65	
	avg	1.05	1.39
	st dev	0.15	0.21
	$N_{test}/N_{Code} \geq 1$	65%	100%
long CCFT	*No. of tests*	95	
	avg	1.10	1.16
	st dev	0.13	0.16
	$N_{test}/N_{Code} \geq 1$	83%	92%
short RCFT	*No. of tests*	52	
	avg	1.17	1.27
	st dev	0.19	0.21
	$N_{test}/N_{Code} \geq 1$	84%	90%
long RCFT	*No. of tests*	32	
	avg	1.03	1.09
	st dev	0.18	0.19
	$N_{test}/N_{Code} \geq 1$	58%	79%

Table 6.2

For CFT columns the EC-4 predicts close to the experimental value with the minimum value of standard deviation than the AISC-2010 prediction. On the other hand the AISC-2010 give a conservative prediction.

6.2 BEAM-COLUMNS WITHIN EC-4 RANGE

beam-columns		short CCFT	long CCFT	short RCFT	long RCFT
No. of specimens for analysis		34	232	66	135
f_y (MPa)	max	363	517	835	750
	min	256	240	262	254
f_{ck} (MPa)	max	113	131.2	96	103
	min	28.8	21	25.4	23
KL/D	max	4	33.4	4	30
	min	3	4.1	2	5.8
δ	max	0.67	0.92	0.84	0.84
	min	0.05	0.1	0.25	0.28

Table 6.3

specimens within EC-4 range(beam-columns)		test/code	
		EC-4	*AISC-2010*
short CCFT	*No. of tests*	11	
	avg	1.50	1.43
	st dev	0.30	0.40
	$N_{test}/N_{Code} \geq 1$	100%	85%
long CCFT	*No. of tests*	133	
	avg	0.99	1.17
	st dev	0.36	0.42
	$N_{test}/N_{Code} \geq 1$	50%	61%
short RCFT	*No. of tests*	5	
	avg	0.79	0.86
	st dev	0.21	0.22
	$N_{test}/N_{Code} \geq 1$	-	-
long RCFT	*No. of tests*	30	
	avg	0.82	0.84
	st dev	0.22	0.23
	$N_{test}/N_{Code} \geq 1$	15%	13%

Table 6.4

For circular CFT beam-columns the EC-4 predicts close to the experimental value with the minimum value of standard deviation while the AISC-2010 predicts conservatively. For rectangular CFT beam-columns both the EC-4 and the AISC-2010 give the same prediction.

6.3 COLUMNS WITH CONCRETE STRENGTH f_{ck} > 50MPa

		columns		beam-columns	
$f_{ck} \geq 50$ MPa		test/code		test/code	
		EC-4	*AISC-2010*	*EC-4*	*AISC-2010*
short CCFT	*No. of tests*	16		-	
	avg	1.32	1.23	-	-
	st dev	0.10	0.09	-	-
	$N_{test}/N_{Code} \geq 1$	100%	100%	-	-
long CCFT	*No. of tests*	20		80	
	avg	1.15	1.02	0.79	0.910
	st dev	0.17	0.08	0.19	0.290
	$N_{test}/N_{Code} \geq 1$	100%	67%	0%	33%
short RCFT	*No. of tests*	92		12	
	avg	1.05	1.03	1.18	1.06
	st dev	0.26	0.25	0.27	0.18
	$N_{test}/N_{Code} \geq 1$	83%	80%	81%	74%
long RCFT	*No. of tests*	23		17	
	avg	1.36	1.40	1.15	1.060
	st dev	0.19	0.19	0.32	0.330
	$N_{test}/N_{Code} \geq 1$	97%	97%	81%	36%

Table 6.5

For CFT columns and beam-columns filled with high strength concrete (f_{ck} > 50 MPa) the prediction by the EC-4 give a good and comparable result with AISC-2010's prediction by introduction of reduction factor for high strength concrete according to EC-2's specification and by ignoring the confinement effect. The AISC-2010 give a good prediction for high strength CFT.

6.4 COLUMNS ABOVE EC-4 STEEL LOCAL BUCKLING LIMIT

		columns		beam-columns	
specimens above EC-4 local buckling limit		test/code		test/code	
		EC-4	*AISC-2010*	*EC-4*	*AISC-2010*
short CCFT	*No. of tests*	54		23	
	avg	1.04	1.15	1.14	1.12
	st dev	0.21	0.17	0.12	0.13
	$N_{test}/N_{Code} \geq 1$	59%	78%	86%	85%
long CCFT	*No. of tests*	22		17	
	avg	1.15	1.22	0.84	0.91
	st dev	0.18	0.19	0.33	0.38
	$N_{test}/N_{Code} \geq 1$	90%	94%	33%	46%
short RCFT	*No. of tests*	78		45	
	avg	0.95	1.03	0.90	0.89
	st dev	0.19	0.21	0.33	0.32
	$N_{test}/N_{Code} \geq 1$	32%	53%	40%	39%
long RCFT	*No. of tests*	28		84	
	avg	1.04	1.13	0.80	0.80
	st dev	0.13	0.16	0.42	0.33
	$N_{test}/N_{Code} \geq 1$	79%	84%	14%	20%

Table 6.6

For CFT column specimens above EC-4 local buckling limit both EC-4 prediction and AISC-2010 prediction give a good result.

RECCOMENDATIONS

- ✓ For circular section columns the Code limitation on concrete cylinder strength could be safely extended higher than 50 MPa for Eurocode.
- ✓ For high strength concrete with f_{ck} > 50 MPa, the effective compressive strength of concrete in accordance with EC2 (EN 1992-1-1, 2004) should be used.
- ✓ Confinement effect should be neglected for higher strength concrete f_{ck} > 50 MPa.

APPENDIX 1:

CFT COLUMNS AND BEAM-COLUMNS DATABASE

1. CIRCULAR CFT COLUMNS

G. Giakoumelis, D. Lam (2003)										EC-4	AISC-2010
test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
1	C3	114.4	4.0	300.0	343.0	31.4	8903.2	1381.0	948	0.96	1.39
2	C4	114.6	4.0	300.0	343.0	93.6	8923.2	1386.1	1308	1.25	1.19
3	C5a	114.4	3.8	300.0	343.0	34.7	8956.8	1327.4	929	0.94	1.35
4	C6a	114.3	3.9	300.0	343.0	97.2	8891.5	1362.2	1359	1.30	1.22
5	C7	114.9	4.9	300.5	365.0	34.7	8668.9	1696.3	1380	1.14	1.64
6	C8	115.0	4.9	300.0	365.0	104.9	8692.0	1702.1	1787	1.53	1.37
7	C9	115.0	5.0	300.5	365.0	57.6	8655.7	1734.8	1413	1.42	1.40
8	C10a	114.5	3.8	299.3	343.0	57.6	8990.3	1304.6	1038	1.20	1.24
9	C11	114.3	3.8	300.0	343.0	57.6	8956.8	1302.3	1067	1.24	1.28
10	C12	114.3	3.9	300.0	343.0	31.9	8924.9	1335.9	998	1.03	1.49
11	C13a	114.1	3.9	300.5	343.0	31.9	8889.8	1333.4	948	0.98	1.42
12	C14	114.5	3.8	300.0	343.0	98.9	8968.5	1335.5	1359	1.30	1.21
13	C15a	114.4	3.9	299.5	343.0	98.9	8936.6	1336.8	1182	1.13	1.05

M.D., O'shea, R.Q., Bridge (1997)										EC-4	AISC-2010
test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
14	S30CS50B	165	2.82	580.5	363.3	48.3	19945.7	1436.8	1662	0.99	1.17
15	S20CS50A	190	1.94	663.5	256.4	41	27206.7	1146.2	1678	1.10	1.26
16	S16CS50B	190	1.52	664.5	306.1	48.3	27452.8	900.0	1695	1.00	1.12
17	S12CS50A	190	1.13	664.5	185.7	41	27682.4	670.5	1377	1.05	1.17
18	S10CS50A	190	0.86	659	210.7	41	27841.9	511.0	1350	1.05	1.16
19	S30CS80A	165	2.82	580.5	363.3	80.2	19945.7	1436.8	2295	1.27	1.14
20	S20CS80B	190	1.94	663.5	256.4	74.7	27206.7	1146.2	2592	1.28	1.19
21	S16CS80A	190	1.52	663.5	306.1	80.2	27452.8	900.0	2602	1.24	1.12
22	S12CS80A	190	1.13	662.5	185.7	80.2	27682.4	670.5	2295	1.16	1.05
23	S10CS80B	190	0.86	663.5	210.7	74.7	27841.9	511.0	2451	1.29	1.21

		Sakino, K., Nakahara, H., Morino, S., Nishiyama (2004)								EC-4	AISC-2010
test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm²)	f_{ck} (N/mm²)	A_c (mm²)	A_s (mm²)	N_{test} (kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
24	CC4-A-2	149	2.96	223.5	308	25.4	16078.6	1358.0	941	0.85	1.17
25	CC4-A-4-1	149	2.96	223.5	308	40.5	16078.6	1358.0	1064	0.79	1.03
26	CC4-A-4-2	149	2.96	223.5	308	40.5	16078.6	1358.0	1080	0.80	1.04
27	CC4-A-8	149	2.96	223.5	308	77	16078.6	1358.0	1781	1.27	1.12
28	CC4-C-2	301	2.96	322.6	279	25.4	68386.4	2771.5	2382	0.77	0.98
29	CC4-C-4-1	300	2.96	322.6	279	41.1	67923.6	2762.2	3277	0.79	0.96
30	CC4-C-4-2	300	2.96	322.6	279	41.1	67923.6	2762.2	3152	0.76	0.92
31	CC4-C-8	301	2.96	322.6	279	80.3	68386.4	2771.5	5540	1.05	0.93
32	CC4-D-2	450	2.96	485.1	279	25.4	154886.1	4157.1	4415	0.74	0.90
33	CC4-D-4-1	450	2.96	485.1	279	41.1	154886.1	4157.1	6870	0.82	0.95
34	CC4-D-4-2	450	2.96	485.1	279	41.1	154886.1	4157.1	6985	0.83	0.97
35	CC4-D-8	450	2.96	485.1	279	85.1	154886.1	4157.1	11665	0.99	0.85
36	CC6-A-2	122	4.54	215.9	576	25.4	10014.6	1675.3	1509	0.86	1.25
37	CC6-A-4-1	122	4.54	215.9	576	40.5	10014.6	1675.3	1657	0.88	1.23
38	CC6-A-4-2	122	4.54	215.9	576	40.5	10014.6	1675.3	1663	0.88	1.24
39	CC6-A-8	122	4.54	215.9	576	77	10014.6	1675.3	2100	1.46	1.24
40	CC6-C-2	239	4.54	315	507	25.4	41518.7	3344.1	3035	0.78	1.13
41	CC6-C-4-1	238	4.54	315	507	40.5	41158.3	3329.8	3583	0.80	1.10
42	CC6-C-4-2	238	4.54	315	507	40.5	41158.3	3329.8	3647	0.81	1.12
43	CC6-C-8	238	4.54	315	507	77	41158.3	3329.8	5578	1.37	1.19
44	CC6-D-2	361	4.54	477.5	525	25.4	97269.7	5084.1	5633	0.80	1.12
45	CC6-D-4-1	361	4.54	477.5	525	41.1	97269.7	5084.1	7260	0.86	1.12
46	CC6-D-4-2	360	4.54	477.5	525	41.1	96717.7	5069.9	7045	0.83	1.10
47	CC6-D-8	360	4.54	477.5	525	85.1	96717.7	5069.9	11505	1.29	1.10
48	CC8-A-2	108	6.47	180.3	853	25.4	7097.2	2063.7	2275	0.83	1.18
49	CC8-A-4-1	109	6.47	180.3	853	40.5	7247.3	2084.0	2446	0.85	1.19
50	CC8-A-4-2	108	6.47	180.3	853	40.5	7097.2	2063.7	2402	0.84	1.19
51	CC8-A-8	108	6.47	180.3	853	77	7097.2	2063.7	2713	1.44	1.20
52	CC8-C-2	222	6.47	261.6	843	25.4	34326.7	4380.9	4964	0.72	1.10
53	CC8-C-4-1	222	6.47	261.6	843	40.5	34326.7	4380.9	5638	0.76	1.13
54	CC8-C-4-2	222	6.47	261.6	843	40.5	34326.7	4380.9	5714	0.77	1.14
55	CC8-C-8	222	6.47	261.6	843	77	34326.7	4380.9	7304	1.41	1.18
56	CC8-D-2	337	6.47	396.2	823	25.4	82478.5	6718.4	8475	0.75	1.13
57	CC8-D-4-1	337	6.47	396.2	823	41.1	82478.5	6718.4	9668	0.77	1.11
58	CC8-D-4-2	337	6.47	396.2	823	41.1	82478.5	6718.4	9835	0.78	1.13
59	CC8-D-8	337	6.47	396.2	823	85.1	82478.5	6718.4	13776	1.36	1.13

										EC-4	AISC-2010
				J. Zeghiche, K. Chaoui (2004)							
test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
60	1	160.1	4.98	2000	280	40	17704.5	2426.9	1261	0.99	1.03
61	2	160.2	4.96	2500	281	41	17737.5	2419.0	1244	1.02	1.07
62	3	160.3	5	3000	270	43	17742.2	2439.4	1236	1.08	1.13
63	4	160.2	4.97	3500	273	41	17732.8	2423.7	1193	1.18	1.20
64	5	159.9	4.98	4000	281	45	17657.3	2423.7	1091	1.21	1.16
65	6	159.8	5.01	2000	283	70	17619.7	2436.3	1650	1.03	1.01
66	7	159.7	5.2	2500	281	71	17506.9	2524.0	1562	1.05	1.02
67	8	159.8	5.1	3000	276	73	17577.3	2478.6	1468	1.11	1.04
68	9	160.1	4.98	3500	276	74	17704.5	2426.9	1326	1.18	1.05
69	10	160.2	5.02	4000	281	71	17709.2	2447.3	1231	1.30	1.12
70	11	160.3	5.03	2000	281	99	17728.1	2453.6	2000	1.16	0.99
71	12	159.8	5.01	2500	275	100	17619.7	2436.3	1818	1.19	0.99
72	13	159.7	4.97	3000	275	101	17615.0	2415.9	1636	1.26	1.00
73	14	159.6	4.98	3500	270	106	17586.7	2419.1	1454	1.39	0.99
74	15	159.8	4.97	4000	270	102	17638.5	2417.5	1333	1.54	1.07

										EC-4	AISC-2010
				J.C.M. Ho, M.H. Lai (2013)							
test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
75	CTN0-5-30	168	5	330	365	36.4	19606.7	2560.4	1908	0.87	1.30
76	CTN0-8-30	168	8	330	365	44.0	18145.8	4021.2	2810	0.92	1.36
77	CTN0-5-80	168	5	330	365	106.8	19606.7	2560.4	2926	1.30	1.16
78	CTN0-8-80	168	8	330	365	94.0	18145.8	4021.2	3101	1.24	1.13

										EC-4	AISC-2010
				J.M. Portoles, E.Serra, M.L. Romero (2013)							
test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
79	C-NSC-0	159	6	2135	394	37.7	16971.7	2884	1535	0.97	1.01
80	C-UHSC-0	159	6	2135	457	120.1	16971.7	2884	2792	1.34	1.07
81	C-UHSC-0b	159	6	2135	487	116	16971.7	2884	2193	1.02	0.84

L.H. Han, G.H. Yao (2004)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
82	scsc1-1	100	3	300	303.5	46.8	6939.8	914.2	708	0.98	1.22
83	scsc1-2	100	3	300	303.5	46.8	6939.8	914.2	820	1.13	1.41
84	sch1-1	100	3	300	303.5	46.8	6939.8	914.2	766	1.06	1.32
85	sch1-2	100	3	300	303.5	46.8	6939.8	914.2	820	1.13	1.41
86	scv1-1	100	3	300	303.5	46.8	6939.8	914.2	780	1.08	1.34
87	scv1-2	100	3	300	303.5	46.8	6939.8	914.2	814	1.12	1.40
88	scsc2-1	200	3	600	303.5	46.8	29559.2	1856.7	2320	1.06	1.25
89	scsc2-2	200	3	600	303.5	46.8	29559.2	1856.7	2330	1.06	1.25
90	sch2-1	200	3	600	303.5	46.8	29559.2	1856.7	2160	0.98	1.16
91	sch2-2	200	3	600	303.5	46.8	29559.2	1856.7	2160	0.98	1.16
92	scv2-1	200	3	600	303.5	46.8	29559.2	1856.7	2383	1.09	1.28
93	scv2-2	200	3	600	303.5	46.8	29559.2	1856.7	2256	1.03	1.21
94	lcsc1-1	200	3	2000	303.5	46.8	29559.2	1856.7	1830	1.01	1.06
95	lcsc1-2	200	3	2000	303.5	46.8	29559.2	1856.7	1806	1.00	1.05
96	lch1-1	200	3	2000	303.5	46.8	29559.2	1856.7	1882	1.04	1.09
97	lch1-2	200	3	2000	303.5	46.8	29559.2	1856.7	2060	1.14	1.20
98	lcv1	200	3	2000	303.5	46.8	29559.2	1856.7	2115	1.17	1.23

M. Dundu (2006)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
99	S1-1	114.9	3	1000	354.05	32.1	9305.7	1054.2	806.4	1.24	1.30
100	S1-2	114.9	3	1500	354.05	32.1	9305.7	1054.2	688.2	1.13	1.18
101	S1-3	114.9	3	2000	354.05	32.1	9305.7	1054.2	632.2	1.15	1.20
102	S1-4	114.9	3	2500	354.05	32.1	9305.7	1054.2	566.1	1.19	1.21
103	S1-5	127.3	3	1000	345.2	32.1	11556.1	1171.5	912.1	1.18	1.26
104	S1-6	127.3	3	1500	345.2	32.1	11556.1	1171.5	848.5	1.19	1.24
105	S1-7	127.3	3	2000	345.2	32.1	11556.1	1171.5	715.8	1.08	1.13
106	S1-8	127.3	3	2500	345.2	32.1	11556.1	1171.5	638.8	1.07	1.11
107	S1-9	139.2	3	1000	361.95	32.1	13934.7	1283.7	1059.8	1.14	1.24
108	S1-10	139.2	3	1500	361.95	32.1	13934.7	1283.7	941.9	1.12	1.15
109	S1-11	139.2	3	2000	361.95	32.1	13934.7	1283.7	868.3	1.08	1.14
110	S1-12	139.2	3	2500	361.95	32.1	13934.7	1283.7	750.7	1.03	1.07
111	S2-1	152.4	3	1000	488.2	25.6	16833.4	1408.1	1463.3	1.26	1.38
112	S2-2	152.4	3	1500	488.2	25.6	16833.4	1408.1	1209.1	1.17	1.20
113	S2-3	152.4	3	2000	488.2	25.6	16833.4	1408.1	1167.3	1.18	1.23

114	S2-4	152.4	3	2500	394.3	25.6	16833.4	1408.1	968.9	1.17	1.23
115	S2-5	165.1	3	1000	438.2	25.6	19880.6	1527.8	1549.5	1.23	1.38
116	S2-6	165.1	3	1500	438.2	25.6	19880.6	1527.8	1338.0	1.20	1.24
117	S2-7	165.1	3	2000	438.2	25.6	19880.6	1527.8	1234.5	1.15	1.20
118	S2-8	165.1	3	2500	430.3	25.6	19880.6	1527.8	1232.0	1.23	1.30
119	S2-9	193.7	3	1000	398.8	25.6	27670.6	1797.3	1999.6	1.26	1.47
120	S2-10	193.7	3.5	1500	398.8	25.6	27376.5	2091.4	1817.1	1.18	1.27
121	S2-11	193.7	3.5	2000	398.8	25.6	27376.5	2091.4	1796.3	1.26	1.30
122	S2-12	193.7	3.5	2500	392.2	25.6	27376.5	2091.4	1620.8	1.18	1.24

O'Shea M.D., Bridge R.Q. (1994)										EC-4	AISC-2010
test No.	spec. No	D	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
		(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)		
123	R12CF1	190	1.15	662	330	110.3	27670.6	682.3	3030	1.32	0.99
124	R12CF2	190	1.15	656	330	110.3	27670.6	682.3	2940	1.28	0.96
125	R12CF3	190	1.15	662	330	110.3	27670.6	682.3	3140	1.37	1.03
126	R12CF4	190	1.15	662	330	94.7	27670.6	682.3	2462	1.11	0.93
127	R12CF5	190	1.15	664	330	110.3	27670.6	682.3	3055	1.33	1.00
128	R12CF7	190	1.15	660	330	110.3	27670.6	682.3	3000	1.30	0.98

Z.W. Wu, F.X. Ding, C.S. Cai (2006)										EC-4	AISC-2010
test No.	spec. No	D	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
		(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)		
129	sz5s4A1a	219	4.78	650	350	41.915	34451.6	3216.9	3400	1.11	1.37
130	sz5s4A1b	219	4.72	650	350	40.4	34491.1	3177.4	3350	1.11	1.38
131	sz5s3A1	219	4.75	650	350	34.08	34471.3	3197.2	3150	1.12	1.42
132	sz5s3A2	219	4.73	650	350	34.08	34484.5	3184.0	3150	1.12	1.42
133	sz3s6A1	165	2.73	510	350	40.4	19990.7	1391.7	2080	1.38	1.67
134	sz3s6B	165	2.81	500	350	61.76	19950.7	1431.8	2160	1.37	1.30
135	sz3s6C	165	2.81	500	350	61.76	19950.7	1431.8	2095	1.33	1.26
136	sz3s6D	165	2.76	500	350	61.76	19975.7	1406.7	2250	1.43	1.36
137	sz3s4A1	165	2.72	510	350	45.6	19995.8	1386.7	1750	1.09	1.31
138	sz3c4A1	165	2.75	510	350	37.04	19980.7	1401.7	1560	1.08	1.32

Gardner, Jacobson (1967)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
139	22	76	1.7	152	363	41	4139.6	396.8	435	1.10	1.43
140	23	76	1.7	152	363	26	4139.6	396.8	372	1.10	1.51
141	19	76	1.7	152	363	25	4139.6	396.8	356	1.06	1.47
142	3	102	3.1	203	605	34	7208.1	963.2	1112	0.98	1.37
143	4	102	3.1	203	605	31	7208.1	963.2	1068	0.96	1.35
144	8	121	4.1	241	451	34	9993.3	1505.7	1201	0.86	1.20
145	9	121	4.1	241	451	30	9993.3	1505.7	1201	0.89	1.25
146	10	121	4.1	241	451	26	9993.3	1505.7	1112	0.84	1.20
147	13	153	3.2	305	415	21	16879.4	1506.0	1201	0.89	1.25
148	14	153	3.2	305	415	23	16879.4	1506.0	1201	0.87	1.21
149	15	153	4.9	305	633	42	16105.6	2279.8	2909	1.02	1.40
150	16	153	4.9	305	633	43	16105.6	2279.8	2913	1.02	1.39

Gardner (1968)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
151	1a	169	2.6	305	298	18	21072.6	1359.2	1326	1.25	1.74
152	2a	169	2.6	305	298	34	21072.6	1359.2	1219	0.88	1.13
153	2a	169	2.6	305	317	37	21072.6	1359.2	1308	0.88	1.12
154	4a	169	2.6	305	317	34	21072.6	1359.2	1330	0.94	1.20
155	5a	168	3.6	305	221	27	20307.8	1859.3	1557	1.27	1.67
156	6a	168	3.6	305	221	33	20307.8	1859.3	1432	1.06	1.37
157	6b	169	3.6	305	221	33	20561.1	1870.6	1463	1.08	1.39
158	7a	169	5	305	260	33	19855.7	2576.1	1966	1.13	1.52
159	7b	169	5	305	260	33	19855.7	2576.1	1966	1.13	1.52
160	8a	169	5	305	260	27	19855.7	2576.1	1984	1.22	1.69
161	8b	169	5	305	260	27	19855.7	2576.1	1984	1.22	1.69

Chapman, Neogi (1966)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
162	A1	356	11.2	1880	355	38	87406.1	12132.1	11458	1.35	1.57
163	A4	356	11.2	1880	355	33	87406.1	12132.1	10711	1.32	1.55
164	A5	356	4.7	1880	276	21	94351.1	5187.1	3517	0.92	1.08
165	A6	356	8	2083	355	23	90792.0	8746.2	7433	1.28	1.49

166	B1	127	1.6	711	371	66	12037.4	630.3	1285	1.40	1.35
167	B1X	127	1.6	711	329	66	12037.4	630.3	1285	1.43	1.38
168	B2	127	2.9	711	371	66	11537.1	1130.6	1305	1.24	1.18
169	B2X	127	2.9	711	329	66	11537.1	1130.6	1305	1.29	1.23
170	DF1	140	9.7	406	265	32	11423.1	3970.7	2949	1.59	2.12
171	DF1X	140	9.7	406	269	32	11423.1	3970.7	2949	1.57	2.09
172	DF2	140	4.9	406	289	33	13314.1	2079.7	1824	1.38	1.80
173	DF2X	140	4.9	406	298	33	13314.1	2079.7	1824	1.35	1.77
174	SC1	168	4.5	813	298	31	19855.7	2311.4	2006	1.33	1.60
175	SC2	168	4.5	813	298	43	19855.7	2311.4	2233	1.30	1.51
176	SC3	168	4.5	813	298	43	19855.7	2311.4	2113	1.23	1.43
177	SC4	168	4.5	813	298	23	19855.7	2311.4	1744	1.28	1.58

Sakino, Hayashi (1991)

test No.	spec. No	D	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
178	L-20-1	178	9	251	283	22	20106.2	4778.4	2922	1.11	1.65
179	L-20-2	178	9	251	283	22	20106.2	4778.4	2853	1.08	1.61
180	H-20-1	178	9	251	283	45	20106.2	4778.4	3216	1.05	1.46
181	H-20-2	178	9	251	283	45	20106.2	4778.4	3177	1.03	1.44
182	L-32-1	179	5.5	251	248	22	22167.1	2997.9	1814	1.04	1.50
183	L-32-2	179	5.5	251	248	24	22167.1	2997.9	1814	1.02	1.45
184	H-32-1	179	5.5	251	248	44	22167.1	2997.9	2040	0.92	1.22
185	H-32-2	179	5.5	251	248	44	22167.1	2997.9	2030	0.92	1.22
186	L-58-1	174	3	251	266	24	22167.1	1611.6	1314	1.04	1.41
187	L-58-2	174	3	251	266	24	22167.1	1611.6	1304	1.03	1.40
188	H-58-1	174	3	251	266	46	22167.1	1611.6	1608	0.93	1.15
189	H-58-2	174	3	251	266	46	22167.1	1611.6	1677	0.97	1.20

Luksha, Nestrovich (1991)

test No.	spec. No	D	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
190	SB-1	159	5.1	478	391	33	17389.8	2465.8	2230	1.14	1.49
191	SB-2	630	7	1890	291	29	298024.0	13700.5	16648	1.14	1.37
192	SB-6	630	7.6	1890	349	28	296864.0	14860.5	17998	1.12	1.39
193	SB-7	630	8.4	1890	350	28	295320.9	16403.6	18598	1.11	1.38
194	SB-3	630	10.2	1890	323	31	291863.5	19861.0	20498	1.11	1.37
195	SB-4	630	11.6	1890	347	37	289188.5	22536.0	24398	1.10	1.37

196	SB-8	720	8.3	2159	311	12	388592.7	18557.7	14999	1.10	1.48
197	SB-5	820	8.9	2461	331	36	505423.2	22678.5	33596	1.15	1.37
198	SB-9	1020	9.7	3061	336	14	786340.9	30787.3	29997	1.11	1.45
199	SB-10	1020	13.3	3061	369	23	775065.1	42063.1	45996	1.12	1.43

Furlong (1967)

test No.	spec. No	D	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
200	1	114	3.2	914	413	29	9093.2	1113.9	712	1.00	1.05
201	2	114	3.2	914	413	29	9093.2	1113.9	756	1.06	1.12
202	3	127	2.4	914	289	35	11728.2	939.5	627	0.91	0.98
203	4	127	2.4	914	289	35	11728.2	939.5	623	0.90	0.98
204	5	127	2.4	914	289	35	11728.2	939.5	658	0.95	1.03
205	6	152	1.5	914	331	21	17436.6	709.2	682	1.06	1.20
206	7	152	1.5	914	331	26	17436.6	709.2	721	1.00	1.11
207	8	152	1.5	914	331	26	17436.6	709.2	733	1.01	1.13

Gardner (1968)

test No.	spec. No	D	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
208	1	169	2.6	1981	298	18	21072.6	1359.2	823	1.12	1.17
209	2	169	2.6	1981	298	34	21072.6	1359.2	916	0.89	0.93
210	3	169	2.6	1981	317	37	21072.6	1359.2	756	0.68	0.72
211	4	169	2.6	1981	317	34	21072.6	1359.2	689	0.65	0.69
212	5	168	3.6	2286	221	27	20307.8	1859.3	947	1.07	1.13
213	6	168	3.6	2286	221	33	20307.8	1859.3	1050	1.06	1.12
214	7	169	5	2286	260	33	19855.7	2576.1	1130	0.93	0.98
215	8	169	5	2286	260	27	19855.7	2576.1	733	0.66	0.69

Gardner, Jacobson (1964)

test No.	spec. No	D	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
216	1	102	3.1	1524	605	34	7208.1	963.2	818	1.21	1.25
217	2	102	3.1	1524	605	34	7208.1	963.2	801	1.19	1.23
218	5	121	4.1	1049	451	34	9993.3	1505.7	1156	1.19	1.23
219	6	121	4.1	1049	451	30	9993.3	1505.7	1092	1.17	1.20

220	7	121	4.1	1049	451	26	9993.3	1505.7	950	1.05	1.09
221	11	153	3.1	1676	415	21	16925.5	1459.9	939	1.06	1.09
222	12	153	3.1	1676	415	21	16925.5	1459.9	881	0.99	1.02
223	18	76	1.7	1524	363	25	4139.6	396.8	245	1.28	1.33
224	20	76	1.7	610	363	41	4139.6	396.8	411	1.35	1.42
225	21	76	1.7	610	363	26	4139.6	396.8	330	1.31	1.40

knowles, Park (1969)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm²)	f_{ck} (N/mm²)	A_c (mm²)	A_s (mm²)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
226	1	89	5.8	1727	400	40	4705.1	1516.0	615	1.03	1.05
227	2	89	5.8	1422	400	40	4705.1	1516.0	712	1.07	1.11
228	3	89	5.8	1118	400	39	4705.1	1516.0	715	1.00	1.04
229	4	89	5.8	813	400	42	4705.1	1516.0	919	1.21	1.24
230	5	89	5.8	508	400	41	4705.1	1516.0	992	1.14	1.29
231	6	83	1.3	1727	482	41	5076.9	333.7	225	0.96	0.95
232	7	83	1.3	1422	482	37	5076.9	333.7	294	1.09	1.14
233	8	83	1.3	1118	482	41	5076.9	333.7	356	1.12	1.18
234	9	83	1.3	813	482	41	5076.9	333.7	400	1.17	1.22
235	10	83	1.3	508	482	41	5076.9	333.7	489	1.30	1.41
236	11	83	1.3	254	482	41	5076.9	333.7	530	1.22	1.49

Kobayashi, konishi (1991)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm²)	f_{ck} (N/mm²)	A_c (mm²)	A_s (mm²)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
237	1A2	191	6	1151	504	56	25164.9	3487.2	3062	0.95	1.02
238	1A4	191	6	2301	504	56	25164.9	3487.2	2611	0.94	0.97
239	1A6	191	6	3449	504	56	25164.9	3487.2	2059	0.92	0.92
240	1G2	191	6	1151	504	48	25164.9	3487.2	3148	1.03	1.12
241	1G6	267	6	3449	504	48	51070.5	4919.7	2133	0.50	0.52
242	2A2	267	7	1600	460	56	50272.6	5717.7	5181	0.93	1.01
243	2A4	267	7	3200	460	56	50272.6	5717.7	4533	0.94	0.98
244	2A6	267	7	4801	460	56	50272.6	5717.7	3625	0.93	0.93
245	2G2	267	7	1600	460	48	50272.6	5717.7	5187	1.00	1.09
246	2G6	267	7	4801	460	48	50272.6	5717.7	3903	1.05	1.07

Jans, Guiaux (1970)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm²)	f_{ck} (N/mm²)	A_c (mm²)	A_s (mm²)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
247	1	218	6	4366	301	31	33329.2	3996.1	1716	0.97	1.01
248	2	218	6.5	3284	301	34	33006.4	4318.9	2063	0.97	1.01
249	3	218	6.3	2205	301	30	33135.3	4190.0	2412	1.13	1.17
250	4	218	6.5	942	301	30	33006.4	4318.9	2755	1.01	1.24
251	5	218	6.4	942	301	31	33070.8	4254.5	2748	1.01	1.23
252	6	219	6	942	280	31	33653.5	4015.0	2804	1.10	1.34
253	8.1	95	3.8	4318	280	31	5999.5	1088.8	202	1.48	1.38
254	8.2	95	3.7	4321	280	34	6027.0	1061.3	155	1.15	1.06
255	8.3	96	3.6	4321	280	34	6193.2	1045.0	140	1.02	0.94
256	9.1	95	3.8	2845	280	34	5999.5	1088.8	279	1.02	0.96
257	9.2	95	3.7	2845	280	30	6027.0	1061.3	282	1.07	1.01
258	9.3	96	3.8	2845	280	30	6137.5	1100.7	291	1.05	1.00
259	10.1	96	3.7	1943	280	30	6165.3	1072.9	363	0.93	0.97
260	10.2	95	3.8	1943	280	34	5999.5	1088.8	407	1.01	1.05
261	10.3	95	3.8	1943	280	34	5999.5	1088.8	407	1.01	1.05
262	11.1	96	3.8	1468	280	34	6137.5	1100.7	444	0.97	1.01
263	11.2	95	3.7	1468	280	34	6027.0	1061.3	441	0.99	1.03
264	11.3	95	3.7	1468	280	34	6027.0	1061.3	495	1.11	1.16
265	12.1	96	3.7	998	280	34	6165.3	1072.9	524	1.09	1.12
266	12.2	95	3.6	993	280	34	6054.5	1033.7	507	1.09	1.12
267	12.3	95	3.7	996	280	34	6027.0	1061.3	533	1.13	1.16
268	13.1	95	3.7	503	280	34	6027.0	1061.3	637	1.11	1.32
269	13.2	96	3.8	503	280	34	6137.5	1100.7	633	1.07	1.27
270	13.3	96	3.7	505	280	34	6165.3	1072.9	667	1.14	1.36

Jans,(1974)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm²)	f_{ck} (N/mm²)	A_c (mm²)	A_s (mm²)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
271	1	406	5	1590	356	35	123163.0	6298.9	7549	1.03	1.21
272	2	406	5	1621	356	38	123163.0	6298.9	7549	0.99	1.14
273	3	406	5	1194	356	35	123163.0	6298.9	7695	1.01	1.22
274	4	406	5	1209	356	36	123163.0	6298.9	7450	0.97	1.16
275	7	356	8.1	1504	321	28	90685.2	8853.0	7059	1.11	1.36
276	8	356	8.2	1499	321	28	90578.5	8959.7	6813	1.06	1.30
277	9	356	7.7	1496	321	28	91112.8	8425.5	7186	1.16	1.42
278	10	275	9	1430	626	28	51874.8	7521.0	8186	1.21	1.38

279	11	275	9	1425	626	28	51874.8	7521.0	8137	1.20	1.37
280	12	275	9	1440	626	28	51874.8	7521.0	8382	1.24	1.42
281	13	275	9	1440	626	28	51874.8	7521.0	8039	1.19	1.36
282	14	275	9	1488	626	28	51874.8	7521.0	8333	1.25	1.41
283	15	275	9	1491	626	28	51874.8	7521.0	8235	1.23	1.39

Han, Yan (2000)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
284	SC154-1	108	4.5	4161	410	26	7697.7	1463.2	342	1.37	1.28
285	SC154-2	108	4.5	4161	410	26	7697.7	1463.2	292	1.18	1.11
286	SC154-3	108	4.5	4161	410	39	7697.7	1463.2	298	1.17	1.07
287	SC154-4	108	4.5	4161	410	39	7697.7	1463.2	280	1.10	1.01
288	SC149-1	108	4.5	4026	410	39	7697.7	1463.2	318	1.18	1.07
289	SC149-2	108	4.5	4026	410	39	7697.7	1463.2	320	1.19	1.08
290	SC141-1	108	4.5	3810	410	26	7697.7	1463.2	351	1.22	1.12
291	SC141-2	108	4.5	3810	410	26	7697.7	1463.2	371	1.29	1.18
292	SC130-1	108	4.5	3513	410	26	7697.7	1463.2	400	1.21	1.11
293	SC130-2	108	4.5	3513	410	26	7697.7	1463.2	391	1.19	1.08
294	SC130-3	108	4.5	3513	410	39	7697.7	1463.2	440	1.29	1.14

2. RECTANGULAR CFT COLUMNS

Sakino, K., Nakahara, H., Morino, S., Nishiyama (2004)

test No.	spec. No	B	H	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
		(mm)	(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)		
1	CR4-A-2	148	148	4.4	224	262	25	19371	2467	1153	1.01	1.08
2	CR4-A-4-1	148	148	4.4	224	262	41	19371	2467	1414	0.99	1.08
3	CR4-A-4-2	148	148	4.4	224	262	41	19371	2467	1402	0.98	1.07
4	CR4-A-8	148	148	4.4	224	262	77	19371	2467	2108	1.09	1.10
5	CR4-C-2	215	215	4.4	323	262	25	42518	3641	1777	0.87	0.95
6	CR4-C-4-1	215	215	4.4	323	262	41	42518	3641	2424	0.90	0.99
7	CR4-C-4-2	215	215	4.4	323	262	41	42518	3641	2393	0.89	0.98
8	CR4-C-8	215	215	4.4	323	262	80	42518	3641	3837	1.00	1.00
9	CR4-D-2	323	323	4.4	485	262	25	98730	5533	3367	0.85	0.94
10	CR4-D-4-1	323	323	4.4	485	262	41	98730	5533	4950	0.90	1.01
11	CR4-D-4-2	323	323	4.4	485	262	41	98730	5533	4830	0.88	0.99
12	CR4-D-8	323	323	4.4	485	262	80	98730	5533	7481	0.91	0.92
13	CR6-A-2	144	144	6.4	216	618	25	17200	3397	2572	1.01	1.04
14	CR6-A-4-1	144	144	6.4	216	618	41	17200	3397	2808	1.00	1.05
15	CR6-A-4-2	144	144	6.4	216	618	41	17200	3397	2765	0.99	1.03
16	CR6-A-8	144	144	6.4	216	618	77	17200	3397	3399	1.05	1.06
17	CR6-C-2	211	211	6.4	315	618	25	39280	5102	3920	0.94	0.98
18	CR6-C-4-1	211	211	6.4	315	618	41	39280	5102	4428	0.93	0.98
19	CR6-C-4-2	211	211	6.4	315	618	41	39280	5102	4484	0.95	1.00
20	CR6-C-8	211	211	6.4	315	618	77	39280	5102	5758	1.00	1.01
21	CR6-D-2	319	319	6.4	478	618	25	93773	7849	6320	0.87	0.92
22	CR6-D-4-1	319	319	6.4	478	618	41	93773	7849	7780	0.89	0.96
23	CR6-D-4-2	319	319	6.4	478	618	41	93773	7849	7473	0.86	0.92
24	CR6-D-8	319	319	6.4	478	618	85	93773	7849	10357	0.91	0.89
25	CR8-A-2	120	120	6.5	180	835	25	11426	2830	2819	1.06	1.08
26	CR8-A-4-1	120	120	6.5	180	835	41	11426	2830	2957	1.05	1.08
27	CR8-A-4-2	120	120	6.5	180	835	41	11426	2830	2961	1.05	1.08
28	CR8-A-8	120	120	6.5	180	835	77	11426	2830	3318	1.06	1.07
29	CR8-C-2	175	175	6.5	262	835	25	26228	4254	4210	1.00	1.02

30	CR8-C-4-1	175	175	6.5	262	835	41	26228	4254	4493	0.97	1.01
31	CR8-C-4-2	175	175	6.5	262	835	41	26228	4254	4542	0.98	1.02
32	CR8-C-8	175	175	6.5	262	835	77	26228	4254	5366	1.01	1.02
33	CR8-D-2	265	265	6.5	396	835	25	63498	6583	6546	0.92	0.96
34	CR8-D-4-1	265	265	6.5	396	835	41	63498	6583	7117	0.88	0.92
35	CR8-D-4-2	265	265	6.5	396	835	41	63498	6583	7172	0.88	0.93
36	CR8-D-8	265	265	6.5	396	835	80	63498	6583	8990	0.92	0.92
37	CR4-A-4-3	210	210	5.5	315	294	39	39591	4406	3183	1.12	1.22
38	CR4-A-9	210	210	5.5	315	294	91	39591	4406	4773	1.15	1.10
39	CR4-C-4-3	210	210	4.5	315	277	39	40384	3647	2713	1.05	1.15
40	CR4-C-9	210	210	4.5	315	277	91	40384	3647	4371	1.11	1.06
41	CR6-A-4-3	211	211	8.8	315	536	39	37313	6940	5898	1.14	1.19
42	CR6-A-9	211	211	8.8	315	536	91	37313	6940	7008	1.09	1.06
43	CR6-C-4-3	204	204	6	306	540	39	36872	4622	4026	1.02	1.08
44	CR6-C-9	204	204	6	306	540	91	36872	4622	5303	1.03	0.99
45	CR8-A-4-3	180	180	9.5	270	825	39	25877	6217	6803	1.11	1.14
46	CR8-A-9	180	180	9.5	270	825	91	25877	6217	7402	1.06	1.04
47	CR8-C-4-3	180	180	6.6	270	824	39	27785	4466	5028	1.05	1.09
48	CR8-C-9	180	180	6.6	270	824	91	27785	4466	5873	1.03	1.01

B. Uy (1998)

test No.	spec. No	B	H	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
49	LB1	126	126	3	300	300	40	14392	1453	950	0.94	1.03
50	LB3	156	156	3	300	300	50	22492	1813	1300	0.78	0.87
51	LB5	186	186	3	300	300	32	32392	2173	1200	0.71	0.78
52	LB7	246	246	3	300	300	38	57592	2893	2200	0.72	0.81
53	LB9	306	306	3	300	300	38	89992	3613	2519	0.56	0.63

C.S. Huang; Y.K. Yeh; G.-Y Liu; H.-T. Hu; K.C. Tsai (2002)

test No.	spec. No	B	H	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(mm)	(N/mm²)	(N/mm²)	(mm²)	(mm²)	(kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
54	SU-040	200	200	5	600	266	27	36079	3836	2312	1.16	1.25
55	CU-040	200	200	5	600	266	27	36079	3836	2013	1.01	1.09
56	SU-070	280	280	4	840	273	31	73970	4375	3401	0.97	1.08
57	CU-070	280	280	4	840	273	31	73970	4375	3025	0.87	0.96
58	SU-150	300	300	2	900	342	27	87613	2374	3062	0.96	1.09
59	CU-150	300	300	2	900	342	27	87613	2374	2608	0.82	0.92

C. Petrus; H.A. Hamid; A. Ibrahim (2010)

test No.	spec. No	B	H	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(mm)	(N/mm²)	(N/mm²)	(mm²)	(mm²)	(kN)	$(N_{test})/(\chi N_{pl,Rd})$	$(P_{test})\ P_n)$
60	LCFT1	200	200	2	550	300	30	38413	1574	1280	0.79	0.89
61	LCFT2	200	200	2	550	236	35	38413	1574	1554	0.91	1.03

D. Liu; W.-M. Gho; J. Yuan (2003)

test No.	spec. No	B	H	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(mm)	(N/mm²)	(N/mm²)	(mm²)	(mm²)	(kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
62	C1-1	100	98.2	4.2	300	550	61	8245	1545	1490	1.13	1.18
63	C1-2	102	101	4.2	300	550	61	8576	1575	1535	1.13	1.18
64	C2-1	101	101	4.2	300	550	72	8595	1576	1740	1.23	1.26
65	C2-2	101	100	4.2	300	550	72	8484	1566	1775	1.26	1.29
66	C3	183	181	4.2	540	550	61	30135	2928	3590	1.07	1.14
67	C4	182	180	4.2	540	550	72	29824	2913	4210	1.20	1.24
68	C5-1	121	80.1	4.2	360	550	61	8044	1564	1450	1.10	1.15
69	C5-2	119	80.6	4.2	360	550	61	7999	1556	1425	1.08	1.14
70	C6-1	120	80.6	4.2	360	550	72	8021	1559	1560	1.14	1.17
71	C6-2	121	80.6	4.2	360	550	72	8086	1566	1700	1.23	1.27
72	C7-1	180	122	4.2	540	550	61	19370	2403	2530	1.04	1.11
74	C8-1	180	120	4.2	540	550	72	19157	2395	2970	1.17	1.21
75	C8-2	179	121	4.2	540	550	72	19280	2397	2590	1.02	1.05
76	C9-1	160	81.4	4.2	480	550	61	11075	1905	1710	1.03	1.08
77	C9-2	161	80.5	4.2	480	550	61	10975	1902	1820	1.10	1.16
78	C10-1	160	81	4.2	480	550	72	11007	1901	1880	1.09	1.12
79	C10-2	161	80.1	4.2	480	550	72	10907	1897	2100	1.22	1.26
80	C11-1	200	101	4.2	600	550	61	17758	2401	2350	1.02	1.08
81	C11-2	200	98.9	4.2	600	550	61	17354	2386	2380	1.05	1.11

82	C12-1	199	102	4.2	600	550	72	17874	2404	2900	1.19	1.23
83	C12-2	200	99.6	4.2	600	550	72	17452	2388	2800	1.17	1.21

D. Liu; W.-M. Gho (2005)											EC-4	AISC-2010
test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
84	A1	120	120	5.8	360	300	83	11722	2563	1697	1.07	1.07
85	A2	120	120	5.8	360	300	106	11722	2563	1919	1.15	1.06
86	A3-1	200	200	5.8	600	300	83	35466	4419	3996	1.06	1.05
87	A3-2	200	200	5.8	600	300	83	35466	4419	3862	1.02	1.02
88	A4-1	100	130	5.8	390	300	83	10438	2447	1601	1.10	1.10
89	A4-2	100	130	5.8	390	300	83	10438	2447	1566	1.07	1.07
90	A5-1	100	130	5.8	390	300	106	10438	2447	1854	1.21	1.12
91	A5-2	100	130	5.8	390	300	106	10438	2447	1779	1.16	1.07
92	A6-1	170	220	5.8	660	300	83	32982	4303	3684	1.03	1.03
93	A6-2	170	220	5.8	660	300	83	32982	4303	3717	1.04	1.04
94	A7-1	100	180	5.8	540	300	83	14858	3027	2059	1.06	1.07
95	A7-2	100	180	5.8	540	300	83	14858	3027	2019	1.04	1.05
96	A8-1	100	180	5.8	540	300	106	14858	3027	2287	1.12	1.04
97	A8-2	100	180	5.8	540	300	106	14858	3027	2291	1.12	1.04
98	A9-1	120	120	4	360	495	55	12530	1815	1739	1.11	1.18
99	A9-2	120	120	4	360	495	55	12530	1815	1718	1.09	1.16
100	A10-1	100	150	4	450	495	55	13050	1895	1815	1.11	1.19
101	A10-2	100	150	4	450	495	55	13050	1895	1763	1.08	1.15
102	A11-1	90	180	4	540	495	55	14090	2055	1725	0.99	1.05
103	A11-2	90	180	4	540	495	55	14090	2055	1742	0.99	1.06
104	A12-1	130	130	4	390	495	55	14870	1975	1963	1.11	1.18
105	A12-2	130	130	4	390	495	55	14870	1975	1988	1.12	1.20
106	A13-1	110	160	4	480	495	55	15490	2055	1947	1.05	1.13
107	A13-2	110	160	4	480	495	55	15490	2055	1912	1.03	1.11
108	A14-1	100	190	4	570	495	55	16730	2215	2035	1.03	1.11
109	A14-2	100	190	4	570	495	55	16730	2215	2138	1.08	1.16

Ge, H.; Usami, T. (1992)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm²)	f_{ck} (N/mm²)	A_c (mm²)	A_s (mm²)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
110	U9C	196	196	4.5	592	266	39	34944	3402	1845	0.81	0.90
111	U12C	263	263	4.5	790	266	40	64488	4611	3070	0.80	0.90
112	U12HC	263	263	4.5	790	266	48	64488	4611	3996	0.92	1.04
113	U15C	329	329	4.5	988	266	41	102370	5801	3275	0.57	0.65

L.-H, Han. (2001)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm²)	f_{ck} (N/mm²)	A_c (mm²)	A_s (mm²)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
114	rc1-1	100	100	2.9	300	228	40	8882	1090	760	1.26	1.39
115	rc1-2	100	100	2.9	300	228	40	8882	1090	800	1.33	1.46
116	rc2-1	120	120	2.9	360	228	40	13053	1319	992	1.21	1.34
117	rc2-2	120	120	2.9	360	228	40	13053	1319	1050	1.28	1.42
118	rc3-1	100	110	2.9	330	228	40	9824	1147	844	1.29	1.43
119	rc3-2	100	110	2.9	330	228	40	9824	1147	860	1.32	1.46
120	rc4-1	135	150	2.9	450	228	40	18645	1576	1420	1.29	1.44
121	rc4-2	135	150	2.9	450	228	40	18645	1576	1340	1.22	1.36
122	rc5-1	70	90	2.9	270	228	40	5410	861	554	1.35	1.47
123	rc5-2	70	90	2.9	270	228	40	5410	861	576	1.40	1.53
124	rc6-1	75	100	2.9	300	228	40	6525	947	640	1.35	1.48
125	rc6-2	75	100	2.9	300	228	40	6525	947	672	1.41	1.55
126	rc7-1	90	120	2.9	360	228	40	9624	1147	800	1.24	1.37
127	rc7-2	90	120	2.9	360	228	40	9624	1147	760	1.18	1.30
128	rc8-1	105	140	2.9	420	228	40	13324	1348	1044	1.25	1.39
129	rc8-2	105	140	2.9	420	228	40	13324	1348	1086	1.30	1.44
130	rc9-1	115	150	2.9	450	228	40	15760	1462	1251	1.30	1.46
131	rc9-2	115	150	2.9	450	228	40	15760	1462	1218	1.27	1.42
132	rc10-1	120	160	7.6	480	194	40	15125	3876	1820	1.35	1.45
133	rc10-2	120	160	7.6	480	194	40	15125	3876	1770	1.31	1.41
134	rc11-1	85	130	2.9	390	228	40	9846	1176	760	1.15	1.28
135	rc11-2	85	130	2.9	390	228	40	9846	1176	820	1.24	1.38
136	rc12-1	80	140	2.9	420	228	40	9967	1205	880	1.31	1.46
137	rc12-2	80	140	2.9	420	228	40	9967	1205	740	1.10	1.22

Han, L.H., Liu, W., Yang, Y.F (2007)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
138	1sp1-1-1	177	177	2	531	345	51	29926	1390	1850	0.93	1.05
139	1sp1-1-2	177	177	2	531	345	51	29926	1390	1850	0.93	1.05
140	1sp1-2-1	177	177	5	531	345	51	27868	3376	1985	0.77	0.84
141	1sp1-2-2	177	177	5	531	345	51	27868	3376	1880	0.73	0.80
142	1sp1-3-1	177	177	9.6	531	345	51	24822	6191	2205	0.65	0.69
143	1sp1-3-2	177	177	9.6	531	345	51	24822	6191	2120	0.62	0.66
144	1sp1-4-1	177	177	12	531	345	51	23285	7549	2300	0.61	0.64
145	1sp1-4-2	177	177	12	531	345	51	23285	7549	2305	0.61	0.64
146	1sp2-1-1	177	177	2	531	345	51	29926	1390	655	0.33	0.37
147	1sp2-1-2	177	177	2	531	345	51	29926	1390	736	0.37	0.42
148	1sp2-2-1	177	177	5	531	345	51	27868	3376	790	0.31	0.33
149	1sp2-2-2	177	177	5	531	345	51	27868	3376	781	0.30	0.33
150	1sp2-3-1	177	177	12	531	345	51	23285	7549	780	0.21	0.22
151	1sp2-3-2	177	177	12	531	345	51	23285	7549	780	0.21	0.22

Mursi, M., Uy, B. (2004)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
152	SL-C110	110	110	5	2174	761	20	9979	2036	1481	1.19	1.23
153	SL-C160	160	160	5	2416	761	20	22479	3036	2126	0.90	0.96
154	SL-C210	210	210	5	2416	761	20	39979	4036	2939	0.82	0.87
155	SL-C260	260	260	5	2817	761	20	62479	5036	3062	0.65	0.69
156	SH-C110	110	110	5	430	761	20	9979	2036	1835	1.05	1.08
157	SH-C160	160	160	5	580	761	20	22479	3036	2831	1.02	1.06
158	SH-C210	210	210	5	730	761	20	39979	4036	3609	0.93	0.97
159	SH-C260	260	260	5	880	761	20	62479	5036	3950	0.77	0.81

Song, Kwon, J.Y, Bong, Y (1997)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
160	US 9	130	130	3.2	390	262	25	15268	1597	1153	1.43	1.55
161	US 12	175	175	3.2	525	262	41	28417	2173	1414	0.82	0.92
162	US 15	220	220	3	660	262	41	45788	2581	1402	0.55	0.63

											EC-4	AISC-2010
					B. Uy, M. Khan, Z. Tao, F. Mashiri (2013)							
test No.	spec. No	B	H	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
		(mm)	(mm)	(mm)	(mm)	(N/mm²)	(N/mm²)	(mm²)	(mm²)	(kN)		
163	CB15-SH(A)	75	75	5	262.5	690	80	4204	1336	1634	1.35	1.37
164	CB15-SH(B)	75	75	5	262.5	690	80	4204	1336	1756	1.45	1.47
165	CB20-SH(A)	100	100	5	350	690	80	8079	1836	2524	1.39	1.41
166	CB20-SH(B)	100	100	5	350	690	80	8079	1836	2632	1.45	1.47
167	CB25-SH(A)	125	125	5	437.5	690	80	13204	2336	3024	1.21	1.22
168	CB25-SH(B)	125	125	5	437.5	690	80	13204	2336	2971	1.18	1.20
169	CB30-SH(A)	150	150	5	525	690	95	19579	2836	4115	1.21	1.18
170	CB30-SH(B)	150	150	5	525	690	95	19579	2836	3968	1.17	1.14
171	CB40-SH(A)	200	200	5	700	690	98	36079	3836	5184	0.97	0.93
172	CB40-SH(B)	200	200	5	700	690	98	36079	3836	5604	1.05	1.00

											EC-4	AISC-2010
					D.M; Lue, J-L; Liu, T, Yen (2006)							
test No.	spec. No	B	H	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
		(mm)	(mm)	(mm)	(mm)	(N/mm²)	(N/mm²)	(mm²)	(mm²)	(kN)		
173	C4 4-1-4a	100	150	4.5	1855	380	29	12814	2117	1345	1.33	1.42
174	C4 4-1-4b	100	150	4.5	1855	380	29	12814	2117	1281	1.27	1.36
175	C4 4-1-4c	100	150	4.5	1855	380	29	12814	2117	1320	1.31	1.40
176	C4 4-1-4d	100	150	4.5	1855	380	29	12814	2117	1368	1.35	1.45
177	C9 6-1-6a	100	150	4.5	1855	380	63	12814	2117	1756	1.38	1.45
178	C9 6-1-6b	100	150	4.5	1855	380	63	12814	2117	1703	1.34	1.40
179	C9 6-1-6c	100	150	4.5	1855	380	63	12814	2117	1763	1.38	1.45
180	C9 6-1-6d	100	150	4.5	1855	380	63	12814	2117	1738	1.36	1.43
181	C9 6-1-6e	100	150	4.5	1855	380	63	12814	2117	1669	1.31	1.37
182	C9 6-1-6f	100	150	4.5	1855	380	63	12814	2117	1706	1.34	1.40
183	C10 6-1-6a	100	150	4.5	1855	380	70	12814	2117	1895	1.45	1.51
184	C10 6-1-6b	100	150	4.5	1855	380	70	12814	2117	1889	1.44	1.50
185	C10 6-1-6c	100	150	4.5	1855	380	70	12814	2117	1886	1.44	1.50
186	C10 6-1-6d	100	150	4.5	1855	380	70	12814	2117	1892	1.45	1.51

187	C10 6-1-6e	100	150	4.5	1855	380	70	12814	2117	1862	1.42	1.48
188	C10 6-1-6f	100	150	4.5	1855	380	70	12814	2117	1890	1.44	1.50
189	C12 6-1-6a	100	150	4.5	1855	380	84	12814	2117	2066	1.52	1.52
190	C12 6-1-6b	100	150	4.5	1855	380	84	12814	2117	2196	1.61	1.62
191	C12 6-1-6c	100	150	4.5	1855	380	84	12814	2117	2096	1.54	1.54
192	C12 6-1-6d	100	150	4.5	1855	380	84	12814	2117	2090	1.53	1.54
193	C12 6-1-6e	100	150	4.5	1855	380	84	12814	2117	2007	1.47	1.48
194	C12 6-1-6f	100	150	4.5	1855	380	84	12814	2117	2084	1.53	1.53

F.R; Mashiri, B. Uy, Z.Zhao (2015)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
195	N1NP1(A)	90	90	3	300	410	32	7048	1021	1067	1.66	1.76
196	N1NP1(B)	90	90	3	300	410	32	7048	1021	1079	1.68	1.78
197	N1NP2(A)	120	120	3	400	410	32	12988	1381	1440	1.47	1.58
198	N1NP2(B)	120	120	3	400	410	32	12988	1381	1432	1.46	1.57
199	N1NP3(A)	150	150	3	500	410	32	20728	1741	1895	1.38	1.49
200	N1NP3(B)	150	150	3	500	410	32	20728	1741	1836	1.33	1.45

Cai, J., He, Z.Q. (2005)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
201	C4	300	300	4	1500	342	40	85250	4695	5300	1.06	1.20
202	C7	300	300	8	1500	388	40	80601	9179	5600	0.83	0.90
203	C10	300	300	12	1500	345	40	76052	13453	6588	0.86	0.92
204	C13	300	300	6	1500	292	40	82913	6963	4370	0.82	0.91

Z., Tao, B., Uy, L.-H., Han

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
205	UNC-L	250	250	2.5	600	338	20	60020	2459	1993	0.97	1.07
206	UFRC-L	250	250	2.5	600	338	20	60020	2459	2020	0.98	1.08

207	SSNC-L	250	250	2.5	600	338	20	60020	2459	2282	1.11	1.22
208	SSFRC-L	250	250	2.5	600	338	20	60020	2459	2270	1.10	1.22
209	DSNC-L	250	250	2.5	600	338	20	60020	2459	2395	1.17	1.28
210	UNC-H	250	250	2.5	600	338	43	60020	2459	3190	0.94	1.07
211	UFRC-H	250	250	2.5	600	338	43	60020	2459	3150	0.93	1.05
212	SSNC-H	250	250	2.5	600	338	43	60020	2459	3520	1.04	1.18
213	SSFRC-H	250	250	2.5	600	338	43	60020	2459	3610	1.07	1.21
214	DSNC-H	250	250	2.5	600	338	43	60020	2459	3865	1.14	1.29
215	DSFRC-H	250	250	2.5	600	338	43	60020	2459	3870	1.14	1.30

knowles, park (1969)											EC-4	AISC-2010
test No.	spec. No	B	H	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
		(mm)	(mm)	(mm)	(mm)	(N/mm²)	(N/mm²)	(mm²)	(mm²)	(kN)		
216	8	76	76	3.3	1727	324	36	4807	932	356	0.96	1.02
217	9	76	76	3.3	1118	324	47	4807	932	423	0.89	0.96
218	10	76	76	3.4	1422	324	34	4779	958	385	0.95	1.02
219	11	76	76	3.4	813	324	45	4779	958	463	0.93	1.00
220	12	76	76	3.4	508	324	41	4779	958	506	1.01	1.09
221	13	76	76	3.4	254	324	41	4779	958	512	1.01	1.08
222	17	76	76	3.3	813	324	41	4807	932	346	0.73	0.78
223	18	76	76	3.3	1422	324	41	4807	932	281	0.66	0.71
224	19	76	76	3.3	813	324	41	4807	932	217	0.46	0.49
225	20	76	76	3.3	1422	324	41	4807	932	157	0.37	0.40

Schneider (1998)											EC-4	AISC-2010
test No.	spec. No	B	H	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
		(mm)	(mm)	(mm)	(mm)	(N/mm²)	(N/mm²)	(mm²)	(mm²)	(kN)		
226	S1	127	127	3.1	610	356	30	14584	1512	917	0.94	1.02
227	S2	127	127	4.3	610	357	26	14003	2063	1095	1.00	1.06
228	S3	127	127	4.5	610	322	24	13907	2153	1112	1.08	1.15
229	S4	125	126	5.7	612	311	24	12991	2648	1201	1.06	1.12
230	S5	127	127	7.5	610	347	24	12496	3440	2068	1.38	1.44
231	R1	77	152	3	612	430	30	10358	1315	818	0.95	1.02
232	R2	76	153	4.5	607	382	26	9631	1928	1006	1.04	1.09
233	R3	102	152	4.3	610	413	26	13378	2063	1144	0.96	1.02
234	R4	103	153	4.6	610	364	24	13470	2216	1224	1.08	1.15
235	R5	101	151	5.7	612	324	24	12480	2659	1334	1.15	1.21
236	R6	102	152	7.3	610	358	24	11963	3358	1690	1.14	1.19

Lin (1998)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm²)	f_{ck} (N/mm²)	A_c (mm²)	A_s (mm²)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
237	D7	150	150	0.7	480	246	22	22082	417	558	0.95	1.09
238	D8	150	150	0.7	800	246	22	22082	417	612	1.04	1.20
239	D10	150	150	1.4	800	247	22	21666	827	712	1.05	1.18
240	D12	150	150	2.1	800	249	22	21254	1231	793	1.02	1.14
241	D13	150	150	0.7	480	247	22	22082	417	795	1.35	1.55
242	D14	150	150	0.7	800	247	22	22082	417	705	1.20	1.39
243	D16	150	150	1.4	800	247	22	21666	827	881	1.29	1.46
244	D18	150	150	2.1	800	248	22	21254	1231	844	1.09	1.22
245	E7	150	150	0.7	480	246	34	22082	417	747	0.88	1.02
246	E10	150	150	1.4	800	247	35	21666	827	974	1.01	1.17
247	E15	150	150	1.4	480	247	34	21666	827	1172	1.25	1.42
248	E18	150	150	2.1	800	248	35	21254	1231	1268	1.21	1.37

Grauers (1993)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm²)	f_{ck} (N/mm²)	A_c (mm²)	A_s (mm²)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
249	1	120	120	5	249	304	47	12079	2236	1440	1.15	1.24
250	2	120	120	5	249	438	46	12079	2236	1690	1.10	1.17
251	3	120	120	5	249	327	96	12079	2236	2040	1.26	1.19
252	4	120	120	5	249	439	96	12079	2236	2240	1.20	1.14
253	5	120	120	8	249	322	39	10761	3419	1550	1.02	1.07
254	6	120	120	8	249	300	46	10761	3419	1670	1.10	1.16
255	7	120	120	8	249	376	47	10761	3419	1990	1.11	1.16
256	8	120	120	8	249	322	103	10761	3419	2270	1.18	1.11
257	9	120	120	8	249	379	103	10761	3419	2680	1.27	1.20
258	10	120	120	8	249	379	39	10761	3419	1800	1.05	1.09
259	11	120	120	8	249	376	93	10761	3419	2820	1.36	1.32
260	12	120	120	8	249	364	93	10761	3419	2710	1.33	1.30
261	13	120	120	8	249	364	80	10761	3419	2300	1.16	1.17
262	14	120	120	8	249	379	80	10761	3419	2290	1.13	1.13
263	15a	120	120	8	249	395	96	10761	3419	2340	1.09	1.05
264	16	120	120	8	249	395	96	10761	3419	1160	0.54	0.52
265	17	120	120	8	249	404	92	10761	3419	1380	0.64	0.62
266	18	120	120	8	249	404	92	10761	3419	1460	0.67	0.66

267	25	120	120	8	249	395	92	10761	3419	2300	1.08	1.05
268	23	120	120	8	249	379	31	10761	3419	1680	1.03	1.07
269	24	120	120	8	249	379	92	10761	3419	2430	1.17	1.14
270	27	250	250	8	249	379	33	54701	7579	4870	1.04	1.11
271	28	250	250	8	249	379	91	54701	7579	8300	1.22	1.17
272	15c	120	120	8	3277	395	96	10761	3419	920	0.76	0.70

Jans (1974)											EC-4	AISC-2010
test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
273	21	330	330	4.5	1318	370	32	103024	5807	4363	0.80	0.89
274	22	331	331	4.5	1328	370	27	103667	5825	4412	0.89	0.98
275	23	331	331	4.5	1321	370	27	103667	5825	4657	0.94	1.04
276	24	331	331	4.5	1318	370	32	103667	5825	4412	0.81	0.89
277	25	333	333	6.4	1318	444	32	102493	8255	5862	0.84	0.92
278	26	331	331	6.3	1318	444	32	101344	8080	5843	0.86	0.93
279	27	331	331	6.3	1318	444	27	101344	8080	5833	0.92	1.00
280	28	331	331	6.3	1321	444	27	101344	8080	5637	0.89	0.96
281	29	331	331	10	1397	389	29	96509	12702	8088	1.05	1.12
282	30	329	329	10	1397	389	29	95145	12739	8137	1.05	1.13
283	31	330	330	10	1397	389	29	95888	12661	7990	1.04	1.11
284	32	333	333	10	1397	389	29	97756	12782	8137	1.04	1.11

Furlong (1967)											EC-4	AISC-2010
test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
285	21	127	127	4.8	914	484	45	13763	2287	1601	0.95	1.01
286	22	102	102	2.1	914	331	23	9561	828	524	1.09	1.18
287	23	102	102	2.1	914	331	23	9561	828	488	1.01	1.10
288	24	102	102	3.2	914	331	29	9131	1238	667	1.02	1.09
289	25	102	102	3.2	914	331	29	9131	1238	676	1.03	1.11

3. CIRCULAR CFT BEAM COLUMNS

Kilpatrick, A. and Rangan B. V., 1997

test No.	spec. No	D	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	M_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)	(kN.m)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
1	SC-16	101.5	2.4	2175	410	96	7344	747	157	7.9	0.47	0.47
2	SC-17	101.5	2.4	2175	410	96	7344	747	282	5.6	0.75	0.86

J. Zeghiche, K. Chaoui (2004)

test No.	spec. No	D	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	M_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)	(kN.m)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
3	16	160	5	2000	271	100	17671	2435	1697	14	1.12	1.13
4	17	160	5	2000	281	100	17671	2435	1394	22	0.99	0.96
5	18	160	5	2000	280	100	17671	2435	1212	29	0.92	0.87
6	19	160	5	2000	276	100	17671	2435	1091	35	0.89	0.81
7	20	160	5	4000	275	100	17671	2435	963	8	1.16	1.38
8	21	160	5	4000	275	100	17671	2435	848	14	1.07	1.17
9	22	160	5	4000	281	100	17671	2435	727	17	0.95	0.99
10	23	160	5	4000	280	100	17671	2435	665	21	0.90	0.89

L.H. Han, G.H. Yao (2004)

test No.	spec. No	D	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	M_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)	(kN.m)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
11	lcsc2-1	200	3	2000	303.5	47	29559	1857	1215	36	0.87	0.95
12	lcsc2-2	200	3	2000	303.5	47	29559	1857	1132	34	0.80	0.87
13	lch2-1	200	3	2000	303.5	47	29559	1857	1291	39	0.95	1.02
14	lch2-2	200	3	2000	303.5	47	29559	1857	1234	37	0.89	0.97
15	lcv2	200	3	2000	303.5	47	29559	1857	1280	38	0.94	1.01

K. Tsuda, C. Matsui, E. Mino

test No.	spec. No	D	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	M_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)	(kN.m)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
16	C4-1	165.2	4.1	660	353	41	19369	2065	1214	25	1.10	1.16
17	C4-3	165.2	4.1	660	353	41	19369	2065	755	47	1.20	1.33
18	C4-5	165.2	4.1	660	353	41	19369	2065	555	57		
19	C8-1	165.2	4.1	1325	353	41	19369	2065	1041	22	0.93	1.00

20	C8-3	165.2	4.1	1325	353	41	19369	2065	656	41	0.80	0.86
21	C8-5	165.2	4.1	1325	353	41	19369	2065	435	45	0.60	0.66
22	C12-1	165.2	4.1	1980	353	41	19369	2065	948	20	0.87	0.99
23	C12-3	165.2	4.1	1980	353	41	19369	2065	572	35	0.67	0.74
24	C12-5	165.2	4.1	1980	353	41	19369	2065	387	40	0.49	0.54
25	C18-1	165.2	4.1	2975	353	41	19369	2065	742	15	0.74	0.94
26	C18-3	165.2	4.1	2975	353	41	19369	2065	461	29	0.56	0.66
27	C18-5	165.2	4.1	2975	353	41	19369	2065	331	34	0.44	0.50
28	C24-1	165.2	4.1	3965	353	41	19369	2065	609	13	0.77	1.06
29	C24-3	165.2	4.1	3965	353	41	19369	2065	351	22	0.50	0.64
30	C24-5	165.2	4.1	3965	353	41	19369	2065	277	29	0.44	0.53
31	C30-1	165.2	4.1	4970	353	41	19369	2065	480	10	0.81	1.30
32	C30-3	165.2	4.1	4970	353	41	19369	2065	309	19	0.59	0.83
33	C30-5	165.2	4.1	4970	353	41	19369	2065	239	25	0.49	0.64

A. Elremaily, A. Azizinamini (2002)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	M_{test} (kN.m)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
34	CFT1	324	6.4	2185	374	100	76062	6386	3172	542	0.63	0.93
35	CFT2	324	9.5	2185	371	104	73062	9386	2140	617	0.35	0.62
36	CFT3	324	9.5	2185	371	104	73062	9386	4280	663	0.71	1.25
37	CFT4	324	9.5	2185	371	40	73062	9386	2628	544	0.49	1.96
38	CFT5	324	9.5	2185	374	40	73062	9386	2514	451	0.47	1.88
39	CFT6	324	6.4	2185	374	70	76062	6386	2382	512	0.51	0.99

G. Muciaccia, F. Giussani, G. Rosati, F. Mola (2010)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	M_{test} (kN.m)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
40	NVC-80-1	139.6	4	1310	374	62	13602	1704	757	19	0.76	0.80
41	NVC-80-2	139.6	4	1230	374	62	13602	1704	875	22	0.92	0.95
42	NVC-200-1	139.6	4	2125	374	62	13602	1704	608	15	0.66	0.75
43	NVC-200-2	139.6	4	2135	374	62	13602	1704	606	15	0.65	0.75
44	NVC-300-1	139.6	4	3270	374	62	13602	1704	556	14	0.87	1.07
45	NVC-300-2	139.6	4	3270	374	62	13602	1704	484	12	0.74	0.93
46	NVC-440-1	139.6	4	4670	374	62	13602	1704	336	8	0.86	1.52

47	NVC-440-2	139.6	4	4670	374	62	13602	1704	333	8	0.85	1.51
48	SCC-80-1	139.6	4	1070	374	62	13602	1704	814	20	0.82	0.84
49	SCC-80-2	139.6	4	1250	374	62	13602	1704	835	21	0.87	0.90
50	SCC-200-1	139.6	4	3040	374	62	13602	1704	610	15	0.89	1.06
51	SCC-200-2	139.6	4	2130	374	62	13602	1704	688	17	0.77	0.87
52	SCC-300-1	139.6	4	3270	374	62	13602	1704	541	14	0.84	1.04
53	SCC-300-2	139.6	4	3270	374	62	13602	1704	569	14	0.90	1.09
54	SCC-440-1	139.6	4	4670	374	62	13602	1704	342	9	0.88	1.55
55	SCC-440-2	139.6	4	4670	374	62	13602	1704	334	8	0.85	1.51
56	SCCE-80-1	139.6	4	1070	374	62	13602	1704	808	20	0.81	0.84
57	SCCE-80-2	139.6	4	1410	374	62	13602	1704	909	23	1.00	1.04
58	SCCE-200-1	139.6	4	2125	374	62	13602	1704	664	17	0.73	0.83
59	SCCE-200-2	139.6	4	2125	374	62	13602	1704	784	20	0.91	1.01
60	SCCE-300-1	139.6	4	3270	374	62	13602	1704	551	14	0.86	1.06
61	SCCE-300-2	139.6	4	3270	374	62	13602	1704	513	13	0.79	0.98
62	SCCE-440-1	139.6	4	4670	374	62	13602	1704	373	9	0.97	1.67
63	SCCE-440-2	139.6	4	4670	374	62	13602	1704	378	9	0.98	1.69

J.M. Portoles, M.L. Romero, F.C. Filippou (2011)											EC-4	AISC-2010
test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	M_{test} (kN.m)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
64	C-1	100	3	2135	322	32.7	6940	914	182	4	0.59	0.77
65	C-2	100	3	2135	322	34.5	6940	914	118	6	0.43	0.54
66	C-3	100	3	2135	322	65.8	6940	914	249	5	0.67	0.78
67	C-4	100	3	2135	322	71.64	6940	914	152	8	0.46	0.51
68	C-5	100	3	2135	322	95.63	6940	914	271	5	0.74	0.76
69	C-6	100	3	2135	322	93.01	6940	914	154	8	0.46	0.44
70	C-9	100	3	3135	322	39.4	6940	914	140	3	0.65	1.03
71	C-10	100	3	3135	322	36.7	6940	914	94	5	0.50	0.71
72	C-11	100	3	3135	322	71.7	6940	914	160	3	0.71	1.09
73	C-12	100	3	3135	322	79.6	6940	914	103	5	0.50	0.64
74	C-13	100	3	3135	322	94.5	6940	914	160	3	0.74	1.06

75	C-14	100	3	3135	322	90.4	6940	914	107	5	0.53	0.64
76	C-18	100	5	2135	322	35.4	6362	1492	270	5	0.66	0.88
77	C-19	100	5	2135	322	30.6	6362	1492	161	8	0.49	0.64
78	C-20	100	5	2135	322	70.2	6362	1492	314	6	0.67	0.86
79	C-21	100	5	2135	322	61	6362	1492	184	9	0.46	0.55
80	C-22	101.6	5	2135	320	95.4	6590	1517	330	7	0.67	0.76
81	C-23	101.6	5	2135	320	81.7	6590	1517	214	11	0.51	0.57
82	C-24	101.6	5	3135	320	38.7	6590	1517	213	4	0.72	1.15
83	C-25	101.6	5	3135	320	39.6	6590	1517	145	7	0.57	0.84
84	C-26	101.6	5	3135	320	71.9	6590	1517	231	5	0.75	1.17
85	C-27	101.6	5	3135	320	72.5	6590	1517	153	8	0.56	0.78
86	C-28	101.6	5	3135	320	86.4	6590	1517	247	5	0.82	1.21
87	C-29	101.6	5	3135	320	96.8	6590	1517	165	8	0.63	0.77
88	C-30	125	5	3135	320	88	10387	1885	474	9	0.86	1.03
89	C-31	125	5	3135	320	97	10387	1885	318	16	0.66	0.66
90	C-32	125	5	3135	320	107.4	10387	1885	490	10	0.92	1.00
91	C-33	125	5	3135	320	97.9	10387	1885	323	16	0.68	0.67
92	C-34	160.1	5.7	3135	320	87.4	17366	2765	1013	20	0.86	0.98
93	C-35	160.1	5.7	3135	320	74.8	17366	2765	642	32	0.63	0.72
94	C-36	160.1	5.7	3135	320	83.1	17366	2765	1012	20	0.86	1.01
95	C-37	160.1	5.7	3135	320	98.5	17366	2765	686	34	0.67	0.65

		J.M. Portoles, E. Serra, M.L. Romero (2013)									EC-4	AISC-2010
test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	M_{test} (kN.m)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
96	C-NSC-20	159	6	2000	377	39.9	16972	2884	851	17	0.65	0.75
97	C-NSC-50	159	6	2000	377	40.1	16972	2884	587	29	0.53	0.60
98	F-NSC-50	159	6	2000	394	35.1	16972	2884	731	37	0.76	0.87
99	F-NSC-50	159	6	2000	457	35.1	16972	2884	753	38	0.69	0.80
100	C-HSC-50	159	6	2000	376	75.7	16972	2884	870	44	0.69	0.75
101	F-HSC-50	159	6	2000	376	89.8	16972	2884	874	44	0.66	0.67
102	F-HSC-50	159	6	2000	461	94.2	16972	2884	934	47	0.63	0.65
103	C-UHSC-20	159	6	2000	380	109.8	16972	2884	1462	29	0.91	0.88
104	C-UHSC-20b	159	6	2000	487	110.7	16972	2884	1525	31	0.85	0.85
105	C-UHSC-50	159	6	2000	444	91.4	16972	2884	1033	52	0.76	0.77
106	F-UHSC-50	159	6	2000	366	122.6	16972	2884	921	46	0.69	0.56
107	F-UHSC-50	159	6	2000	366	131.2	16972	2884	905	45	0.68	0.52

L.H. Han, G.H. Yao (2003)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	M_{test} (kN.m)	EC-4 (N_{test})/($\chi N_{pl,Rd}$)	AISC-2010 (P_{test}/P_n)
108	S-2	120	2.7	360	340	16	10333	977	533	7	1.68	1.74
109	S-4	120	2.7	360	340	28.8	10333	977	600	8	1.37	1.44
110	L-1	120	2.7	1400	340	28.8	10333	977	590	8	1.45	1.62
111	L-5	120	2.7	1400	340	28.8	10333	977	412	13	1.33	1.44

S-H Lee, B Uy, S-H Kim, Y-H Choi, S-M Choi (2011)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	M_{test} (kN.m)	EC-4 (N_{test})/($\chi N_{pl,Rd}$)	AISC-2010 (P_{test}/P_n)
112	E24-30	240	6	1400	489	31.5	40828	4411	1277	153	0.80	0.86
113	E36-30	360	6	1760	498	31.5	95115	6673	4294	258	0.99	1.06
114	E48-30	480	6	2120	468	31.5	172021	8935	3323	798	0.71	0.78
115	E60-30	600	6	2480	517	31.5	271547	11197	4590	1377	0.58	0.63
116	E24-60	240	6	1400	489	59	40828	4411	1438	173	0.59	0.60
117	E36-60	360	6	1760	498	59	95115	6673	2537	457	0.48	0.49
118	E48-60	480	6	2120	468	59	172021	8935	3895	935	0.52	0.53
119	E30-30	300	12	1580	479	31.5	59828	10857	3683	552		
120	E4830	480	12	2120	489	31.5	163313	17643	5135	1232	0.80	0.86

Furlong (1967)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	M_{test} (kN.m)	EC-4 (N_{test})/($\chi N_{pl,Rd}$)	AISC-2010 (P_{test}/P_n)
121	1	114	3.2	914	413	29	9093	1114	445	11	1.04	1.13
122	2	114	3.2	914	413	29	9093	1114	400	12	0.99	1.06
123	3	114	3.2	914	413	29	9093	1114	334	15	0.98	1.06
124	4	114	3.2	914	413	29	9093	1114	222	10	0.49	0.53
125	5	114	3.2	914	413	29	9093	1114	111	16	0.37	0.39
126	6	152	1.5	1016	331	26	17437	709	568	10	1.12	1.20
127	7	152	1.5	1016	331	26	17437	709	422	18	1.64	1.87
128	8	152	1.5	1016	331	26	17437	709	286	17	1.00	1.13
129	9	152	1.5	1016	331	21	17437	709	136	16	0.60	0.68
130	10	152	1.5	1016	331	21	17437	709	135	15	0.46	0.51
131	11	127	2.4	1067	289	35	11728	939	568	9	1.17	1.26
132	12	127	2.4	1067	289	35	11728	939	534	13	1.33	1.42

133	13	127	2.4	1067	289	35	11728	939	400	16	1.97	2.16
134	14	127	2.4	1067	289	35	11728	939	351	16	1.62	1.87
135	15	127	2.4	1067	289	35	11728	939	349	14	1.11	1.22
136	16	127	2.4	1067	289	35	11728	939	345	16	1.63	1.86
137	17	127	2.4	1067	289	35	11728	939	306	17		
138	18	127	2.4	1067	289	35	11728	939	267	18		
139	19	127	2.4	1067	289	35	11728	939	261	17		
140	20	127	2.4	1067	289	35	11728	939	175	16		
141	21	127	2.4	1067	289	35	11728	939	89	16	0.43	0.48
142	22	127	2.4	1067	289	35	11728	939	44	15	0.16	0.17

Neogi, Seri, Chapman(1969)											EC-4	AISC-2010
test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	M_{test} (kN.m)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
143	M1	169	5.1	3327	303	44	19806	2626	611	29	0.68	0.81
144	M2	169	5.3	3327	303	43	19706	2726	689	26	0.73	0.89
145	M3	169	5.7	3327	289	34	19508	2924	591	28	0.72	0.87
146	M4	168	6.6	3327	293	30	18821	3347	613	29	0.74	0.91
147	M5	169	7.2	3327	306	26	18772	3660	641	31	0.75	0.93
148	M6	169	7.3	3327	306	27	18723	3708	725	28	0.78	0.98
149	M7	169	8.8	3327	317	26	18003	4429	744	36	0.76	0.96
150	M8	140	9.6	3327	268	33	11461	3933	538	17	0.71	0.95
151	M9	140	9.8	3327	268	22	11385	4009	538	17	0.77	1.04
152	M10	141	5	3327	288	34	13478	2136	409	13	0.73	0.94
153	C5	127	1.8	1410	189	67	11960	708	947	6	1.38	1.38
154	C6	127	2.8	1410	265	67	11575	1093	1027	6	1.30	1.36
155	C7	127	1.7	1715	189	60	11998	669	836	5	1.36	1.44
156	C8	127	3	1715	265	60	11499	1169	790	5	1.05	1.18
157	C9	127	1.8	2032	189	40	11960	708	347	6	0.81	0.90
158	C10	127	3.3	2032	265	40	11385	1282	517	8	0.97	1.14
159	C11	127	1.6	2032	189	43	12037	630	338	7	0.87	0.94
160	C12	127	3.3	2032	265	43	11385	1282	494	11	0.97	1.10

Knowles, Park(1969)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm²)	f_{ck} (N/mm²)	A_c (mm²)	A_s (mm²)	N_{test} (kN)	M_{test} (kN.m)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
161	1	89	5.8	813	400	41	4705	1516	554	4	0.91	0.99
162	2	89	5.8	1422	400	41	4705	1516	469	4	0.85	1.06
163	3	89	5.8	813	400	41	4705	1516	195	5	0.33	0.36
164	4	89	5.8	1118	400	41	4705	1516	191	5	0.34	0.39
165	5	89	1.4	813	482	41	5836	385	302	2	0.92	1.02
166	6	89	1.4	813	482	41	5836	385	89	2	0.27	0.30

Rangan, Joyce (1992)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm²)	f_{ck} (N/mm²)	A_c (mm²)	A_s (mm²)	N_{test} (kN)	M_{test} (kN.m)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
167	1	102	1.6	808	214	67	7667	505	430	4	0.95	0.92
168	2	102	1.6	808	214	67	7667	505	235	7	0.78	0.73
169	3	102	1.6	1313	214	67	7667	505	350	4	0.82	0.83
170	4	102	1.6	1313	214	67	7667	505	190	6	0.55	0.52
171	5	102	1.6	1565	214	67	7667	505	315	3	0.78	0.81
172	6	102	1.6	1819	214	67	7667	505	279	3	0.76	0.80
173	7	102	1.6	1819	214	67	7667	505	140	4	0.41	0.39
174	8	102	1.6	2322	214	67	7667	505	220	2	0.77	0.84
175	9	102	1.6	2322	214	67	7667	505	126	4	0.48	0.44

Cai (1991)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm²)	f_{ck} (N/mm²)	A_c (mm²)	A_s (mm²)	N_{test} (kN)	M_{test} (kN.m)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
176	PB1-1	166	5	665	313	41	19113	2529	1470	29	1.35	1.43
177	PB1-2	166	5	665	313	41	19113	2529	1517	30	1.42	1.50
178	PB1-3	166	5	665	313	41	19113	2529	1240	50	1.92	2.12
179	PB1-4	166	5	665	313	41	19113	2529	1245	50	1.96	2.18
180	PB1-5	166	5	665	313	41	19113	2529	707	71		
181	PB1-6	166	5	665	313	41	19113	2529	637	64		
182	PB2-1	166	5	1496	298	41	19113	2529	1465	29	1.46	1.58
183	PB2-2	166	5	1496	298	41	19113	2529	1431	29	1.41	1.53
184	PB2-3	166	5	1496	298	41	19113	2529	1093	44	1.41	1.51
185	PB2-4	166	5	1496	298	41	19113	2529	1147	46	1.60	1.73
186	PB2-5	166	5	1496	298	41	19113	2529	582	58		
187	PB2-6	166	5	1496	298	41	19113	2529	568	57		

188	PB3-1	166	5	1989	277	41	19113	2529	1225	25	1.23	1.37
189	PB3-2	166	5	1989	277	41	19113	2529	1156	23	1.14	1.27
190	PB3-3	166	5	1989	277	41	19113	2529	916	37	1.13	1.23
191	PB3-4	166	5	1989	298	41	19113	2529	896	36	1.04	1.13
192	PB3-5	166	5	1989	298	41	19113	2529	477	48	0.81	0.90
193	PB3-6	166	5	1989	277	41	19113	2529	515	52		
194	PC1-1	166	5	2990	285	28	19113	2529	1022	20	1.34	1.61
195	PC1-2	166	5	2990	289	28	19113	2529	1094	22	1.46	1.76
196	PC1-3	166	5	2240	285	28	19113	2529	860	34	1.41	1.55
197	PC1-4	166	5	2240	304	28	19113	2529	907	36	1.46	1.62
198	PC1-5	166	5	1331	290	41	19113	2529	1460	29	1.46	1.57
199	PC1-6	166	5	1331	290	41	19113	2529	1568	31	1.63	1.75
200	PC1-7	166	5	1331	290	41	19113	2529	657	66		
201	PC1-8	166	5	1989	290	41	19113	2529	809	49	1.70	1.94
202	PC1-9	166	5	1989	290	41	19113	2529	882	53		

Kloppel, Goder (1957)

test No.	spec. No	D	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	M_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)	(kN.m)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
203	7	95	13	1420	274	20	3805	3284	947	1	1.12	1.32
204	8	95	13	1420	272	20	3805	3284	938	2	1.16	1.36
205	9	95	13	1420	272	20	3805	3284	907	2	1.12	1.32
206	10	95	13	861	274	20	3805	3284	1018	2	1.17	1.25
207	11	95	13	861	272	20	3805	3284	1008	2	1.16	1.25
208	12	95	13	861	272	20	3805	3284	1034	2	1.20	1.28
209	13	95	13	1981	278	20	3805	3284	886	4	1.31	1.75
210	14	95	13	1981	276	20	3805	3284	907	2	1.25	1.68
211	15	95	13	1981	279	20	3805	3284	917	2	1.26	1.69
212	41	95	3.7	861	327	25	6027	1061	656	1	1.48	1.61
213	42	95	3.7	861	386	25	6027	1061	686	1	1.33	1.45
214	43	95	3.4	861	335	25	6110	978	656	1	1.47	1.61
215	44	95	3.9	1420	327	25	5972	1116	567	1	1.29	1.53
216	45	95	3.9	1420	386	25	5972	1116	606	1	1.28	1.55
217	46	95	3.6	1420	335	25	6055	1034	576	1	1.35	1.61
218	47	95	3.8	1981	327	25	5999	1089	536	2	1.53	2.02
219	48	95	3.8	1981	386	25	5999	1089	566	1	1.44	1.96
220	49	95	3.5	1981	335	25	6082	1006	488	1	1.44	1.91
221	63	216	4.1	2220	285	23	33914	2729	1023	3	0.71	0.78
222	64	216	4.1	2220	299	23	33914	2729	1834	6	1.27	1.40
223	65	216	4.1	2220	288	30	33914	2729	2289	5	1.39	1.55

224	66	216	4.1	2220	286	30	33914	2729	2239	9	1.41	1.56
225	69	216	6	2220	389	23	32685	3958	2462	7	1.20	1.32
226	70	216	6	2220	393	23	32685	3958	2421	12	1.21	1.33
227	71	216	6.5	2220	295	30	32365	4278	2804	6	1.38	1.51
228	72	216	6.3	2220	404	30	32493	4150	2932	6	1.23	1.36
229	73	95	3.9	1981	332	24	5972	1116	498	1	1.34	1.79
230	74	95	3.4	1981	337	24	6110	978	473	1	1.38	1.85
231	76	95	3.7	1981	327	24	6027	1061	413	1	1.18	1.57
232	83	121	3.7	1049	295	21	10136	1363	695	1	1.19	1.32
233	84	121	3.7	1049	327	21	10136	1363	746	4	1.33	1.46
234	85	121	3.8	1049	308	24	10100	1399	837	3	1.43	1.57
235	86	121	4	1049	327	24	10029	1470	867	4	1.44	1.57
236	89	121	5.6	1049	344	21	9469	2030	998	5	1.32	1.44
237	90	121	5.4	1049	343	21	9538	1961	1018	2	1.26	1.38
238	91	121	5.5	1049	330	24	9503	1996	1099	7	1.56	1.70
239	92	121	5.6	1049	322	24	9469	2030	1079	5	1.48	1.61
240	95	121	3.7	2311	295	21	10136	1363	641	1	1.28	1.64
241	96	121	3.8	2311	327	21	10100	1399	629	2	1.23	1.59
242	97	121	3.7	2311	308	24	10136	1363	695	1	1.35	1.73
243	98	121	3.9	2311	327	24	10064	1435	755	2	1.38	1.78
244	101	121	5.7	2311	344	21	9434	2065	786	3	1.17	1.52
245	102	121	5.5	2311	343	21	9503	1996	816	1	1.16	1.51
246	103	121	5.6	2311	330	24	9469	2030	874	1	1.23	1.59
247	104	121	5.4	2311	322	24	9538	1961	865	2	1.30	1.67

											EC-4	AISC-2010
		Kvedaras, Tomaszewicz (1994)										
test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	M_{test} (kN.m)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
248	S1	250	2	2202	240	73	47529	1558	2602	60	1.05	0.98
249	S2	250	2	2202	240	71	47529	1558	2002	92	1.15	1.06
250	S3	250	2	2202	240	70	47529	1558	2402	43	0.87	0.84
251	S4	250	2	2202	240	41	47529	1558	1699	51	1.06	1.14
252	S5	250	2	2202	240	36	47529	1558	1299	39	0.79	0.85

O'Shea, Bridge (2000)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	M_{test} (kN.m)	EC-4 (N_{test})/($\chi N_{pl,Rd}$)	AISC-2010 (P_{test}/P_n)
253	S30E250B	165	2.8	582	363	48	19956	1427	1524	11	1.10	1.23
254	S20E250A	190	1.9	660	256	41	27230	1123	1532	14	1.22	1.32
255	S12E250A	190	1.1	663	185	41	27700	653	1229	10	1.07	1.14
256	S10E250A	190	0.9	663	211	41	27818	535	1218	9	1.05	1.12
257	S30E150B	165	2.8	579	363	48	19956	1427	1123	19	0.95	1.01
258	S20E150A	190	1.9	663	256	41	27230	1123	1283	21	1.12	1.19
259	S16E150B	190	1.5	663	306	48	27465	888	1259	19	0.92	0.99
260	S12E150A	190	1.1	663	185	41	27700	653	1023	19	1.16	1.26
261	S10E150A	190	0.9	663	211	41	27818	535	1016	14	0.99	1.06
262	S30E280A	165	2.8	579	363	80	19956	1427	1939	17	1.19	1.12
263	S20E280B	190	1.9	663	256	75	27230	1123	2202	22	1.21	1.14
264	S10E280B	190	0.9	665	211	75	27818	535	1909	17	1.15	1.07
265	S30E180A	165	2.8	579	363	80	19956	1427	1653	30	1.16	1.06
266	S20E180B	190	1.9	663	256	75	27230	1123	1729	36	1.23	1.12
267	S16E180A	190	1.5	663	306	80	27465	888	1924	27	1.10	0.98
268	S10E180B	190	0.9	665	211	75	27818	535	1531	28	1.13	1.02
269	S30E210B	165	2.8	579	363	113	19956	1427	2245	16	1.22	0.95
270	S20E210B	190	1.9	660	256	113	27230	1123	2682	19	1.24	0.92
271	S10E210B	190	0.9	660	211	113	27818	535	2111	8	0.98	0.72
272	S30E110B	165	2.8	579	363	113	19956	1427	1879	28	1.15	0.85
273	S20E110B	190	1.9	665	256	113	27230	1123	2385	41	1.48	0.98
274	S16E110B	190	1.5	660	306	113	27465	888	2419	31	1.29	0.89
275	S12E110B	190	1.1	663	185	113	27700	653	1924	33	1.24	0.81

Jung (1994)

test No.	spec. No	D (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	M_{test} (kN.m)	EC-4 (N_{test})/($\chi N_{pl,Rd}$)	AISC-2010 (P_{test}/P_n)
276	5	267	5.3	1300	244	33	51633	4357	999	63	0.46	0.50
277	6	267	5.3	1300	244	33	51633	4357	999	61	0.46	0.49
278	7	267	5.3	1300	244	33	51633	4357	1499	67	0.71	0.76
279	8	267	5.3	1300	244	33	51633	4357	999	67	0.47	0.51
280	9	267	5.3	1300	244	33	51633	4357	999	69	0.48	0.51
281	11	267	5.3	1300	244	44	51633	4357	1499	70	0.57	0.60
282	12	267	5.3	1300	244	44	51633	4357	999	74	0.38	0.41
283	17	267	4	1300	244	33	52685	3305	999	49	0.48	0.51
284	18	267	4	1300	244	33	52685	3305	1499	49	0.72	0.77

285	20	267	4	1300	244	44	52685	3305	999	50	0.38	0.40
286	21	267	4	1300	244	44	52685	3305	1499	49	0.57	0.60

4. RECTANGULAR BEAM COLUMNS

D. Liu (2005)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
1	S1	120	120	4	360	495	60	12530	1815	1294	1.05	1.13
2	S2	120	120	4	360	495	60	12530	1815	1125	1.22	1.16
3	S3	120	120	4	360	495	60	12530	1815	949	1.04	0.98
4	S4	120	120	4	360	495	60	12530	1815	810	1.14	1.00
5	S5	100	150	4	360	495	60	13050	1895	1422	1.15	1.14
6	S6	100	150	4	360	495	60	13050	1895	1190	1.26	1.18
7	S7	100	150	4	360	495	60	13050	1895	964	1.23	1.10
8	S8	100	150	4	360	495	60	13050	1895	763	1.03	0.91
9	S9	90	180	4	360	495	60	14090	2055	1491	1.16	1.14
10	S10	90	180	4	360	495	60	14090	2055	1319	1.18	1.13
11	S11	90	180	4	360	495	60	14090	2055	1208	1.26	1.16
12	S12	90	180	4	360	495	60	14090	2055	1051	1.19	1.07
13	S13	130	130	4	360	495	60	14870	1975	1472	1.11	1.11
14	S14	130	130	4	360	495	60	14870	1975	1305	1.20	1.15
15	S15	130	130	4	360	495	60	14870	1975	1022	1.13	1.04
16	S16	130	130	4	360	495	60	14870	1975	789	0.93	0.84
17	L1	100	150	4	360	495	60	13050	1895	1130	0.85	0.86
18	L2	100	150	4	360	495	60	13050	1895	884	0.78	0.76
19	L3	100	150	4	360	495	60	13050	1895	711	0.70	0.67
20	L4	100	150	4	360	495	60	13050	1895	617	0.67	0.63

D. Liu (2004)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
21	E01	150	150	4.2	870	550	60.8	20047	2393	1678	0.82	1.10
22	E02	150	150	4.2	870	550	72.1	20047	2393	1850	1.23	1.13
23	E03	150	150	4.2	2170	550	60.8	20047	2393	1330	0.92	0.91
24	E04	150	150	4.2	2170	550	72.1	20047	2393	1020	0.85	0.76
25	E05	180	120	4.2	1040	550	60.8	19147	2393	1950	1.97	1.72
26	E06	180	120	4.2	1040	550	72.1	19147	2393	1140		
27	E07	120	80	4.2	1740	550	60.8	7983	1557	660	1.15	1.05
28	E08	120	80	4.2	1740	550	72.1	7983	1557	855	1.69	1.45
29	E09	200	100	4.2	1150	550	60.8	17547	2393	1310		
30	E10	200	100	4.2	1150	550	72.1	17547	2393	1800		
31	E11	160	80	4.2	2310	550	60.8	10848	1892	670		

32	E12	160	80	4.2	2310	550	72.1	10848	1892	1020	3.26	2.27

L.H. Guo, Y.Y. Wang, S.M. Zhang (2011)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
34	S-1	150	150	3.6	1350	283	46	20381	2075	1552	1.37	1.17
35	R-1	100	200	3.6	1800	283	47	17881	2075	1461	1.19	1.18
36	R-5	100	200	3.6	1800	283	47	17881	2075	655	0.6525	0.646

L.H. Han, G.H. Yao (2004)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
37	Issc 2-1	200	200	3	2310	304	49	37628	2341	1450	0.67	0.87
38	Issc 2-2	200	200	3	2310	304	49	37628	2341	1415	0.75	0.85
39	Ish 2-1	200	200	3	2310	304	49	37628	2341	1502	0.81	0.91
40	Ish 2-2	200	200	3	2310	304	49	37628	2341	1535	0.83	0.94
41	Isv 2	200	200	3	2310	304	49	37628	2341	1620	0.89	1.01

Y.F. Yang, L.H. Han (2011)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
42	Ss-1	150	150	3	450	324	50	20728	1741	846	0.60	0.59
43	Ss-2	150	150	3	450	324	50	20728	1741	797	0.57	0.63
44	Ss-3	150	150	3	450	324	50	20728	1741	778	0.64	0.71
45	Scfst-1	150	150	3	450	324	50	20728	1741	1618	1.01	1.13
46	Scfst-2	150	150	3	450	324	50	20728	1741	1260	0.98	1.09
47	Scfst-3	150	150	3	450	324	50	20728	1741	1244	1.26	1.41
48	Rp-1	120	180	3	540	324	50	19828	1741	957	0.61	0.68
49	Rp-2	120	180	3	540	324	50	19828	1741	929	0.70	0.78
50	Rp-3	120	180	3	540	324	50	19828	1741	843	0.75	0.84
51	Rr-1	120	180	3	540	324	50	19828	1741	811	0.52	0.58
52	Rr-2	120	180	3	540	324	50	19828	1741	752	0.55	0.61
53	Rr-3	120	180	3	540	324	50	19828	1741	747	0.64	0.71
54	Rcfst-1	120	180	3	540	324	50	19828	1741	1476	0.95	1.05
55	Rcfst-2	120	180	3	540	324	50	19828	1741	1140	1.17	1.31

											EC-4	AISC-2010
		Baba, Fujimoto, Mukai, Nishiyama (1995)										
test No.	spec. No	B	H	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
		(mm)	(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)		
56	ER4-A-4-19	149	149	4.4	447	262	41.1	19651	2484	778		
57	ER4-A-4-57	148	148	4.4	447	262	41.1	19371	2467	264	0.34	0.37
58	ER4-C-2-25	215	215	4.4	648	262	25.4	42518	3641	1152		
59	ER4-C-2-56	214	214	4.4	648	262	25.4	42107	3623	514	0.43	0.46
60	ER4-C-4-21	215	215	4.4	648	262	41.1	42518	3641	1392		
61	ER4-C-4-38	215	215	4.4	648	262	41.1	42518	3641	1037	0.96	0.94
62	ER4-C-4-51	215	215	4.4	648	262	41.1	42518	3641	573	0.33	0.37
63	ER4-C-8-33	214	214	4.4	648	262	80.3	42107	3623	2029	1.14	1.35
64	ER4-C-8-46	215	215	4.4	648	262	80.3	42518	3641	1455	0.78	0.66
65	ER4-D-4-27	323	323	4.4	973	262	41.1	98730	5533	3322	1.58	1.70
66	ER4-D-4-60	323	323	4.4	973	262	41.1	98730	5533	1495	0.37	0.42
67	ER6-A-4-22	144	144	6.4	431	618	41.1	17200	3397	1691		
68	ER6-A-4-61	144	144	6.4	431	618	41.1	17200	3397	630	0.64	0.52
69	ER6-C-2-25	211	211	6.4	630	618	25.4	39280	5102	2425	6.09	2.50
70	ER6-C-2-58	210	210	6.4	442	618	25.4	38885	5076	1045	0.54	0.47
71	ER6-C-4-18	210	210	6.4	630	618	41.1	38885	5076	2730		
72	ER6-C-4-44	210	210	6.4	630	618	41.1	38885	5076	2107	1.29	1.07
73	ER6-C-4-57	209	209	6.4	630	618	41.1	38491	5051	862	0.37	0.34
74	ER6-C-8-24	210	210	6.4	630	618	80.3	38885	5076	3409	0.82	1.91
75	ER6-C-8-54	210	210	6.4	630	618	80.3	38885	5076	1515	0.52	0.44
76	ER6-D-4-23	319	319	6.4	955	618	41.1	93773	7849	4180	2.05	0.52
77	ER6-D-4-47	319	319	6.4	955	618	41.1	93773	7849	2003	0.40	0.40
78	ER8-A-4-08	121	121	6.5	252	835	40.5	11641	2856	232		
79	ER8-C-2-38	175	175	6.5	366	835	25.4	26228	4254	1664	3.41	1.50
80	ER8-C-2-57	175	175	6.5	366	835	25.4	26228	4254	2518	1.91	1.42

81	ER8-C-4-24	175	175	6.5	366	835	40.5	26228	4254	1164	13.83	0.26
82	ER8-C-4-38	175	175	6.5	366	835	40.5	26228	4254	1863	2.37	1.37
83	ER8-C-4-57	176	176	6.5	366	835	40.5	26553	4280	2749	1.44	1.21
84	ER8-C-8-39	176	176	6.5	366	835	77	26553	4280	2241	1.79	1.11
85	ER8-C-8-58	175	175	6.5	366	835	77	26228	4254	3362	1.59	1.24
86	ER8-D-4-40	265	265	6.5	554	835	40.5	63498	6583	3320	1.13	0.94
87	ER8-D-4-60	265	265	6.5	554	835	40.5	63498	6583	4939	1.13	1.05

K. Tsuda, C. Matsui, E. Mino											EC-4	AISC-2010
test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
88	C4-1	150	150	4.5	600	412	41	19864	2567	1185	0.75	0.98
89	C4-3	150	150	4.5	600	412	41	19864	2567	735	0.94	0.96
90	C4-5	150	150	4.5	600	412	41	19864	2567	515	0.86	0.95
91	C8-1	150	150	4.5	1200	412	41	19864	2567	1134	0.89	0.94
92	C8-3	150	150	4.5	1200	412	41	19864	2567	666	0.77	0.80
93	C8-5	150	150	4.5	1200	412	41	19864	2567	484	0.74	0.76
94	C12-1	150	150	4.5	1800	412	41	19864	2567	1026	0.81	0.86
95	C12-3	150	150	4.5	1800	412	41	19864	2567	632	0.73	0.76
96	C12-5	150	150	4.5	1800	412	41	19864	2567	445	0.62	0.64
97	C18-1	150	150	4.5	2700	412	41	19864	2567	847	0.70	0.74
98	C18-3	150	150	4.5	2700	412	41	19864	2567	554	0.63	0.66
99	C18-5	150	150	4.5	2700	412	41	19864	2567	894		
100	C24-1	150	150	4.5	3600	412	41	19864	2567	706	0.67	0.69
101	C24-3	150	150	4.5	3600	412	41	19864	2567	441	0.52	0.53
102	C24-5	150	150	4.5	3600	412	41	19864	2567	327	0.44	0.44
103	C30-1	150	150	4.5	4500	412	41	19864	2567	589	0.71	0.67
104	C30-3	150	150	4.5	4500	412	41	19864	2567	373	0.54	0.50

Furlong (1967)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
105	1	127	127	4.8	914	484	45	13763	2287	1112	0.81	0.88
106	2	127	127	4.8	914	484	45	13763	2287	667	0.93	0.83
107	3	127	127	4.8	914	484	45	13763	2287	667	1.22	1.00
108	4	127	127	4.8	914	484	45	13763	2287	445	0.34	0.35
109	5	102	102	2.1	914	331	23	9561	828	374	1.04	1.12
110	6	102	102	2.1	914	331	23	9561	828	374	1.04	1.12
111	7	102	102	2.1	914	331	23	9561	828	242	1.09	1.15
112	8	102	102	2.1	914	331	23	9561	828	90	0.49	0.53
113	9	102	102	2.1	914	331	23	9561	828	89		
114	10	102	102	3.2	914	331	29	9131	1238	438		
115	11	102	102	3.2	914	331	29	9131	1238	306		
116	12	102	102	3.2	914	331	29	9131	1238	302		
117	13	102	102	3.2	914	331	29	9131	1238	261		
118	14	102	102	3.2	914	331	29	9131	1238	129		
119	15	102	102	3.2	914	331	29	9131	1238	128		
120	16	102	102	3.2	914	331	29	9131	1238	40		

knowles, park (1969)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
121	17	76	76	3.3	813	324	41	4807	932	346	0.88	0.99
129	18	76	76	3.3	1422	324	41	4807	932	281	0.79	0.86
123	19	76	76	3.3	813	324	41	4807	932	217	0.74	0.83
124	20	76	76	3.3	1422	324	41	4807	932	157	0.51	0.57

Bridge (1976)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
125	21	200	200	10	2131	291	30	32314	7342	1956	0.93	1.01
126	22	150	150	6.5	3051	254	35	18733	3622	680	0.71	0.76
127	23	150	150	6.5	3051	254	35	18733	3622	513	0.60	0.64

Grauers (1993)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
128	1	120	120	5	3195	304	47	12079	2236	610	0.91	0.94
129	2	120	120	5	3195	438	46	12079	2236	700	1.00	0.93
130	3	120	120	5	3195	327	96	12079	2236	710	0.92	0.80
131	4	120	120	5	3195	439	96	12079	2236	830	1.06	0.88
132	5	120	120	8	3195	322	39	10761	3419	740	0.90	0.91
133	6	120	120	8	3195	300	46	10761	3419	770	0.94	0.96
134	7	120	120	8	3195	376	47	10761	3419	870	1.01	0.97
135	8	120	120	8	3195	322	103	10761	3419	820	0.84	0.75
136	9	120	120	8	3195	379	103	10761	3419	1000	1.05	0.90
137	10	120	120	8	3195	379	39	10761	3419	820	0.98	0.93
138	11	120	120	8	3195	376	93	10761	3419	1030	1.11	0.97
139	12	120	120	8	3195	364	93	10761	3419	960	1.03	0.90
140	13	120	120	8	3195	364	80	10761	3419	1160	1.37	1.21
141	14	120	120	8	3195	379	80	10761	3419	1110	1.26	1.87
142	16	120	120	8	3195	395	96	10761	3419	1040	1.10	0.95
143	17	120	120	8	3195	404	92	10761	3419	1010	1.06	0.92
144	18	120	120	8	3195	404	92	10761	3419	750	0.73	0.65
145	25	120	120	8	3195	395	92	10761	3419	960	1.00	0.87
146	23	120	120	8	1697	379	31	10761	3419	1080	1.12	1.42
147	24	120	120	8	1697	379	92	10761	3419	1300	1.03	1.31
148	27	250	250	8	3195	379	33	54701	7579	3400	0.95	0.99
149	28	250	250	8	3195	379	91	54701	7579	5300	1.05	0.98

Hardika, Gardner (2004)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
150	SNL-1	203	203	4.4	1801	390	44	37697	3446	3920		
151	SNL-2	203	203	4.4	1801	390	44	37697	3446	553	0.21	0.22
152	SNL-3	203	203	4.4	1801	390	44	37697	3446	1113	0.41	0.45
153	SCL-1	203	203	9	1801	393	44	34155	6775	7715		
154	SCL-2	203	203	9	1801	393	44	34155	6775	1390		
155	SCL-3	203	203	9	1801	393	44	34155	6775	2662		
156	SNH-1	203	203	9	1801	411	99	34155	6775	6082	2.09	1.68
157	SNH-2	203	203	9	1801	411	99	34155	6775	10178	4.21	3.15
158	SNH-3	203	203	9	1801	411	99	34155	6775	8434	3.50	2.61
159	SCH-1	203	203	9	1801	378	83	34155	6775	9349		

160	SCH-2	203	203	9	1801	378	87	34155	6775	15592		
161	SCH-3	203	203	9	1801	378	89	34155	6775	11449		

Han, Yao (2002)

test No.	spec. No	B	H	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
161	M-4-1	195	130	2.5	780	340	23	23745	1584	872	1.00	1.12
162	M-4-2	195	130	2.5	780	340	23	23745	1584	812	0.94	1.02
163	H-4-1	195	130	2.5	780	340	23	23745	1584	732	0.82	0.90
164	H-4-2	195	130	2.5	780	340	23	23745	1584	740	0.83	0.91
165	M-5-1	195	130	2.5	780	340	23	23745	1584	646	0.92	1.00
166	M-5-2	195	130	2.5	780	340	23	23745	1584	610	0.84	0.92
167	H-5-1	195	130	2.5	780	340	23	23745	1584	500	0.63	0.69
168	H-5-2	195	130	2.5	780	340	23	23745	1584	514	0.66	0.72
169	M-7-1	195	130	2.5	2339	340	23	23745	1584	670	0.84	0.92
170	M-7-2	195	130	2.5	2339	340	23	23745	1584	635	0.79	0.87
171	H-7-1	195	130	2.5	2339	340	23	23745	1584	525	0.63	0.70
172	H-7-2	195	130	2.5	2339	340	23	23745	1584	500	0.60	0.66

Uy. B (2000)

test No.	spec. No	B	H	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
180	HSS3	110	110	5	3000	750	30	9979	2036	1554	2.60	2.44
181	HSS4	110	110	5	3000	750	30	9979	2036	1281	3.91	2.82
182	HSS7	160	160	5	3000	750	30	22479	3036	1307	0.91	0.87
183	HSS10	160	160	5	3000	750	30	22479	3036	2023	1.39	1.33
184	HSS11	160	160	5	3000	750	30	22479	3036	1978	2.63	2.08
185	HSS16	210	210	5	3000	750	32	39979	4036	3105	1.14	1.15
186	HSS17	210	210	5	3000	750	32	39979	4036	2616	1.27	1.21

Seo, Chung (2002)

test No.	spec. No	B	H	t	KL	f_y	f_{ck}	A_c	A_s	N_{test}	EC-4	AISC-2010
		(mm)	(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)	(kN)	$(N_{test})/(\chi N_{pl,Rd})$	(P_{test}/P_n)
187	C4-1	125	125	3.2	500	451	96	14057	1533	1257	1.00	0.93
188	C4-3	125	125	3.2	500	451	96	14057	1533	658	0.70	0.63
189	C8-1	125	125	3.2	1001	451	96	14057	1533	1186	0.97	0.90

190	C8-3	125	125	3.2	1001	451	96	14057	1533	614	0.61	0.56
191	C12-1	125	125	3.2	1501	451	96	14057	1533	1129	0.96	0.90
192	C12-3	125	125	3.2	1501	451	96	14057	1533	570	0.57	0.53
193	C18-1	125	125	3.2	2250	451	96	14057	1533	827	0.76	0.72
194	C18-3	125	125	3.2	2250	451	96	14057	1533	446	0.47	0.44
195	C24-1	125	125	3.2	3000	451	96	14057	1533	666	0.78	0.69
196	C24-3	125	125	3.2	3000	451	96	14057	1533	361	0.47	0.41
197	C30-3	125	125	3.2	3749	451	96	14057	1533	287	0.49	0.41

Seo (2002)

test No.	spec. No	B (mm)	H (mm)	t (mm)	KL (mm)	f_y (N/mm^2)	f_{ck} (N/mm^2)	A_c (mm^2)	A_s (mm^2)	N_{test} (kN)	EC-4 $(N_{test})/(\chi N_{pl,Rd})$	AISC-2010 (P_{test}/P_n)
198	C04-1-00	125	125	3	500	444	69	14153	1441	1128	1.03	1.10
199	C04-1-15	125	125	3	500	444	69	14153	1441	1060	0.74	0.78
200	C04-1-30	125	125	3	500	444	69	14153	1441	1089	0.82	0.87
201	C04-1-45	125	125	3	500	444	69	14153	1441	1060	0.86	0.91
202	C04-3-00	125	125	3	500	444	69	14153	1441	612	0.88	0.99
203	C04-3-15	125	125	3	500	444	69	14153	1441	590	0.44	0.46
204	C04-3-30	125	125	3	500	444	69	14153	1441	577	0.48	0.51
205	C04-3-45	125	125	3	500	444	69	14153	1441	588	0.56	0.60
206	C08-1-00	125	125	3	1001	444	64	14153	1441	1024	0.97	1.05
207	C08-1-15	125	125	3	1001	444	64	14153	1441	1016	0.76	0.81
208	C08-1-30A	125	125	3	1001	453	64	14153	1441	927	0.73	0.77
209	C08-1-30B	125	125	3	1001	435	64	14153	1441	957	0.77	0.82
210	C08-1-45	125	125	3	1001	444	64	14153	1441	1003	0.86	0.93
211	C08-3-00	125	125	3	1001	444	64	14153	1441	551	0.67	0.74
212	C08-3-15	125	125	3	1001	444	64	14153	1441	537	0.42	0.45
213	C08-3-30	125	125	3	1001	444	64	14153	1441	548	0.49	0.52
214	C08-3-45	125	125	3	1001	444	64	14153	1441	539	0.53	0.58
215	C12-0-00	125	125	3	1501	444	67	14153	1441	882	0.83	0.88

216	C12-1-15	125	125	3	1501	444	67	14153	1441	917	0.71	0.75
217	C12-1-30	125	125	3	1501	444	67	14153	1441	893	0.73	0.77
218	C12-1-45	125	125	3	1501	444	67	14153	1441	894	0.78	0.83
219	C12-3-00	125	125	3	1501	444	67	14153	1441	487	0.56	0.61
220	C12-3-15	125	125	3	1501	444	67	14153	1441	497	0.40	0.42
221	C12-3-30	125	125	3	1501	444	67	14153	1441	489	0.44	0.46
222	C12-3-45	125	125	3	1501	444	67	14153	1441	493	0.49	0.52
223	C18-1-00	125	125	3	2250	444	67	14153	1441	792	0.83	0.87
224	C18-1-15	125	125	3	2250	444	67	14153	1441	799	0.71	0.73
225	C18-1-30	125	125	3	2250	444	67	14153	1441	804	0.75	0.78
226	C18-1-45	125	125	3	2250	444	67	14153	1441	785	0.77	0.80
227	C18-3-00	125	125	3	2250	444	67	14153	1441	438	0.55	0.59
228	C18-3-15	125	125	3	2250	444	67	14153	1441	452	0.42	0.43
229	C18-3-30	125	125	3	2250	444	67	14153	1441	424	0.42	0.44
230	C18-3-45	125	125	3	2250	444	67	14153	1441	435	0.48	0.50
231	C24-1-00	125	125	3	3000	444	67	14153	1441	602	0.78	0.75
232	C24-1-15	125	125	3	3000	444	67	14153	1441	596	0.68	0.65
233	C24-1-30	125	125	3	3000	444	67	14153	1441	583	0.69	0.66
234	C24-1-45	125	125	3	3000	444	67	14153	1441	571	0.70	0.67
235	C24-3-00	125	125	3	3000	444	67	14153	1441	337	0.50	0.48
236	C24-3-15	125	125	3	3000	444	67	14153	1441	337	0.39	0.38
237	C24-3-30	125	125	3	3000	444	67	14153	1441	333	0.42	0.40
238	C24-3-45	125	125	3	3000	444	67	14153	1441	331	0.44	0.42
239	C30-1-00	125	125	3	3749	444	67	14153	1441	492	0.85	0.74
240	C30-1-15	125	125	3	3749	444	67	14153	1441	496	0.77	0.67
241	C30-1-30	125	125	3	3749	444	67	14153	1441	492	0.79	0.69
242	C30-1-45	125	125	3	3749	444	67	14153	1441	496	0.82	0.72

243	C30-3-00	125	125	3	3749	444	67	14153	1441	298	0.58	0.51
244	C30-3-15	125	125	3	3749	444	67	14153	1441	292	0.47	0.41
245	C30-3-30	125	125	3	3749	444	67	14153	1441	290	0.49	0.43
246	C30-3-45	125	125	3	3749	444	67	14153	1441	286	0.51	0.45

APPENDIX 2

Resistance to Axial compression of circular CFT columns(CCFT) according to EC-4

spec. No	D	t	D/t	local buckling check	local buckling limit	kL	kL/D	F_y	f_{ck}	f_{cd}	f_{yd}	A_c
	(mm)	(mm)				(mm)		(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(mm²)
1	160.1	4.98	32.1	75.5	ok	2000	12.5	280	40	40	280	17704.5

A_s	I_c	I_a	E_c	E_s	EI_{eff}	N_{cr}	$N_{pl,Rk}$	λ	η_a	η_c	N_c	N_s
(mm²)	(mm⁴)	(mm⁴)	(N/mm²)	(N/mm²)	(N-mm²)	(kN)	(kN)				(kN)	(kN)
2426.9	2E+07	7E+06	35220	213000	2.08E+12	5141	1388	0.52	1	0	708.2	679.5

$N_{pl,Rd}$	δ	α	ϕ	χ	$\chi N_{pl,Rd}$	N_{test}	$(N_{test})/(\chi N_{pl,Rd})$
(kN)					(kN)	(kN)	
1388	0.49	0.21	0.67	0.92	1274	1261	0.9898

Resistance to Axial compression of circular CFT columns(CCFT) according to AISC-2010

spec. No	D	t	D/t	compact λ_P	section type	r_i	kL	kL/D	D_c	f_{yk}	f_{ck}	F_c
	(mm)	(mm)				(mm)	(mm)		(mm)	(N/mm²)	(N/mm²)	(N/mm²)
1	160.1	4.98	32.1	114.1	ok	9.96	2000	12.5	150.1	280	40	38

E_c	E_s	A_c	A_s	I_c	I_s	C_3	EI_{eff}	P_e	P_c	P_s
(N/mm²)	(N/mm²)	(mm²)	(mm²)	(mm⁴)	(mm⁴)		(N-mm²)	(kN)	(kN)	(kN)
33994	213000	17704	2426.9	2.E+07	7.31E+06	0.84	2.3E+12	5600.01	672.8	679.5

P_{no}	P_{no}/P_e	P_n	$\Phi_c P_n$	P_{test}	$(P_{test})/(P_n)$
(kN)		(kN)	(kN)	(kN)	
1352.3	0.241	1222.3	1222.3	1261	1.031666

Resistance to compression of Rectangular CFT columns(RCFT) according to EC-4

test No.	spec. No	B (mm)	H (mm)	t (mm)	B/t	H/t	λ	local buckling	r_i (mm)	kL (mm)	B_c (mm)	H_c (mm)	F_y (N/mm²)
1	CR4-A-2	148	148	4.38	33.8	33.8	49.2	ok	8.76	223.5	139.2	139.24	262

f_{yd} (N/mm²)	f_{ck} (N/mm²)	f_{cd} (N/mm²)	E_c (N/mm²)	E_s (N/mm²)	A_c (mm²)	A_s (mm²)	$I_{c,y}$ (mm⁴)	$I_{s,y}$ (mm⁴)	$I_{c,z}$ (mm⁴)	$I_{s,z}$ (mm⁴)	$EI_{eff,y}$ (N-mm²)
262	25.4	25.4	31589.78	200000	19371.3	2467	3.1E+07	9E+06	3E+07	9E+06	2.3E+12

$EI_{eff,Z}$ (N-mm²)	$N_{pl,Rk}$ (kN)	$N_{cr,Y}$ (kN)	$N_{cr,Z}$ (kN)	λ_{cr}	N_c (kN)	N_s (kN)	$N_{pl,Rd}$ (kN)	δ	α	Φ	χ	$\chi N_{pl,Rd}$ (kN)	N_{test} (kN)	$N_{test}/\chi N_{pl,Rd}$
2.33E+12	1138	459447	459447	0.05	492	646.3	1138.3	0.57	0.21	0.49	1	1138.3	1153	1.013

Resistance to compression of Rectangular CFT columns(RCFT) according to AISC-2010

spec. No	B (mm)	H (mm)	t (mm)	B/t	H/t	compact λ_P	section type	r_i (mm)	kL (mm)	kL/H	B_c (mm)	H_c (mm)	F_y (N/mm²)	f_{ck} (N/mm²)
CR4-A-2	148	148	4.38	33.8	33.8	62.4	ok	8.8	223.5	1.5	139.2	139.2	262	25.4

F_c (N/mm²)	E_c (N/mm²)	E_s (N/mm²)	A_c (mm²)	A_s (mm²)	$I_{c,y}$ (mm⁴)	$I_{s,y}$ (mm⁴)	$I_{c,z}$ (mm⁴)	$I_{s,z}$ (mm⁴)	C_3	$EI_{eff,y}$ (N-mm²)	$EI_{eff,Z}$ (N-mm²)	$P_{e,y}$ (kN)	$P_{e,z}$ (kN)
21.6	27089.1	200000	19371.3	2466.8	3.1E+07	9E+06	3.1E+07	9E+06	0.83	2.43E+12	2.43E+12	480610.6	480610.6

P_c (kN)	P_s (kN)	P_{no} (kN)	$P_{no}/P_{e,y}$	$P_{no}/P_{e,z}$	$P_{n,y}$ (kN)	$P_{n,z}$ (kN)	P_n (kN)	P_n (kN)	P_{test} (kN)	P_{test}/P_n
418.2	646.3	1064.5	0.0022	0.0022	1063.5	1063.5	1063.5	1063.5	1153	1.08

Resistance of Circular CFT beam-columns(CCFT) according to EC-4

spec. No	D (mm)	t (mm)	D/t	local buckling check	local buckling limit	kL (mm)	kL/D	D_c (mm)	F_y (N/mm²)	f_{ck} (N/mm²)	f_{cd} (N/mm²)	f_{yd} (N/mm²)	A_c (mm²)	A_s (mm²)	I_c (mm⁴)
Icsc2-1	200	3	66.7	69.7	ok	2000	10	194	303.5	47	47	303.5	29559	1857	7E+07

I_a (mm⁴)	E_c (N/mm²)	E_s (N/mm²)	EI_{eff} (N-mm²)	N_{cr} (kN)	$N_{pl,Rk}$ (kN)	λ	η_{ao}	η_{co}	η_a	η_c	δ	α	ϕ	χ
9E+06	36688.6	206500	3.39E+12	8367	1953	0.48	1	0	1	0	0.29	0.21	0.65	0.93

at point A				at point D				at point B					at point C	
N_c (kN)	N_s (kN)	$N_{pl,Rd}$ (kN)	$\chi N_{pl,Rd}$ (kN)	W_c (mm³)	W_s (mm³)	$M_{at\text{-}D}$ (kNm)	$N_{pl\ at\text{-}D}$ (kN)	h_n (mm)	W_{cn} (mm³)	W_{sn} (mm³)	$M_{at\text{-}B}$ (kNm)	$N_{pl\ at\text{-}B}$ (kN)	$M_{at\text{-}c}$ (kNm)	$N_{pl\ at\text{-}C}$ (kN)
1389.3	563.5	1952.8	1815.0	1216897	116436	63.94	645.6	54.44	641957	98654	45.0	0	45.0	1291

N_{test} (kN)	M_{test} (kNm)	e (mm)	μ_{dy}	Linear Interpolation $N_{A\text{-}C}$ (kN)	$N_{C\text{-}D}$ (kN)	$N_{D\text{-}B}$ (kN)	$N_{C\text{-}D}$ (kN)	$N_{D\text{-}B}$ (kN)	N_{EC4} (kN)	N_{test}/N_{EC4}
1215	36.5	30	0.81	1391.0	0	0	0	0	1391.0	0.87

Resistance of Circular CFT beam-columns(CCFT) according to AISC-2010

spec. No	D	t	D/t	compact λ_P	check	r_i	kL	D_c	F_y	f_{ck}	F_c	E_c	E_s	A_c	A_s
	(mm)	(mm)				(mm)	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(N/mm^2)	(N/mm^2)	(N/mm^2)	(mm^2)	(mm^2)
lcsc2-1	200	3	66.7	102.1	ok	6	2000	194	303.5	47	45	36849	206500	29559	1857

I_c	I_s	C_3	EI_{eff}	P_e	P_c	P_s	P_{no}	P_{no}/P_e	P_n	P_n	λ
(mm^4)	(mm^4)		(N-mm^2)	(kN)	(kN)	(kN)	(kN)		(kN)	(kN)	
69530735	9009082	0.72	3.7E+12	9130.6	1319.8	563.5	1883.3	0.21	1727.6	1727.6	0.92

point A	at point D				at point B					at point C			
$P_{n\ at\ A}$	Z_c	Z_s	M_{at-D}	P_{at-D}	h_n	Z_{cn}	Z_{sn}	M_{at-B}	P_{at-B}	Z_{cn}	Z_{sn}	M_{at-c}	P_{at-c}
(kN)	(mm^3)	(mm^3)	(kNm)	(kN)	mm	(mm^3)	(mm^3)	(kNm)	(kN)	(mm^3)	(mm^3)	(kNm)	(kN)
1584.7	1216897	116436	62.51	605.3	53.6	658849.2	99176.8	45	0	658849	99177	45	1211

P_{test}	M_{test}	e	P_{A-C}	P_{C-D}	P_{D-B}	P_{C-D}	P_{D-B}	P_{AISC}	P_{test}/P_{AISC}
(kN)	(kNm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	
1215	36.5	30	1280	0	0	0	0	1280	0.95

Resistance of Rectangular CFT beam-columns(RCFT) according to EC-4

spec. No	B (mm)	H (mm)	t (mm)	B/t	H/t	λ	check	r_i (mm)	kL (mm)	B_c (mm)	H_c (mm)	F_y (N/mm²)	f_{yd} (N/mm²)	f_{ck} (N/mm²)	f_{cd} (N/mm²)	E_c (N/mm²)	E_s (N/mm²)
S1	120	120	4	30	30	35.829	ok	8	360	112	112	495	495	60	57	39100	206000

A_c (mm²)	A_s (mm²)	$I_{c,y}$ (mm⁴)	$I_{s,y}$ (mm⁴)	$I_{c,z}$ (mm⁴)	$I_{s,z}$ (mm⁴)	$EI_{eff,y}$ (N-mm²)	$EI_{eff,Z}$ (N-mm²)	$N_{pl,Rk}$ (kN)	$N_{cr,Y}$ (kN)	$N_{cr,Z}$ (kN)	λ_{cr}	δ	α	Φ	χ
12530	1815	1E+07	4E+06	1E+07	4E+06	1.2E+12	1.2E+12	1612.5	88803.1	88803.1	0.13	0.56	0.21	0.50	1

at point A				at point D						
N_c (kN)	N_s (kN)	$N_{pl,Rd}$ (kN)	$\chi N_{pl,Rd}$ (kN)	$W_{c,y}$ (mm³)	$W_{s,y}$ (mm³)	$W_{c,Z}$ (mm³)	$W_{S,Z}$ (mm³)	$M_{at\text{-}D,y}$ (kNm)	$M_{at\text{-}D,z}$ (kNm)	$N_{pl\ at\text{-}D}$ (kN)
714.2	898.3	1612.5	1612.5	351232	80768	351232	80768	50	50	357

at point B									at point C						
$h_{n,y}$ (mm)	$h_{n,z}$ (mm)	$W_{cn,y}$ (mm³)	$W_{sn,y}$ (mm³)	$W_{cn,Z}$ (mm³)	$W_{sn,Z}$ (mm³)	$M_{at\text{-}B,y}$ (kNm)	$M_{at\text{-}B,z}$ (kNm)	$N_{pl\ at\text{-}B}$ (kN)	$M_{at\text{-}c,y}$ (kNm)	$M_{at\text{-}c,Z}$ (kNm)	$N_{pl\ at\text{-}C}$ (kN)	N_{test} (kN)	$M_{test,y}$ (kNm)	μ_{dy}	e mm
25	25	281423	75782	281423	75782	46	46	0	46	46	714	1294	19.41	0.43	15

linear interpolation				
$N_{A\text{-}C}$ (kN)	$N_{C\text{-}D}$ (kN)	$N_{D\text{-}B}$ (kN)	$N_{EC4,Y}$ (kN)	N_{test}/N_{EC4}
1230	0	0	1230	1.05

Resistance of Rectangular CFT beam-columns(RCFT) according to AISC-2010

spec. No	B (mm)	H (mm)	t (mm)	B/t	H/t	compact,B λ_P	compact,H λ_P	limit check,B	limit check,H	r_i (mm)	kL (mm)	kL/H	B_c (mm)	H_c (mm)	F_y (N/mm²)
S1	120	120	4	30	30	46.1	61.2	ok	ok	8	360	3	112	112	495

f'_c (N/mm²)	F_c (N/mm²)	E_c (N/mm²)	E_s (N/mm²)	A_c (mm²)	A_s (mm²)	$I_{c,y}$ (mm⁴)	$I_{s,y}$ (mm⁴)	$I_{c,z}$ (mm⁴)	$I_{s,z}$ (mm⁴)	C_3	$EI_{eff,y}$ (N-mm²)	$EI_{eff,Z}$ (N-mm²)	$P_{e,y}$ (kN)	$P_{e,z}$ (kN)	P_c (kN)
60	51	41635	206000	12530	1815	1.3E+07	4E+06	1E+07	4E+06	0.85	1.32E+12	1.32E+12	100841	100841	639

P_s (kN)	P_{no} (kN)	$P_{no}/P_{e,y}$	$P_{no}/P_{e,z}$	$P_{n,y}$ (kN)	$P_{n,z}$ (kN)	P_n (kN)	$\Phi_c P_n$ (kN)	λ	point A $P_{n,A}$ (kN)	at point D $Z_{c,y}$ (mm³)	$Z_{s,y}$ (mm³)	$Z_{c,Z}$ (mm³)	$Z_{S,Z}$ (mm³)	$M_{at-D,y}$ (kNm)	$M_{at-D,z}$ (kNm)	P_{at-D} (kN)
898	1537	0.0152	0.0152	1527.6	1527.6	1527.6	1527.6	0.99	1528	351232	80768	351232	80768	49	44	318

at point B $h_{n,y}$ (mm)	$h_{n,z}$ (mm)	$Z_{cn,y}$ (mm³)	$Z_{sn,y}$ (mm³)	$Z_{cn,Z}$ (mm³)	$Z_{sn,Z}$ (mm³)	$M_{at-B,y}$ (kNm)	$M_{at-B,z}$ (kNm)	P_{at-B} (kN)	at point C $Z_{cn,y}$ (mm³)	$Z_{sn,y}$ (mm³)	$Z_{cn,Z}$ (mm³)	$Z_{sn,Z}$ (mm³)	$M_{at-c,y}$ (kNm)	$M_{at-c,Z}$ (kNm)	P_{at-C} (kN)
23	23	289700	76373	289700	76373	45	45	0	289700	76373	289700	76373	45	45	635

P_{test} (kN)	$M_{test,y}$ (kNm)	e_y mm	e_z mm	P_{A-C} (kN)	P_{C-D} (kN)	P_{D-B} (kN)	P_{AISC} (kN)	P_{test}/P_{AISC}
1294	19	15	0	1144	0	0	1144	1.13

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