

**STRENGTHENING OF CONCRETE-FILLED COLD
FORMED STAINLESS STEEL TUBULAR COLUMNS**

BY

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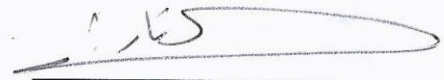


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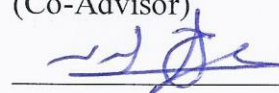
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To
My Beloved Parents, Wife, Brothers, Sisters
and my Children (Osamah, Mohammad and
Azhar)

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TABLE OF CONTENTS

ACKNOWLEDGMENTS.....	III
LIST OF TABLES	X
LIST OF FIGURES	XII
LIST OF ABBREVIATIONS.....	XVIII
ABSTRACT	XXV
ملخص الرسالة.....	XXVII
1 INTRODUCTION	1
1.1 Stainless Steel.....	1
1.2 Concrete-Filled Tubes	3
1.3 Need of Research	4
1.4 Objectives.....	5
1.5 Dissertation Outline.....	6
2 LITERATURE REVIEW.....	8
2.1 Structural Performance of CFSTCs.....	8
2.1.1 Structural Performance of CFCSTCs.....	8
2.1.2 Structural Performance of CFSSTCs.....	9
2.2 Strengthening of CFST.....	11
2.2.1 Strengthening Using CFRP	11
2.2.2 Strengthening Using Carbon Steel with Stainless Steel	14
3 MATERIAL PROPERTIES	18
3.1 Stainless Steel.....	18
3.2 Carbon Steel Material	24

3.3	Concrete Material	25
3.2.1	Concrete Compression Test	25
3.4	CFRP Material	28
4	BEHAVIOUR OF CFRP-WRAPPED HSSTCS UNDER AXIAL LOADING	29
4.1	Experimental Investigation	29
4.1.1	Test Specimens	30
4.1.2	Specimens Preparation.....	32
4.1.3	Instrumentation and Testing.....	33
4.2	Experimental Results and Discussion	34
4.2.1	Load-Displacement Responses	34
4.2.2	Failure Modes.....	38
4.2.3	Effect of CFRP on the Load Carrying Capacity.....	40
4.2.4	Effect of CFRP Strengthening On the Axial Shortening Capacity.....	40
4.3	Finite Element Modeling	41
4.3.1	Finite Element Types.....	41
4.3.2	Boundary Conditions and Load Application	42
4.3.3	Initial Imperfections.....	42
4.3.4	Material Modeling.....	43
4.3.5	Validation of the FE Models.....	45
4.4	FE Parametric Study	49
4.5	Analytical Study	49
4.5.1	Prediction of Load Carrying Capacity of Unstiffened HSSTCs.....	49
4.5.2	Proposing a Strength Model for CFRP-stiffened HSSTCs.....	52
4.6	Concluding Remarks.....	59
5	BEHAVIOUR OF CFRP-CONFINED CFSSTCS UNDER AXIAL LOADING	61

5.1	Experimental Investigation	61
5.1.1	Specimens Preparation.....	64
5.1.2	Instrumentation and Testing.....	65
5.2	Results and Discussion.....	67
5.2.1	Force-Deformation Response	67
5.2.2	Failure Modes	71
5.2.3	Effect of CFRP Strengthening On the Load Carrying Capacity	74
5.2.4	Effect of CFRP Strengthening On the Axial Shortening Capacity	75
5.3	Finite Element Modeling	76
5.3.1	Finite Element Types	76
5.3.2	Boundary Conditions and load Application.....	78
5.3.3	Initial Imperfections	78
5.3.4	Material Modeling.....	78
5.3.5	FE Interaction Modeling	81
5.3.6	Validation of the FE Models.....	82
5.3.7	Comparison of Stainless Steel and Carbon Steel Columns	85
5.4	FE Parametric Study	86
5.5	Analytical Study.....	87
5.5.1	Proposed Model	87
5.6	Concluding Remarks	97
6	BEHAVIOUR OF CFRP-CONFINED CFSSTCS UNDER ECCENTRIC LOADING	99
6.1	Experimental Investigation	100
6.2	Stub Column Specimens.....	102
6.2.1	Specimens Preparation.....	102
6.2.2	Instrumentation and Testing.....	103
6.3	Experimental Results and Discussion.....	105

6.3.1	Load Carrying Capacity Results.....	105
6.3.2	Load Eccentricity Effect.....	106
6.3.3	Force-displacement Response	107
6.3.4	Failure Modes.....	111
6.4	Finite Element Analysis.....	113
6.4.1	Finite Element Types.....	114
6.4.2	Boundary Conditions and Load Application	115
6.4.3	Initial Imperfections	115
6.4.4	Material Modeling	115
6.4.5	FE Interaction Modeling	115
6.4.6	Validation of FE Models.....	116
6.4.7	FE Parametric Study.....	121
6.5	Analytical Study	123
6.5.1	Assumptions of the Proposed Model.....	123
6.5.2	Modeling Of Stainless Steel Tube.....	123
6.5.3	Modeling of Concrete Core	124
6.5.4	Proposed P-M Interaction Model.....	127
6.5.5	Verification of the Proposed P-M Interaction Model	132
6.6	Concluding Remarks.....	134
7	BEHAVIOUR OF BARS-WELDED HSSTCS AND CFSSTCS UNDER AXIAL LOADING	135
7.1	Experimental Investigation	135
7.1.1	Preparation of Stub Column Specimens.....	137
7.1.2	Instrumentation and Testing.....	139
7.2	Experimental results and discussion	140
7.2.1	Unfilled Specimens	140
7.2.2	Filled Specimens	142

7.3	Finite Element Modeling	145
7.3.1	Initial Geometric Imperfections	146
7.3.2	Materials Modeling.....	147
7.3.3	FE Interaction Modeling	147
7.3.4	FE Model Validation	148
7.3.5	FE Parametric Study	151
7.4	Simplified Evaluation of the Ultimate Load Carrying Capacity	156
7.4.1	ACI-318 code	157
7.4.2	Eurocode 4.....	158
7.4.3	Continuous Strength Method	158
7.5	Results Verification of Simplified Methods.....	159
7.5.1	Unfilled Specimens	160
7.5.2	Filled Specimens	161
7.6	Concluding Remarks	163
8	CONCLUSIONS AND FUTURE RECOMMENDATIONS	164
8.1	Conclusions.....	164
8.2	Future Recommendations.....	167
	REFERENCES	168
	VITAE	174

LIST OF TABLES

Table 1-1: Composition and strength of austenitic, duplex and ferritic grades of stainless steel [2].....	2
Table 3-1. Chemical Compositions (% by weight) of Test Material	20
Table 3-2. Average Measured Tensile Material Properties used in the Ramberg-Osgood model.....	23
Table 4-1: Measured Geometric Dimensions and Test Variables of HSSTs	31
Table 4-2. Test results of load carrying capacity and ultimate end shortening	37
Table 4-3. CFRP modeling parameters	44
Table 4-4. Comparison of the test and FE load carrying capacity and ultimate deformation	46
Table 4-5. Comparison of the FE parametric study results and the proposed model results	54
Table 5-1: Measured Geometric Dimensions and Test Variables of CFSSTCs.....	63
Table 5-2. Test results of load carrying capacity and ultimate end shortening	71
Table 5-3. Comparison of the test and FE load carrying capacity and ultimate deformation	83
Table 5-4. Comparison of the FE parametric study results and the proposed model results	91
Table 6-1: Test specimens and variables.....	101
Table 6-2: Experimental key results	106
Table 6-3: comparison of experimental and numerical load carrying capacity results	117

Table 6-4: FE parametric study results	121
Table 7-1: Numbers and sizes of carbon steel bars for strengthening.....	137
Table 7-2: Experimental results of load carrying capacity and maximum axial displacement of unfilled specimens	141
Table 7-3: Experimental results of load carrying capacity of filled specimens....	143
Table 7-4: Experimental and FE results for different levels of imperfection amplitudes.....	148
Table 7-5: Maximum thickness (t) reduction of stainless steel tube and added area of carbon steel bars (A_b) to maintain the initial load carrying capacity	155
Table 7-6: Comparison of Numerical results with design models for unfilled specimens.....	159
Table 7-7: Comparison of Numerical results with design models for filled specimens.....	160
Table 7-8: Comparison of experimental results with design models for unfilled specimens.....	161
Table 7-9: Comparison of experimental results with design models for filled specimens.....	162

LIST OF FIGURES

Fig. 1-1: stainless steel and carbon steel stress–strain relationship	3
Fig. 1-2: Concrete-filled steel tube with different shapes	4
Fig. 1-3: Flow chart shows the dissertation objectives	6
Fig. 3-1: Experimental cold-rolled stainless steel Specimens	19
Fig. 3-2: Chemical composition analysis of stainless steel material	20
Fig. 3-3: Tensile coupon locations.....	21
Fig. 3-4: Cutting of stainless steel coupons by waterjet machine.....	22
Fig. 3-5: Tensile test of stainless steel coupons.....	22
Fig. 3-6: Stress-strain curves for CHSSTs material.....	23
Fig. 3-7: Tensile test setup of carbon steel bars.....	24
Fig. 3-8: Average stress-strain relationship of the carbon steel material	25
Fig. 3-9: Concrete compression strength (a) Preparation of concrete cylinders for compression test (b) test setup	26
Fig. 3-10: Concrete Split test.....	27
Fig. 3-11: Three points test setup.....	27
Fig. 4-1: Cross-section of CFRP-stiffened HSST column	30
Fig. 4-2: CFRP arrangements	32
Fig. 4-3. Stub columns preparation (a) stainless steel tubes (b) stainless steel tubes before CFRP wrapping (c) stainless steel tubes after CFRP wrapping.	33
Fig. 4-4. Axially loaded stub column test arrangement: (a) test Setup (b) schematic Diagram of Experimental Setup.	34

Fig. 4-5. Experimental force-end shortening of series G1 specimens.....	36
Fig. 4-6. Experimental force-end shortening of series G2 specimens.....	36
Fig. 4-7. Experimental force-end shortening of series G3 specimens.....	37
Fig. 4-8: Failure Mode of: (a) series G1 specimens (b) series G2 specimens (c) series G3 specimens.....	39
Fig.4-9. Finite Element modeling (a) stainless steel tube elements (b) CFRP elements.....	41
Fig. 4-10. Experimental and numerical force-end shortening for some selected specimens	47
Fig.4-11. General failure mode of (a) unwrapped specimens (b) wrapped by one CFRP ply specimens (c) wrapped by two or three CFRP plies specimens (d) partial wrap specimens.	48
Fig. 4-12: Stress-strain relationship of stainless steel material adopted in CSM....	51
Fig.4-13. Comparison of the load carrying capacity obtained from the FE parametric study and the proposed model.....	54
Fig. 5-1. CFRP-wrapped CFSSTCs scheme (a) Cross Section (b) CFRP rrangement.....	63
Fig. 5-2. Stub columns preparation (a) long cold-rolled stainless steel tubes (b) stub columns with the bottom welded plate (c) Stub columns concrete casting (d) applying a thin gypsum capping.	65
Fig. 5-3. Axially loaded stub column test arrangement: (a) test Setup (b) schematic Diagram of Experimental Setup.....	66
Fig. 5-4. Experimental force-end shortening of series G1 specimens.....	69

Fig. 5-5. Experimental force-end shortening of series G2 specimens.	69
Fig. 5-6. Experimental force-end shortening of series G3 specimens.	70
Fig. 5-7: Comparison of fully and partially wrapped specimens with D_s/t_s of 50.....	70
Fig. 5-8: Failure Mode of: (a) series G1 specimens (b) series G2 specimens (c) series G3 specimens.	74
Fig. 5-9: Failure mode of the partially wrapped specimen.....	74
Fig. 5-10. Finite Element modeling (a) rigid plates with reference points (b) stainless steel tube elements (c) concrete core elements (d) CFRP elements.	77
Fig.5-11: Effect of mesh size on the load carrying capacity of CFRP-wrapped CFSSTCs	78
Fig. 5-12: stress-strain model of concrete material (a) compression (b) tension ...	81
Fig. 5-13. Experimental and numerical force-end shortening for some selected specimens.....	84
Fig.5-14. General failure mode of (a) unwrapped specimens (b) wrapped specimens.....	85
Fig. 5-15: stainless steel and carbon steel stress–strain relationships.....	86
Fig. 5-16: comparison of force-displacement relationships of stainless steel and carbon steel	86
Fig.5-17. Confinement pressure provided by the CFRP and the Stainless steel. ...	89
Fig.5-18. Comparison of the load carrying capacity obtained from the FE parametric study and the proposed model.	90

Fig. 5-19. Variation of P_{FE} to P_n ratio as a function of (a) D/t ratio	
(b) $\sigma_{0.2} / f'_c$ ratio (c) t_f / t_s ratio.	96
Fig. 6-1. Cross Section of CFRP-wrapped CFSSTC.....	100
Fig. 6-2. Stub columns preparation (a) stub columns with the	
bottom-welded plate (b) Stub columns concrete casting (c) applying a thin	
gypsum capping (d) stub columns with the two thick plates.	103
Fig. 6-3. Eccentrically loaded stub column test arrangement: (a) test Setup (b)	
schematic Diagram of Experimental Setup.....	104
Fig. 6-4: Ultimate load carrying capacity versus load eccentricity ratio	107
Fig. 6-5: Experimental force-axial displacement of the specimens under axial	
loading	110
Fig. 6-6: Experimental force-lateral displacement of the specimens under eccentric	
loading	111
Fig. 6-7: Failure modes (a) axially loaded specimens (b) unwrapped eccentrically	
loaded specimens (c) wrapped eccentrically loaded specimens	113
Fig. 6-8: Finite Element modeling (a) rigid plates with eccentric reference points	
(b) stainless steel tube elements (c) concrete core elements (d) CFRP	
elements.	114
Fig. 6-9: Experimental and numerical force-axial displacement of the specimens	
under eccentric loading	118
Fig. 6-10: Experimental and numerical force-lateral displacement of the specimens	
under eccentric loading	119

Fig. 6-11: Failure modes of (a) the unwrapped axially loaded specimen (b) the wrapped axially loaded specimens (c) the unwrapped eccentrically loaded specimens (d) the wrapped eccentrically loaded specimens.	120
Fig. 6-12: Stress–strain model of FRP-confined concrete proposed by Lam and Teng [67]	126
Fig. 6-13: Stress and strain distributions over the cross section (a) column cross-section (b) strain distribution (c) confined concrete stress distribution (d) stainless steel stress distribution.....	128
Fig. 6-14: Representative simplified P-M interaction diagram	130
Fig. 6-15 : Comparison between the predicted and experimental results.	133
Fig. 6-16: Comparison between the predicted and FE results.	134
Fig. 7-1: Strengthened cross-sections by carbon steel bars (a) HSSTC (b) CFSSTC	136
Fig. 7-2: Stub column specimen preparation (a) stainless steel tube cutting (b) welding of carbon steel bars to stainless steel tube (c) stub column specimens with bottom thick plates (d) concrete casting	139
Fig. 7-3: Axially loaded stub column: (a) test Setup (b) schematic drawing for measurements	140
Fig. 7-4: Experimental axial load vs. Axial displacement of unfilled specimens	141
Fig. 7-5: Failure mode of unfilled specimens.....	142
Fig. 7-6: Experimental axial load vs. axial displacement of the filled specimens	143

Fig. 7-7: Failure mode of the filled specimens	145
Fig. 7-8: Finite Element modeling (a) stainless steel tube with longitudinal bars elements (c) concrete core elements (c) rigid plates with reference points	146
Fig. 7-9: Experimental and numerical axial load vs. axial displacement curves for some selected specimens	150
Fig. 7-10: General failure mode of (a) strengthened unfilled specimen (b) control unfilled specimen (c) strengthened filled specimen (d) control filled specimen.....	150
Fig. 7-11: Experimental and FE ultimate load carrying capacity results for unfilled specimens	151
Fig. 7-12: Experimental and FE ultimate load carrying capacity results for filled specimens	151
Fig. 7-13: Equivalent Carbon Steel Bars Configuration.....	152
Fig. 7-14: Axial load vs. axial displacement for different configurations of carbon steel bars.	153
Fig. 7-15: Area of carbon steel bars required to compensate for the thickness reduction of stainless steel tube.	154
Fig. 7-16: Effect of D/t ratio on the axial load vs. axial displacement response of (a) unfilled specimens (b) filled specimens.....	156

LIST OF ABBREVIATIONS

HSSTC	Hollow stainless steel tubular stub column
CFSTC	Concrete-filled steel tubular stub column
CFCSTC	Concrete-filled carbon steel tubular stub column
CFSSTC	Concrete-filled stainless steel tubular stub column
CHSST	Circular hollow stainless steel tube
CFRP	Carbon fiber reinforced polymer
FRP	Fiber reinforced polymer
COV	coefficient of variation
SD	Standard deviation
FEM	Finite element modeling
FEMs	Finite element models
FE	Finite element
DOF	Degree of freedom
CSM	Continues Strength Method
CDPM	Concrete Damaged Plasticity Model

ACI	American concrete Institute
ASTM	American Standards Of Testing And Materials
LVDT	Linear Variable Displacement Transducers
E_0	Initial Elastic Modulus Of Stainless Steel
$E_{0.2}$	Elastic Modulus At 0.2% Proof Stress,
ν	Poisson's Ratio
$\sigma_{0.2}$	0.2% Proof Stress
$\sigma_{1.0}$	1.0% Proof Stress
$n_0, n_{0.2,1}$	Strain-Hardening Exponents Determined From The Stress- Strain Curves Fitting
f_b	Yield Stress Of The Carbon Steel Bars
f'_c	Unconfined Compressive Strength
f_t	Concrete Tensile Strength
D	Outer Stainless Steel Diameter
t_s	Stainless Steel Tube Thickness
t_f	CFRP Jacket Thickness
P_{test}	Experimental Load Carrying Capacity

δ_{test}	Experimental Ultimate End-Shortening Cross-Ponding To P_{test}
P_{FE}	Finite Element Load Carrying Capacity
δ_{FE}	Finite Element End-Shortening Cross-Ponding To P_{FE}
σ_{Eng}	Engineering Stress
ϵ_{Eng}	Engineering Strain
σ_{true}	True Stress
ϵ_{true}	True Strain
L/D	Stainless Steel Tube Length To Diameter Ratio
E_{sh}	Strain Hardening Modulus
A_s	Cross-Section Area Of Stainless Steel Tube
σ_{LB}	Inelastic Local Buckling Stress
P_s	Stainless Steel Load Carrying Capacity
ϵ_{LB}	Inelastic Local Buckling Strain
ϵ_u	Stainless Steel Ultimate Strain
$\epsilon_{0.2}$	0.2% Proofing Strain
λ_c	Cross-Section Slenderness
σ_{cr}	Elastic Local Critical Stress

P_n	Predicted Ultimate Load Carrying Capacity
Ψ_h	A Dimensionless Parameter
σ_f	Tensile Strength Of Carbon Fiber
f'_{cc}	Ultimate Confined Compressive Strength
ϵ_{ccu}	Ultimate Confined Compressive Strain Corresponding To F_{cc}
E_{cc}	Confined Modulus Of Elasticity
P_{core}	Predicted Concrete Core Load Carrying Capacity
A_c	Cross-Section Of Concrete Core
Ψ	Confinement Parameter
f_l	The Lateral Confining Pressure
σ_θ	Hoop Stress In The Stainless Steel Tube
e	Load Eccentricity
e/D	Eccentricity Diameter Ratio
M_i	Primary Bending Moment
M_s	Secondary Bending Moment
δ_h	Lateral Displacement Measure At The Mid-Height
$f'_{cc,f}$	FRP-Confined Ultimate Concrete Strength

$f_{l,a}$	Actual Confining Pressure Of CFRP
$f'_{cc,t}$	Tube-Confined Concrete
ε_{cc}	Axial Compression Strain Of The FRP-Confined Concrete
ε'_t	Transition Strain
E_c	Initial Concrete Elastic Modulus
E_f	FRP Elastic Modulus
ε_{ef}	Effective Strain In The FRP Wrap At Failure
ε_{co}	Ultimate Unconfined Concrete Strain
r^*	Radius Measured To The Center Of The Stainless Steel Tube
d	Diameter Measured From Center-To-Center Of The Stainless Steel Tube
c	Depth Of The Neutral Axis
a	Effective Depth Of The Neutral Axis
K_1	Tube-Confining Pressure Parameter
$f_{l,steel}$	Confining Pressure Of Steel Tube
ε_{frp}	CFRP Tensile Strain At The Rupture
γ	Concrete Equivalent Rectangular Stress Parameter

A_{conc}	Concrete Area In Compressive Zone
r	Inner Radius
A_{sc}	Stainless Steel Areas Subjected To Compressive Stress
A_{st}	Stainless Steel Areas Subjected To Tensile Stress
R	Outer Radius
y_{conc}	Distance From Neutral Axis To The Centroid Of Concrete In Compressive Zone
y_{sc}	Distance From Neutral Axis To The Centroid Of Tube In Compressive Zone
y_{st}	Distance From Neutral Axis To The Centroid Of Tube In Tensile Zone
ϵ_{bot}	Strain At The Bottom Face
ϵ_{top}	Strain At The Top Face
C_{conc}	Confined Concrete Compressive Internal Force
C_{tube}	Stainless Steel Tube Compressive Internal Force
T_{tube}	Stainless Steel Tube Tensile Internal Force
SE	Strength Enhancement
$N_{u, ACI}$	Predicted ACI Strength

$N_{u, EC4}$	Predicted Eurocode Strength
$N_{u, CSM}$	Predicted CSM Strength
A_b	Cross-Section Area Of Carbon Steel Bars
Ω	Reduction Factor For CSM
η_s and η_c	Factors Proposed By Eurocode To Take In Consideration The Interaction Between The Steel Tube And Concrete Core

ABSTRACT

FULL NAME : [GALAL MOHAMMED TAHER AL-MEKHLAFI]

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Recently, stainless steel material has been used as a construction material due to its advantages over carbon steel which include its aesthetic appearance, high resistance to corrosion, ease of maintenance and high fire resistance. Concrete-filled stainless steel tubular stub columns (CFSSTCs) offer high ductility, stiffness, strength and full usage of construction materials. CFSSTCs can be utilized for the construction of bridge piers, multi-story buildings and other supporting members due to their advantages compared with ordinary reinforced concrete columns.

This study aims at investigation of two strengthening techniques for enhancing the ultimate load carrying capacity of hollow stainless steel tubular columns (HSSTCs) and CFSSTCs. One of these techniques was utilizing carbon fiber reinforced polymer (CFRP) jacketing to strengthen the HSSTCs and CFSSTCs that were already constructed and the other technique was directed to the HSSTCs and CFSSTCs newly designed by using longitudinal carbon steel bars welded to the

inner stainless steel surfaces. This research has three major components: an experimental work, numerical simulation and proposing design models.

For experimental work, forty-three stub circular stub columns were tested under monotonic compressive loading to evaluate the effectiveness of the CFRP jacketing and longitudinal carbon steel bars. The experimental stub column specimens were categorized into four groups regarding their structural behavior: CFRP-stiffened HSSTCs under axial load, CFRP-confined CFSSTCs under axial load, CFRP-confined CFSSTCS under eccentric load and carbon bars-welded HSSTCs and CFSSTCs under axial loading. Experimental results exhibited that using CFRP jacketing and longitudinal carbon bars as strengthening techniques were effective in terms of enhancing the load carrying capacity. Moreover, three dimensional finite element models (FEMs) were developed and validated against the experimental results using ABAQUS software. Thereafter, the validated FEMs were employed to perform extensive parametric studies; more than 300 FEMs were analyzed to obtain the load carrying capacity results. Finally, the experimental and FE results were utilized to propose design models to predict the load carrying capacity for the four groups considered in this research.

**DOCTOR OF PHILOSOPHY
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ملخص الرسالة

الاسم الكامل: جلال محمد طاهر المخلافي

عنوان الرسالة: تقوية الأعمدة الفولاذية المقاومة للصدأ و المشكله على البارد و المملوءه بالخرسانة

التخصص: هندسة مدنية

تاريخ الدرجة العلمية: [ديسمبر 2018]

في الآونة الأخيرة ، تم استخدام مادة الفولاذ المقاوم للصدأ كمادة بناء بسبب مزاياها مقارنة بالفولاذ الكربوني والتي تشمل مظهرها الجمالي ، ومقاومة عالية للتآكل ، وسهولة الصيانة ومقاومة عالية للحريق. أنابيب الفولاذ المقاوم للصدأ المملوءة بالخرسانة توفر ليونة عالية ، وصلابة ، قوة واستخدام كامل لمواد البناء. يمكن استخدام أنابيب الفولاذ المقاوم للصدأ المملوءة بالخرسانة لبناء اعمدة الجسور والمباني متعددة الطوابق وغيرها من الأعضاء الداعمة بسبب مزاياها مقارنة باعمدة الخرسانية المسلحة العادية.

تهدف هذه الدراسة الى التحقق من طريقتين تقنيتين تستخدمان لزيادة القدرة على التحمل ، إحدى هذتين التقنيتين استخدام قمصان من الياف الكربون المعززة بالبولمر لزيادة التحمل في أنابيب الصلب غير القابل للصدأ المفرغة وأنابيب الفولاذ المقاوم للصدأ المملوءة بالخرسانة و التي سبق انشاؤه و التقنية الاخرى موجهة لاستخدام قضبان الفولاذ الكربوني الملحومة الى السطح الداخلي لانابيب الفولاذ المفرغ و المملوءة. يحتوي هذا البحث على ثلاثة مكونات رئيسية: العمل التجريبي ، والمحاكاة العددية ، واقتراح نماذج التصميم

لغرض العمل التجريبي ، تم اختبار ثلاثة وأربعين عموداً قصيراً دائرياً تحت تحميل الضغط لتقييم فعالية قميص البوليمر المقوى بألياف الكربون وقضبان الفولاذ الكربوني الطولية. تم تصنيف عينات الاعمدة إلى أربع مجموعات تتعلق بسلوكه الانشائي: أنابيب الصلب غير القابل للصدأ المفرغة المقواة باستخدام قميص البوليمر المقواة بألياف الكربون تحت التحميل المحوري ، أنابيب الفولاذ المقاوم للصدأ المملوءة بالخرسانة المحصورة بواسطة قميص البوليمر المقواة بألياف الكربون تحت التحميل المحوري ، أنابيب مملوءة بالخرسانة من الصلب غير القابل للصدأ محصورة بالقميص البوليمرية المقواة بألياف الكربون تحت التحميل غير المركزي ، وأنابيب الصلب غير القابل للصدأ المفرغة وأنابيب الفولاذ المقاوم للصدأ المملوءة بالخرسانة المقواة بقضبان ملحومة طولية تحت التحميل المحوري. أظهرت النتائج التجريبية أن استخدام قمصان البوليمر المقوى بألياف الكربون والقضبان الطولية كطرق تقوية كانت فعالة من حيث زيادة القدرة على الحمل.

علاوة على ذلك ، تم عمل نماذج العناصر المحددة ثلاثية الأبعاد والتحقق من صحتها مقارنة بالنتائج التجريبية

بعد ذلك ، تم توظيف نماذج العناصر المحددة المحققة من صحتها لإجراء دراسات عوامل أخرى. تم تحليل .

أكثر من 300 من نماذج العناصر المحددة للحصول على نتائج قدرة التحمل. أخيرًا ، تم استخدام نتائج العناصر التجريبية والعديدية لاقتراح نماذج تصميم للتنبؤ بقدرة التحمل للمجموعات الأربع التي تم بحثها في هذا الرسالة.

درجة الدكتوراه
جامعة الملك فهد للبترول والمعادن
الظهران ، المملكة العربية السعودية

CHAPTER 1

INTRODUCTION

1.1 Stainless Steel

Stainless steel is a steel alloy with chromium and Nickel elements. Compared to conventional carbon steel, Stainless steel has very high corrosion resistance, ease of maintenance, aesthetic appearance and high fire resistance [1].

Over the last 20 years, the yearly consumption of stainless steel material has enlarged at a growth rate of 5%, exceeding the growth rate of other materials. The rate of increasing of stainless steel which is used for construction purposes has been even faster. In 2006, approximately four million tons of stainless steel were used in construction applications worldwide, 14 percent of the total quantity consumed. For example, in China more than 20% goes into construction[2].

The initial cost of stainless steel is higher than carbon steel however if we calculate the long term cost for both materials, they will be close since stainless steel does not need additional maintenance cost and corrosion coating cost.

Depending on their microstructure, stainless steel material is divided into five categories: austenitic, duplex, ferritic, martensitic and precipitation hardening. The duplex and austenitic categories are most extensively used in construction. Chromium is still the key alloying element which provides the material the ability

to struggle corrosion[3]. **Table 1-1** compares the mechanical properties and composition of the three widely groups of stainless steel: austenitic, duplex and ferritic stainless steels [2].

Table 1-1: Composition and strength of austenitic, duplex and ferritic grades of stainless steel [2]

Type of stainless steel	Grade		Composition (EN 10088)			0.2% proof strength (MPa)	Strain at failure (%)
	EN	AISI/AS TM	Cr	Ni	Mo		
Austenitic	1.4301	304	17.5-19.5	8-10.5	-	230	45
	1.4401	316	16.5-18.5	10-13	2-2.5	240	40
Duplex	1.4162	S32101	21.0	1.5	0.3	530	30
	1.4362	S32304	22.0-24.0	3.5-5.5	0.1-0.5	450	20
	1.446	S32205/S31803	21.0-23.0	4.5-6.5	2.5-3.5	500	20
Ferritic	1.4510	439	16.0-18.0	-	-	240	23
	1.4509	441	17.5-18.5	-	-	250	18
	1.4521	444	17.0-20.0	-	1.8-2.5	320	20

Stainless steel displays nonlinear stress–strain relationship with no defined yield point, unlike carbon steel. **Fig. 1-1** shows a comparison of stress-stain relationship between carbon steel and stainless steel.

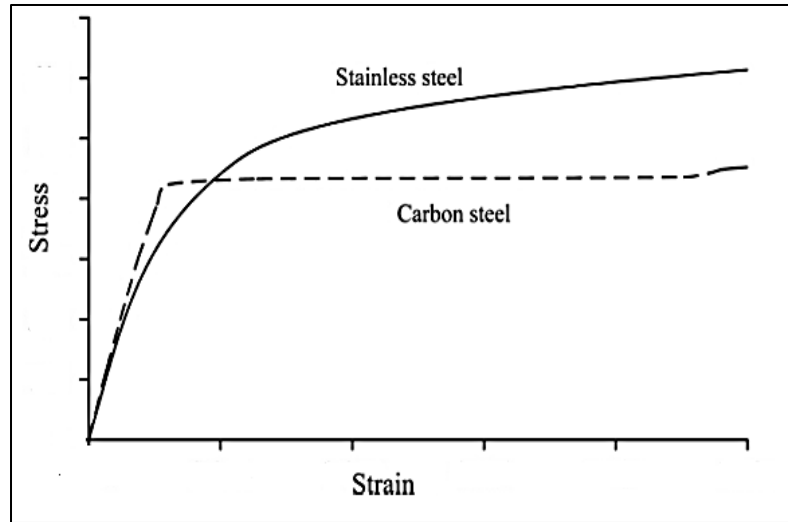


Fig. 1-1: stainless steel and carbon steel stress–strain relationship

1.2 Concrete-Filled Tubes

Concrete-filled steel tube is a composite structural member made of two materials: concrete and steel, as shown in **Fig. 1-2**. CFST columns offer high ductility, high strength, high stiffness and full use of construction materials. The steel tubes surrounding the concrete core are used as formwork which decreases construction cost [4]. Use of CFST columns reduces its size, which leads to significant cost savings. Reduction in size of columns is particularly helpful where floor space is at a premium, for example in office blocks or car parks. The concrete core in CFST columns resists the compressive force and prevents inward local buckling of the steel tube. The steel tube provides confinement to the concrete core and reinforces the concrete core to carry tensile and shear forces [5]. Therefore, CFST columns are extensively used for bridge piers, multi-story buildings and other supporting

structures. These CFST columns have significant advantages compared with conventional reinforced concrete as they can enhance space efficiency by using smaller size [6].

Recently, stainless steel tube structures have become popular as a result to aesthetic appearance, great corrosion resistance and ease of maintenance [4]. Use of CFSSTCS is relatively innovative and new due to providing the advantages outlined above and offering high durability associated with stainless steel [5].

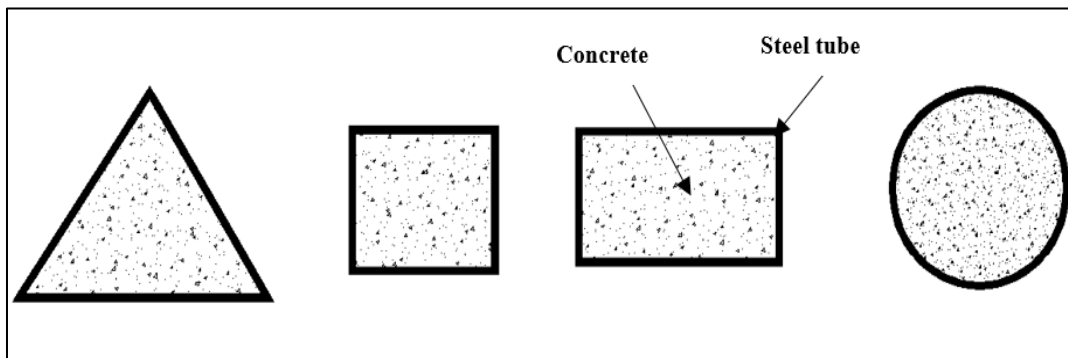


Fig. 1-2: Concrete-filled steel tube with different shapes

1.3 Need of Research

A review of literature clearly shows that investigations to use carbon fiber reinforced polymer (CFRP) material to strengthen unfilled or filled stainless steel tubular stub columns which were already constructed under axial loading or eccentric loading is not yet been investigated. Furthermore, using longitudinal welded carbon steel bars to strengthen new designed unfilled or filled stainless steel tubular stub columns under axial loading also is not yet been investigated. Thus, a comprehensive research

is required to investigate the structural performance of circular CFSSTCs confined with CFRP or stiffened by longitudinal bars.

1.4 Objectives of this Dissertation

The main objectives of this study are:

1. Undertake a planned experimental program to evaluate the structural performance of CFRP-wrapped HSSTCs under axial compressive loading, CFRP-confined CFSSTCs under axial loading, CFRP-confined CFSSTCs under eccentric loading and longitudinal bars-welded HSSTCs and CFSSTCs under axial loading.
2. Using commercially available software ABAQUS to perform three-dimensional FE models for the specimens considered in the experimental program.
3. Using the validated FE models to perform parametric studies and generating extensive data.
4. Using the results obtained from the experimental work and numerical work to propose simplified mathematical models to predict the ultimate load carrying capacity for CFRP-wrapped HSSTCs, CFRP-confined CFSSTCs and bars-welded HSSTCs and CFSSTCs.

Fig. 1-3 shows the dissertation objectives using a simple flow chart.

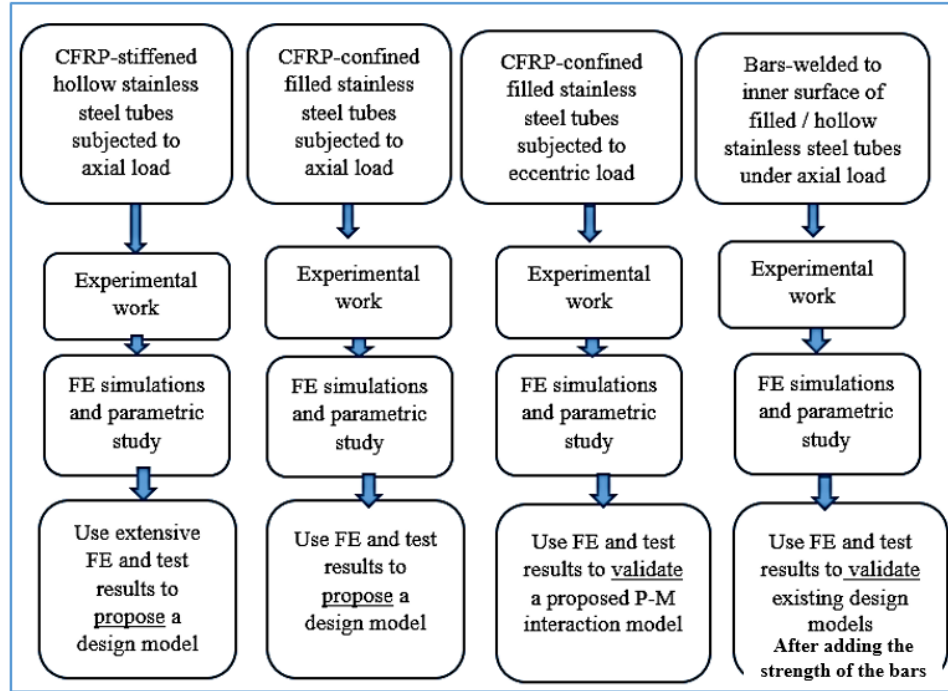


Fig. 1-3: Flow chart shows the dissertation objectives

1.5 Dissertation Outline

Chapter 2 provides a comprehensive literature review of that are relevant to this study.

Chapter 3 provides material properties of stainless steel, carbon steel and normal concrete materials.

Chapter 4 presents experimental, numerical and analytical investigations of the structural behavior of CFRP-wrapped HSSTCs. The experimental and numerical studies are completely reported. The results obtained from the tests and FE simulations were used to propose a simplified model to predict the load carrying capacity.

Chapter 5 and 6 provide experimental, numerical and analytical investigations of the structural performance of CFRP-wrapped CFSSTCs under axial and eccentric loadings, respectively. The experimental and numerical studies were completely reported. The results obtained from the test and FE simulations were used to propose a simplified model to predict the load carrying capacity of CFRP-wrapped CFSSTCs under axial and eccentric loading.

Chapter 7 presents experimental and numerical investigations of the structural behavior of bars-welded HSSTCs and CFSSTCs. The experimental and numerical studies were completely reported. The results obtained from the tests and FE simulations were used to evaluate available famous design codes.

Chapter 8 provides the conclusions and recommendations for future investigations.

CHAPTER 2

LITERATURE REVIEW

The literature review was conducted under the following subheadings:

1. Structural Performance of concrete-filled carbon steel tubular stub columns
CFCSTCs
2. Structural performance of CFSSTCs
3. Structural Performance of CFCSTCs confined with CFRP
4. Structural performance of CFSSTCs stiffened with longitudinal carbon bars

2.1 Structural Performance of Concrete-Filled Steel Tubes

2.1.1 Structural Performance of CFCSTCs

Schneider [7] conducted an experimental and numerical investigation on the performance of CFCSTCs under compression load. Steel tube shape and tube thickness were studied as test variables. From the experimental and numerical results, circular tubes provide substantial post-yield strength and ductility, unlike square and rectangular cross sections.

The ultimate capacity of rectangular CFCSTCs was experimentally investigated by Liu et al. [8]. From the results, the ultimate strength of rectangular CFCSTCs decreases with increase of cross-sectional aspect ratios.

Giakoumelis & Lam [9] investigated ultimate strength of circular CFCSTCs subjected to axial load. Confinement of concrete, carbon steel tube thickness and bond between concrete core and steel tube were examined. The results obtained were compared with three code standards. Giakoumelis & Lam [9] concluded that values calculated based on the code standards were lower than what obtained from experiments.

Faxing et al [10] conducted experimental investigation of round- ended CFCSTCs subjected to axial compression. Width–thickness ratio was evaluated as a test variable. Faxing et al [10] concluded that increase width–thickness ratio leads to a shorter elastic–plastic stage of load–strain curves.

Ding et al. [11] conducted both experimental and numerical study on the performance of square and circular CFCSTCs under local compression. The numerical results agreed well with experimental results.

2.1.2 Structural Performance of CFSSTCs

Young & Ellobody [4] conducted an experimental study of CFSSTCs with high strength stainless steel. The performance of the columns was explored by using several concrete compressive strengths ranged from 40 N/mm² to 80 N/mm². From the test results, some recommendations were proposed to use for CFSSTCs with high strength stainless steel.

Lam & Gardner [5] studied experimentally performance of circular and square CFSSTCs subjected to axial load. The experimental research was carried out by

using concrete strengths ranged from 30 N/mm² to 100 N/mm². Experimental results and other accessible results from the literature together were compared with available design methods for CFCSTC sections. It was found that using carbon composite columns guidance can be applied to stainless steel composite columns safely but it leads to over-conservative.

Dabaon et al. [12] conducted an experimental exploration on CFSSTCs. Experimental tests were carried out to inspect the influences of concrete strength and cross-section shape on the performance of stiffened CFCSTCs. The experimental results indicated that the design guidelines in the European and ASCE standards are highly conservative for rectangular and square stiffened CFSSTCs.

Uy et al. [13] performed a series of experimental tests on slender and CFSSTCs to investigate their behaviour under axial compressive force or under axial force plus bending moment. The results displayed that the performance of CFSSTCs was good and they have the potential to use widely as structural members.

Ellobody & Ghazy [1] developed an experimental investigation of plain and fiber reinforced (FR) CFSSTCs loaded axially and eccentrically. The results showed that FR CFSSTCs provide a significant increase in composite column ductility compared with plain CFSSTCs.

Tam et al., [14] conducted an experimental and numerical investigations on performance of recycled aggregate CFSSTCs to combine the advantages of both recycled aggregate concrete and stainless steel. Finite element analysis was developed to simulate the test results.

Tokgoz [15] conducted experimentally structural behavior of steel fiber and plain CFSSTCs subjected to both load cases axial compression and biaxial bending moment. Several parameters on the structural performance of plain and steel fiber CFSSTCs were investigated such as concrete strength, load eccentricity, cross-section, slenderness and steel fiber material were examined. The test results showed significant observations to describe the structural behaviour of steel fiber and plain CFSSTCs.

2.2 Strengthening of Concrete-Filled Steel Tubes

2.2.1 Strengthening Using CFRP

Investigations for using CFRP for strengthening of stainless steel tube columns against outward local buckling are not available in literature review. On the other hand, Limited investigations are existing in literature review to use CFRP for strengthening CFCSTC against outward local buckling.

FRP composites are commonly used in repair and rehabilitation of reinforced concrete structures. Many investigations have been conducted in this field. Using of FRP composites for repair and rehabilitation of steel structures is still at elementary level. Numerous types of FRP composites, with their preferable properties such as good resistance to corrosion and high strength to weight ratio, are considered as an ideal material for external retrofitting [16].

CFCSTC are commonly used as columns in several structural systems. The common mode of failure of such tube columns is outward local buckling close to column end.

Using of FRP jackets to delay local buckling has recently been suggested by limited experimental results to be used in both strengthening of existing structures and for new construction. Those limited test results were directed to strengthen carbon steel tubes[17].

Xiao et al. model [18] developed an experimentally study validate an innovative CFCSTC, named as confined CFCSTCs, for enhanced seismic design of CFCSTCs. Based on mechanics, the concept is directed to control the outward local buckling of the carbon steel tubes in the potential regions of a CFCSTC. As confirmed from the test results of axial compression loading and seismic loading, CFCSTC reinforced with FRP can lead to improve seismic performance. The local buckling and confinement mechanisms were inspected using a proposed analytical model.

Teng & Hu [19] reported test results of an investigation of CFRP full confinement of unfilled carbon steel tubes subjected to axial compression load. Numerical modelling for these experimental tests has been discussed. The numerical and experimental results indicated that FRP jackets are a very encouraging technique to retrofit carbon steel tubes against local buckling.

Hu et al. [17] reported results of an experimental investigation into the performance of CFRP-reinforced circular CFCSTC under axial load. Two parameters were studied which are stiffness of the FRP wrap and thickness of the steel tube. From the results, CFRP wrap can considerably delay and even totally suppress local buckling deformation in the steel tube.

Haedir & Zhao [20] conducted an experimental evaluation for externally bonded CFRP sheets for strengthening circular hollow carbon tube column subjected to axial compression. The experimental results showed that improvement of the axial capacity is possible by CFRP.

Sundarraja & Prabhu [21] investigated experimentally and analytically the structural enhancements of CFCSTC with normal strength concrete externally reinforced with CFRP sheets. CFRP were used as horizontal strips with some variables such as spacing of strips, number of layers and width of CFRP sheets. Tests were undertaken until the failure to completely understand the effect of CFRP on the compressive performance of square CFCSTC. Experimental results showed that CFRP strips offers external confinement pressure which can delay or even prevent the local buckling of steel tube and improves the load capacity as well.

Lu et al. [22] reported experimental results of the performance of CFCSTC confined by FRP. Eleven CFCSTCs were tested to examine the influences of steel tube thickness, FRP layer number and concrete compressive strength on their capacity. The experimental results showed that the FRP can effectively delay the outward local buckling of the steel tube.

Yu et al. [23] studied modelling and behavior of FRP-confined CFCSTCs under cyclic axial compression. An analytical model for confined concrete was proposed and it showed a good agreement with experiments results.

Punitha Kumar & Senthil [24] investigated the performance of CFRP-confined HSCT under both axial cyclic loading and axial static loading. Test results showed

that the axial capacity of HSCT was improved up to 39% in case of axial static loading and 41% in case of axial cyclic loading as a result to use CFRP.

Alam et al. [25] developed a finite element model for CFCSTC strengthened with CFRP to investigate CFRP bond length under transverse impact loading. The finite element model was verified by comparing with results from literature.

2.2.2 Using Carbon Steel with Stainless Steel

The initial high cost of stainless steel material still preventing the wide spreading. Now, the challenge of how we can utilize the excellent advantages of the stainless steel tubes either filled or unfilled at the same time with a rational cost. Using stainless steel elements together with carbon steel elements can be used to mitigate the high initial cost and utilize the desirable advantages of stainless steel material. For the literature, use of stainless steel and carbon steel elements together in the columns may take one of these ways: as bimetallic tubes, a carbon steel tube inside a stainless steel tube or by using longitudinal stiffeners welded to the inner stainless steel tube surface.

Using bimetallic tubes made of exterior stainless steel tube to be durable and internal tube made of carbon steel, where the internal tube is concrete-filled. This new system was investigated experimentally and analytically by Ye et al. [26,27] and investigated numerically was performed by Patel et al.[28]. These studies presented the bimetallic concrete-filled system as a solution to mitigate the high initial stainless steel cost. The behavior of bimetallic concrete filled columns exhibited, based on these investigations, a ductile behavior. Ye et al. [27] proposed a simplified rules to

predict the load carrying capacity of such circular bimetallic concrete-filled tubular columns. Because the square tubular columns obviously show a different behavior than the circular ones under axial compressive loading, Ye et al.[29] conducted an experimental and numerical investigation to study the square bimetallic concrete-filled columns as well as Ye et al.[29] proposed a simplified calculation to predict the load carrying capacity of these square bimetallic columns.

Another system, which is combines the both stainless steel and carbon steel filled-tubes together in one tubular column was introduced in literature as a proposed form to obtain the higher strength capacity, lower cost and higher corrosion resistance. This composite system made of concrete-filled carbon steel placed inside concrete-filled stainless steel tube. This filled tube-in filled tube composite columns were proposed by Chang et al.[30]. Chang et al. [30] conducted experimental and numerical investigation for such composite columns under axial compressive loading. Hassanein et al.[31] carried out a numerical investigation for these composite columns.

Use of longitudinal stiffeners welded to the inner surface of the steel tubes were investigated as presented in the following studies. Tao et al [32,33] conducted experimental work of square CFCSTCs to evaluate effect of internal stiffeners on the structural behavior. Five experimental parameters were studied which are stiffener height, using saw-shaped stiffeners, stiffener number, welding anchor bars, and adding fibers to concrete. The specimens were tested under axial compression to the failure stage. From the experimental results, internal stiffeners delayed

effectively the local buckling of square steel tubes leading to increase the ultimate strength.

Tao et al. [34] investigated experimentally square CFCSTCs reinforced by outer and inner longitudinal stiffeners. The composite stub columns were subjected to concentric axial compression. The experimental variables were stiffener rigidity and height to thickness ratio. It was concluded that ultimate strength of square CFCSTC was developed due to delaying the local buckling, however the ductility was not increased. The experimental results showed that the outer and inner stiffened samples behaviour almost is the same.

Dabaon et al.[35] performed a comparative investigation experimentally between unstiffened and inner stiffened stainless steel hollow stub tubes under concentric loading. Rectangular and square sections with constant thickness were used to evaluate the effect of cross section shape. Experimental finding showed that the ultimate strength capacity of stiffened stainless steel hollow stub tubes was developed compared with unstiffened ones. However, this development in the ultimate capacity is associated with a reasonably slight increase in the cross-section area.

Wang et al.[36] proposed several innovative types of inner stiffeners put for square CFCSTCs. Experimental, numerical and analytical investigations were performed. It was clear that local buckling of stiffened samples was delayed therefore considerable ultimate strength was achieved. Reasonably, the numerical model was

verified with experimental results. A simple analytical model for stiffened specimens was suggested and reasonably confirmed.

Huang et al. [37] carried out experimental and numerical investigation to evaluate stiffened square CFCSTCs with the width/thickness ratio between 40 up to 150 under axial loading. An effective stiffening scheme was proposed to enhance the mechanical properties. The investigation parameters were width-to-thickness ratio and stiffening arrangements. From the results, suggested stiffening scheme can improve the ultimate strength and ductility. Moreover, as the spacing between longitudinal stiffeners decreases, the improvement of ductility increases.

Cai and He [38] proposed a method to estimate ultimate capacity of stiffened square CFCSTCs under axial loading. The proposed method was verified by experimental results conducted by Cai and He [38] and experimental data from literature review. The results showed that the diameter and spacing of binding bars influence considerably on the ultimate capacity and concrete confinement.

From the above-mentioned investigations, the welded longitudinal stiffeners were used in non-circular filled steel tubes to increase the concrete confinement. The axial compressive behavior of carbon steel longitudinal stiffeners welded to the inner surfaces of circular stainless steel tubular columns has not yet been investigated.

CHAPTER 3

MATERIAL PROPERTIES

This chapter presents the testing of material properties of all materials used in this research. These tests include chemical composition analysis of stainless steel material, stainless steel tensile test, carbon steel tensile test, concrete compressive strength test, and indirect concrete tensile tests.

3.1 Stainless Steel

Long cold-formed Stainless steel pipes with four different diameters and thicknesses have been cut into small specimens to represent short columns as shown in **Fig. 3-1**. The dimensions of the tested specimens were designed to meet two requirements. First, these specimens must be short enough to avoid occurring of flexural buckling. The second requirement that the strength of the specimens should be less than the capacity of the hydraulic jack available in the laboratory. **Fig. 3-1** shows the specimens which were tested to achieve the objectives of this study. In the following chapters, more details are available.



Fig. 3-1: Experimental cold-rolled stainless steel Specimens

Experiments were conducted on four different circular hollow stainless steel (CHSS) cross-sectional sizes: CHSS 101×2 , CHSS 101×1.5 , CHSS 114.3×3.05 and CHSS 141.3×3.4 , where the first number refers to the tube's outer diameter and the second number represents the tube thickness.

3.2.1 Chemical Composition Analysis

Like carbon steel material, stainless steel material has various grades due to variations in its chemical composition as well as heat treatment. Five main groups are available to classify stainless steel material regarding its metallurgical structure that are namely austenitic, ferritic, duplex, martensitic and precipitation hardening. In this study, a chemical composition analysis was carried out in a material science laboratory to identify the stainless steel material used in test specimens. The chemical composition analysis was conducted on three small samples taken from

the CHSSTs, as shown in **Fig. 3-2**. From the chemical analysis of the test specimens shown in Error! Reference source not found., the stainless steel material used in this study was austenitic stainless steel Grade 304.

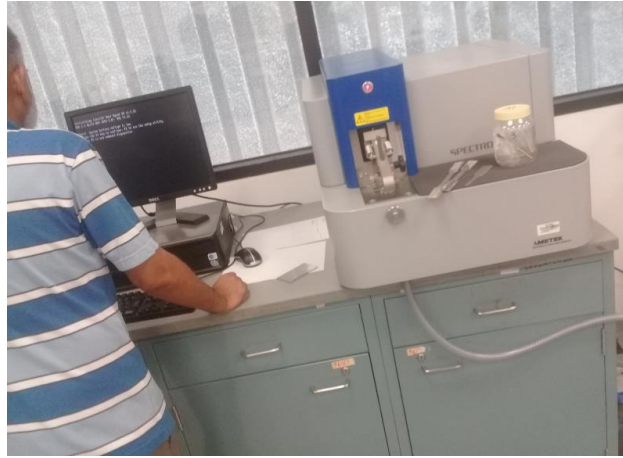


Fig. 3-2: Chemical composition analysis of stainless steel material

Table 3-1. Chemical Compositions (% by weight) of Test Material

specimen	Fe (%)	Cr (%)	Ni (%)	C (%)	Si (%)	Mn (%)	Nb (%)	P (%)	S (%)	Mo (%)
1	72	18.9	7.8	0.045	0.284	0.0048	<0.0040	<0.0005	<0.0005	0.115
2	72	18.5	7.9	0.047	0.531	0.004	<0.0040	<0.0005	<0.0005	0.101
3	70	18.7	7.7	0.021	0.445	1.81	<0.0040	<0.0005	<0.0005	0.328

3.2.2 Mechanical Properties

To determine the material stress-strain relationship of the tested stainless steel material, tensile coupon experiments were conducted. Two longitudinal coupons for each cross-section were extracted at 90° from the weld location to obtain the average bending residual stresses formed due to the cold-rolling process, as illustrated in **Fig.**

3-3. The stainless steel tensile coupons were cut from the specimens using the waterjet machine in a local factory as shown in **Fig. 3-4**. Both ends of the tensile coupon were flattened in order to be gripped by the test machine. For the four cross-sections, eight tensile coupons were tested according to the requirements of ASTM E8M [39], as shown in **Fig. 3-5**. The obtained stress-strain curves of the tested tensile coupons are shown in **Fig. 3-6**, where the stainless steel material displays a curved stress-strain relationship with no specific yield stress point. Commonly, the stress-strain relationship of stainless steel can be represented by the Ramberg-Osgood model [40] modified by Hill [41] as illustrated in **Table 3-2**, where: E_0 , $E_{0.2}$, ν , $\sigma_{0.2}$ and $\sigma_{1.0}$ are, respectively, initial elastic modulus, elastic modulus at 0.2% proof stress, Poisson's ratio, 0.2% proof stress and 1.0% proof stress. The coefficients (n_0) and ($n_{0.2,1}$) are strain-hardening exponents determined from the stress-strain curves fitting.

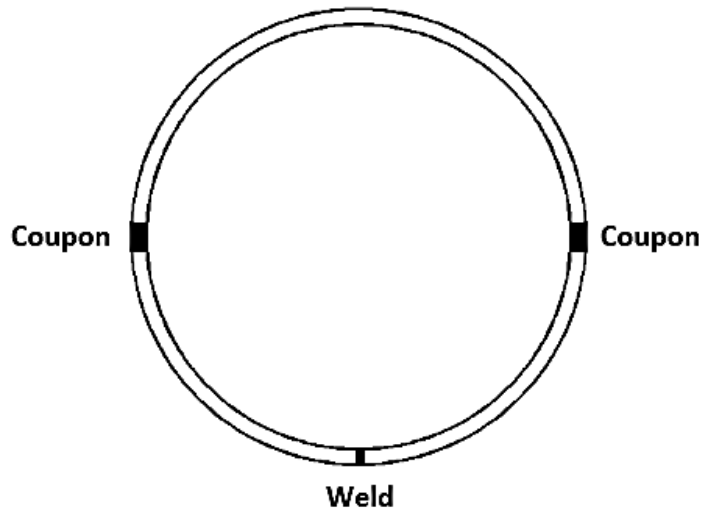


Fig. 3-3:Tensile coupon locations.



Fig. 3-4: Cutting of stainless steel coupons by waterjet machine



Fig. 3-5: Tensile test of stainless steel coupons

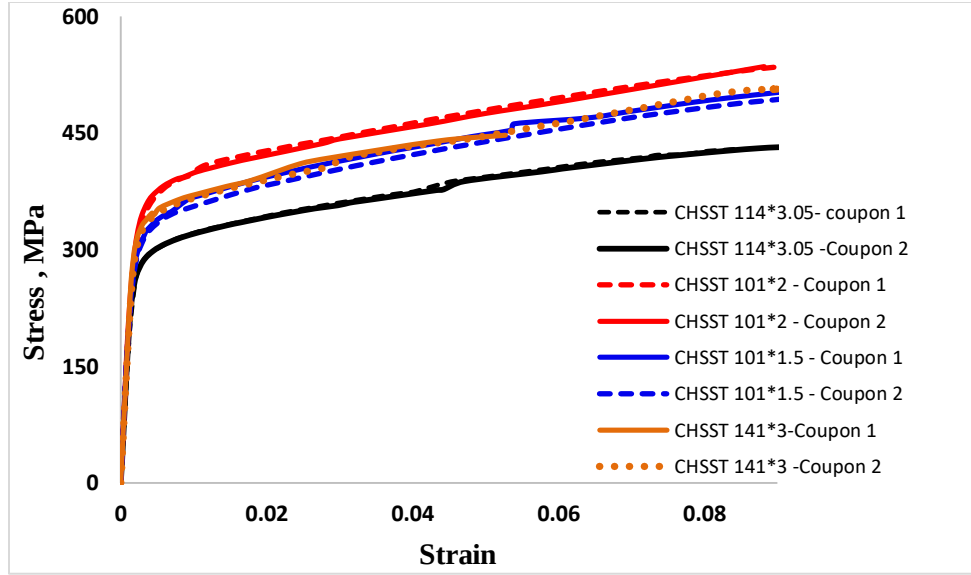


Fig. 3-6: Stress-strain curves for CHSSTs material.

Table 3-2. Average Measured Tensile Material Properties used in the Ramberg-Osgood model

Specimen	E0 (GPa)	ν	E0.2 (MPa)	σ 0.2 (MPa)	σ 1.0 (MPa)	Ramberge– Osgood coefficients	
						n_0	$n_{0.2,1}$
CHSST 101×2	208	0.33	17747	357	401	9.21	1.89
CHSST 101×1.5	204	0.30	16285	322	365	9.09	1.69
CHSST 114.3×3.05	189	0.33	13611	293	332	10	1.91
CHSST 141.3×3.4	192	0.3	15584	339	373.3	10	1.62

3.2 Carbon Steel Material

Mechanical properties of carbon steel were obtained by testing the longitudinal stiffeners using a Universal Testing Machine according to the ASTM-A615 requirements as shown in **Fig. 3-7**. For each cross section size, three specimens were tested. Two strain gauges were affixed to the specimens orthogonally to measure the longitudinal and lateral strains. The average elastic properties and yielding strength were 200 GPa, 0.3, 348 for the elastic modulus, Poisson's ratio and yielding strength of the carbon steel bars (f_b), respectively. **Fig. 3-8** shows the average stress-strain relationship of the carbon steel material obtained from the tensile test.

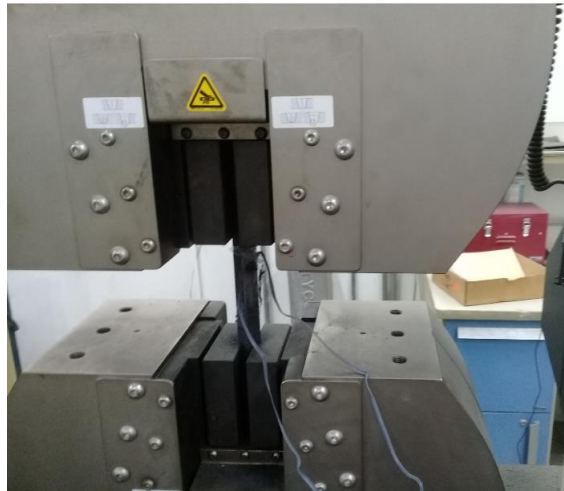


Fig. 3-7: Tensile test setup of carbon steel bars

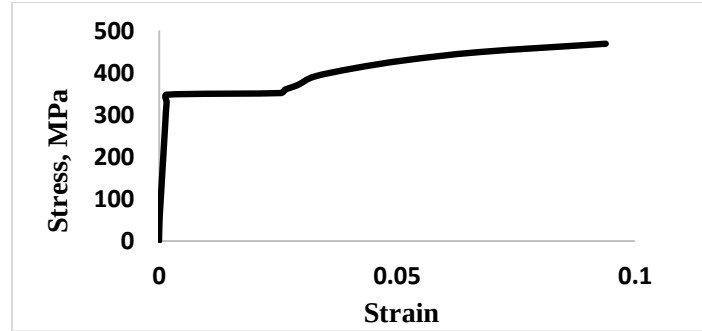


Fig. 3-8: Average stress-strain relationship of the carbon steel material

3.3 Concrete Material

The concrete material was cast by using local commercial constituents that were available to produce normal concrete with typical mixing and curing, and the mix ratios was 1:2.1:3.1 by weight. After several initial trials, one concrete mix was chosen to obtain the appropriate strength using ordinary Portland cement.

3.2.1 Concrete Compression Test

The compressive strengths (f_c') of concrete were determined by testing cylindrical specimens in accordance with ASTM C39 [42] with a dimension of 75×150 mm, as shown in **Fig. 3-9** . The average of six cylinders tested for each batch was taken as the applicable value of f_c' for that batch. The average of f_c' at 28-day was determined as 43.6 MPa with standard deviation (SD) and coefficient of variation (COV), respectively, which were 1.14 and 0.026.

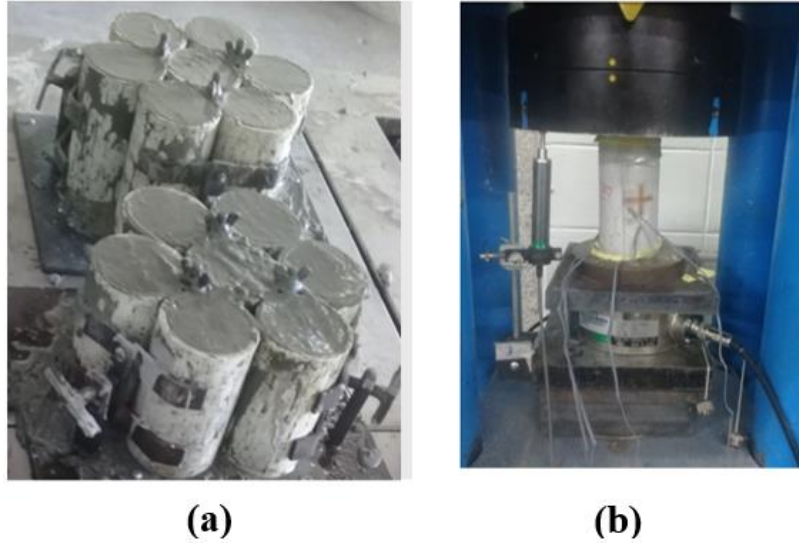


Fig. 3-9: Concrete compression strength (a) Preparation of concrete cylinders for compression test (b) test setup

3.3.1 Split Test (Indirect Tensile Test)

To evaluate the concrete tensile strength, spit test was conducted as an indirect tensile test. The tensile test was conducted according to ASTM C496 standards. Six 150×300 cylinders have been tested to find the splitting strength, as shown in **Fig. 3-10**. The average of the splitting strength was determined as 2.8 MPa with standard deviation (SD) and coefficient of variation (COV), respectively, which were 0.42 and 0.15.



Fig. 3-10: Concrete Split test

3.3.2 Three Points Bend Test

Three points test was conducted to determine concrete flexural strength. For this aim, six prismatic samples with dimensions shown in **Fig. 3-11** according to ASTM C293. The average of the flexural strength was determined as 6.7 MPa with standard deviation (SD) and coefficient of variation (COV), respectively, which were 0.31 and 0.05.

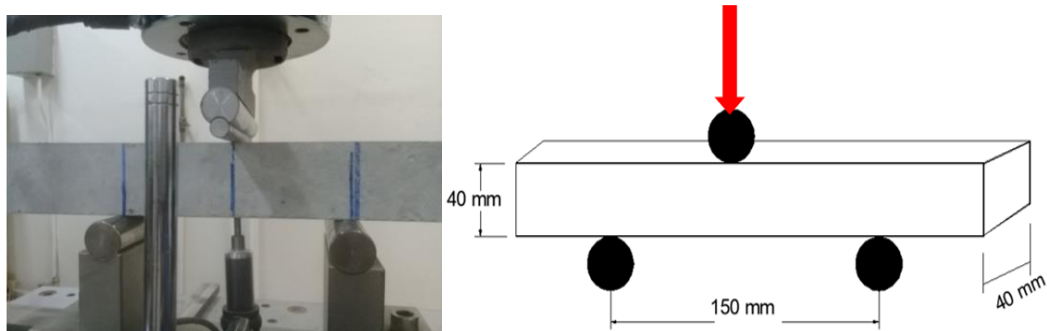


Fig. 3-11: Three points test setup

3.4 CFRP Material

The CFRP is made of two materials: carbon fiber and epoxy adhesive. Carbon fiber and its epoxy were brought from well-known SIKA Company. The mechanical properties of unidirectional CFRP material with a thickness of 0.29 mm per ply were taken from the manufacturer's manual. The modulus of elasticity and ultimate tensile strength in the fiber direction are, respectively, 220 GPa and 3000 MPa.

CHAPTER 4

BEHAVIOUR OF CFRP-WRAPPED HSSTCs UNDER AXIAL LOADING

This chapter presents experimental work, finite element simulation, parametric study and proposing a simplified design formula for CFRP-stiffened hollow stainless steel tubular stub columns under axial loading. Conclusions are given at the end of this chapter.

4.1 Experimental Investigation

The experimental work was conducted to study the structural performance of CFRP-stiffened circular HSSTCs under axial compression loading by using two cross-section sizes. The experimental work involves different tests, such as stainless steel material tensile coupon, concrete uniaxial compressive, and eleven stub column tests under axial loading. The preparation of the experimental specimens and test setup used for the experimental work is described and reported in the following sections.

4.1.1 Test Specimens

The experimental program was designed and performed to evaluate the structural performance of CFRP-wrapped HSSTCs under axial monotonic compressive loading. **Fig. 4-1** shows a cross-section for CFRP-stiffened HSSTCs. Experiments were conducted on two different circular hollow stainless steel (CHSS) cross-sectional sizes: CHSS 101×2 and CHSS 114×3 , where the first number refers to the tube's outer diameter and the second number represents the tube thickness. All stub columns were 290 mm in length for two reasons: first to ensure that the specimens are suitably short to avoid overall flexural buckling, while the second reason relates to the available height in the hydraulic testing machine. The test specimens and parameters are summarized in **Table 4-1**. The CFRP arrangements were full and partial arrangements. In the partially wrapped specimens, three configurations were used for the CFRP arrangement, as shown in Error! Reference source not found.

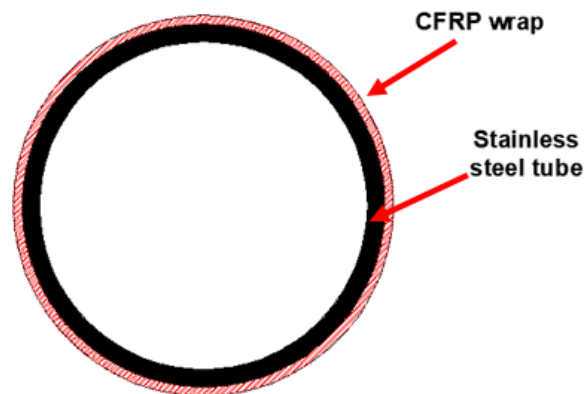


Fig. 4-1: Cross-section of CFRP-stiffened HSST column

Table 4-1: Measured Geometric Dimensions and Test Variables of HSSTs

Series	Specimen label	Stainless Steel Tube			CFRP		
		D (mm)	t_s (mm)	D/t_s	No. of plies	t_f (mm)	arrangement
G1	H0-37	114.3	3.05	37.5	0	0	-
	H1-37	114.3	3.05	37.5	1	0.29	Full wrapping
	H2-37	114.3	3.05	37.5	2	0.58	Full wrapping
	H3-37	114.3	3.05	37.5	3	0.87	Full wrapping
G2	H0-50	101	2	50.1	0	0	-
	H1-50	101	2	50.1	1	0.29	Full wrapping
	H2-50	101	2	50.1	2	0.58	Full wrapping
	H3-50	101	2	50.1	3	0.87	Full wrapping
G3	H2-50-P1	101	2	50.1	2	0.58	Partial Wrapping
	H2-50-P2	101	2	50.1	2	0.58	Partial Wrapping
	H2-50-P3	101	2	50.1	2	0.58	Partial Wrapping

Each specimen was given a distinctive label starting with the letter H followed by a number referring to the number of CFRP plies. The letter H stands for the HSSTCs. The second number represents the D/t_s ratio. For the partial wrapping, P letter was added to mean partial wrapping followed by a number that refers to the CFRP configuration, as shown in **Fig. 4-2**. For example, the label H2-50-P3 refers to a HSSTC that has a D/t_s ratio of 50, wrapped partially by two CFRP plies and the partial arrangement with configuration 3.

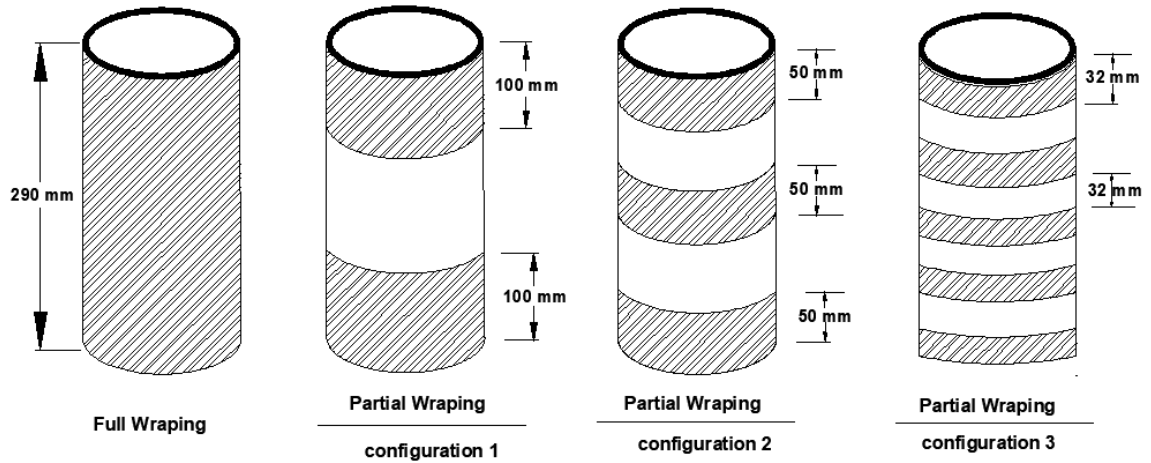


Fig. 4-2.CFRP arrangements

4.1.2 Specimens Preparation

Eleven stainless steel circular tubes with the dimensions given in **Table 4-1** were cut from three long cold-rolled tubes as shown in **Fig. 4-3 a**. Subsequently, two thick steel plates were welded to both ends of the tubes as shown in **Fig. 4-3b**. Subsequently, the outer surface of the stainless steel tubes was cleaned to remove any dirt and dust. To increase adhesion with CFRP, the outer surface of the stainless steel tubes was roughened. A continuous carbon fiber sheet 300 mm in width was wrapped with a sufficient amount of glue around the stainless steel tubes with the required number of layers, as shown in **Fig. 4-3c**. An appropriate overlapping length was added to satisfy circumferential continuity.

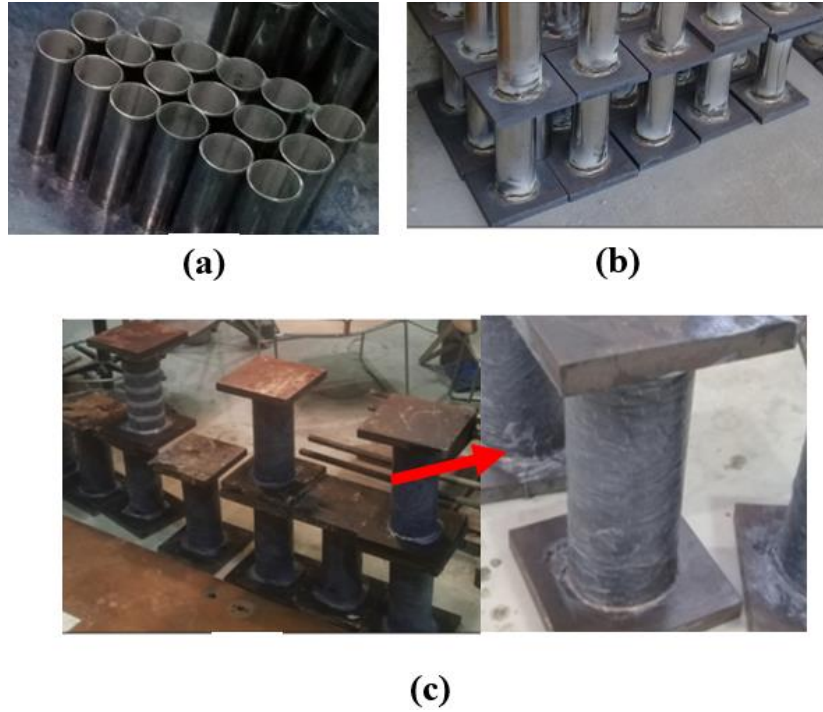


Fig. 4-3. Stub columns preparation (a) stainless steel tubes (b) stainless steel tubes before CFRP wrapping (c) stainless steel tubes after CFRP wrapping.

4.1.3 Instrumentation and Testing

Eleven HSSTC tests were conducted, two without a CFRP wrap were employed as control specimens. For each specimen, an axial compressive test with monotonic loading was performed to determine the load-deformation response. **Fig. 4-4** shows the test setup, comprised of four linear variable displacement transducers (LVDTs) which were located vertically to measure the average end shortening. Two strain gauges were glued longitudinally to all the specimens at mid-height and two strain gauges were pasted horizontally at mid-height. For the wrapped specimens, strain gauges were affixed to the outer surface of the CFRP composite. A MATEST 3000 kN hydraulic testing machine was used to apply an axial compressive load with a

loading rate of 1 kN/s. The LVDTs, strain gauges and load cell readings were connected to the data logger.

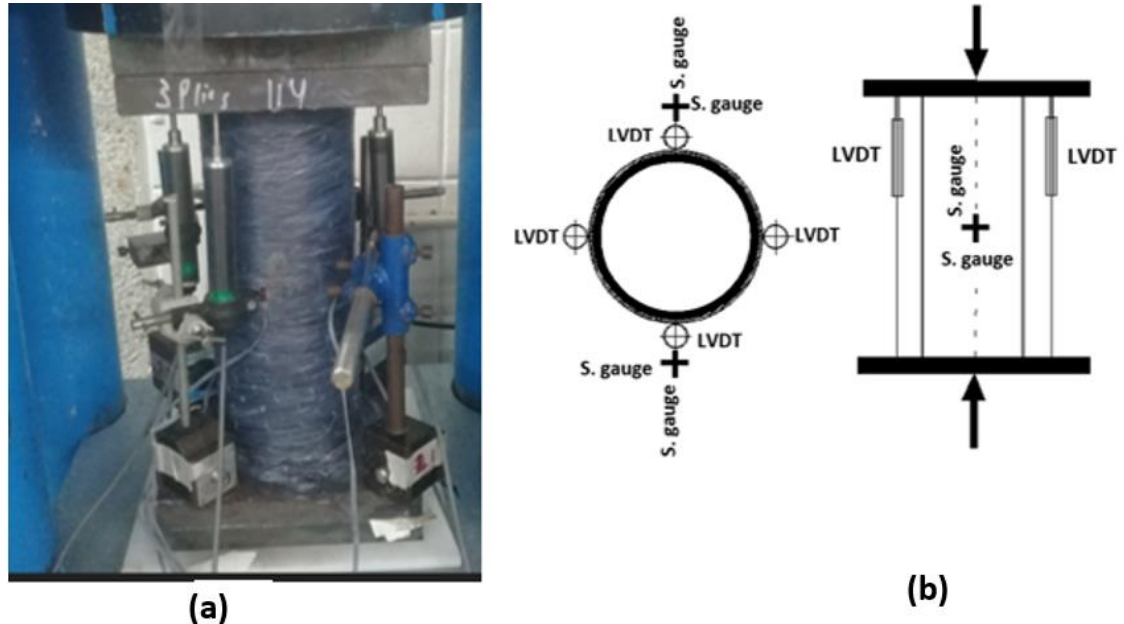


Fig. 4-4. Axially loaded stub column test arrangement: (a) test Setup (b) schematic Diagram of Experimental Setup.

4.2 Experimental Results and Discussion

4.2.1 Load-Displacement Responses

The axial load-end shortening curves for all of the stub column specimens are displayed in **Figs. 4-5, 4-6 and 4-7**, where the end shortening was taken as the average value of the four LVDTs. Vertical strain gauge readings were employed to correct the force-end shortening curves in the initial loading stage to obtain true end shortening values. At the initial loading stage, the effect of end platen deformation leads to a slightly shift in the end shortening values. The vertical mid-height stain gauge readings were multiplied by the specimen length to obtain the average end

shortening. The force-end shortening curves shown in **Figs. 4-5 and 4-6** represent the true force-end shortening after the correction.

By comparing the results of the CFRP-fully wrapped HSSTCs with the unwrapped ones, it is clear that the CFRP wraps provide a good improvement in terms of load carrying capacity, as well as in the axial shortening capacity as shown in **Figs. 4-5 and 4-6**. The load carrying capacity and axial shortening capacity of the stub specimens are summarized in **Table 4-2** as P_{test} and δ_{test} , respectively.

The axial load-end shortening curves of the partially wrapped specimens (H2-50-P1, H2-50-P2 and H2-50-P3) and unwrapped specimen (H0-50) are displayed in **Fig. 4-7**. It is clear that partial wrapping is not effective and does not contribute to improving the load carrying capacity. Furthermore, the partial wrapping configurations is not effective. To explain this behavior, the role of CFRP wrapping is to increase the stainless steel stiffness in the hoop direction and this provides more stability against the local buckling. The axial compressive loading provides a uniform stress distribution along the stainless steel tube height except at the end supports where these supports provide stresses in the other directions. Therefore, the resultant of the stresses at the ends becomes a little larger and this is likely to force the failure to occur at tube end. In the partial wrapping, any region not covered by CFRP is likely to be buckled.

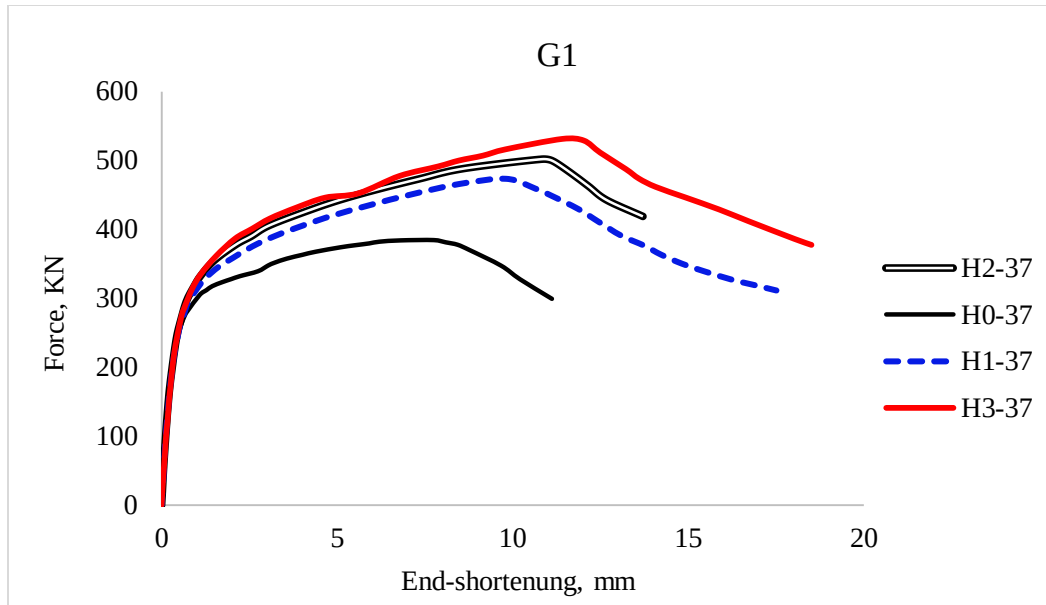


Fig. 4-5. Experimental force-end shortening of series G1 specimens.

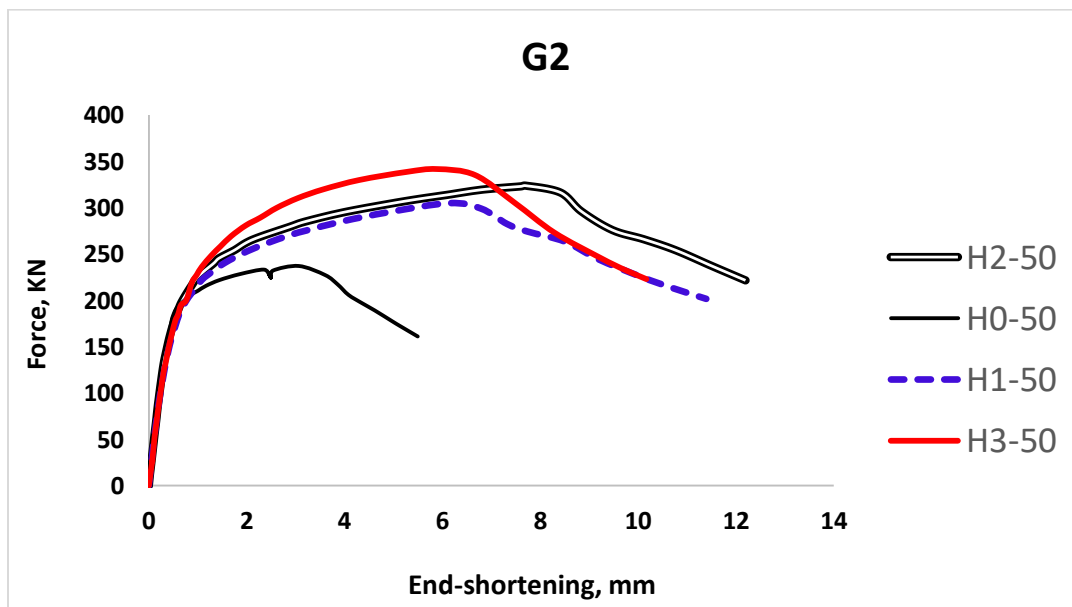


Fig. 4-6. Experimental force-end shortening of series G2 specimens.

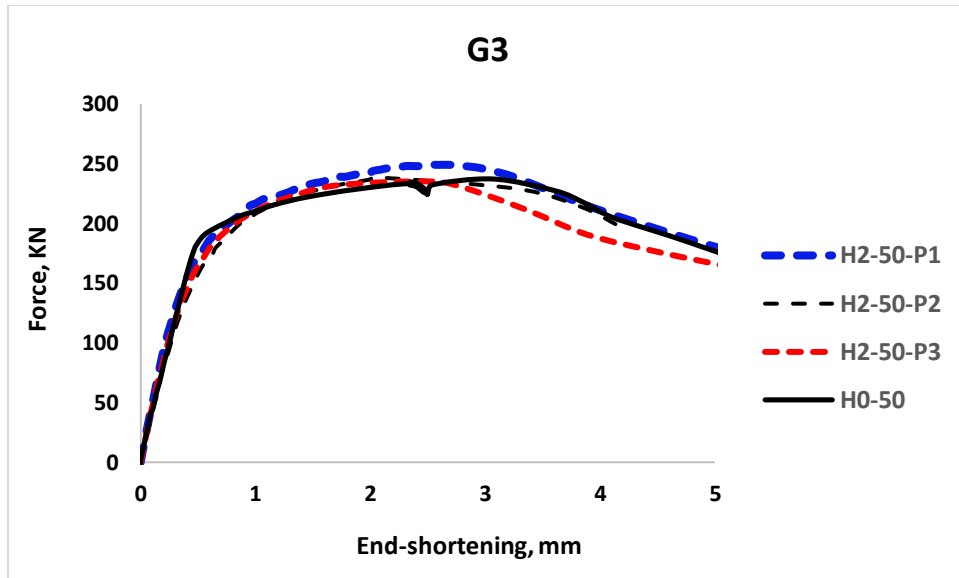


Fig. 4-7. Experimental force-end shortening of series G3 specimens.

Table 4-2. Test results of load carrying capacity and ultimate end shortening

series	Specimen Label	P_{test} , (kN)	δ_{test} , (mm)
G1	H0-37	384	7.8
	H1-37	473	9.8
	H2-37	501	10.8
	H3-37	531	11.4
G2	H0-50	237	2.53
	H1-50	305	6.14
	H2-50	324	7.7
	H3-50	341	5.9
G3	H2-50-P1	249	2.51
	H2-50-P2	238	2.03
	H2-50-P3	235	2.53

4.2.2 Failure Modes

The failure mode of the unwrapped specimens (H-50 and H0-37) was outward local buckling at the tube end as shown in **Fig. 4-8**. This failure mode of the unwrapped specimens agrees with what reported in the literature[5,19,43,44]. The stub columns partially wrapped failed by outward local buckling, as displayed in **Fig. 4-8**.

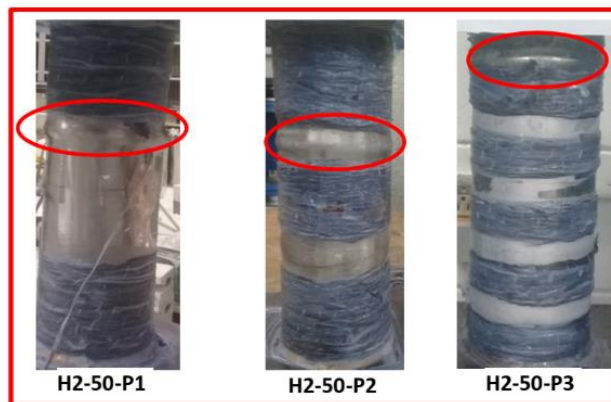
The failure mode of the specimens fully wrapped by one ply (H1-37 and H1-50) was inelastic outward local buckling deformation causing CFRP jacket rupture under hoop tensile stress. The location of the outward local buckling in the specimen of H1-37 was at the tube mid-height as shown in **Fig. 4-8**. In the specimen of H1-50, the outward local buckling occurred at the tube end accompanying with inward local buckling at the mid-height. The mode of failure of the specimens fully wrapped by two or three plies was inelastic inward local buckling deformation not associated with CFRP rupture, as illustrated in **Fig. 4-8**. It can be concluded that the increase in CFRP thickness leads to change the mode of failure from outward to inward local buckling because the thicker CFRP jacket prevents the tube to buckle outward.



(a)



(b)



(c)

Fig. 4-8: Failure Mode of: (a) series G1 specimens (b) series G2 specimens (c) series G3 specimens.

4.2.3 Effect of CFRP on the Load Carrying Capacity

By adding the CFRP wrap, the load carrying capacity of the HSSTCs specimens was enhanced compared with the control specimens. For the HSSTCs made of stainless steel tube with a D/t_s ratio of 37, the load carrying capacity enhancement by adding one CFRP ply, two CFRP plies and three CFRP plies was 23%, 30% and 38%, respectively. Regarding the specimens with D/t_s of 50, the enhancement was 29%, 36% and 43% due to using one CFRP ply, two CFRP plies and three CFRP plies, respectively. From the above results, CFRP jacket provides a little enhancement once the failure governed by the inward local buckling deformation. Furthermore, the results reflect that steel tubes with a higher D/t ratio are more susceptible to the local buckling, therefore the benefits from CFRP jacket becomes greater.

4.2.4 Effect of CFRP Strengthening On the Axial Shortening Capacity

Inspection of ultimate end-shortening capacity values in Table 2 displays an increase in the ultimate end-shortening capacity of the H1-37, H2-37 and H3-37 stub columns by 26%, 38% and 46%, respectively, compared to H0-37 column. Table 2 also shows that the ultimate end-shortening capacity for the H1-50, H2-50 and H3-50 stub columns exceeded that of the control specimen, respectively, by 140%, 200% and 130%. Experimental results confirm that degree of enhancement in deformation capacity of the specimens with D/t ratio of 50 higher than those specimens have D/t ratio of 37.

4.3 Finite Element Modeling

Finite element modeling (FEM) was performed in parallel with the experimental work. The aim of the FEM is to validate the models and thereafter perform the parametric study. This parametric study reduces the cost and saves time compared with conducting a large number of experiments. The parametric study results were exploited to propose an analytical model based on curve fitting parameters as described later. The nonlinear FEMs were carried out using standard ABAQUS software [45].

4.3.1 Finite Element Types

The axially loaded tubular columns were modeled using shell elements obtainable in the element library of the ABAQUS software [45] as shown in **Fig. 4-9**. The circular stainless steel tube was modeled using S4R shell elements that have six degrees of freedom (DOF) at each node making them able to provide accurate predictions for buckling modes. Furthermore, S4R shell elements were employed to model the CFRP wrap.

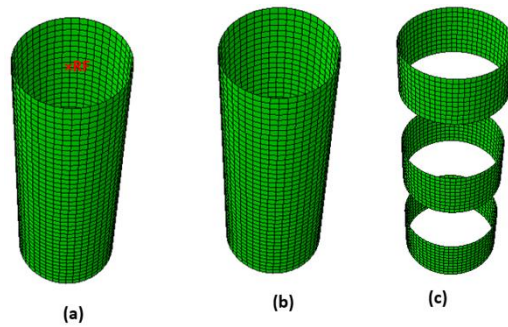


Fig.4-9. Finite Element modeling (a) stainless steel tube elements (b) CFRP elements

4.3.2 Boundary Conditions and Load Application

Each end of the stainless steel tube was coupled to a reference point assigned at the centroid of the tube cross section so that all end degrees of freedom (DOFs) were applied to the reference points as point DOFs. At the top end, all DOFs were restrained, except for translation in the longitudinal direction. On the other hand, all DOF at the bottom end were restrained (fully fixed end). The compressive loading was applied as a displacement control to capture the strain softening behavior. The modified Riks method was employed to solve the materially and geometrically stub column models.

4.3.3 Initial Imperfections

Initial geometric imperfections are out-of-plane deflections introduced into thin-walled structural elements during the manufacturing process [47]. The structural performance of thin-walled members can be strongly influenced by the initial geometric imperfections depending on how thin the element is. Therefore, it is necessary to include these initial geometric imperfections with appropriate patterns and amplitudes into the FE models to obtain an acceptable match with the measured experimental results. The initial geometric imperfection pattern along the member length was taken as the elastic local buckling mode that provides a symmetric half-sine wave shape under axial compressive loading. This was carried out by performing an elastic buckling analysis and linking the required elastic buckling mode shape deformation results with the actual nonlinear FE model using a short subroutine available in the ABAQUS software [45] called IMPERFECTION. The

amplitude to be multiplied by the imperfection pattern was taken as a percentage of the stainless steel tube thickness. In this study, the amplitude was taken as (t/10, t/50 and t/100) for each model to evaluate the effect of the initial geometric imperfections.

4.3.4 Material Modeling

4.3.4.1 Stainless Steel Material

An elastic-plastic model with the Von Mises yield criterion was employed to define the constitutive behavior of the stainless steel material. The engineering stress-strain relationship (σ_{Eng} , ϵ_{Eng}) of stainless steel material obtained from tensile testing reported in Chapter 3 was converted to a true stress-true strain (σ_{true} , ϵ_{true}) format using Eqs. 4-1 and 4-2.

$$\sigma_{true} = \sigma_{Eng} (1 + \epsilon_{Eng}) \quad (4 - 1)$$

$$\epsilon_{true} = \ln(1 + \epsilon_{Eng}) \quad (4 - 2)$$

4.3.4.2 CFRP Material and Contact Modeling

CFRP wraps were modeled in the ABAQUS software environment [45] as a linear elastic composite laminate. The failure mode observed in the experimental work of the CFRP wraps was the tensile rupture. Hence, it was essential to model the damage behavior of CFRP in the FE modeling. To model the CFRP wrap properly, it is necessary to define the laminate elastic, strength and damage evolution properties. The laminate elastic properties in the fibers' direction were taken from

the manufacturer as mentioned earlier, but in the other direction they were taken as a percentage of the properties in the fibers' direction. The Hashin Damage Model [46], which is available in the ABAQUS software [45], was used to model the strength and damage properties. The tensile strength value in the fibers' direction was given from the manufacturer, while the other values of strength shown in **Table 4-3** were assumed small values by trials to make the FE results match closely with the tests results. The values of damage evolution shown in **Table 4-3** were adopted as given in reference [47]. The contact between the stainless steel tube and the CFRP jacket was represented using a tie interaction where the stainless steel surface was assigned to be the master surface.

Table 4-3. CFRP modeling parameters

Parameters	Unit	Value
Laminate Elastic Properties		
Poisson's ratio		0.3
E1 (fiber direction), E2	GPa	220, 10
G12, G13	GPa	5, 5
Hashin damage model		
<u>Strength</u>		
Tensile strength (fiber direction), Tensile strength (transverse direction)	MPa	3000, 10
compressive strength (fiber direction), compressive strength (transverse direction)	MPa	10, 10
Shear (fiber direction), Shear (transverse direction)	MPa	10, 10
<u>Damage Evolution</u>		
Tensile fracture energy (fiber direction), Tensile fracture energy (transverse direction)	mJ/mm ²	92, 1.1
compressive fracture energy (fiber direction), compressive fracture energy (transverse direction)	mJ/mm ²	1.1, 0.2

4.3.5 Validation of the FE Models

An evaluation of the accuracy of the FE models was performed by comparing the results obtained from the experimental work and the FE models in terms of the load carrying capacity, force-displacement curves and the mode of failure. The ratios of the FE to the test load carrying capacity for the three imperfection levels are reported in **Table 4-4**. It is clear from **Table 4-4** that the predicted FE failure loads and deformations for the three considered imperfection amplitudes ($t/10$, $t/50$ and $t/100$) were not sensitive to the change in imperfection amplitudes. Thus, an imperfection amplitude of $t/100$ was adopted for verification with the experimental results. A good agreement between the FE force-displacement curves and those obtained from the experimental work was observed as illustrated in **Fig. 4-10**. For the comparison between the test and FE studies in terms of failure modes, a good agreement was observed, as shown in **Fig. 4-11**. In general, it can be concluded that the FE models carried out using the ABAQUS software[45] are able to predict the load carrying capacity of CFRP-confined CFSSTCs accurately, to capture the modes of failure observed in testing and to provide force-deformation curves that match well with the test curves.

Table 4-4. Comparison of the test and FE load carrying capacity and ultimate deformation

Specimen	P_{FE} / P_{test}			$\delta_{FE} / \delta_{test}$		
	Imperfection Amplitudes			Imperfection Amplitudes		
	t/10	t/50	t/100	t/10	t/50	t/100
H0-37	1.09	0.98	0.97	0.95	0.79	0.82
H1-37	1.15	0.99	0.98	1.81	0.94	0.92
H2-37	1.14	1.01	1.01	2.15	1.04	0.95
H3-37	1.15	1.04	1.03	1.37	0.64	0.61
H0-50	1.01	1.01	1.01	1.40	1.53	1.30
H1-50	0.97	1.03	1.00	1.08	1.28	1.18
H2-50	1.09	0.98	0.97	1.41	0.91	0.92
H3-50	1.09	1.01	1.00	1.26	0.93	0.90
H2-50-P1	0.89	0.99	1.03	0.59	0.80	1.00
H2-50-P2	0.94	1.03	1.06	0.98	0.99	1.30
H2-50-P3	0.95	1.04	1.07	0.68	0.80	0.99
Mean	1.04	1.01	1.01	1.24	0.97	0.99
COV	0.09	0.02	0.03	0.35	0.25	0.20

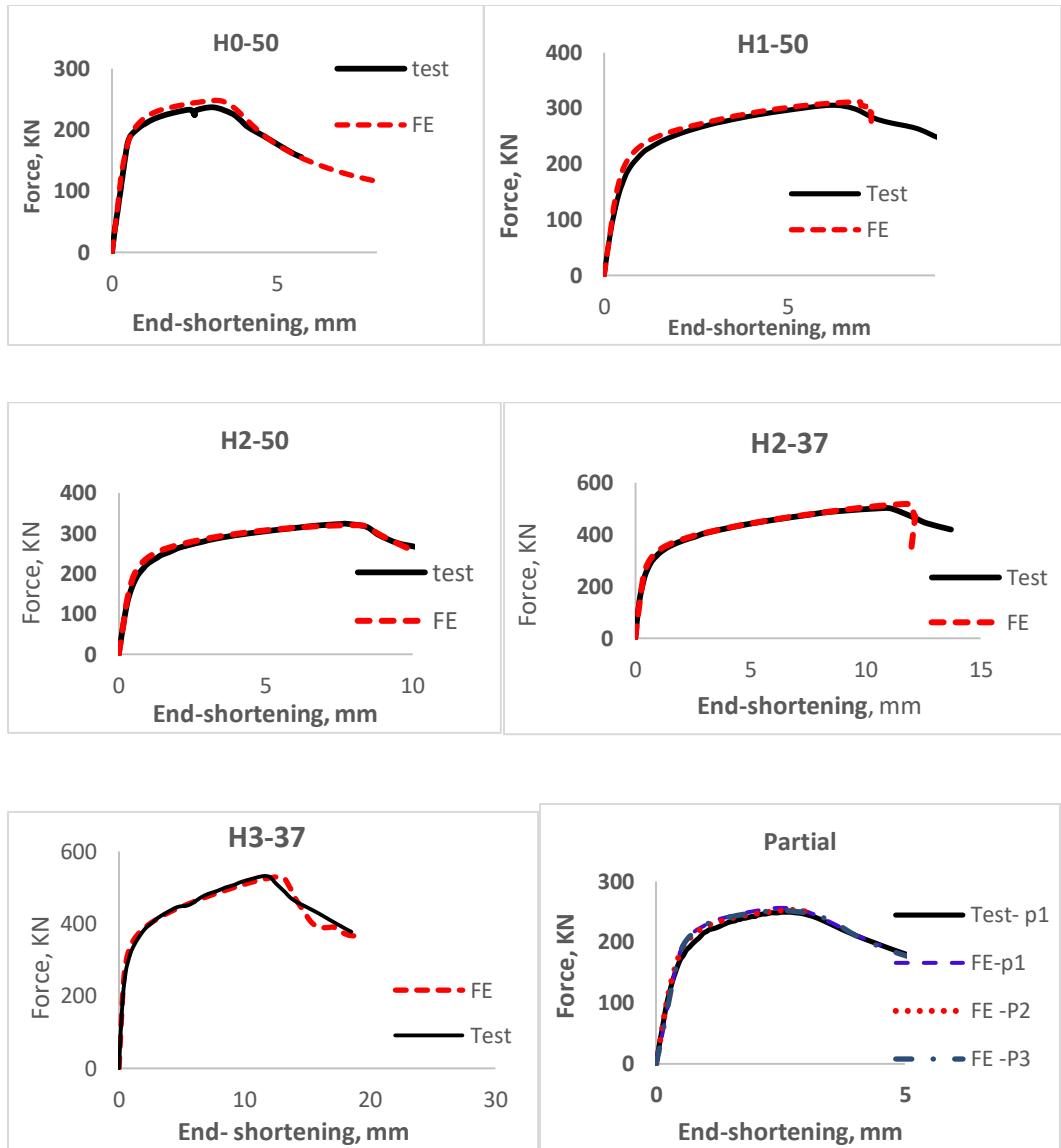


Fig. 4-10. Experimental and numerical force-end shortening for some selected specimens

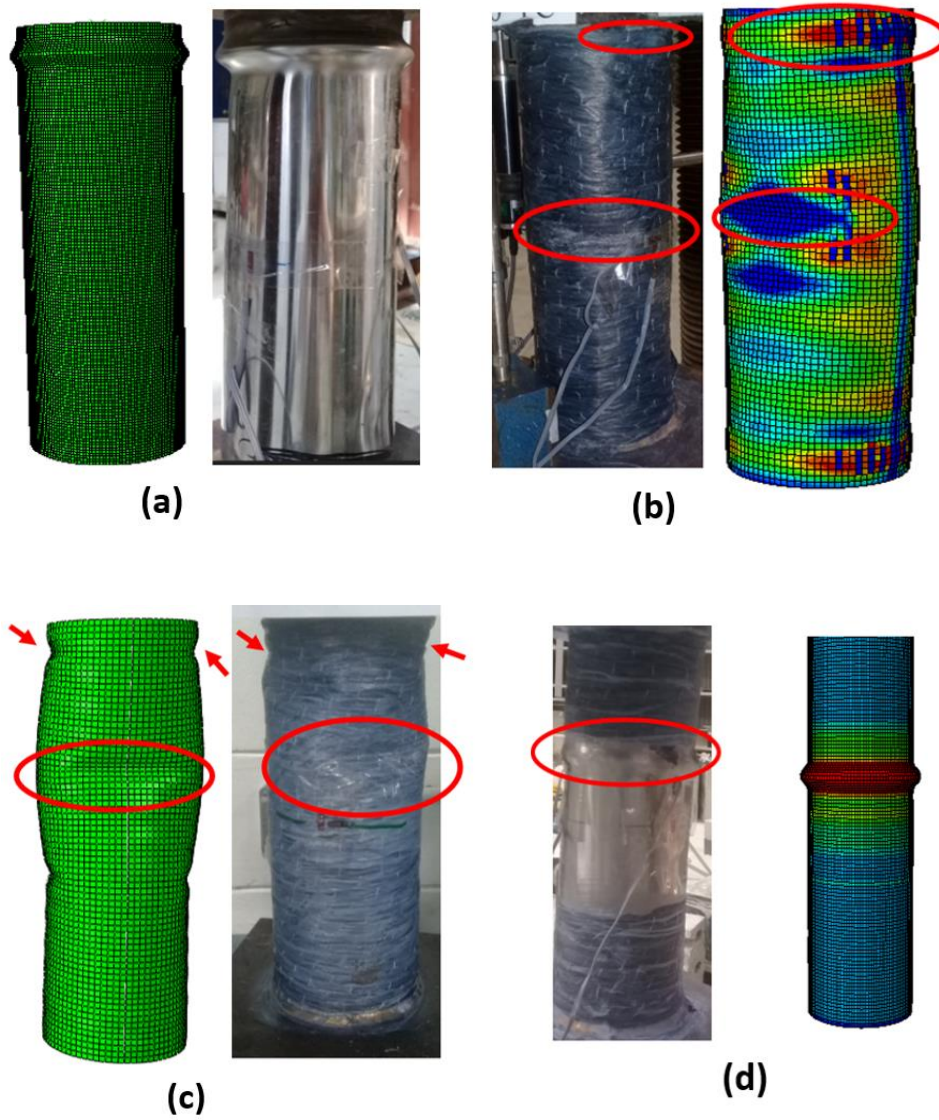


Fig.4-11. General failure mode of (a) unwrapped specimens (b) wrapped by one CFRP ply specimens (c) wrapped by two or three CFRP plies specimens (d) partial wrap specimens.

4.4 FE Parametric Study

Using the validated FE models, 106 models were analyzed using FE to obtain the load carrying capacity (P_{FE}) of CFRP-wrapped HSSTCs. P_{FE} data was employed to develop a proposed model to predict the load carrying capacity. The austenitic stainless steel material properties of CHSST 101×2 and CHSST 114×3.05 shown in **Table 3-2** were adopted in the FE parametric study. To avoid any flexural global buckling, the L/D ratio of the stainless steel tubes was taken as 3. The parametric study variables as well as the P_{FE} data are summarized in **Table 4-5**.

4.5 Analytical Study

A mathematical model to predict the load carrying capacity of CFRP-wrapped HSSTCs under axial compression is not available in the literature. Therefore, in this study, an analytical model based on curve fitting parameters to predict the load carrying capacity of CFRP-wrapped HSSTCs was proposed. The stress-strain characteristics of the stainless steel material are different from those of carbon steel material as discussed in the introduction.

4.5.1 Prediction of Load Carrying Capacity of Unstiffened HSSTCs

In this study, the unstiffened HSSTCs was designed using a new approach called Continues Strength Method (CSM)

Many investigations have been conducted into the structural behavior HSSTCs. Important early reported studies to explore the structural performance of HSSTC sections were performed by Rasmussen and Hancock [48]. Subsequently, a series of studies have been performed to understand more about these tubular sections [49–52]. These studies have contributed greatly to the expansion of information on the structural performance of stainless steel elements and at the same time have highlighted some shortcomings in the existing design standards that for the sake of simplification disregarded some mechanical characteristics observed in stainless steel, such as strong strain hardening and the rounded stress-strain relationship. This simplification method, adopted in the existing standard codes, includes using an elastic-perfectly plastic bilinear material model that leads to a significant degree of conservatism. Instead of considering that the maximum design stress limit is 2% proof stress, as in the existing standards, a new method called the continuous strength method (CSM) [43,44,53–56] has been developed as a deformation-based design method to exploit the noticeable strain hardening in the determination of stainless steel cross-section resistances. The stress-strain relationship of the stainless steel material is idealized to a bi-linear relationship as shown in **Fig. 4-12**.

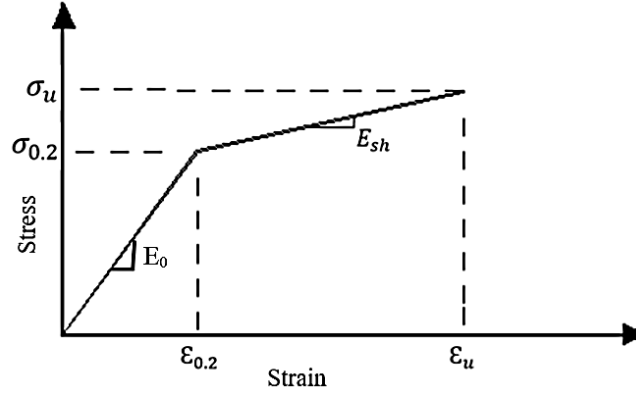


Fig. 4-12: Stress-strain relationship of stainless steel material adopted in CSM

The load carrying capacity of HSSTCs (P_{ss}) can be determines as follows:

$$P_s = A_s \sigma_{LB} \quad (4 - 3)$$

where: A_s is the cross sectional area of the stainless steel tube and σ_{LB} is the local buckling stress. The local buckling stress can be calculated as follows.

$$\sigma_{LB} = E_0 \varepsilon_{LB} \quad \frac{\varepsilon_{LB}}{\varepsilon_{0.2}} < 1 \quad (4 - 4)$$

$$\sigma_{LB} = \sigma_{0.2} + E_{sh} (\varepsilon_{LB} - \varepsilon_{0.2}) \quad \frac{\varepsilon_{LB}}{\varepsilon_{0.2}} \geq 1 \quad (4 - 5)$$

The CSM approach replaces the cross-sectional classification with a continuous relationship between the deformation capacity and the cross-section slenderness for the non-slender and slender sections, respectively, as follows:

$$\frac{\varepsilon_{LB}}{\varepsilon_{0.2}} = \frac{4.44 \times 10^{-3}}{\lambda_c^{4.5}} \leq \min(15, \frac{0.1 \varepsilon_u}{\varepsilon_{0.2}}) \quad \lambda_c \leq 0.3 \quad (4 - 6a)$$

$$\frac{\varepsilon_{LB}}{\varepsilon_{0.2}} = \left(1 - \frac{0.224}{\lambda_c^{0.342}}\right) \frac{1}{\lambda_c^{0.342}} \quad 0.3 \leq \lambda_c \leq 0.6 \quad (4 - 6b)$$

where: λ_c , ε_{LB} , $\varepsilon_{0.2}$, and ε_u are defined as the cross-section slenderness, local buckling strain, equivalent 2% proof strain (equivalent yield strain) and ultimate strain, respectively. The cross-section slenderness is expressed as:

$$\lambda_c = \sqrt{\frac{\sigma_{0.2}}{\sigma_{cr}}} \quad (4-7)$$

$$\sigma_{cr} = \frac{E}{\sqrt{3(1-\nu^2)}} \frac{2t_s}{D_s} \quad (4-8)$$

where: σ_{cr} is the elastic local critical stress.

$$E_{sh} = \frac{\sigma_u - \sigma_{0.2}}{c_2 \varepsilon_u - \varepsilon_{0.2}} \quad (4-9)$$

$$\varepsilon_u = 1 - \frac{\sigma_{0.2}}{\sigma_u} \quad (4-10)$$

where: c_1 and c_2 are parameters depending on the stainless steel grade, for austenitic stainless steel grade considered in this study as discussed earlier in chapter 3 c_1 and c_2 are 0.1 and 0.16, respectively. For the other grades, they are reported in the references [43,44,53–56].

4.5.2 Proposing a Strength Model for CFRP-stiffened HSSTCs

The role of CFRP wrap is to increase the stainless steel stiffness in the hoop direction and this leads to postponing occurring of the stainless steel local buckling. The results of load carrying capacity obtained from the parametric study shown in **Table 4-5** were employed to develop a simple model to predict the load carrying capacity, P_n ,

of CFRP-wrapped HSSTCs under axial compressive load. From an equilibrium state, the load carrying capacity, P_n , is defined as:

$$P_n = P_s \quad (4 - 11)$$

where: P_s refers to the axial ultimate strength of the stainless steel tube which can be expressed using Equation 4-3, as discussed in the previous section. To introduce the CFRP effect on the ultimate load carrying capacity, a dimensionless parameter (ψ_h) was added in the equilibrium equation to be as follows:

$$P_n = P_s (1 + \psi_h) \quad (4 - 12)$$

Equation (4-5) can be rewritten in the following form.

$$P_n = A_s \sigma_{LB} (1 + \psi_h) \quad (4 - 13)$$

The dimensionless parameter ψ_h can be formulated as an equation involving a number of variables that can affect.

A nonlinear dimensionless equation was proposed to obtain the best fitting with the load carrying capacity results that are summarized in **Table 4-5** as follows:

$$\psi_h = 2 \left(\frac{t_f}{t_s} \right)^{0.5} \left(\frac{t_s}{D_s} \right)^{0.4} \left(\frac{\sigma_f}{\sigma_{0.2}} \right)^{0.1} \quad (4 - 14)$$

where: t_f , t_s , D_s , and σ_f are, respectively, CFRP thickness, stainless steel tube thickness, tube diameter and CFRP failure strength. Regarding to **Table 4-5**, the proposed model provides accurate estimations, where the COV of the ratio P_{FE} / P_n is 0.05.

A comparison of the load carrying capacity obtained from the FE parametric study, P_{FE} , and that obtained from the proposed model is shown in **Fig. 4-13**.

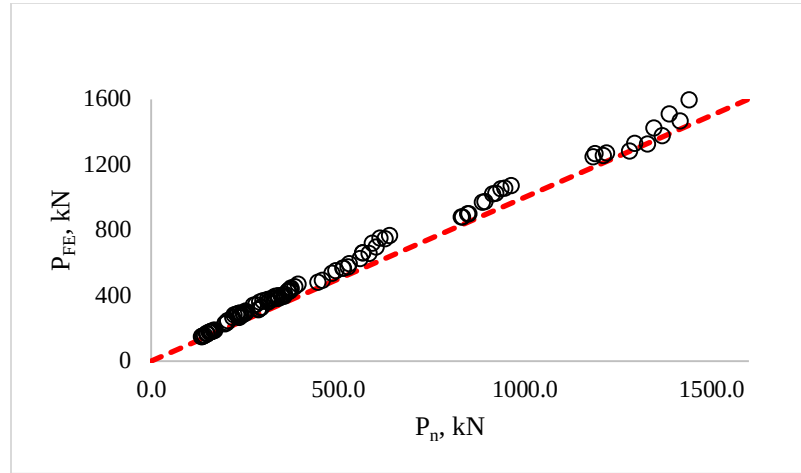


Fig.4-13. Comparison of the load carrying capacity obtained from the FE parametric study and the proposed model.

Table 4-5. Comparison of the FE parametric study results and the proposed model results

	Stainless steel tubes				<i>CFRP jacket</i>		P_{FE} (KN)	P_{FE}/P_n
	D (mm)	t_s (mm)	D/t	$\sigma_{0.2}$ (MPa)	t_f (mm)	σ_{frp} (MPa)		
S1	140	7	20	293	0.050	3200	1261	1.06
S2	140	7	20	293	0.100	3200	1264	1.03
S3	140	7	20	293	0.300	3200	1324	1.02
S4	140	7	20	293	0.500	3200	1420	1.05
S5	140	7	20	293	0.700	3200	1506	1.08
S6	140	7	20	293	1.000	3200	1592	1.10
S7	120	4	30	293	0.050	3200	562	1.09

S8	120	4	30	293	0.100	3200	589	1.11
S9	120	4	30	293	0.300	3200	656	1.15
S10	120	4	30	293	0.500	3200	716	1.21
S11	120	4	30	293	0.700	3200	744	1.21
S12	120	4	30	293	1.000	3200	760	1.19
S13	100	2	50	293	0.050	3200	226	1.13
S14	100	2	50	293	0.100	3200	240	1.16
S15	100	2	50	293	0.300	3200	275	1.23
S16	100	2	50	293	0.500	3200	284	1.21
S17	100	2	50	293	0.700	3200	282	1.16
S18	100	2	50	293	1.000	3200	289	1.14
S19	150	2	75	293	0.050	3200	310	1.07
S20	150	2	75	293	0.100	3200	323	1.08
S21	150	2	75	293	0.300	3200	363	1.14
S22	150	2	75	293	0.500	3200	376	1.13
S23	150	2	75	293	0.700	3200	378	1.10
S24	150	2	75	293	1.000	3200	390	1.09
S25	300	3	100	293	0.050	3200	876	1.05
S26	300	3	100	293	0.100	3200	896	1.05
S27	300	3	100	293	0.300	3200	968	1.08
S28	300	3	100	293	0.500	3200	1019	1.10
S29	300	3	100	293	0.700	3200	1051	1.11
S30	150	1	150	293	0.050	3200	144	1.05
S31	150	1	150	293	0.100	3200	149	1.05

S32	150	1	150	293	0.300	3200	163	1.07
S33	150	1	150	293	0.500	3200	170	1.07
S34	150	1	150	293	0.700	3200	176	1.07
S35	150	1	150	293	1.000	3200	182	1.06
S36	140	7	20	293	0.050	1500	1244	1.05
S37	140	7	20	293	0.100	1500	1252	1.03
S38	140	7	20	293	0.300	1500	1276	0.99
S39	140	7	20	293	0.500	1500	1320	0.99
S40	140	7	20	293	0.7	1500	1372	1.00
S41	140	7	20	293	1	1500	1463	1.03
S42	120	4	30	293	0.05	1500	560	1.09
S43	120	4	30	293	0.1	1500	568	1.07
S44	120	4	30	293	0.3	1500	620	1.10
S45	120	4	30	293	0.5	1500	652	1.11
S46	120	4	30	293	0.7	1500	692	1.14
S47	120	4	30	293	1	1500	740	1.18
S48	100	2	50	293	0.05	1500	220	1.10
S49	100	2	50	293	0.1	1500	228	1.11
S50	100	2	50	293	0.3	1500	259.6	1.18
S51	100	2	50	293	0.5	1500	280	1.21
S52	100	2	50	293	0.7	1500	288	1.21
S53	150	2	75	293	0.05	1500	308	1.07
S54	150	2	75	293	0.1	1500	320.4	1.08
S55	150	2	75	293	0.3	1500	356	1.13

S56	150	2	75	293	0.5	1500	376	1.15
S57	150	2	75	293	0.7	1500	378	1.12
S58	150	2	75	293	1	1500	388	1.11
S59	300	3	100	293	0.05	1500	876	1.05
S60	300	3	100	293	0.1	1500	896	1.06
S61	300	3	100	293	0.3	1500	964	1.08
S62	300	3	100	293	0.5	1500	1016	1.11
S63	300	3	100	293	0.7	1500	1048	1.12
S64	300	3	100	293	1	1500	1068	1.11
S65	150	1	150	293	0.05	1500	143.6	1.05
S66	150	1	150	293	0.1	1500	148	1.05
S67	150	1	150	293	0.3	1500	164	1.09
S68	150	1	150	293	0.5	1500	170	1.09
S69	150	1	150	293	0.7	1500	176	1.09
S70	150	1	150	293	1	1500	181.2	1.08
S71	120	4	30	293	0.300	3200	656	1.15
S72	101	2	50.5	357	0.050	3200	271	1.12
S73	101	2	50.5	357	0.100	3200	285	1.14
S74	101	2	50.5	357	0.150	3200	298	1.17
S75	101	2	50.5	357	0.030	2500	261	1.10
S76	101	2	50.5	357	0.050	2500	268	1.11
S77	101	2	50.5	357	0.100	2500	280	1.12
S78	101	2	50.5	357	0.150	2500	293	1.15
S79	114.3	3.05	37.5	292	0.030	3200	394	1.12

S80	114.3	3.05	37.5	292	0.050	3200	403	1.12
S81	114.3	3.05	37.5	292	0.100	3200	424	1.15
S82	114.3	3.05	37.5	292	0.150	3200	442	1.17
S83	101	3	33.7	357	0.030	3200	420	1.14
S84	101	3	33.7	357	0.050	3200	427	1.14
S85	101	3	33.7	357	0.100	3200	449	1.16
S86	101	3	33.7	357	0.150	3200	463	1.17
S87	114.3	3.05	37.5	292	0.030	3200	394	1.12
S88	114.3	3.05	37.5	292	0.050	3200	401	1.12
S89	114.3	3.05	37.5	292	0.100	3200	418	1.13
S90	114.3	3.05	37.5	292	0.150	3200	434	1.15
S91	200	2	100	357	0.050	3200	476	1.06
S92	200	2	100	357	0.100	3200	489	1.06
S93	200	2	100	357	0.290	3200	532	1.09
S94	200	2	100	357	0.400	3200	545	1.10
S95	150	1	150	293	0.150	3200	151	1.04
S96	150	1	150	293	0.300	3200	160	1.05
S97	150	1	150	293	0.600	3200	173	1.07
S98	150	1	150	293	0.900	3200	180	1.06
S99	101	2	50.5	357	0.400	3200	336	1.22
S100	101	2	50.5	357	0.500	3200	341	1.21
S101	101	2	50.5	357	0.700	3200	355	1.21
S102	101	2	50.5	357	0.900	3200	361	1.20
S103	101	2	50.5	357	1.200	3200	370	1.18

S104	101	2	50.5	357	1.500	3200	380	1.17
S105	101	2	50.5	357	1.800	3200	388	1.16
S106	101	2	50.5	357	2.100	3200	394	1.15
COV								0.05

4.6 Concluding Remarks

Experimental, numerical and analytical investigations were presented in this chapter to study the behavior of CFRP-wrapped HSSTCs under axial compression loading. External CFRP wrapping was used to restrain the local buckling deformation of the stainless steel tubes. Based on experimental, numerical and analytical investigations, the following conclusions can be drawn:

1. Mode of failure of the specimens considered in this chapter can be summarized as follows.
 - (a) For the unwrapped specimens, the failure mode was outward inelastic local buckling deformation.
 - (b) For the wrapped specimens by one ply, they failed due to outward inelastic local buckling followed by CFRP rupture with some inward local buckling deformation in different locations but the dominated failure mode was the outward inelastic local buckling deformation.
 - (c) For the specimens wrapped by two and three plies, they failed because of inward inelastic local buckling with no CFRP rupture.

2. The CFRP jacketing is effective in improving the axial compressive behavior of HSSTCs in terms of the ultimate strength and the axial shortening capacity. However, when the dominated failure mode is inward inelastic local buckling, the ultimate strength can be improved slightly compared to the specimens dominated by outward local buckling.
3. Partial wrapping has not shown any improvement in the load carrying capacity as well as deformation capacity.
4. The FE analysis showed that the CFRP-wrapped HSSTCs models were sensitive to the geometric imperfections.
5. A large number of results obtained from the FE parametric studies were employed to propose a simplified model to predict the load carrying capacity of CFRP-wrapped HSSTCs by a coefficient of variation of 0.04 and a standard deviation of 1.06.

CHAPTER 5

BEHAVIOUR OF CFRP-CONFINED CFSSTCs UNDER AXIAL LOADING

This chapter displays an experimental work, finite element simulation, parametric study and proposing a design model for CFRP-confined concrete-filled stainless steel tubular stub columns under axial loading. Conclusions are given at the end of this chapter.

5.1 Experimental Investigation

The experimental work was conducted to study the structural performance of CFRP-confined circular CFSSTCs under axial compression loading by using three cross-section sizes. The experimental work involves different tests, such as stainless steel material tensile coupon, concrete uniaxial compressive, and thirteen stub column tests under axial loading. The preparation of the experimental specimens and test setup used for the experimental work is described and reported in the following sections.

The experimental program was designed and performed to evaluate the structural performance of CFRP wrapped CFSSTCs under axial monotonic compressive loading. **Fig. 5-1a** shows a cross-section of CFRP-confined CFSSTCs. Experiments were conducted on three different circular hollow stainless steel (CHSS) cross-sectional sizes: CHSS 101×2 , CHSS 114×3 and CHSS 101×1.5 , where the first number refers to the tube's outer diameter and the second number represents the tube thickness. For each section size, four stub column tests were carried out, one of them without CFRP and the other three with CFRP. Normal concrete was used. The main variable parameters in the tests were the stainless steel outer diameter to thickness ratio (D/t_s) and the CFRP jacket thickness (t_f). All of the stub columns were 290 mm in length for two reasons: first to ensure that the specimens are suitably short to avoid overall flexural buckling, while the second reason relates to the available height in the hydraulic testing machine. The test specimens and parameters are summarized in **Table 5-1**. The CFRP arrangement was full wrapping, except one specimen that was a partial arrangement, as displayed in **Fig. 5-1b**. In the partially wrapped specimen, two-thirds of the length were confined by CFRP wrapping. The middle third was free from CFRP wrapping. This partial wrapping is limited to a diameter to thickness ratio of 50; it was used only in the experimental program as a control to compare with the full wrap one. It was not included in the numerical and mathematical parts later.

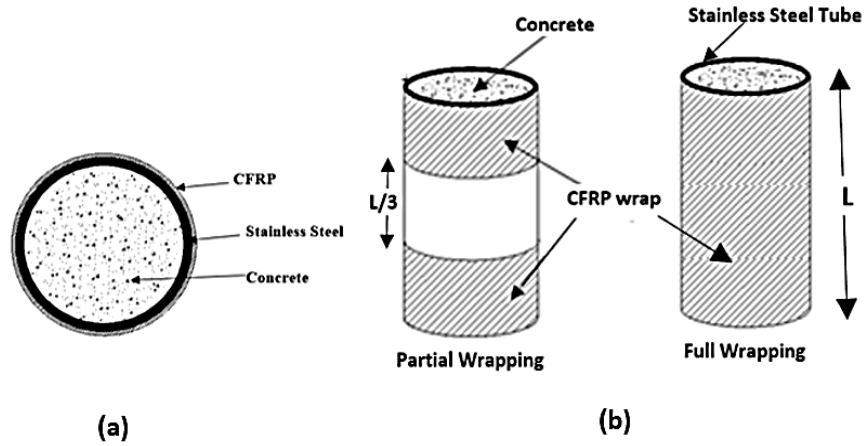


Fig. 5-1. CFRP-wrapped CFSSTCs scheme (a) Cross Section (b) CFRP arrangement

Table 5-1: Measured Geometric Dimensions and Test Variables of CFSSTCs

Series	Specimen label	Stainless Steel Tube			CFRP		arrangement
		D (mm)	t _s (mm)	D/t _s	No. of plies	t _f (mm)	
G1	C0-37	114.3	3.05	37.5	0	0	-
	C1-37	114.3	3.05	37.5	1	0.29	Full wrapping
	C2-37	114.3	3.05	37.5	2	0.58	Full wrapping
	C3-37	114.3	3.05	37.5	3	0.87	Full wrapping
G2	C0-50	101	2	50.1	0	0	-
	C1-50	101	2	50.1	1	0.29	Full wrapping
	C2-50	101	2	50.1	2	0.58	Full wrapping
	C3-50	101	2	50.1	3	0.87	Full wrapping
G3	C0-67	101	1.5	67.3	0	0	-
	C1-67	101	1.5	67.3	1	0.29	Full wrapping
	C2-67	101	1.5	67.3	2	0.58	Full wrapping
	C3-67	101	1.5	67.3	3	0.87	Full wrapping
G4	C2-50-partial	101	2	50.1	2	0.58	Partial wrapping

Each specimen was given a distinctive label starting with the letter C followed by a number referring to the number of CFRP plies. The letter C stands for the CFSSTCs. The second number represents the D/t_s ratio. For example, the label C2-37 refers to a CFSSTC that has a D/t_s ratio of 37.5 and is wrapped by two CFRP plies, respectively.

5.1.1 Specimens Preparation

Thirteen stainless steel circular tubes with the dimensions given in **Table 4-1** were cut from three long cold-rolled tubes as shown in **Fig. 5-2a**. Subsequently, one thick steel plate was welded to the bottom end of the tubes as shown in **Fig. 5-2b**, while the top end was left without a plate to cast the concrete in. The tubes were filled with normal concrete with appropriate shaking to avoid any concrete segregation, as displayed in **Fig. 5-2c**. After curing, the outer surface of the stainless steel tubes was cleaned to remove any fragments stuck during the concrete casting. To increase adhesion with CFRP, the outer surface of the stainless steel tubes was roughened by sandpapers. A continuous carbon fiber sheet 300 mm in width was wrapped with a sufficient amount of glue around the stainless steel tubes with the required number of layers. An appropriate overlapping length was added to satisfy circumferential continuity. To ensure uniform compressive loading on the specimens, a thin gypsum layer was applied to the top surface to fill any voids, as illustrated in **Fig. 5-2d**. A minimum amount of gypsum was used to reduce its effect on the early stages of loading.

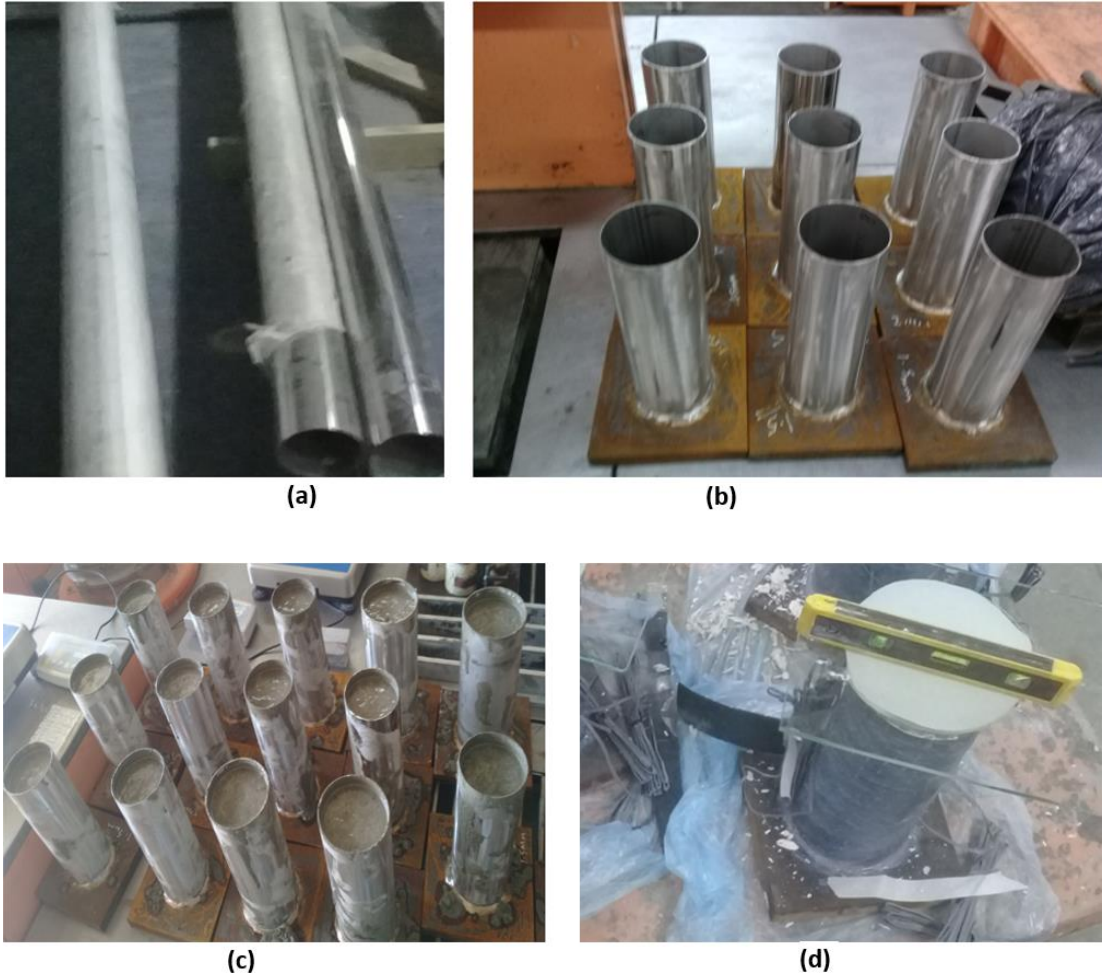


Fig. 5-2. Stub columns preparation (a) long cold-rolled stainless steel tubes (b) stub columns with the bottom welded plate (c) Stub columns concrete casting (d) applying a thin gypsum capping.

5.1.2 Instrumentation and Testing

For each specimen, an axial compressive test with monotonic loading was performed to determine the load-deformation response. The top end of the samples was clamped with an appropriate steel belt, while the bottom end was welded with a thick steel plate. This clamping for both ends is required to avoid any possibility of failure at the end surfaces due to out of flatness. The steel clamp at the top end

was placed slightly away from the top end to prevent any load transferred from the steel clamp to the stainless steel tube. **Fig. 5-3** shows the test setup, comprised of four LVDTs which were located vertically to measure the average end shortening. Two strain gauges were glued longitudinally to all the specimens at mid-height while two strain gauges were pasted horizontally at mid-height. For the wrapped specimens, strain gauges were affixed to the outer surface of the CFRP composite. A MATEST 3000 kN hydraulic testing machine was used to apply an axial compressive load with a loading rate of 1 kN/s. The LVDTs, strain gauges and load cell readings were connected to the data logger.

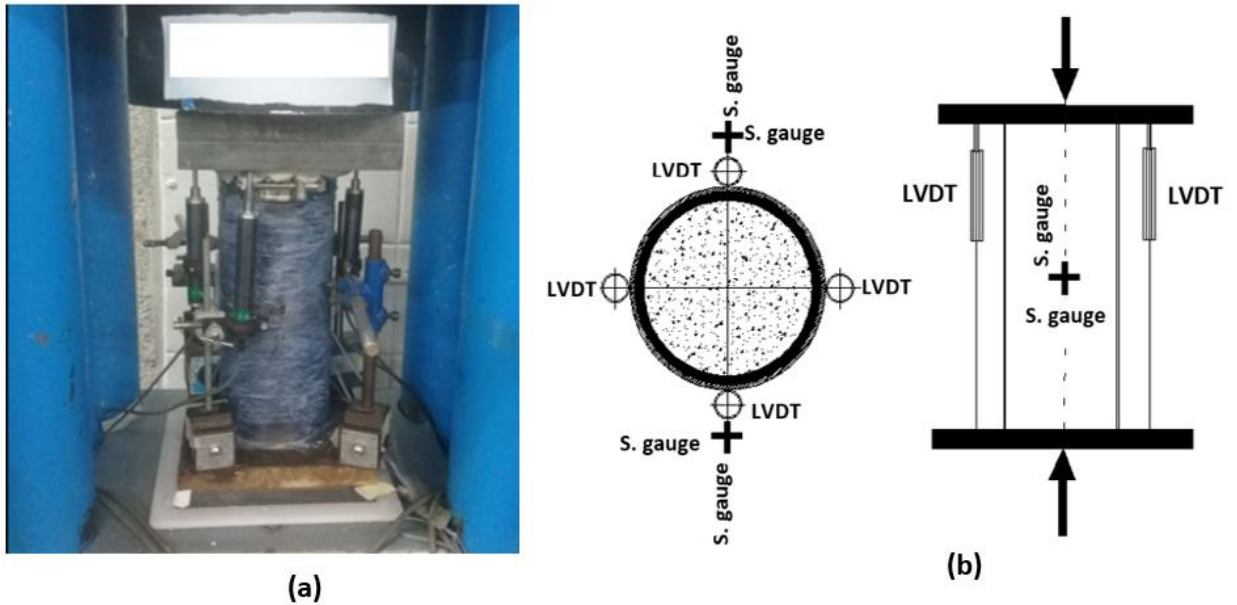


Fig. 5-3. Axially loaded stub column test arrangement: (a) test Setup (b) schematic Diagram of Experimental Setup.

5.2 Results and Discussion

5.2.1 Force-Deformation Response

The axial load-end shortening curves for all of the stub column specimens are displayed in **Fig. 5-4** to **Fig. 5-6**, where the end shortening was taken as the average value of the four LVDTs. Vertical strain gauge readings were employed to correct the force-end shortening curves in the initial loading stage to obtain true end shortening values. At the initial loading stage, the effect of end platen deformation and the gypsum layer deformation lead to a shift in the end shortening values. The vertical mid-height strain gauge readings were multiplied by the specimen length to obtain the average end shortening. The force-end shortening curves shown in **Fig. 5-4** to **Fig. 5-6** represent the true force-end shortening after the correction.

The full force-end shortening curves of the wrapped specimens can be divided into three stages. In the first stage, the stainless steel and concrete core behave elastically with no composite action between them since the lateral expansion of the stainless steel tube is greater than the lateral expansion of the concrete core due to the larger Poisson's ratio of the stainless steel material. In this stage, as shown in **Fig. 5-4** to **Fig. 5-6**, the force-end shortening relationships of the unwrapped and the wrapped specimen are matching because of the absence of the CFRP role at this stage. In the second stage, with the increase in the loading, the concrete core starts to behave inelastically and the concrete lateral expansion becomes greater than that of the stainless steel tube. The concrete core becomes confined by the stainless steel tube and the CFRP wrap. As clarified in **Fig. 5-4** to **Fig. 5-6**, the strength of the wrapped

specimens becomes greater than that of the unwrapped specimens. In this stage, the increase in the strength of the wrapped specimens is driven by the CFRP confinement and this is clear where the force-end shortening relationships become straight lines, as shown in **Fig. 5-4** to **Fig. 5-6**. In the third stage, when the CFRP wrap reaches its ultimate strength, CFRP rupture suddenly occurs leading to a sudden drop in the load carrying capacity, as shown in **Fig. 5-4** to **Fig. 5-6**. After that, the wrapped specimens behave like the unwrapped specimens and their force-end shortening relationships come into agreement again.

By comparing the results of the CFRP-wrapped CFSSTCs with the unwrapped ones, it is clear that the CFRP wraps provide a significant improvement in terms of load carrying capacity, as well as in the axial shortening capacity as shown in **Fig. 5-4** to **Fig. 5-6**. The load carrying capacity and axial shortening capacity of the stub specimens are summarized in **Table 5-2** as P_{test} and δ_{test} , respectively.

The axial load-end shortening curves of the partially wrapped, fully wrapped and unwrapped specimens for the same D_s/t_s ratio are displayed in **Fig.5-7**. For the same CFRP thickness and layers, the carrying load capacity of the fully wrapped specimen is much greater than the load carrying capacity of the partially wrapped specimens, as shown clearly in **Fig.5-7**. Although two-thirds of the specimen length were covered by CFRP, the strength enhancement compared with the unwrapped specimen was 10%, while the fully wrapped specimen enhanced the strength by 96%. Therefore, it is not recommended to use partial wrapping for CFSSTCs.

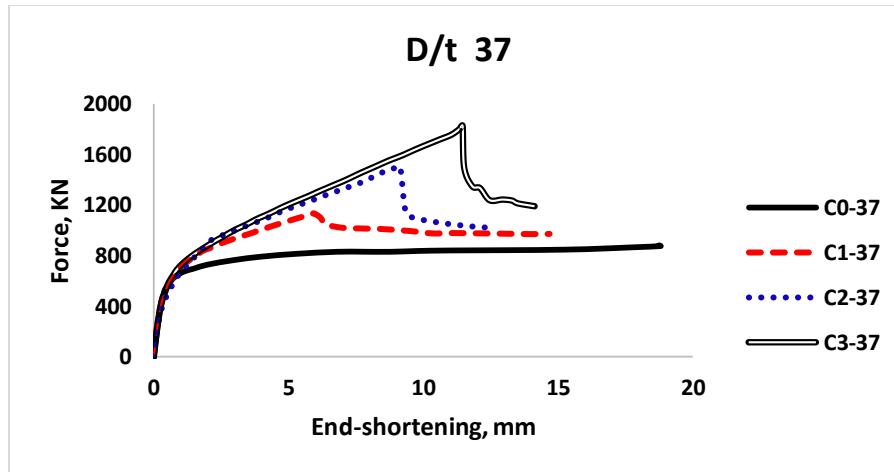


Fig. 5-4. Experimental force-end shortening of series G1 specimens.

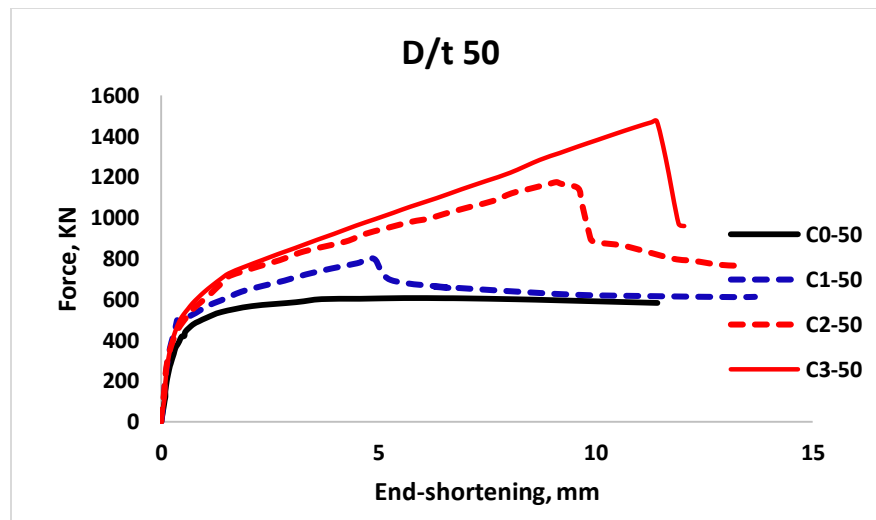


Fig. 5-5. Experimental force-end shortening of series G2 specimens.

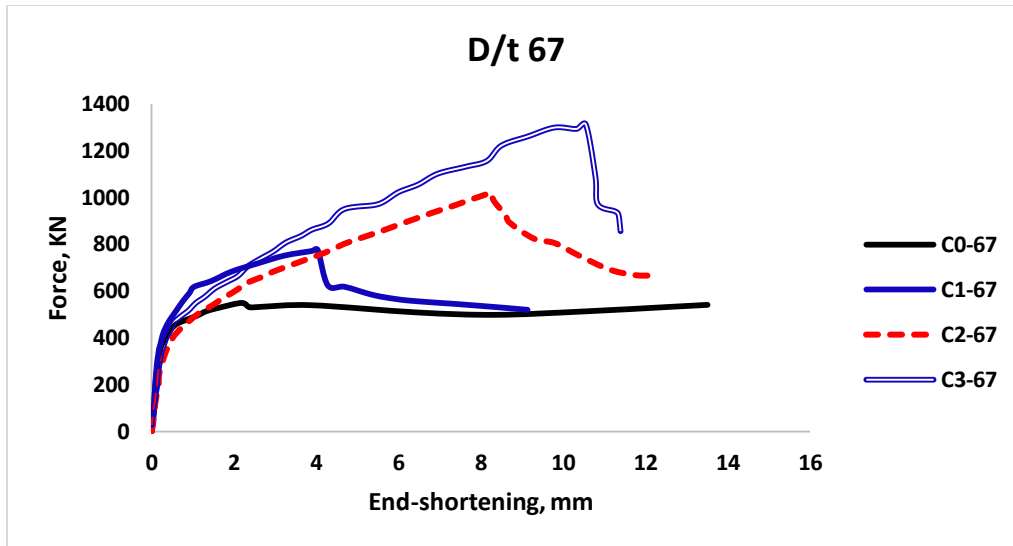


Fig. 5-6. Experimental force-end shortening of series G3 specimens.

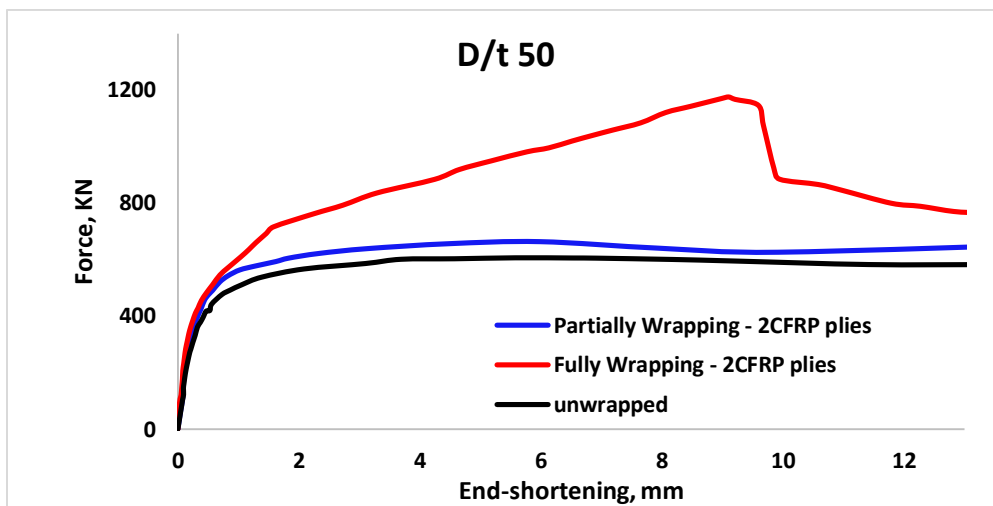


Fig. 5-7: Comparison of fully and partially wrapped specimens with D_s/t_s of 50

Table 5-2. Test results of load carrying capacity and ultimate end shortening

series	Specimen Label	Confinement ratio	P_{test} , (kN)	δ_{test} , (mm)
G1	C0-37	0	831	3.7
	C1-37	0.43	1133	6.3
	C2-37	0.85	1502	9.1
	C3-37	1.28	1825	11.5
G2	C0-50	0	600	3.6
	C1-50	0.38	799	4.9
	C2-50	0.76	1176	9.1
	C3-50	1.13	1474	11.4
G3	C0-67	0	562	3.7
	C1-67	0.38	775	5.6
	C2-67	0.76	1020	8.2
	C3-67	1.13	1311	10.6
G4	C2-50-Partial	0.76	662	5.9

5.2.2 Failure Modes

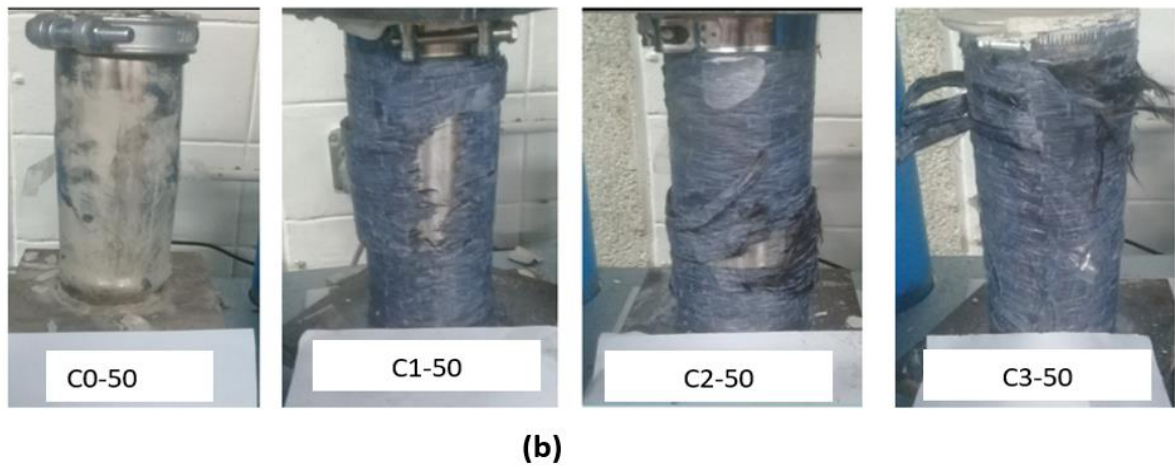
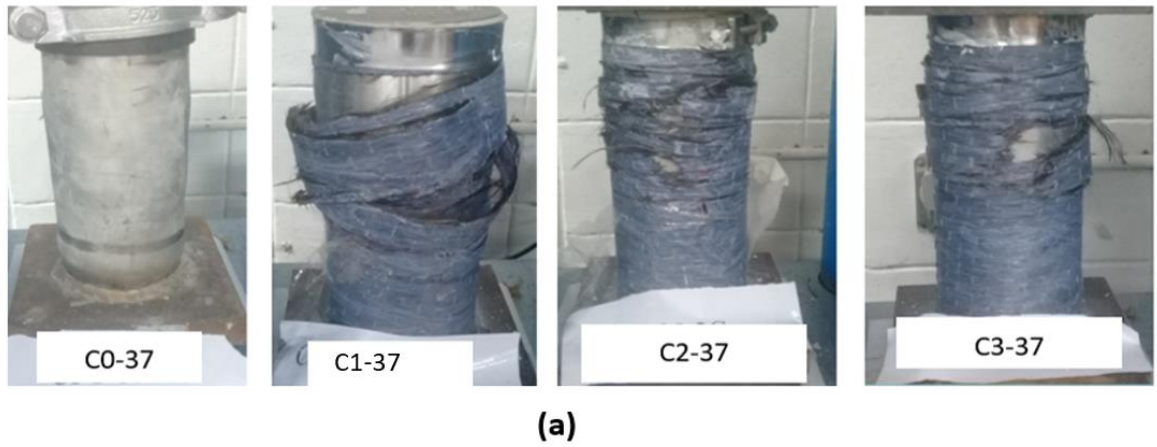
The function of the stainless steel, concrete infill and CFRP wrap in the CFRP-confined CFSSTCs can be explained as follows. The stainless steel tube has two main functions: contributing, based on its axial stiffness, to carrying the axial load and confining the concrete. For the concrete core, it carries a major part of the axial compressive load, as well as preventing inward local buckling of the stainless steel

tube. The CFRP wrap postpones the occurrence of outward local buckling of the stainless steel tube and increases the concrete confinement.

All of the unwrapped specimens experienced continuous dilation and localized outward buckling of the stainless steel tube close to one end, as shown in **Fig. 5-8**. Because of the initial imperfections, the inelastic local buckling cannot occur simultaneously near to both ends, but only at one end. When the outward local buckling of the stainless steel tubes occurs, the load carrying capacity drops slightly and then rises again. This re-increasing of the strength takes place because of the redistribution of the load carrying where the stainless steel tubes stop carrying the load and the entire loading takes place on the confined concrete core. The ultimate state of the non-confined specimens is defined as the state when the outward local buckling of the stainless steel tube occurs associated with excess inelastic deformation; regardless of the load increasing that takes place after inelastic local buckling.

All CFRP-confined specimens failed because of the sudden rupture of the CFRP wrap due to the lateral concrete core expansion at the mid-height as displayed in **Fig. 5-8**. This concrete lateral expansion makes the carbon CFRP expand until it reaches its ultimate strain. This sudden rupture of the CFRP leads to a rapid drop in load carrying capacity. The ultimate state of the CFRP-confined specimens is defined as the state when the rupture of the CFRP jacket occurs followed by a sudden load drop.

The mode of failure of the partially wrapped specimen was the outward local buckling of the stainless steel just after CFRP confinement in the middle-third, as shown in **Fig.5-9**.





(c)

Fig. 5-8: Failure Mode of: (a) series G1 specimens (b) series G2 specimens (c) series G3 specimens.



Fig. 5-9: Failure mode of the partially wrapped specimen.

5.2.3 Effect of CFRP Strengthening On the Load Carrying Capacity

By adding the CFRP wrap, the load carrying capacity of the CFSSTC specimens was increased significantly compared with the control specimens. For the CFSSTCs made of stainless steel tubes with a D/t_s ratio of 37, the load carrying capacity enhancement by adding one CFRP ply, two CFRP plies and three CFRP plies was

36%, 81% and 120%, respectively. Regarding the specimens with a D/t_s of 50, the enhancement was 33%, 96% and 145% due to using one CFRP ply, two CFRP plies and three CFRP plies, respectively. For specimens with a D/t_s of 67, the improvement was 38%, 81% and 133% compared with the control specimen, by using a CFRP wrap with one ply, two plies and three plies, respectively.

Test results indicate that the relationship between percentage increase of the axial capacity compared to bare column and varying of D/t is not clear. For example, the percentage increase of the axial capacity of specimens with D/t of 37 and 67 are close to each other.

A confinement ratio can also be used to represent the effect of the CFRP thickness. It can be defined as the ratio between confinement pressure (f_l) at the CFRP rupture to the unconfined concrete compressive strength (f'_c). The confinement ratios for the tested specimens are summarized in **Table 5-2**. It is clear that the confinement ratio is directly proportional to the amount of strength enhancement.

$$f_l = \frac{2 t_f \sigma_f}{D} \quad (5-1)$$

where: σ_f is the tensile strength of CFRP jacket at CFRP rupture.

5.2.4 Effect of CFRP Strengthening On the Axial Shortening Capacity

The CFRP-wrapped CFSSTCs exhibited a significant enhancement in the axial shortening capacity. For the wrapped specimens with a D/t_s ratio of 37, the axial shortening capacity enhancement by adding one, two and three CFRP plies with respect to the unwrapped specimen was, respectively, 70%, 145% and 210%. For

the wrapped specimens with a D/t_s ratio of 50, the axial shortening capacity enhancement was 36%, 150% and 216% due to the addition of one, two and three CFRP plies with respect to the unwrapped specimen, respectively. For the specimens with a D/t_s of 67, the axial shortening capacity improvement was 50%, 120% and 180% compared with the unwrapped one, by using a CFRP jacket with one, two and three plies with respect to the unwrapped specimen, respectively.

5.3 Finite Element Modeling

FEM was performed in parallel with the experimental work. The aim of the FEM is to validate the models and thereafter perform a parametric study. This parametric study reduces the cost and saves time compared with conducting a large number of experiments. The parametric study results were exploited to propose an analytical model based on the curve fitting parameters as described later. The nonlinear finite element models were carried out using the ABAQUS software [45].

5.3.1 Finite Element Types

The axially loaded composite columns were modeled using shell and solid elements obtainable in the element library of the ABAQUS software [45] as shown in **Fig.5-10**. The concrete core was represented using 3D C3D8 solid elements. The circular stainless steel tube was modeled using S4R shell elements that have six DOF at each node making them able to provide accurate predictions for buckling modes. Furthermore, S4R shell elements were employed to model the CFRP wrap. A rigid

shell plate was modeled and placed at both ends of the tubular columns to transfer the axial load uniformly.

The results of mesh size sensitivity are presented in **Fig.5-11** where each point refers to the mesh size. It indicates that an approximate mesh size of 7 mm provides accurate predictions with a reasonable running time.

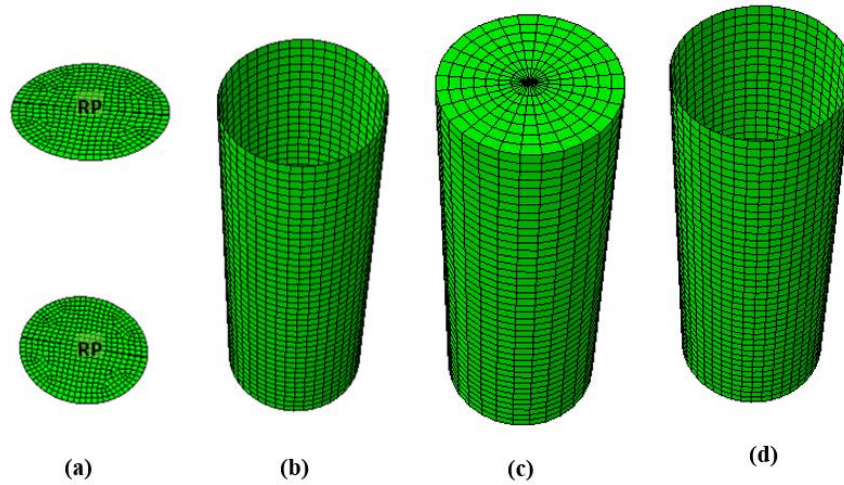


Fig. 5-10. Finite Element modeling (a) rigid plates with reference points (b) stainless steel tube elements (c) concrete core elements (d) CFRP elements.

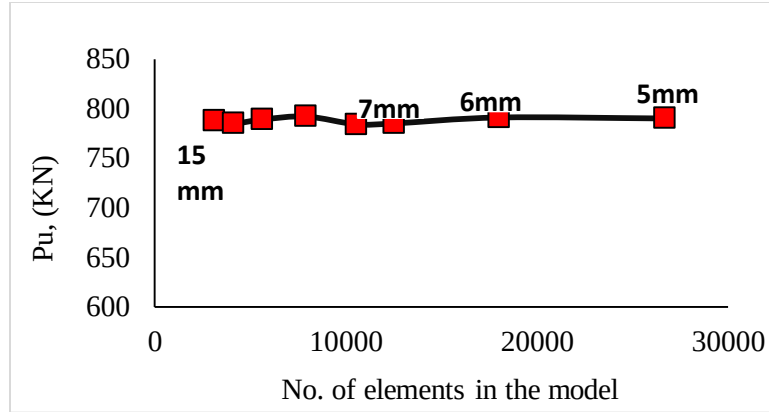


Fig.5-11: Effect of mesh size on the load carrying capacity of CFRP-wrapped CFSSTCs

5.3.2 Boundary Conditions and load Application

Each end of the rigid plate was coupled to a reference point so that all end DOFs were applied to the reference points as point DOFs. At the upper rigid plate, all DOFs were restrained, except for translation in the longitudinal direction. On the other hand, all DOFs at the lower rigid plate were restrained (fully fixed end). The compressive loading was applied as a displacement control to capture the strain softening behavior.

5.3.3 Initial Imperfections

Modeling of Initial geometric imperfections is similar to what discussed earlier in chapter 3, section 4.3.

5.3.4 Material Modeling

The stainless steel and CFRP materials were modeled in ABAQUS as discussed earlier in sections 4.3.4.1 and 4.3.4.2, respectively.

5.3.4.1 Concrete Material

In concrete-filled stainless steel tubes, the concrete infill has to be modeled as confined concrete, while the mechanical properties testing was conducted on uniaxial unconfined concrete. Therefore, an equivalent tube-confined concrete stress-strain curve has to be used as reported in [57]. The tube-confined stress-strain curve was modeled using the approach reported in reference [58]. The stress-strain curves for FRP-confined and unconfined concrete are shown in **Fig. 5-12a** where f'_c , $f'_{cc,t}$, ε'_c and $\varepsilon'_{cc,t}$ are, respectively, unconfined concrete compressive strength, confined concrete compressive strength, unconfined compressive strain corresponding to f'_c and confined compressive strain corresponding to $f'_{cc,t}$. The confined concrete strength of the concrete core subjected to a confining pressure from circular steel tube ($f'_{cc,t}$) is determined by Mander et al. [59] model as follows:

$$f'_{cc,t} = f'_c + k_1 f_{l,steel} \quad (5 - 2a)$$

where: the parameter factor (k_1) is given as 4.1 as suggested by Richart et al. [60]. The confining pressure of the steel tube ($f_{l,steel}$) was modeled by Hu et al. [58] as follows:

$$\frac{f_{l,steel}}{f_y} = \begin{cases} 0.006241 - 0.0000357 \frac{D}{t_s} & 47 \leq \frac{D}{t_s} \leq 150 \\ 0.006241 - 0.0000357 \frac{D}{t_s} & 21 \leq \frac{D}{t_s} \leq 47 \end{cases} \quad (5 - 2b)$$

where, f_y is the yield strength which is equivalent to $\sigma_{0.2}$ in stainless steel material.

The constitutive description of the concrete material in the elastic linear region, before $0.5 f'_{cc,t}$ as shown in **Fig. 5-12a**, needs only two elastic parameters to describe it: the initial modulus of elasticity (E_c) and the Poisson's ratio which was taken as 0.2. On the other side, the constitutive description behavior of the concrete material in the plastic region includes the description of the yield surface function, the flow rule and the hardening/softening laws. The plasticity of the concrete material was modeled in the ABAQUS using the Concrete Damaged Plasticity Model (CDPM) [54]. In this study, to achieve a good match with the experimental results, 0.8 and 1.16 values were used to define K_c and f_{bo}/f'_c , respectively. In addition, two parameters are needed to describe the non-associated flow rule: the dilation angle and the eccentricity parameter, and they were introduced in the ABAQUS with the values 15° and 0.10, respectively.

Uniaxial tensile stress-strain response as shown in **Fig. 5-12b** is required to define in CDPM. In this study, two indirect tests were conducted to determine the concrete tensile strength, f_t , as reported in chapter 3. The concrete tensile strength resulting from the formula adopted by ACI-318 was used beside the results obtained from the indirect tensile tests to evaluate which one can provide results that are more accurate.

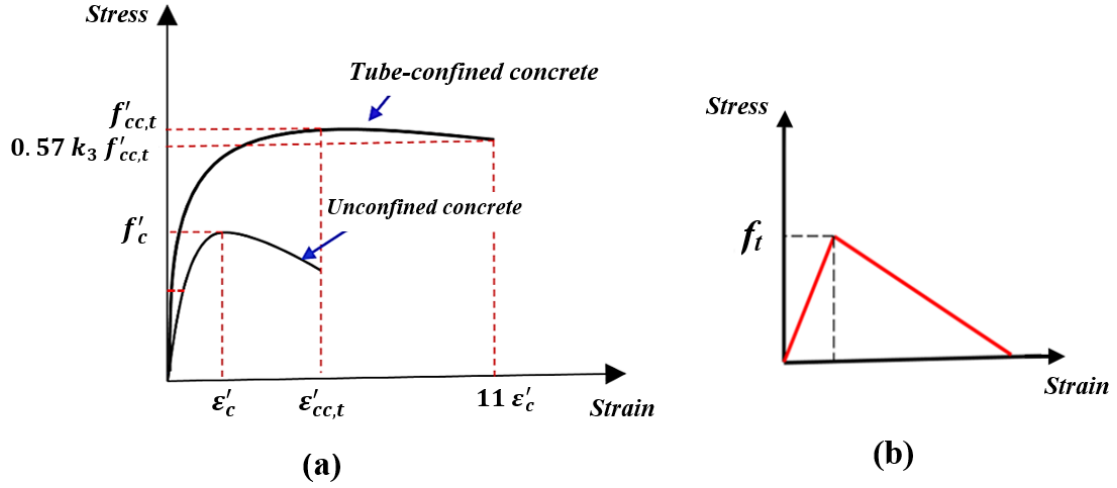


Fig. 5-12: stress-strain model of concrete material (a) compression (b) tension

5.3.5 FE Interaction Modeling

The interaction modeling between the stainless tube and concrete infill was specified as a friction interaction in the tangential direction with a friction coefficient of 0.25 as reported in [61], while the interaction in the normal direction was assigned as a hard contact to prevent any penetration between the two surfaces. The concrete core and stainless steel tube were assigned as master and slave surfaces, respectively. The contact between the stainless steel tube and the CFRP jacket was represented using a tie interaction where the stainless steel surface was assigned to be the master surface. The interaction between the rigid plate surfaces and the steel tube surfaces was specified as a tie contact where the rigid plates were assigned to be the master surfaces. On the other hand, the interaction between the rigid plates and the concrete infill surfaces was introduced using a friction interaction in the tangential direction with a friction coefficient of 0.35 as recommended in [30] and hard contact in the normal direction.

5.3.6 Validation of the FE Models

An evaluation of the accuracy of the FE models was performed by comparing the results obtained from the experimental work and the FE models in terms of the load carrying capacity, force-displacement curves and the mode of failure. The ratios of the FE to the test load carrying capacity for the three imperfection levels are reported in **Table 5-3**. It is clear from **Table 5-3** that the predicted FE failure loads and deformations for the three considered imperfection amplitudes ($t/10$, $t/50$ and $t/100$) were not sensitive to the change in imperfection amplitudes. Thus, an imperfection amplitude of $t_s/100$ was adopted for validation with the experimental results. The FE model results showed a good agreement with the experimental results up to the ultimate load, thereafter, the FE model showed some variation from the experimental results. This may attribute to the used values of CFRP damage evolution modeling. A good agreement between the FE force-displacement curves and those obtained from the experimental work was observed as illustrated in **Fig.5-13**. For the comparison between the test and FE studies in terms of failure modes, a good agreement was observed, as shown in **Fig. 5-14**. In general, it can be concluded that the FE models carried out using the ABAQUS software[45] are able to predict the load carrying capacity of CFRP-confined CFSSTCs accurately, to capture the modes of failure observed in testing and to provide force-deformation curves that match well with the test curves.

Table 5-3. Comparison of the test and FE load carrying capacity and ultimate deformation

Specimen	P_{FE} / P_{test}			$\delta_{FE} / \delta_{test}$		
	Imperfection Amplitudes			Imperfection Amplitudes		
	t/10	t/50	t/100	t/10	t/50	t/100
C0-37	1.02	1.02	1.02	0.99	0.99	0.99
C1-37	1.05	1.06	1.06	0.98	0.91	0.91
C2-37	1.02	1.03	1.03	1.05	1.01	1.01
C3-37	1.04	1.04	1.04	0.98	0.94	0.94
C0-50	1.04	1.05	1.05	1.04	1.04	1.04
C1-50	1.13	1.14	1.14	1.15	1.12	1.12
C2-50	1.02	1.02	1.02	1.04	1.02	1.01
C3-50	1.02	0.99	1.01	0.96	0.95	0.94
C0-67	0.93	0.93	0.93	0.98	0.98	0.98
C1-67	1.03	1.03	1.03	1.01	1.01	1.01
C2-67	1.04	1.04	1.04	1.12	1.09	1.09
C3-67	1.03	1.02	1.02	1.00	0.99	0.99
Mean	1.03	1.03	1.03	1.03	1.00	1.00
COV	0.04	0.05	0.05	0.06	0.06	0.06

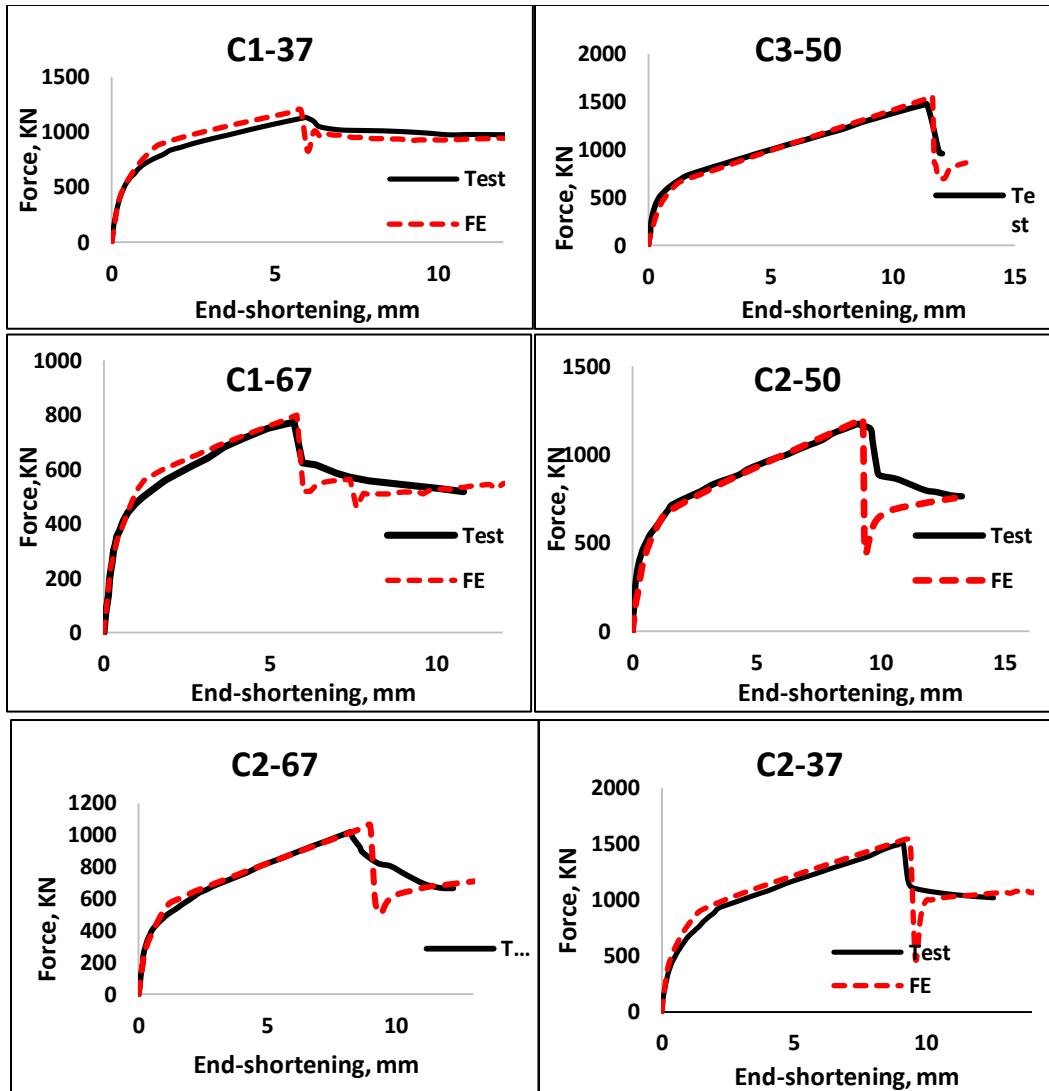


Fig. 5-13. Experimental and numerical force-end shortening for some selected specimens

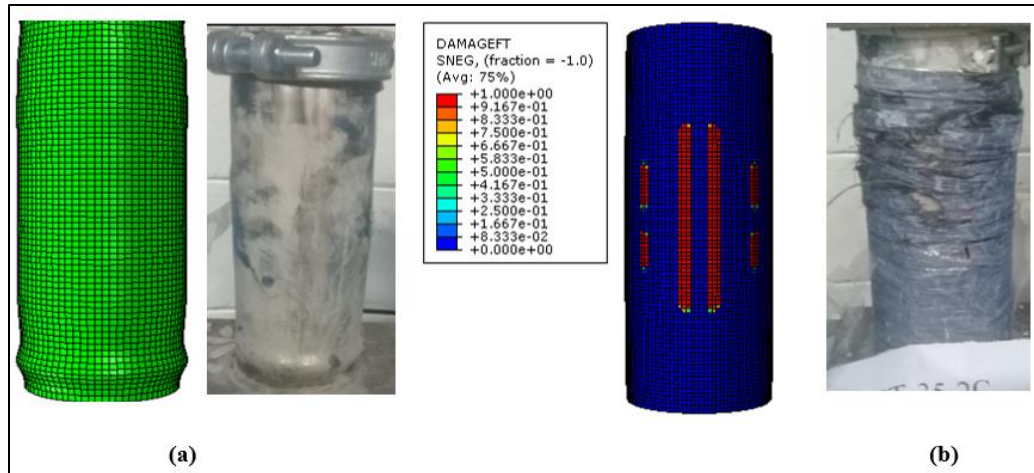


Fig.5-14. General failure mode of (a) unwrapped specimens (b) wrapped specimens.

5.3.7 Comparison of Stainless Steel and Carbon Steel Columns

A comparison is made between CFRP-wrapped CFSSTCs and CFCSTCs. Identical columns, one using stainless steel and the other carbon steel tubes, were structurally evaluated. The stress-strain diagrams for stainless steel are shown in **Fig. 5-15**. The load versus end shortening for both columns are shown in **Fig. 5-16**. The two columns showed identical behavior within the elastic range similar to their stress-strain diagrams. However, stainless steel showed better performance in the inelastic range, as expected from the mechanical properties.

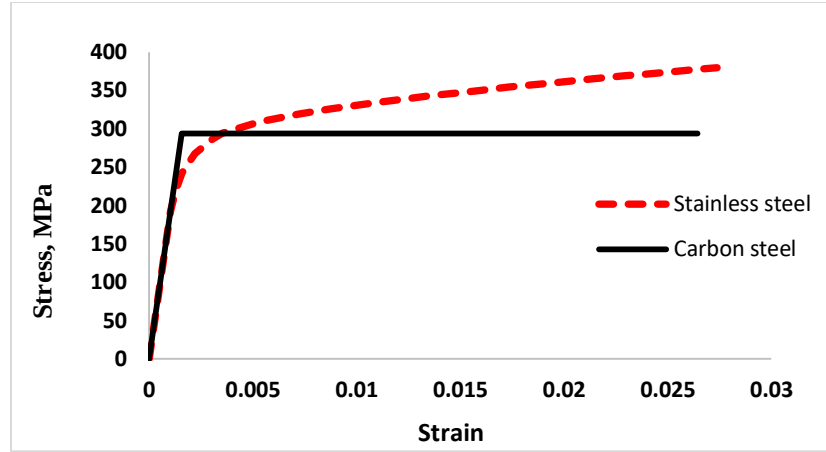


Fig. 5-15: stainless steel and carbon steel stress–strain relationships

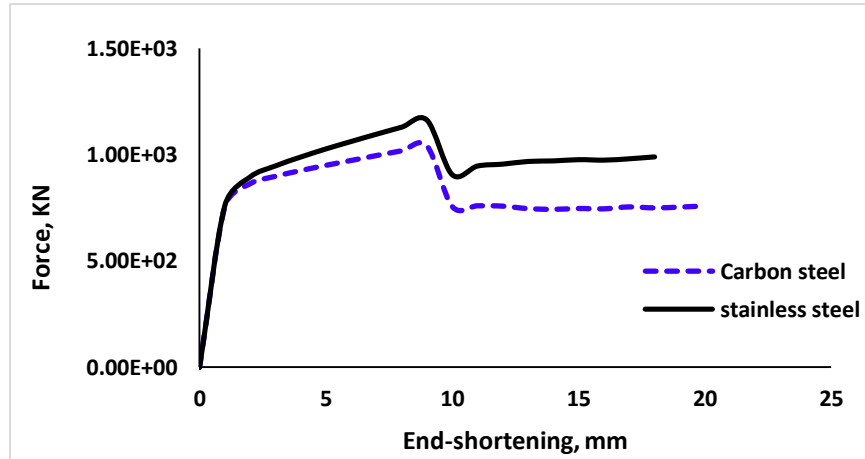


Fig. 5-16: comparison of force-displacement relationships of stainless steel and carbon steel

5.4 FE Parametric Study

Using the validated FE models, 108 models were analyzed using FE to obtain the load carrying capacity (P_{FE}) of CFRP-wrapped CFSSTCs. P_{FE} data was employed to develop a proposed model to predict the load carrying capacity. The austenitic stainless steel material properties of CHSST 101×1.5 and CHSST 114×3.05 were

adopted in the FE parametric study. The normal concrete compressive strength was extended in this FE parametric study to be three levels: 25 MPa, 43.6 MPa and 60 MPa, as well as the D/t_s ratio of the stainless steel tubes which was taken as six levels: 37.5, 50.5, 67.3, 100, 150 and 200. The number of CFRP plies (n) were grouped in four levels of one, two, three and six plies, where the thickness of one ply is 0.29 mm. The ultimate strength of unidirectional CFRP in the fiber direction (σ_f) was 3000 MPa and 1500 MPa. To avoid any flexural global buckling, the L/D ratio of the stainless steel tubes was taken as 3. The parametric study variables discussed above, as well as the P_{FE} , are summarized in **Table 5-4**.

5.5 Analytical Study

A mathematical model to predict the load carrying capacity of CFRP-wrapped CFSSTCs under axial compression is not available in the literature. Therefore, in this study, an analytical model based on curve fitting parameters to predict the load carrying capacity of CFRP-wrapped CFSSTCs was proposed. The stress-strain characteristics of the stainless steel material are different from those of carbon steel material, as discussed in the introduction.

5.5.1 Proposed Model

The load carrying capacity results obtained from the parametric study, as shown in **Table 4-5**, were employed to develop a simple model to predict the load carrying capacity analytically (P_u) of CFRP-confined CFSSTCs under axial compressive load. From an equilibrium state, the load carrying capacity (P_u) is defined as:

$$P_u = P_s + P_{core} \quad (5-3)$$

The CSM approach has been utilized to calculate the local buckling stress as discussed earlier in **Chapter 4**.

$$P_s = A_s \sigma_{LB} \quad (5-4)$$

where A_s is the cross sectional area of the stainless steel tube and σ_{LB} is the local buckling stress.

The axial strength capacity of the concrete infill was defined as:

$$P_{core} = A_c f'_{cc} \quad (5-5)$$

where A_c is the cross-sectional area of the concrete core. The concrete fill is subjected to confinement pressure provided by the CFRP wrap and the stainless steel tube, as illustrated in **Fig. 5-17**. Over the past two decades, many investigations have been conducted to understand the behavior of axial compressive FRP-confined concrete. Ozbakkaloglu et al. [62] summarized, classified and evaluated 88 models proposed, in the literature, to predict the behavior of FRP-confined concrete in circular sections. Many of these existing design-oriented models for FRP-confined concrete take the following simple form that was adopted in this study:

$$f'_{cc} = f'_c + \psi f_l \quad (5-6)$$

where f_l is the lateral confining pressure and ψ is a confinement parameter that was obtained using the curve fitting for the P_{FE} results obtained from the FEMs. The confining pressure generated by the stainless steel tube and the CFRP wrap can be

derived from the force equilibrium of the hoop stresses in these two components, as shown in **Fig. 5-17**, to be expressed as:

$$f_l = \frac{2 \sigma_{\theta} t_s}{D} + \frac{2 \sigma_f t_f}{D} \quad (5-7)$$

where: σ_{θ} is the hoop stress in the stainless steel tube where ($\sigma_{\theta} = 0.1 \sigma_{0.2}$), σ_f is the ultimate tensile strength of the CFRP jacket and t_f is the CFRP jacket thickness.

Eq.5-6 can then be rewritten in the form shown below.

$$f'_{cc} = f'_c + \frac{\psi}{D} (\sigma_{\theta} t_s + \sigma_f t_f) \quad (5-8)$$

Eq. 5-3 can then be rewritten in the following form.

$$P_u = A_s \sigma_{LB} + A_c \left(f'_c + \frac{\psi}{D} (\sigma_{\theta} t_s + \sigma_f t_f) \right) \quad (5-9)$$

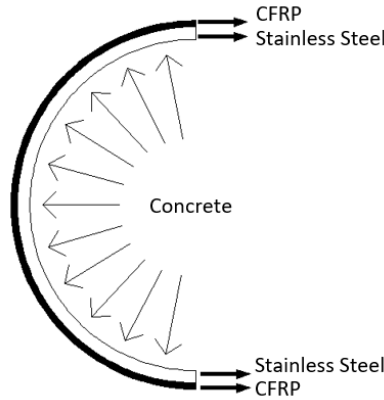


Fig.5-17. Confinement pressure provided by the CFRP and the Stainless steel.

The 108 results of P_{FE} summarized in **Table 5-4** were used to develop a best fit for **Eq. 5-10**. The best representation of the ψ parameter was not a constant or linear

expression; it was a nonlinear formula. The nonlinear expression shown in **Eq.5-10** was obtained from a regression analysis.

$$\psi = 7.2 \times \left(\frac{t_s}{D}\right)^{0.43} \times \left(\frac{\sigma_{0.2}}{f'_c}\right)^{0.3} \times \left(\frac{t_f}{t_s}\right)^{0.08} \quad (5 - 10)$$

Eqs. 5-9 and 5-10 represent the final proposed model to predict the axial load carrying capacity of CFRP-wrapped CFSSTCs. From **Table 5-4**, the proposed analytical model provides accurate estimations, where the mean and COV of the ratio P_{FE}/P_n are, respectively, 1.01 and 0.07. A comparison of the load carrying capacity obtained from the FE parametric study and that obtained from the proposed model is shown in **Fig. 5-18**.

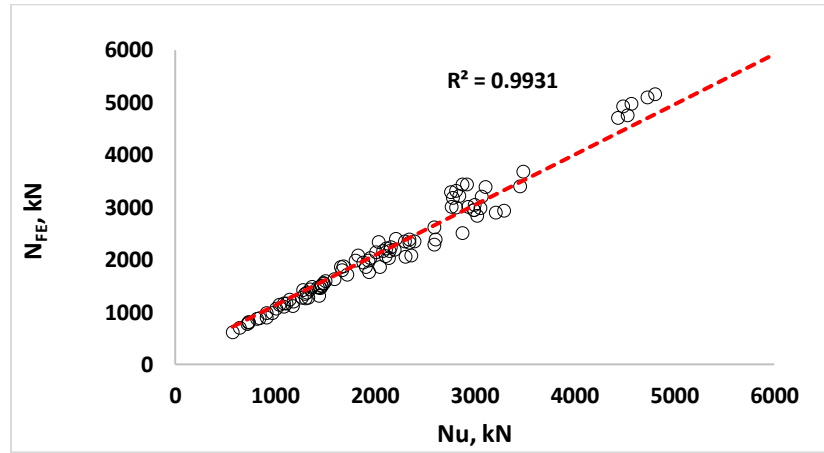


Fig.5-18. Comparison of the load carrying capacity obtained from the FE parametric study and the proposed model.

Table 5-4. Comparison of the FE parametric study results and the proposed model results

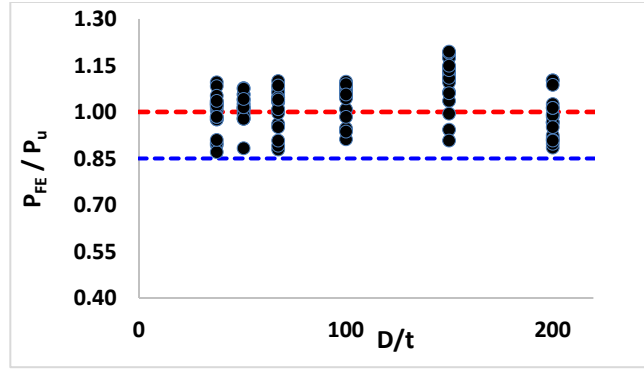
Specimen	D (mm)	t_s (mm)	D/ t_s	f'_c (MPa)	Stainless steel		CFRP		P_{FE} (kN)	P_{FE}/P_u
					$\sigma_{0.2}$ (MPa)	$\sigma_{1.0}$ (MPa)	f_{frp} (MPa)	n		
S1	114.3	3.05	37.5	25	322	365	3000	1	1142	1.08
S2	114.3	3.05	37.5	25	322	365	3000	2	1507	1.01
S3	114.3	3.05	37.5	25	322	365	3000	3	1862	1.00
S4	114.3	3.05	37.5	25	322	365	3000	6	2938	0.88
S5	114.3	3.05	37.5	43.6	322	365	3000	1	1241	1.06
S6	114.3	3.05	37.5	43.6	322	365	3000	2	1585	1.04
S7	114.3	3.05	37.5	43.6	322	365	3000	3	1944	1.03
S8	114.3	3.05	37.5	43.6	322	365	3000	6	2981	0.98
S9	114.3	3.05	37.5	60	322	365	3000	1	1274	0.98
S10	114.3	3.05	37.5	60	322	365	3000	2	1630	1.01
S11	114.3	3.05	37.5	60	322	365	3000	3	1985	1.01
S12	114.3	3.05	37.5	60	322	365	3000	6	3044	1.01
S13	101	2	50.1	25	322	365	3000	1	792	1.08
S14	101	2	50.1	25	322	365	3000	2	1101	1.01
S15	101	2	50.1	25	322	365	3000	3	1450	1.00
S16	101	2	50.1	25	322	365	3000	6	2290	0.88
S17	101	2	50.1	43.6	322	365	3000	1	865	1.06
S18	101	2	50.1	43.6	322	365	3000	2	1162	1.04
S19	101	2	50.1	43.6	322	365	3000	3	1465	1.03
S20	101	2	50.1	43.6	322	365	3000	6	2349	0.98

S21	101	2	50.1	60	322	365	3000	1	898	0.98
S22	101	2	50.1	60	322	365	3000	2	1198	1.01
S23	101	2	50.1	60	322	365	3000	3	1487	1.01
S24	101	2	50.1	60	322	365	3000	6	2380	1.01
S25	101	1.5	67.3	25	322	365	3000	1	698	1.08
S26	101	1.5	67.3	25	322	365	3000	2	979	1.01
S27	101	1.5	67.3	25	322	365	3000	3	1256	0.96
S28	101	1.5	67.3	25	322	365	3000	6	2077	0.88
S29	101	1.5	67.3	43.6	322	365	3000	1	813	1.10
S30	101	1.5	67.3	43.6	322	365	3000	2	1061	1.05
S31	101	1.5	67.3	43.6	322	365	3000	3	1341	1.03
S32	101	1.5	67.3	43.6	322	365	3000	6	2189	1.00
S33	101	1.5	67.3	60	322	365	3000	1	876	1.04
S34	101	1.5	67.3	60	322	365	3000	2	1162	1.07
S35	101	1.5	67.3	60	322	365	3000	3	1434	1.06
S36	101	1.5	67.3	60	322	365	3000	6	2235	1.03
S37	200	2	100	25	322	365	3000	1	1797	1.07
S38	200	2	100	25	322	365	3000	2	2394	1.08
S39	200	2	100	25	322	365	3000	3	3011	1.09
S40	200	2	100	25	322	365	3000	6	4756	1.05
S41	200	2	100	43.6	322	365	3000	1	2024	0.95
S42	200	2	100	43.6	322	365	3000	2	2620	1.01
S43	200	2	100	43.6	322	365	3000	3	3207	1.04
S44	200	2	100	43.6	322	365	3000	6	4966	1.09

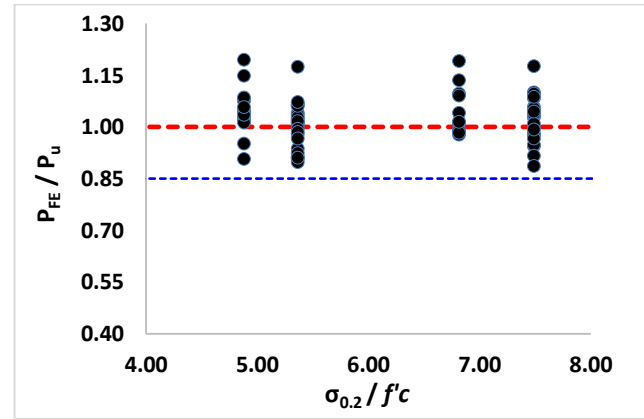
S45	200	2	100	60	322	365	3000	1	2379	0.91
S46	200	2	100	60	322	365	3000	2	2832	0.94
S47	200	2	100	60	322	365	3000	3	3400	0.98
S48	200	2	150	60	322	365	3000	6	5152	1.07
S49	150	1	150	25	322	365	3000	1	977	1.06
S50	150	1	150	25	322	365	3000	2	1420	1.11
S51	150	1	150	25	322	365	3000	3	1862	1.12
S52	150	1	150	25	322	365	3000	6	3208	1.13
S53	150	1	150	43.6	322	365	3000	1	1111	0.94
S54	150	1	150	43.6	322	365	3000	2	1539	1.04
S55	150	1	150	43.6	322	365	3000	3	1978	1.10
S56	150	1	150	43.6	322	365	3000	6	3312	1.18
S57	150	1	150	60	322	365	3000	1	1309	0.91
S58	150	1	150	60	322	365	3000	2	1711	0.99
S59	150	1	150	60	322	365	3000	3	2134	1.06
S60	150	1	150	60	322	365	3000	6	3431	1.17
S61	600	3	200	25	322	365	3000	1	9576	0.97
S62	600	3	200	25	322	365	3000	2	10921	0.98
S63	600	3	200	25	322	365	3000	3	12658	1.03
S64	600	3	200	25	322	365	3000	6	17855	1.10
S65	600	3	200	43.6	322	365	3000	1	14226	0.97
S66	600	3	200	43.6	322	365	3000	2	14413	0.92
S67	600	3	200	43.6	322	365	3000	3	14871	0.89
S68	600	3	200	43.6	322	365	3000	6	19922	0.99

S69	600	3	200	60	322	365	3000	1	18706	0.97
S70	600	3	200	60	322	365	3000	2	18721	0.92
S71	600	3	200	60	322	365	3000	3	19030	0.90
S72	600	3	200	60	322	365	3000	6	22020	0.910
S73	114.3	3.05	37.8	25	293	332	3000	6	2893	0.90
S74	114.3	3.05	37.5	43.6	293	332	3000	6	2939	0.98
S75	114.3	3.05	37.5	60	293	332	3000	6	3000	1.02
S76	101	2	50.5	25	293	332	3000	6	0	1.06
S77	101	2	50.5	43.6	293	332	3000	6	2322	0.99
S78	101	2	50.5	60	293	332	3000	6	2351	1.02
S79	101	1.5	50.5	25	293	332	3000	6	2056	0.89
S80	101	1.5	50.5	43.6	293	332	3000	6	2164	1.01
S81	101	1.5	50.5	60	293	332	3000	6	2212	1.05
S82	200	2	100	25	293	332	3000	6	4701	1.06
S83	200	2	100	43.6	293	332	3000	6	4917	1.10
S84	200	2	100	60	293	332	3000	6	5094	1.08
S85	150	1	150	25	293	332	3000	6	3183	1.14
S86	150	1	150	43.6	293	332	3000	6	3287	1.19
S87	150	1	150	60	293	332	3000	6	3433	1.19
S88	600	3	200	25	293	332	3000	6	17619	1.10
S89	600	3	200	43.6	293	332	3000	6	19702	0.99
S90	600	3	200	60	293	332	3000	6	21783	0.91
S91	114.3	3.05	37.8	25	293	332	1500	6	1763	0.91
S92	114.3	3.05	37.5	43.6	293	332	1500	6	2939	0.98

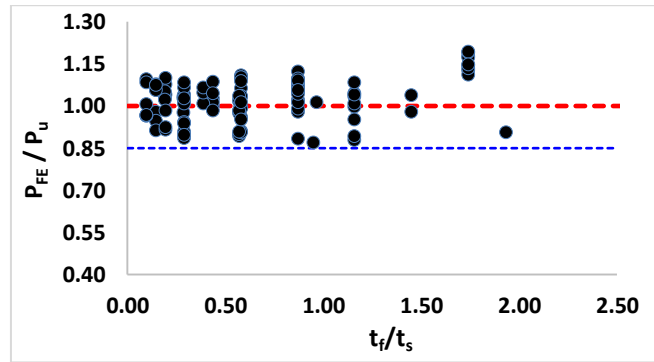
S93	114.3	3.05	37.5	60	293	332	1500	6	2026	1.04
S94	101	2	50.5	25	293	332	1500	6	0	1.07
S95	101	2	50.5	43.6	293	332	1500	6	1469	1.01
S96	101	2	50.5	60	293	332	1500	6	1551	1.04
S97	101	1.5	50.5	25	293	332	1500	6	1266	0.95
S98	101	1.5	50.5	43.6	293	332	1500	6	1372	1.04
S99	101	1.5	50.5	60	293	332	1500	6	1483	1.08
S100	200	2	100	25	293	332	1500	6	2996	1.07
S101	200	2	100	43.6	293	332	1500	6	3386	1.09
S102	200	2	100	60	293	332	1500	6	3682	1.06
S103	150	1	150	25	293	332	1500	6	1873	1.11
S104	150	1	150	43.6	293	332	1500	6	2077	1.13
S105	150	1	150	60	293	332	1500	6	2335	1.15
S106	600	3	200	25	293	332	1500	6	13513	1.09
S107	600	3	200	43.6	293	332	1500	6	17092	1.01
S108	600	3	200	60	293	332	1500	6	20229	0.951
Mean	-	-	-	-	-	-	-	-	-	1.01
COV	-	-	-	-	-	-	-	-	-	0.07



(a)



(b)



(c)

Fig. 5-19. Variation of P_{FE} to P_n ratio as a function of (a) D/t ratio (b) $\sigma_{0.2} / f'_c$ ratio (c) t_f / t_s ratio.

The variations in the load carrying capacity obtained from the FE analysis, P_{FE} , with the load carrying capacity calculated by the proposed model, P_u , versus D/t , $\sigma_{0.2}/f'_c$ and t_f/t_s ratios are displayed in **Fig. 5-19**. From the variations illustrated in **Fig. 5-19**, a reduction factor, Ω , of 0.85 should be taken for P_n to make the proposed model conservative. Therefore, **Eq. 5-9** becomes as follows:

$$\Omega P_u = 0.85 [A_s \sigma_{LB} + A_c \left(f_c + \frac{\psi}{D_s} (\sigma_{\theta} t_s + \sigma_f t_f) \right)] \quad (5 - 11)$$

The above-developed equation is limited to predicting the load carrying capacity of CFRP-wrapped CFSSTCs. In the case of the CFRP-wrapped CFCSTCs, σ_{LB} in **Eq.5-11** should be replaced by the yielding strength of carbon steel, and the confinement parameter, ψ , should be evaluated from the curve fitting data of the CFRP-wrapped filled carbon steel tubes.

5.6 Concluding Remarks

Experimental, numerical and analytical investigations were presented in this chapter to study the behavior of CFRP-wrapped concrete-filled stainless steel tubes under axial compression. External CFRP wrapping was used to restrain the outward local buckling deformation of the stainless steel tubes, as well as to provide additional confinement to the concrete infill. Based on experimental, numerical and analytical investigations, the following conclusions can be drawn:

1. CFRP confined CFSSTC specimens failed because of the rupture of the CFRP due to the lateral expansion of concrete, however, the non-confined

CFSSTC specimens failed due to excess outward local buckling inelastic deformation near one column end.

2. Partial wrapping has not shown noticeable improvement in load carrying capacity.
3. The CFRP jacketing is highly effective in improving the axial compressive behavior of CFSSTCs, in terms of the ultimate strength and the axial shortening capacity.
4. The increase in the CFRP jacket thickness lead to a significant improvement in the ultimate load carrying capacity and the axial shortening capacity.
5. The FE analysis showed that the CFRP-wrapped CFSSTCs were not sensitive to the geometric imperfections.
6. A large number of results obtained from the FE parametric study for many variables were considered in this study in order to propose an effective model for predicting the load carrying capacity of CFRP wrapped CFSSTCS. The predictions of the proposed model differ from the results of the FE by a coefficient of variation of 0.07 and a standard deviation of 1.01. The difference between the FE results and the results obtained from the proposed model is acceptable.

CHAPTER 6

BEHAVIOUR OF CFRP-CONFINED CFSSTCs UNDER ECCENTRIC LOADING

This chapter presents the results of experimental, numerical and analytical investigations of the behavior of circular CFSSTCs confined by CFRP wrap and subjected to eccentric compression loading. The experimental investigation comprised three series of tests. The main variables tested were the load eccentricity-diameter ratio (e/D) and the thickness of the CFRP wrap. 3D finite element (FE) model was developed for CFRP-wrapped CFSSTCs using ABAQUS software and was evaluated with the test results. The validated FE model was employed to generate many results with some variables not included in the experimental work. It was shown from the experimental and FE results that CFRP jacketing was effective in improving the ultimate load carrying capacity under eccentric loading. Finally, an analytical P-M interaction model for CFRP-confined CFSSTCs was proposed and validated against the test and generated FE results to ensure that the predictions are enough conservative.

6.1 Experimental Investigation

The experimental program was designed and performed to evaluate the structural behavior of CFRP-wrapped CFSSTCs under eccentric monotonic compressive loading. **Fig. 6-1** shows a cross-section for CFRP-confined CFSSTCs. Experiments were conducted on a circular hollow stainless steel cross-sectional size of 101×2 , where the first number refers to the tube's outer diameter and the second number represents the tube thickness. The tubular columns were filled by normal concrete. The main variable parameters in the tests were the load eccentricity-diameter ratio (e/D) and the CFRP jacket thickness (t_f). All stub columns were 290 mm in length to ensure that the specimens are suitably short to avoid overall flexural buckling. The test specimens and variables are summarized in **Table 6-1**. The axial loaded specimens were taken from **Chapter 5** to use them as control specimens.

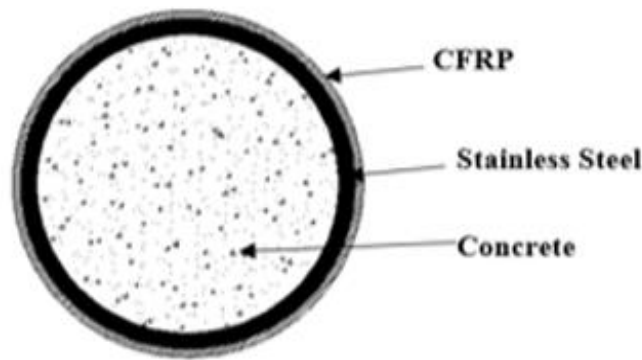


Fig. 6-1. Cross Section of CFRP-wrapped CFSSTC

Table 6-1: Test specimens and variables

Series	Specimen label	Load eccentricity (e) (mm)	e/D	CFRP wrap	
				No. of plies	t _f (mm)
G1	C0-e0	0	0	0	0
	C0-e1	23	0.23	0	0
	C0-e2	33	0.33	0	0
	C0-e3	53	0.52	0	0
G2	C1-e0	0	0	1	0.29
	C1-e1	23	0.23	1	0.29
	C1-e2	33	0.33	1	0.29
	C1-e3	53	0.52	1	0.29
G3	C2-e0	0	0	2	0.58
	C2-e1	23	0.23	2	0.58
	C2-e2	33	0.33	2	0.58
	C2-e3	53	0.52	2	0.58

Each specimen was given a distinctive label starting with the letter C followed by a number referring to the number of CFRP plies. The letter C stands for the CFSSTCs. The second term represents the load eccentricity, where e1, e2 and e3 refer to load eccentricity of 23mm, 33 mm and 53mm, respectively. For example, the label C1-e2 refers to a CFSSTC wrapped by one CFRP ply and subjected to load eccentricity of 33 mm.

6.2 Stub Column Specimens

6.2.1 Specimens Preparation

Twelve stainless steel circular tubes with dimensions of 290mm×101mm×2 mm as mentioned earlier were cut from long cold-rolled tubes. Subsequently, one thick steel plate with dimensions of 200×200×25 mm was welded to the bottom end of the tubes as shown in **Fig.6-2a**, while the top end was left without a plate to cast the concrete in. The tubes were filled with normal concrete with appropriate shaking to avoid any concrete segregation, as displayed in **Fig.6-2b**. After curing, the outer surface of the stainless steel tubes was cleaned to remove any fragments stuck during the concrete casting. To increase adhesion with CFRP, the outer surface of the stainless steel tubes was roughened using sandpapers. A continuous carbon fiber sheet 300 mm in width was wrapped with a sufficient amount of glue around the stainless steel tubes with the required number of layers. An appropriate overlapping length was added to satisfy circumferential continuity. To ensure uniform compressive loading on the specimens, a thin gypsum layer was applied to the top surface to fill any voids, as illustrated in **Fig.6-2c**. A minimum amount of gypsum was used to reduce its effect on the early stages of loading. Carefully, another thick steel plate was welded to the top end of the tubes, as shown in **Fig.6-2d**.



(a)



(b)



(c)



(d)

Fig. 6-2. Stub columns preparation (a) stub columns with the bottom-welded plate (b) Stub columns concrete casting (c) applying a thin gypsum capping (d) stub columns with the two thick plates.

6.2.2 Instrumentation and Testing

Three specimens were subjected to axial loading and nine specimens were subjected to monotonic eccentric loading. **Fig.6-3** shows the test setup, comprised of four

LVDTs: one was located laterally at the mid-height to measure the lateral displacement and the others were located vertically to measure the axial displacements. Two strain gauges were glued longitudinally to all the specimens at mid-height. For the wrapped specimens, strain gauges were affixed to the outer surface of the CFRP composite. A MATEST 3000 kN hydraulic testing machine was used to apply compressive load with a loading rate of 1 kN/s. The LVDTs, strain gauges and load cell readings were connected to the data logger.

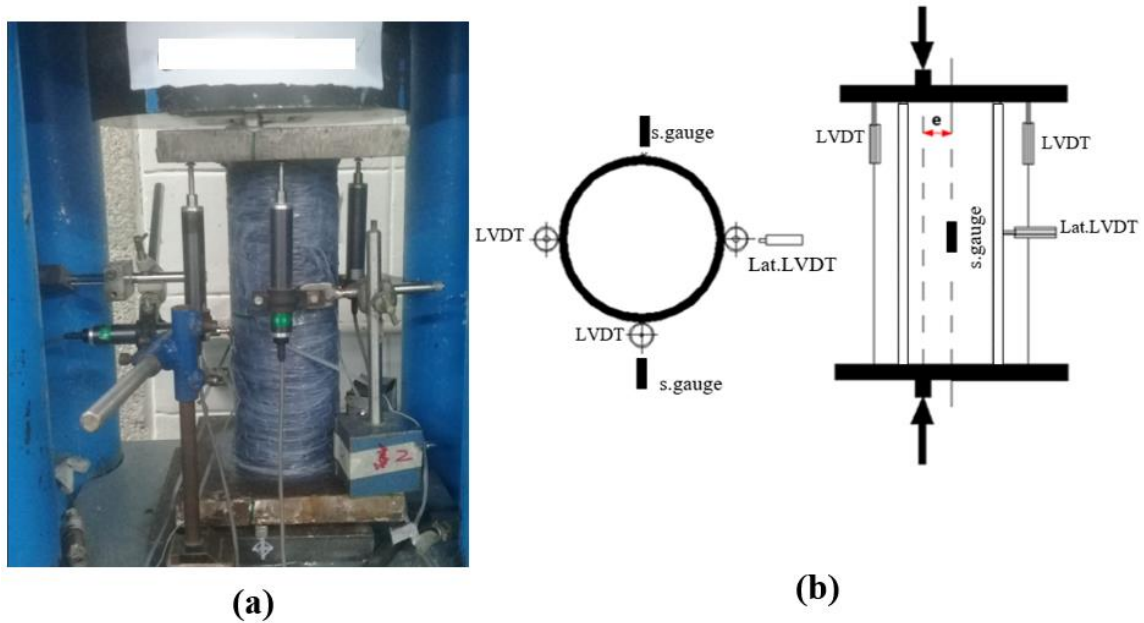


Fig. 6-3. Eccentrically loaded stub column test arrangement: (a) test Setup (b) schematic Diagram of Experimental Setup.

6.3 Experimental Results and Discussion

In this section, the main results of the tested specimens are presented such as load and the ultimate load carrying capacity, axial load versus axial displacement, mid-height lateral displacement versus axial and the failure modes.

6.3.1 Load Carrying Capacity Results

The ultimate load carrying capacities of the wrapped and unwrapped specimens under eccentric loading resulting from the experimental results are summarized in **Table 6-2**. **Table 6-2** presents the ultimate load carrying capacities resulting from experiments (P_{test}), the initial load eccentricity (e), the lateral displacements at the mid-height of the columns (δ_h) corresponding to the ultimate load carrying capacity, the initial bending moment (M_i) calculated as given in **Eq.6-1a** and the secondary bending moment (M_s) calculated as given in **Eq.6-1b**. To highlight the load eccentricity effect on the structural performance of the wrapped specimens, **Table 6-2** includes as well the wrapped and unwrapped specimens subjected to axial loading as reference specimens. The total moment (M_t) summarized in **Table 6-2** refers to the summation of the initial and secondary moments.

$$M_i = P_n \times e \quad (6-1a)$$

$$M_s = P_n \times \delta_h \quad (6-1b)$$

Table 6-2: Experimental key results

series	Specimen	e/D	P _{test} (kN)	δ_h (mm)	M _i kN.m	M _s (kN.m)	M _t (kN.m)
G1	C0-e0	0	600	-	-	-	-
	C0-e1	0.23	341.2	3.43	7.8	1.2	9.0
	C0-e2	0.33	287.3	4.55	9.5	1.3	10.8
	C0-e3	0.52	203.5	3.32	10.8	0.7	11.5
G2	C1-e0	0	800	-	-	-	-
	C1-e1	0.23	500.5	9.60	11.5	4.8	16.3
	C1-e2	0.33	417.7	9.37	13.8	3.9	17.7
	C1-e3	0.52	249.0	7.15	13.2	1.8	15
G3	C2-e0	0	1173	-	-	-	-
	C2-e1	0.23	544.2	8.00	12.5	4.4	16.9
	C2-e2	0.33	436.7	8.00	14.4	2.4	16.8
	C2-e3	0.52	268.0	5.22	14.2	1.4	15.6

6.3.2 Load Eccentricity Effect

The ultimate load carrying capacity versus the load eccentricity ratio (e/D) for all the tested specimens are shown in **Fig. 6-4**. It is seen that the load eccentricity leads to a significant decrease in the ultimate load carrying capacity of the eccentrically loaded specimens compared with corresponding axially loaded specimens. For the axially loaded specimens ($e/D = 0$), the enhancement in the load carrying capacity due to adding one and two CFRP plies was 43 % and 96%, respectively, compared with the unwrapped specimen. These enhancement ratios were 46% and 59% for the eccentrically loaded specimens with e/D of 0.23. For the eccentrically loaded

specimens with e/D of 0.33, the improvement due to bonding one and two CFRP plies was 45% and 52%, respectively, compared with the unwrapped specimen. The improvement of the wrapped specimens compared with the unwrapped specimen were 22% and 32%, respectively, due to adding one and two CFRP plies under load eccentricity ratio of 0.52. As expected, **Fig. 6-4** depicts that the amount of improvement in the load carrying capacity due to adding CFRP wrap decreases with increase in the load eccentricity. This is because that increase of e / D ratio leads to shifting in the neutral axis where the area exposed to compressive stresses becomes lesser.

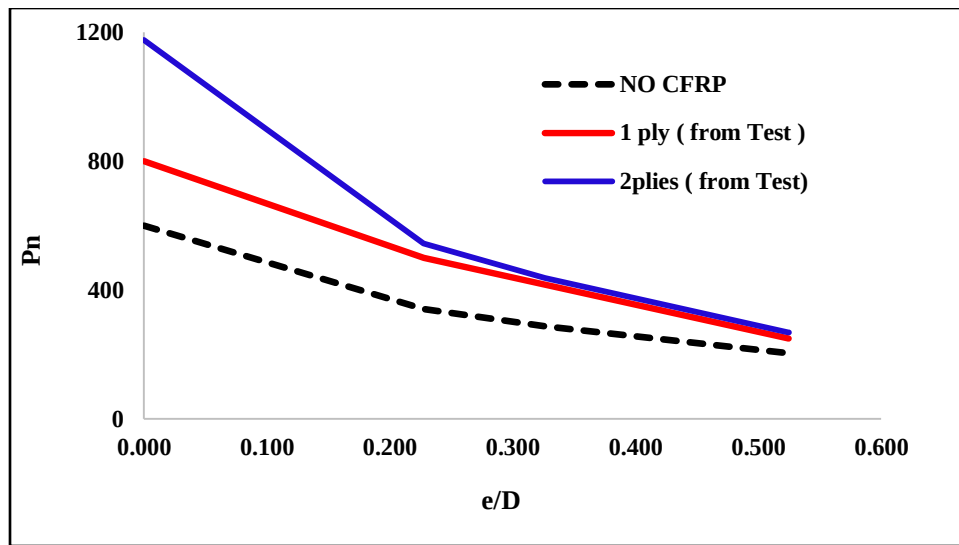


Fig. 6-4: Ultimate load carrying capacity versus load eccentricity ratio

6.3.3 Force-displacement Response

Fig.6-5 shows the force- axial displacement curves of the specimens subjected to axial loading resulting from the experiments. The axial displacement readings were taken from the vertical LVDTs. The vertical strain gauges readings were used to

correct the axial displacement results obtained by vertical LVDTs at the initial loading phase. In the initial loading phase, the end platen and the thin gypsum layer deformations can cause a slight shift in the vertical LVDTs readings. For the unwrapped specimen (C0-e0), load- axial displacement relationship consists of three stages: the first part is a linear line and at this stage, the stainless steel tube and the concrete core behave elastically. The second stage represents a transitional nonlinear part between the linear line and the third portion that is represents the softening part. Similarly, the wrapped specimens under axial loading (C1-e0 and C2-e0) exhibit three stages as shown in **Fig.6-5**: the first portion is a linear part followed by a hardening part because of the high confining pressure applied to the concrete core. The brittle failure of the CFRP jacket appears in the third part as a sudden drop in the load carrying capacity.

Fig.6 -6 shows the force-lateral displacement curves resulting from the experiments of the specimens subjected to eccentric loading. The lateral displacements were measured by the lateral LVDT located at the mid-height. The force-lateral displacement curve of the unwrapped specimen (C0-e1) consists of three stages: The first stage is a linear part representing the elastic behavior of the CFSSTCs. The second stage is a nonlinear part resulting due to the plastic behavior of the CFSSTCs. The third stage represents the softening behavior that was associated with a large end rotation and this softening part was due to the outward local buckling of the stainless steel tube. From the experimental observations, Increasing end rotation of the specimens C0-e2 and C0-e3 led to contact between the tip of the specimen thick plate and the machine end platen. This contact prevented any further rotation.

Therefore, the softening part was not captured as shown in **Fig.6-6**. However, the softening part is not important for this study.

For the wrapped specimens under load eccentricity, the load carrying capacity increases up to the ultimate value and then it drops suddenly due to the CFRP rupture as shown in **Fig.6-6** for the specimens C1-e1 and C1-e2. The force-displacement relationships can be divided into *three stages*. In the first stage, the concrete core and the stainless steel tube behave elastically with no interaction between them because the lateral strain of the concrete core is less than the lateral strain of stainless steel tube as a result that the Poisson's ratio of the concrete core is smaller than that of stainless steel. The force-displacement curves of the unwrapped and wrapped specimens are matching at this first stage as shown in **Fig.6-6** because of the absence of the confining pressure coming from the CFRP wrap and stainless steel tube. In the second stage, the concrete core behaves inelastically and the lateral strain of the concrete core becomes larger than that of the stainless steel tube. The concrete core becomes under confining pressure coming from the stainless steel tube and the CFRP wrap. This second stage appears in the force-displacement relationship as a nonlinear part. The third stage represents the brittle failure of the CFRP wrap and it appears in the force-displacement curves as a sudden drop in the load carrying capacity as shown in **Fig.6-6**. This sudden drop was not captured for the specimens C1-e3, C2-e1, C2-e2 and C2-e3 because of the contact problem between the specimen plate and machine end platen as discussed earlier.

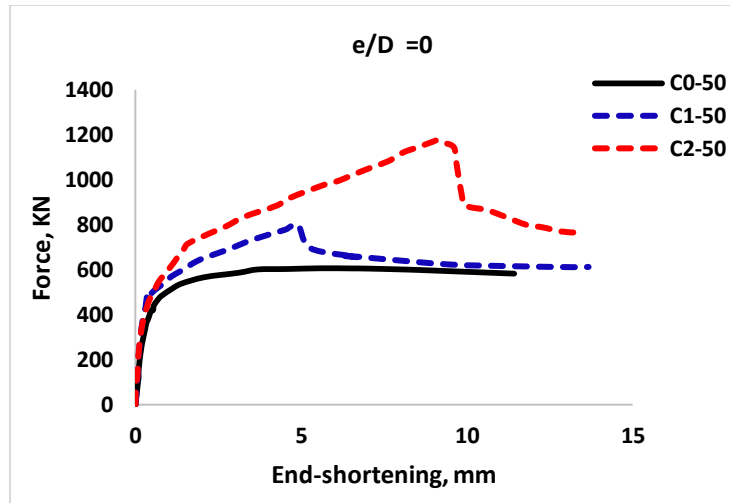
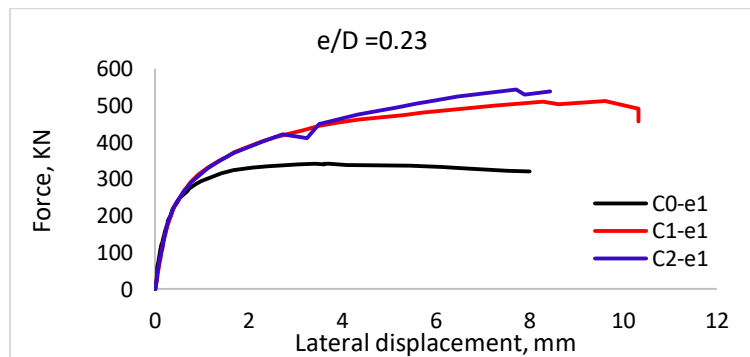
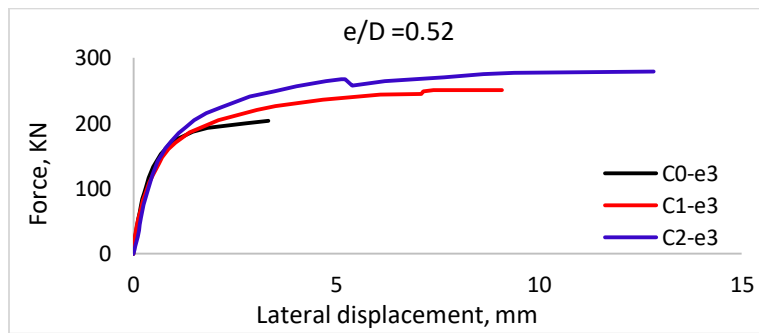


Fig. 6-5: Experimental force-axial displacement of the specimens under axial loading



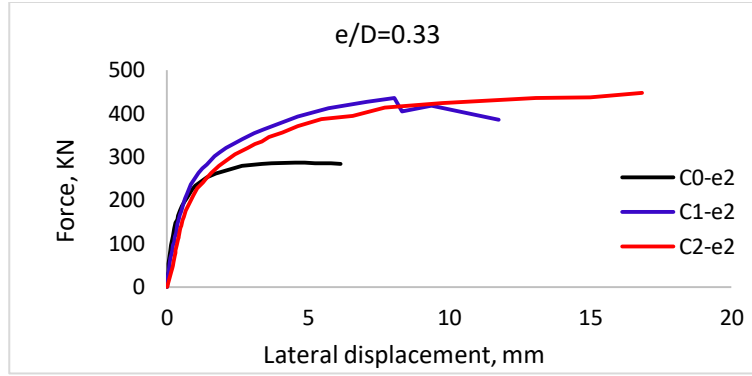


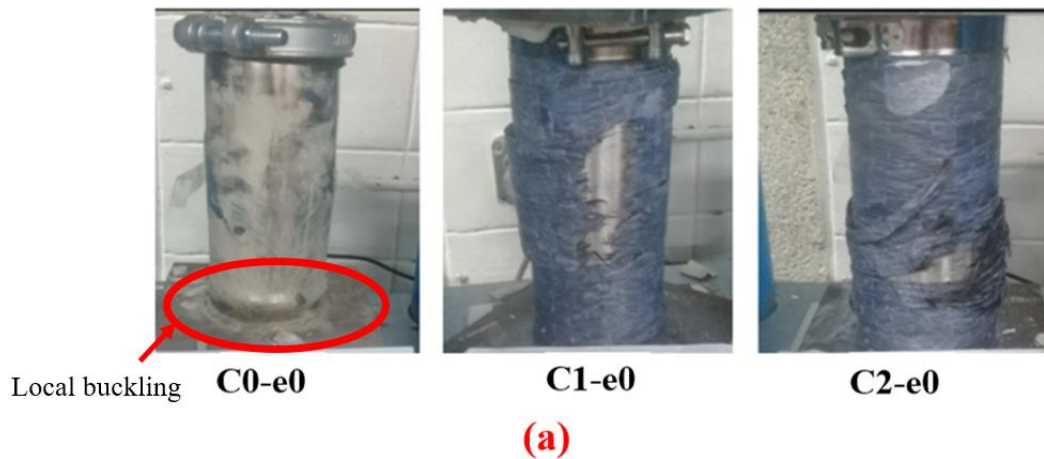
Fig. 6-6: Experimental force-lateral displacement of the specimens under eccentric loading

6.3.4 Failure Modes

The role of the stainless steel, concrete core and CFRP wrap in the CFRP-wrapped CFSSTCs under eccentric loading can be clarified as follows. The stainless steel tube has two main functions: resisting the flexural stresses and confining the concrete that is under compressive stress. For the concrete core, it resists the axial compressive stress, as well as preventing inward local buckling of stainless steel tube. The CFRP wrap postpones the occurrence of outward local buckling of the stainless steel tube and increases confining the concrete that is under compressive stresses. The failure usually occurs when any element losses its function. **Fig.6-7** depicts the failure modes observed in the experiments for the considered specimens in this study. The unwrapped specimen (C0-e0) subjected to axial loading failed due to outward local buckling deformation of the stainless steel tubes, as displayed in **Fig.6-7a**. For the two wrapped specimens (C1-e0, C2-e0) subjected to axial loading, the failure mode was the rupture of the CFRP jacket at the mid height region due to concrete lateral expansion, as indicted in **Fig.6-7a**.

Under eccentric loading, the cross-sections of the unwrapped specimens were subjected to compressive and tensile stresses and this flexural behavior leads to the specimen end rotation. As loading increase, this end rotation continues to increase until the outward local buckling occurs at the specimen mid-height, as shown in **Fig. 6-7b**. The ultimate state of the unwrapped stub specimens can be defined as the state when the outward local buckling of the stainless steel tube occurs at the mid-height associated with excess end rotation.

For the wrapped specimens under eccentric loading, existing of the CFRP wrap leads to postponing the outward local buckling of the stainless steel tubes, as shown in **Fig.6-7c**. It was observed that the wrapped specimens experienced a large end rotation before the CFRP rupture. Therefore, the ultimate state of the wrapped stub column specimens can be considered as the state when the CFRP rupture occurs.



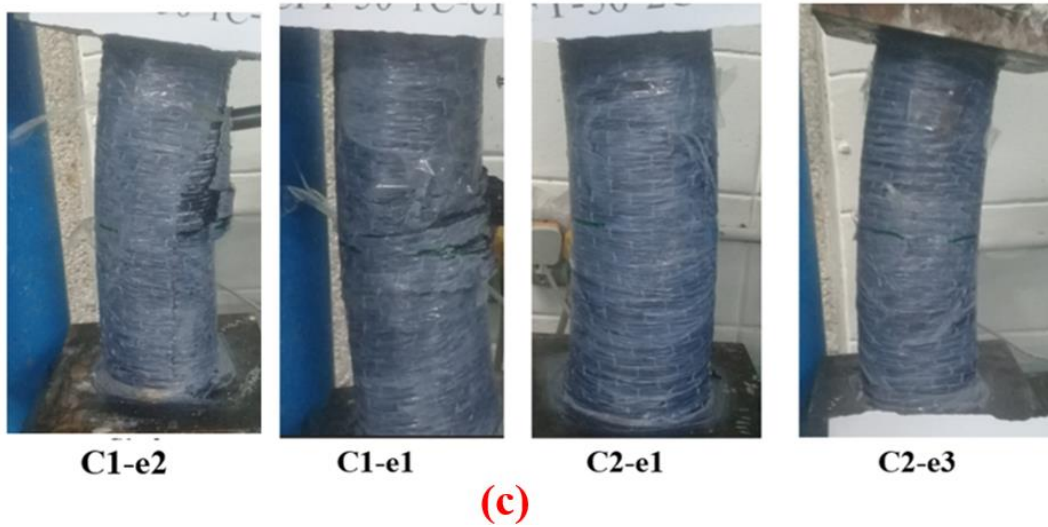
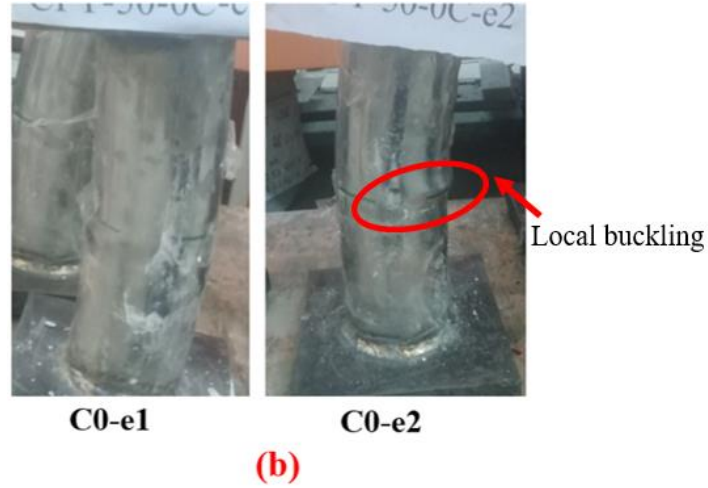


Fig. 6-7: Failure modes (a) axially loaded specimens (b) unwrapped eccentrically loaded specimens (c) wrapped eccentrically loaded specimens

6.4 Finite Element Analysis

The FEA was used to provide more results that were employed together with the test results to evaluate the proposed analytical P-M interaction model presented in section 5. The nonlinear finite element models were carried out using ABAQUS software [45].

6.4.1 Finite Element Types

The axially loaded composite columns were modeled using shell and solid elements obtainable in the element library of the ABAQUS as shown in **Fig. 6-8**. The concrete core was represented using 3D C3D8 solid elements. The circular stainless steel tube was modeled using S4R shell elements which have six DOF at each node making them able to provide accurate predictions for buckling modes. Furthermore, S4R shell elements were employed to model the CFRP wrap. A rigid shell plate was modeled and placed at both ends of the tubular columns to transfer the eccentric loading.

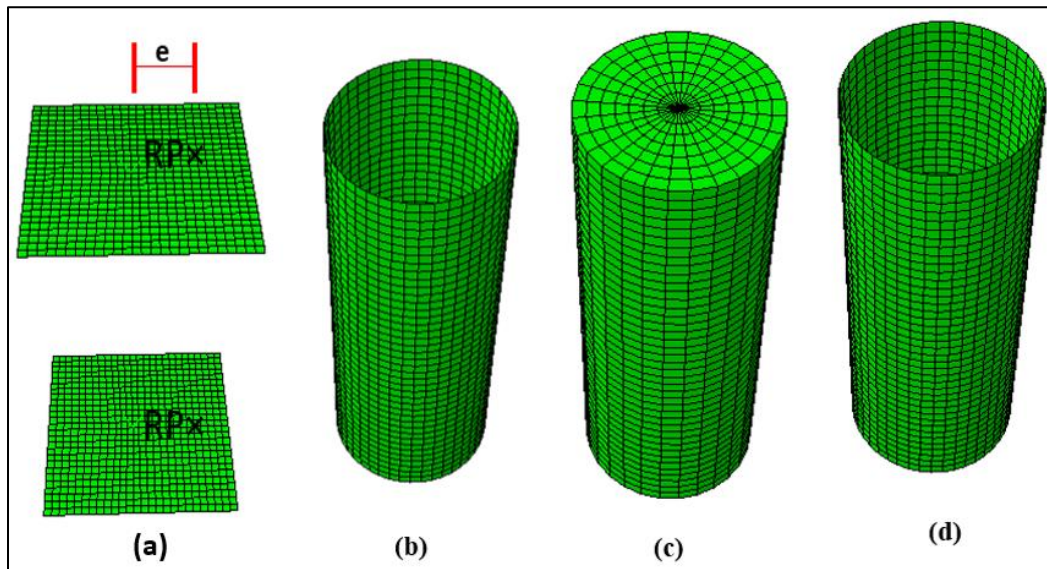


Fig. 6-8: Finite Element modeling (a) rigid plates with eccentric reference points (b) stainless steel tube elements (c) concrete core elements (d) CFRP elements.

6.4.2 Boundary Conditions and Load Application

Each end of the rigid plate was coupled to an eccentric reference point so that all end DOFs were applied to the reference points as point DOFs. At the upper rigid plate, only the rotation about buckling axis and the longitudinal translation were released to simulate pin-ended boundary conditions. On the other hand, all DOFs at the lower rigid plate were restrained except the rotation about buckling axis. The eccentric compressive loading was applied as a displacement control to capture the strain softening behavior.

6.4.3 Initial Imperfections

Modeling of Initial geometric imperfections is similar to what discussed earlier in chapter 3, section 4.3.

6.4.4 Material Modeling

The stainless steel and CFRP materials were modeled in ABAQUS as discussed earlier in sections 4.3.4.1 and 4.3.4.2, respectively. The concrete material was modeled as a tube-confined concrete similar to what is presented earlier in section 5.3.4.1

6.4.5 FE Interaction Modeling

The interaction modeling between the stainless tube and concrete infill was specified as a friction interaction in the tangential direction with a friction coefficient of 0.25 as reported in [61], while the interaction in the normal direction was assigned as a

hard contact to prevent any penetration between the two surfaces. The contact between the stainless steel tube and the CFRP jacket was represented using a tie interaction where the stainless steel surface was assigned to be the master surface. The interaction between the rigid plate surfaces and the steel tube surfaces was specified as a tie contact where the rigid plates were assigned to be the master surfaces. On the other hand, the interaction between the rigid plates and the concrete infill surfaces was introduced using a friction interaction in the tangential direction with a friction coefficient of 0.35 as recommended in [30] and hard contact in the normal direction.

6.4.6 Validation of FE Models

An evaluation of the accuracy of the FE models was performed by comparing the results obtained from the experimental work and the FE models in terms of the ultimate load carrying capacity, force-axial displacement curves, force-lateral displacement curves and the modes of failure. The ratio of the FE to the test load carrying capacity was reported in **Table 6-3**. It is clear from **Table 6-3** that the load carrying capacity results predicted by FE models are very close to the experimental results. A good agreement between the FE force- deformation curves and those obtained from the experimental work was observed as illustrated in **Figs.6-9 and 6-10** for the force-axial displacement and force-lateral displacement curves, respectively. For the comparison between the test and FE studies in terms of failure modes, a good agreement was observed, as shown in **Fig.6-11(a-d)** for the unwrapped axially loaded specimen, wrapped axially loaded specimens, unwrapped

eccentrically loaded specimens and wrapped eccentrically loaded specimens, respectively. In general, it can be concluded that the FE models carried out using the ABAQUS are able to predict the load carrying capacity of CFRP-confined CFSSTCs accurately, to capture the modes of failure observed in testing and to provide force-deformation curves that match well with the test curves.

Table 6-3: comparison of experimental and numerical load carrying capacity results

series	Specimen	e/D (mm)	P _{test} (kN)	P _{FE} (kN)	P _{FE} / P _{test}
G1	C0-e0	0	600	-	-
	C0-e1	0.23	341.2	3.43	7.8
	C0-e2	0.33	287.3	4.55	9.5
	C0-e3	0.52	203.5	3.32	10.8
G2	C1-e0	0	800	-	-
	C1-e1	0.23	500.5	9.60	11.5
	C1-e2	0.33	417.7	9.37	13.8
	C1-e3	0.52	249.0	7.15	13.2
G3	C2-e0	0	1173	-	-
	C2-e1	0.23	544.2	8.00	12.5
	C2-e2	0.33	436.7	8.00	14.4
	C2-e3	0.52	268.0	5.22	14.2

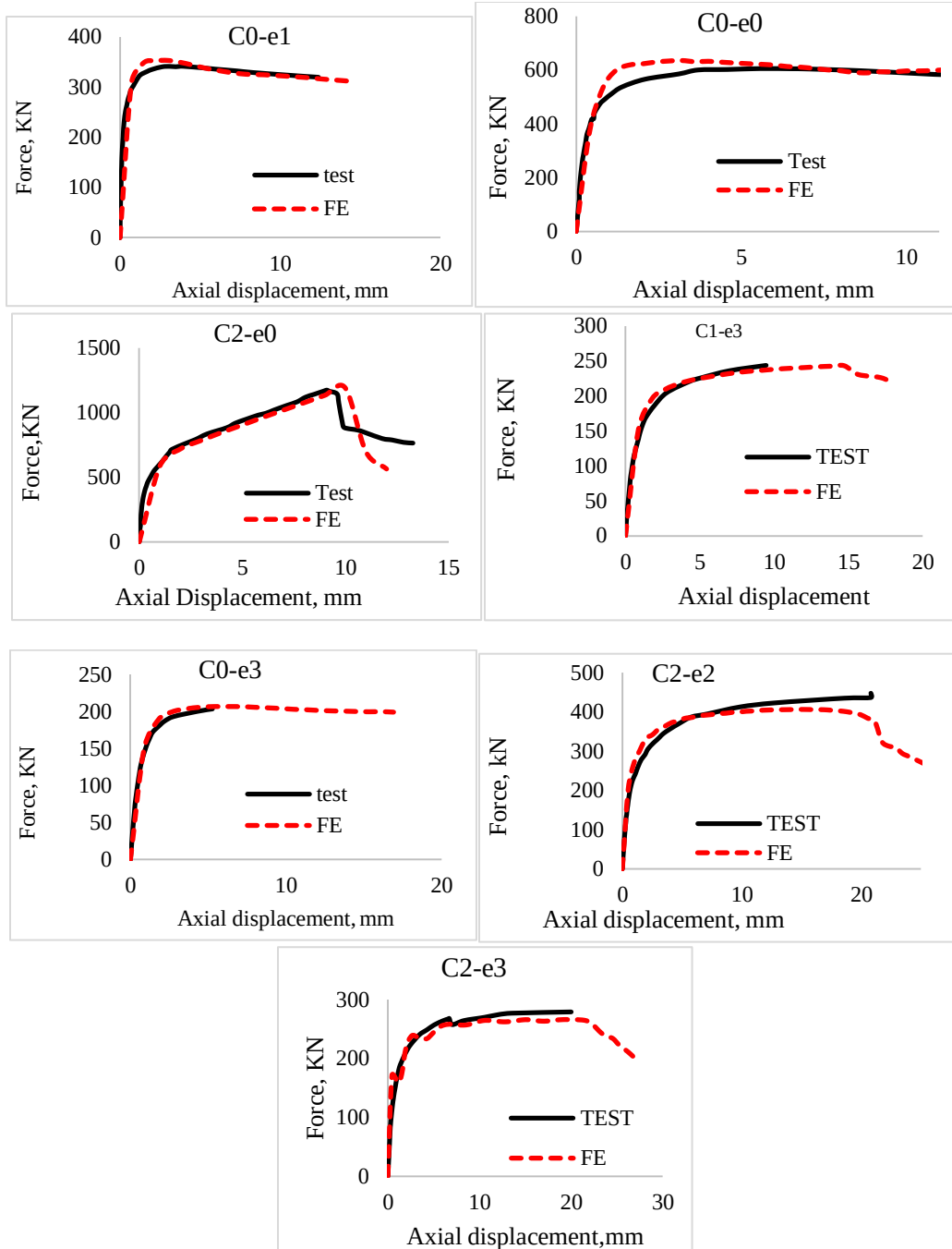


Fig. 6-9: Experimental and numerical force-axial displacement of the specimens under eccentric loading

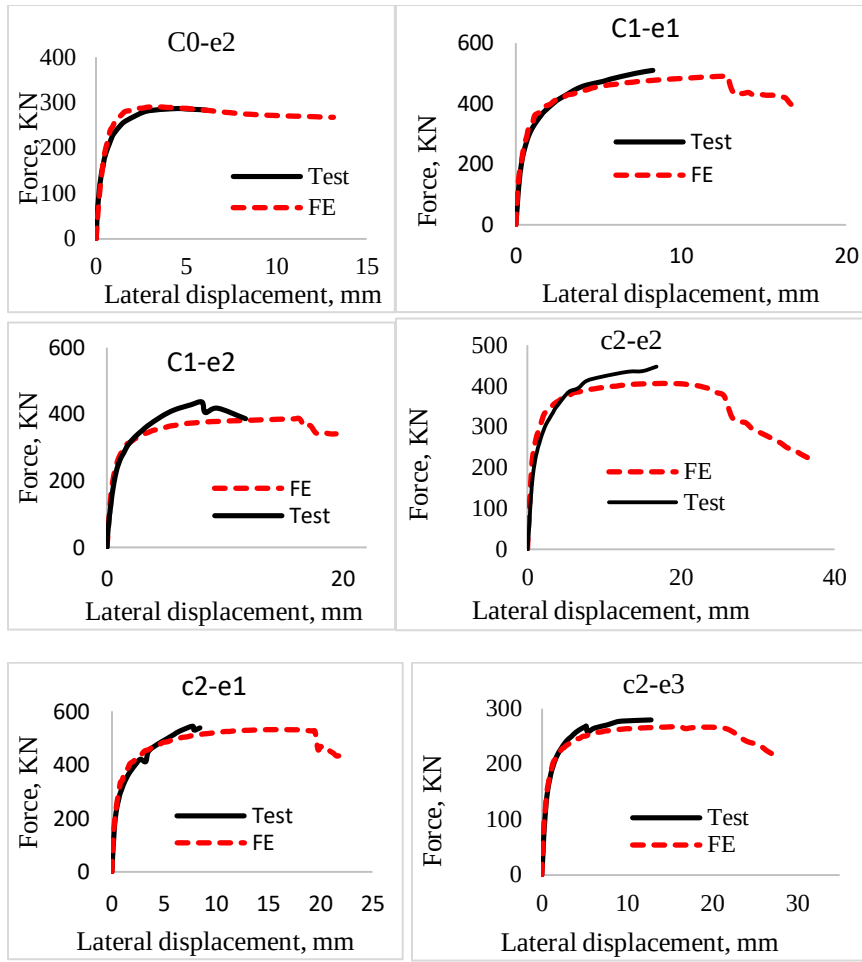


Fig. 6-10: Experimental and numerical force-lateral displacement of the specimens under eccentric loading

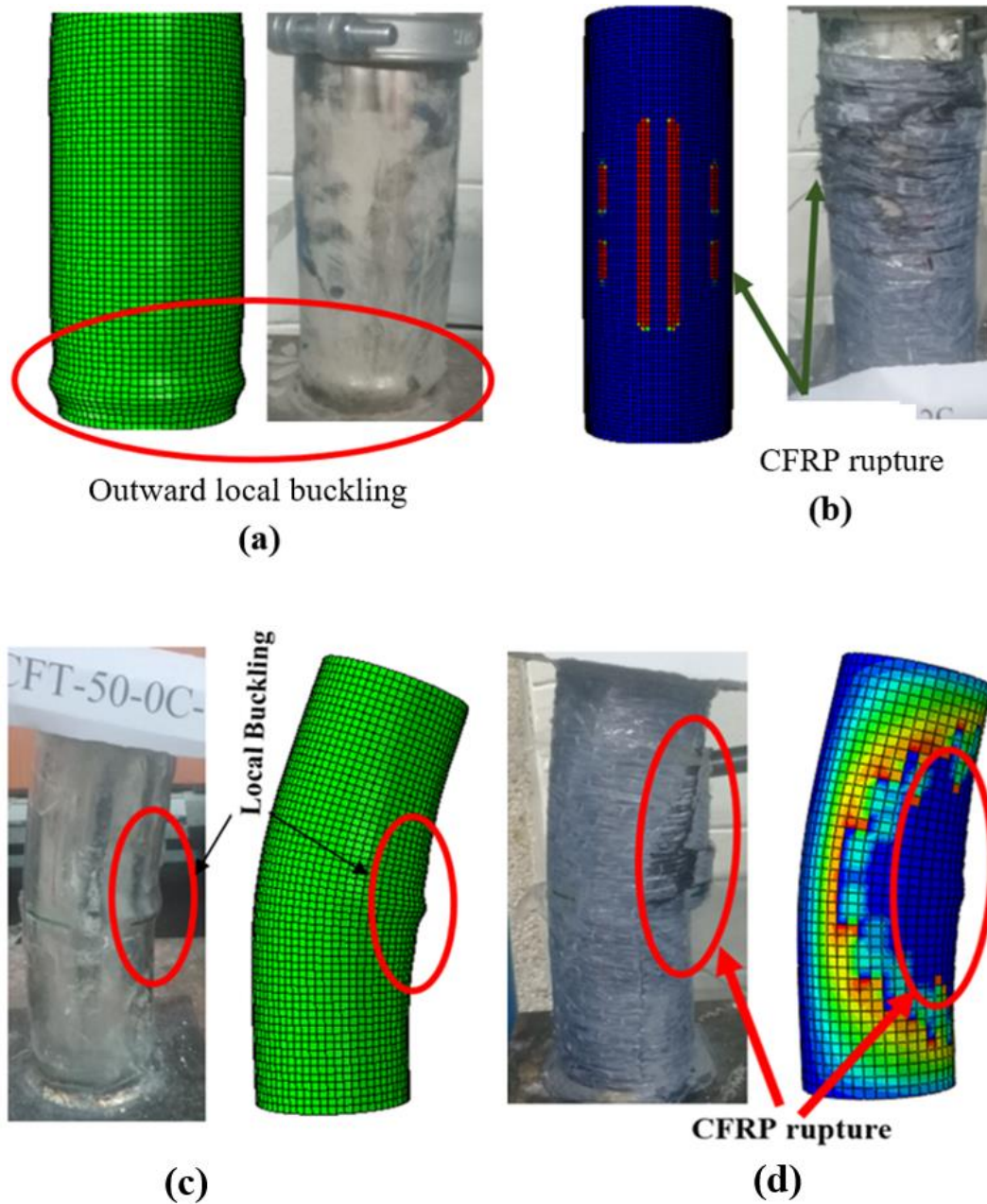


Fig. 6-11: Failure modes of (a) the unwrapped axially loaded specimen (b) the wrapped axially loaded specimens (c) the unwrapped eccentrically loaded specimens (d) the wrapped eccentrically loaded specimens.

6.4.7 FE Parametric Study

Using the validated FE models, extensive FE models with different levels of variables were analyzed using FE analysis to obtain the load carrying capacity (P_{FE}). The FE and experimental results were employed to verify that the proposed P-M interaction model discussed in the next section could provide conservative predictions. The parametric study variables were D/t ratio, e/D ratio as well as CFRP thickness. The material properties of the concrete, stainless steel and CFRP materials were taken from experimental work. **Table 6-4** shows the FE parametric study variables and the obtained load carrying capacity results.

Table 6-4: FE parametric study results

label	Stainless steel			e/D	CFRP wrap		P_{FE} (kN)	M_i (kN.m) $P_{FE} \times e$
	D (mm)	t_s (mm)	D/ t_s		No. of plies	t_f (mm)		
F1	150	4	37.5	0	1	0.29	1972	0
	150	4	37.5	0.10	1	0.29	1439	21.59
	150	4	37.5	0.30	1	0.29	935	42.08
	150	4	37.5	0.50	1	0.29	645	48.38
	150	4	37.5	0.75	1	0.29	461	51.85
F2	150	4	37.5	0	2	0.58	2398	0
	150	4	37.5	0.10	2	0.58	1685	25.28
	150	4	37.5	0.30	2	0.58	1060	47.7
	150	4	37.5	0.50	2	0.58	715	53.63
	150	4	37.5	0.75	2	0.58	502	56.44

F3	150	4	37.5	0	4	1.16	3263	0
	150	4	37.5	0.10	4	1.16	2144	32.16
	150	4	37.5	0.30	4	1.16	1243	55.94
	150	4	37.5	0.50	4	1.16	796	59.72
	150	4	37.5	0.75	4	1.16	555	62.44
F4	200	3	66.6	0	1	0.29	2738	0
	200	3	66.6	0.10	1	0.29	2008	40.15
	200	3	66.6	0.30	1	0.29	1237	74.2
	200	3	66.6	0.50	1	0.29	818	81.76
	200	3	66.6	0.75	1	0.29	551	82.69
F5	200	3	66.6	0	2	0.58	3388	0
	200	3	66.6	0.10	2	0.58	2360	47.19
	200	3	66.6	0.30	2	0.58	1379	82.77
	200	3	66.6	0.50	2	0.58	886	88.57
	200	3	66.6	0.75	2	0.58	595	89.31
F6	200	3	66.6	0	4	1.16	4582	0
	200	3	66.6	0.10	4	1.16	3035	60.69
	200	3	66.6	0.30	4	1.16	1591	95.43
	200	3	66.6	0.50	4	1.16	951	95.13
	200	3	66.6	0.75	4	1.16	625	93.69

6.5 Analytical Study

6.5.1 Assumptions of the Proposed Model

Two simplified approaches were utilized to construct P-M interaction diagrams for reinforced concrete beam-column elements based on sectional analysis: numerical integration approach as reported in [63,64] and strain compatibility and forces equilibrium approach, as referred in [64–66]. In this study, the strain compatibility and force equilibrium approach was employed to develop the P-M interaction model for CFRP-confined CFSSTCs. Thereafter, the results obtained from the developed P-M interaction model were compared with those obtained from experimental and FE results to ensure that the developed P-M interaction model can provide conservative results, as discussed later. The compatibility and equilibrium method was used under the following assumptions:

- (a) Plane sections before bending remain plane sections after bending
- (b) tensile strength of the concrete is ignored
- (c) Perfect bond between the stainless steel tube and the CFRP jacket
- (d) Distribution of the compressive stresses in the concrete was idealized by equivalent rectangular block.

6.5.2 Modeling Of Stainless Steel Tube

Unlike carbon steel, the stainless steel material shows a nonlinear stress-strain relationship with no strictly defined yielding point, as discussed earlier in the

experimental work. To simplify the calculations, the stress-strain relationship of the stainless steel material was idealized as a linear elastic-perfectly plastic model.

6.5.3 Modeling of Concrete Core

Concrete core is subjected to confining pressure from the stainless steel tube and the CFRP jacket, therefore concrete core should be modeled as a confined concrete. Thus, the ultimate total confined concrete strength (f'_{cc}) is required to model the concrete core.

The stress-strain model shown in **Fig.6-12** which was proposed by Lam and Teng [67] for FRP-confined concrete columns under axial loading was adopted in this study to calculate the ultimate FRP-confined concrete strength ($f'_{cc,f}$). Jiang and Teng [68] reported that the Lam and Teng model can be used for the columns subjected to eccentric loading. Lam and Teng stress-strain model is given by the following equations:

$$f_{cc,f} = \begin{cases} E_c \varepsilon_{cc} - \frac{(E-E_2)^2}{4 f'_c} (\varepsilon_{cc})^2, & 0 \leq \varepsilon_{cc} \leq \varepsilon'_t \\ f'_c + E_2 \varepsilon_{cc}, & \varepsilon'_t \leq \varepsilon_{cc} \leq \varepsilon_{ccu} \end{cases} \quad (6-2a)$$

$$\varepsilon'_t = \frac{2 f'_c}{E_c - E_2} \quad (6-2b)$$

$$E_2 = \frac{f'_{cc,f} - f'_c}{\varepsilon_{ccu}} \quad (6-2c)$$

where: $f'_{cc,f}$, ε_{cc} and ε'_t are, respectively, axial compression stress of the FRP-confined concrete, axial compression strain of the FRP-confined concrete and the transition

strain between the nonlinear portion and the second linear portion as displayed in **Fig.6-12**. E_c and E_2 are the modulus of elasticity of the unconfined concrete and the tangent of the second linear part, respectively. The ultimate FRP-confined concrete strength, $f'_{cc,f}$, for a circular cross-section is given as follows:

$$f'_{cc,f} = f'_c + 3.3 f_{l,a} \quad (6 - 3)$$

where: $f_{l,a}$ is the actual confining pressure due to CFRP wrap. $f_{l,a}$ was determined as suggested by Lam and Teng as follows:

$$f_{l,a} = \frac{2 t_f E_f \varepsilon_{ef}}{D} \quad (6 - 4)$$

where: t_f , E_f and ε_{ef} are, respectively, CFRP jacket thickness, the elastic modulus of the FRP material and the effective strain in the FRP wrap at failure and can be determined as given in **Eq.6-5**:

$$\varepsilon_{ef} = 0.586 \frac{\sigma_f}{E_f} \quad (6-5)$$

where: σ_f is the ultimate tensile strength of the CFRP coupons as reported by Lam and Teng. The coefficient, 0.586, is used only for the CFRP composite not for the other FRP materials. For a member subjected to eccentric compression loading, the effective strain in the FRP wrap at failure, ε_{ef} , should be not greater than 0.004 as recommended by ACI-440 [69]. Based on experiments conducted by Lam and Teng, the ratio $\frac{f_{l,a}}{f'_c}$ should be greater than 0.08.

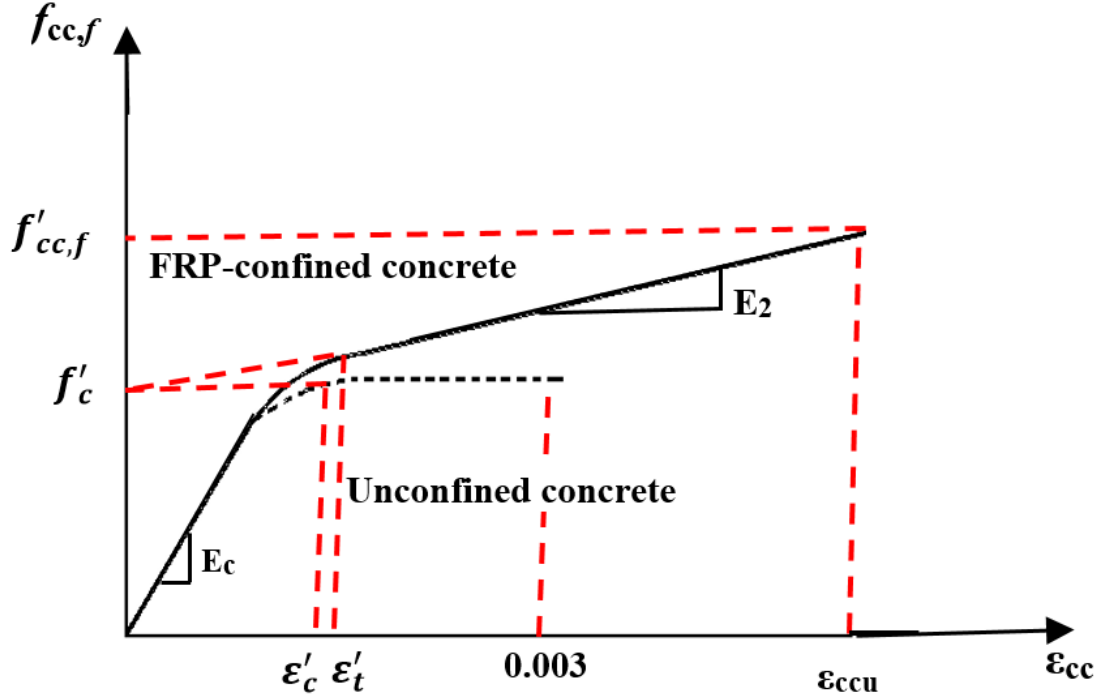


Fig. 6-12: Stress–strain model of FRP-confined concrete proposed by Lam and Teng [67]

The ultimate confined concrete strain (ε_{ccu}) is determined as suggested by Lam and Teng as follows:

$$\varepsilon_{ccu} = \varepsilon_{co} \left(1.75 + 5.53 \frac{f_{l,a}}{f'_c} \left(\frac{\varepsilon_{ef}}{\varepsilon_{co}} \right)^{0.45} \right) \quad (6-6)$$

where: ε_{co} is the ultimate unconfined concrete strain that is taken as 0.003. For a member subjected to a pure compression, ε_{ccu} should not exceed 0.01 to prevent excessive cracking as recommended by ACI-440. When value of ε_{ccu} exceeds 0.01, the corresponding $f'_{cc,f}$ should be recalculated **from Eq.6-2** to be smaller at this strain level.

The ultimate tube-confined concrete strength, $f'_{cc,t}$, was given earlier in Chapter 5 ,**Eq.5-2**. In this study, the concrete core is subjected to confining pressures coming from the CFRP jacket as well as stainless steel tube. Therefore, the total ultimate confined concrete strength (f'_{cc}) is given as follows.

$$f'_{cc} = f'_c + (3.3 f_{l,a} + 4.1 f_{l,steel}) \quad (6 - 7)$$

6.5.4 Proposed P-M Interaction Model

The three elements in the cross-section of the CFRP-confined CFSSTCs were reduced into two components: the confined concrete core and the stainless steel tube. The ultimate state of the CFRP-confined CFSSTCs was considered when the confined concrete reaches (ϵ_{ccu}).

The nonlinear stress distribution of the concrete was represented as a uniform rectangular distribution using two parameters: 0.85 and γ , as shown in **Fig. 6-13**.

The parameter, γ , can be determined using **Eq. 6 – 8**, as reported in ACI 318-11.

$$\gamma = 0.85 - 0.05 \left(\frac{f'_c - 28 \text{ MPa}}{7 \text{ MPa}} \right), \quad 0.65 \leq \gamma \leq 0.85 \quad (6 - 8)$$

From **Fig. 6-13**, the relationships between the angles (θ and Φ) and distances (c and a) are given as follows.

$$\theta = 2 \cos^{-1} \left(1 - \frac{c}{r^*} \right) \quad (6 - 9a)$$

$$\Phi = 2 \cos^{-1} \left(1 - \frac{a}{r^*} \right) \quad (6 - 9b)$$

$$a = \gamma \times c \quad (6-9c)$$

$$r^* = \frac{r+R}{2} = \frac{d}{2} \quad (6-9d)$$

Where: c and a are the depth of the neutral axis and the effective depth of the neutral axis, respectively. R , r , d and r^* are, respectively, the outer radius, inner radius, the diameter measured from center-to-center of the stainless steel tube and the radius measured to the center of the stainless steel tube. The area of the confined concrete subjected to compressive stresses, shown in **Fig. 6-13** as a hatched area, can be calculated from the geometrical relationships as given in **Eq.6-10a**.

$$A_{\text{conc}} = \frac{r^2}{2} [\Phi - \sin \Phi] \quad (6-10a)$$

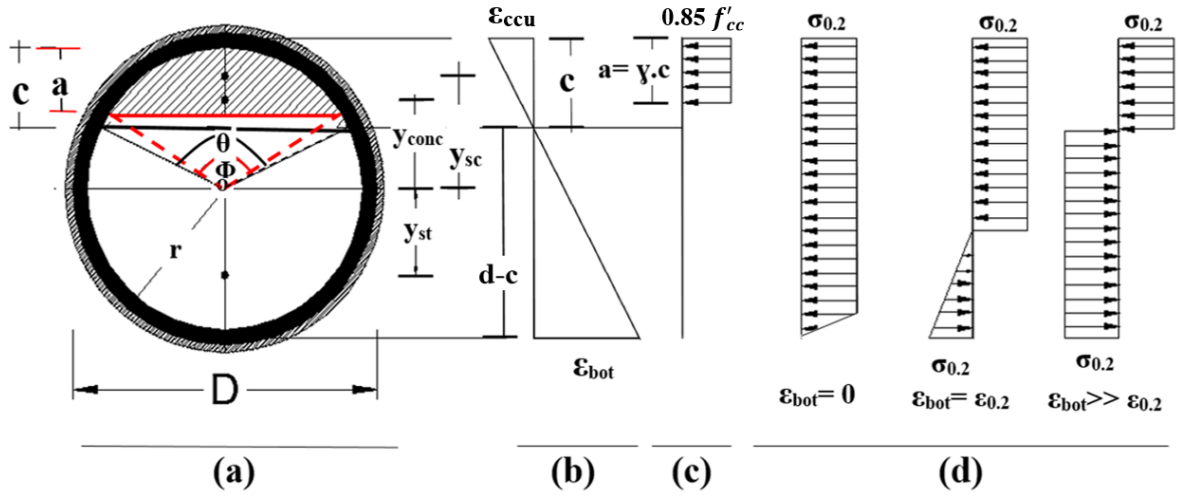


Fig. 6-13: Stress and strain distributions over the cross section (a) column cross-section (b) strain distribution (c) confined concrete stress distribution (d) stainless steel stress distribution.

The stainless steel areas subjected to compressive stresses (A_{sc}) and tensile stresses (A_{st}) were calculated by **Eq. (6 – 10b)** and **Eq. (6 – 10c)**, respectively.

$$A_{sc} = \theta(r^*)t_s \quad (6 - 10b)$$

$$A_{st} = (2\pi - \theta)(r^*)t_s \quad (6 - 10c)$$

The distances from the center of the column cross-section (o) to the centers of A_{conc} , A_{sc} and A_{st} areas were determined using geometrical relationships as follows, respectively.

$$y_{conc} = \frac{4}{3}r \left[\frac{(\sin \frac{\phi}{2})^3}{\phi - \sin \phi} \right] \quad (6 - 11a)$$

$$y_{sc} = r^* \left[\frac{\sin(\frac{\theta}{2})}{\frac{\theta}{2}} \right] \quad (6 - 11b)$$

$$y_{st} = r^* \left[\frac{\sin(\frac{2\pi - \theta}{2})}{\frac{2\pi - \theta}{2}} \right] \quad (6 - 11c)$$

As shown in **Fig.6-13**, the strain at the bottom (ϵ_{bot}) can be easily determined by the similarity of triangles as follows.

$$\epsilon_{bot} = \epsilon_{ccu} \left(\frac{d-c}{c} \right) \quad (6 - 12)$$

For simplified calculation, instead of a continuous curve, a conservative P-M interaction diagram was proposed as straight lines joining the nominal axial load and bending moment values corresponding to five characteristic points as illustrated in **Fig. 6-14**.

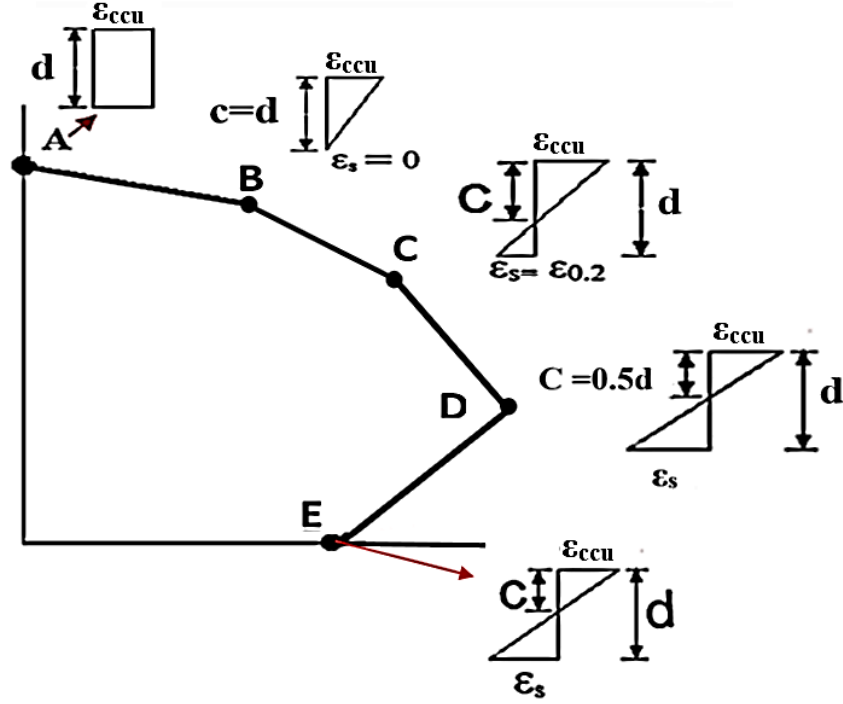


Fig. 6-14: Representative simplified P-M interaction diagram

- **Point A:** the point corresponding to the pure axial load and zero bending moment of. The nominal axial load, $P_{n,A}$, can be found using **Eq. 6-13**.

$$P_{n,A} = 0.85 f'_{cc} A_c + \sigma_{0.2} A_s \quad (6-13)$$

Where: A_c and A_s are the gross cross-section areas of the concrete core and stainless steel tube, respectively.

- **Point B:** this point characterizes the strain distribution corresponding to a maximum compressive strain ϵ_{ccu} at the top and zero strain at the bottom. The stress distribution of the stainless steel tube is shown in **Fig.6-13**. The nominal axial load $P_{n,B}$ and bending moment $M_{n,B}$ are given in **Eqs.6-14a and 6-14b**, respectively. The depth of the neutral axis, c , equals d , as displayed in **Fig.6-14**.

$$P_{n,B} = (0.85 f'_{cc} \times A_{conc}) + (A_{sc} \times \sigma_{0.2}) \quad (6 - 14a)$$

$$M_{n,B} = (0.85 f'_{cc} \times A_{conc}) y_{conc} + (A_{sc} \times \sigma_{0.2}) y_{sc} \quad (6-14b)$$

- **Point C:** this point characterizes the strain distribution corresponding to the balanced failure with a maximum compressive strain ϵ_{ccu} at the top and a yielding tensile strain $\epsilon_{0.2}$ at the bottom. The tensile stress distribution of the stainless steel tube is a triangle shape, as shown in **Fig.6-13**. The nominal axial load $P_{n,C}$ and bending moment $M_{n,C}$ are given in **Eqs.6-15a and 6-15b**, respectively. The depth of the neutral axis, c , should be determined as $c = d \left(\frac{\epsilon_{ccu}}{\epsilon_{ccu} + \epsilon_{0.2}} \right)$.

$$P_{n,C} = (0.85 f'_{cc} \times A_{conc}) + (A_{sc} \times \sigma_{0.2}) - 0.5 (A_{st} \times \sigma_{0.2}) \quad (6 - 15a)$$

$$M_{n,C} = (0.85 f'_{cc} \times A_{conc}) y_{conc} + (A_{sc} \times \sigma_{0.2}) y_{sc} + 0.5 (A_{st} \times \sigma_{0.2}) y_{st} \quad (6-15b)$$

- **Point D:** this point corresponding to the maximum bending moment. The nominal axial load $P_{n,D}$ and bending moment $M_{n,D}$ are given in **Eqs.6-16a and 6-16b**, respectively. By iteration, the depth of the neutral axis should be found where it makes $M_{n,D}$ a maximum value. By analysis of many cases in this study, the maximum value of $M_{n,D}$ can be found approximately at $c=0.5d$.

$$P_{n,D} = (0.85 f'_{cc} \times A_{conc}) + (A_{sc} \times \sigma_{0.2}) - (A_{st} \times \sigma_{0.2}) \quad (6-16a)$$

$$M_{n,D} = (0.85 f'_{cc} \times A_{conc}) y_{conc} + (A_{sc} \times \sigma_{0.2}) y_{sc} + (A_{st} \times \sigma_{0.2}) y_{st} \quad (6 - 16b)$$

- **Point E:** this point corresponding to the pure bending moment and zero axial load. The nominal axial load $P_{n,E}$ and bending moment $M_{n,E}$ are given in **Eqs.6-**

17a and 6-17b, respectively. By iteration, the depth of the neutral axis, c , should be found where it makes $P_{n,E}$ equals zero.

$$P_{n,E} = (0.85 f'_{cc} \times A_{conc}) + (A_{sc} \times \sigma_{0.2}) - (A_{st} \times \sigma_{0.2}) = 0 \quad (6-17a)$$

$$M_{n,E} = (0.85 f'_{cc} \times A_{conc}) y_{conc} + (A_{sc} \times \sigma_{0.2}) y_{sc} + (A_{st} \times \sigma_{0.2}) y_{st} \quad (6-17b)$$

6.5.5 Verification of the Proposed P-M Interaction Model

As mentioned earlier, the experimental and FE results shown in **Tables 6-2 and 6-4**, respectively, were utilized to verify that the proposed model can provide safe predictions. The secondary bending moments were not taken into account. The comparison between measured (FE and experimental) and predicted results was conducted graphically by plotting the predicted P-M interaction diagram and measured result points, as shown in **Figs.6-15 and 6-16**. It can be seen that the proposed P-M model provides conservative predictions. The inclusion of secondary bending moments in the measured values makes the predicted values more conservative.

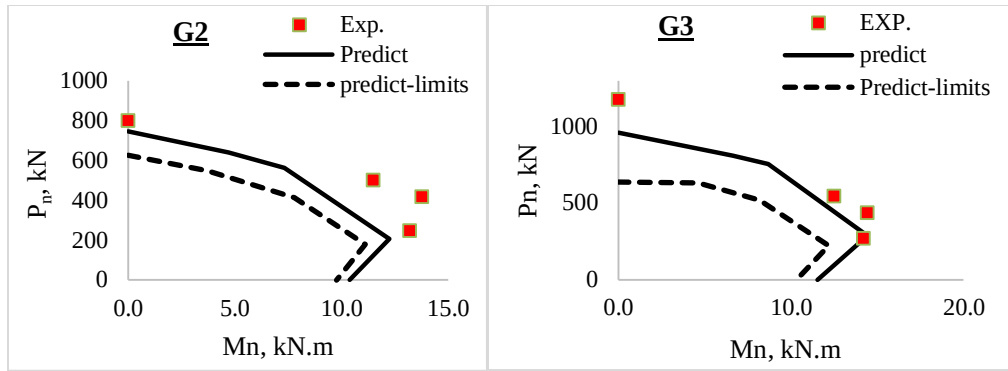
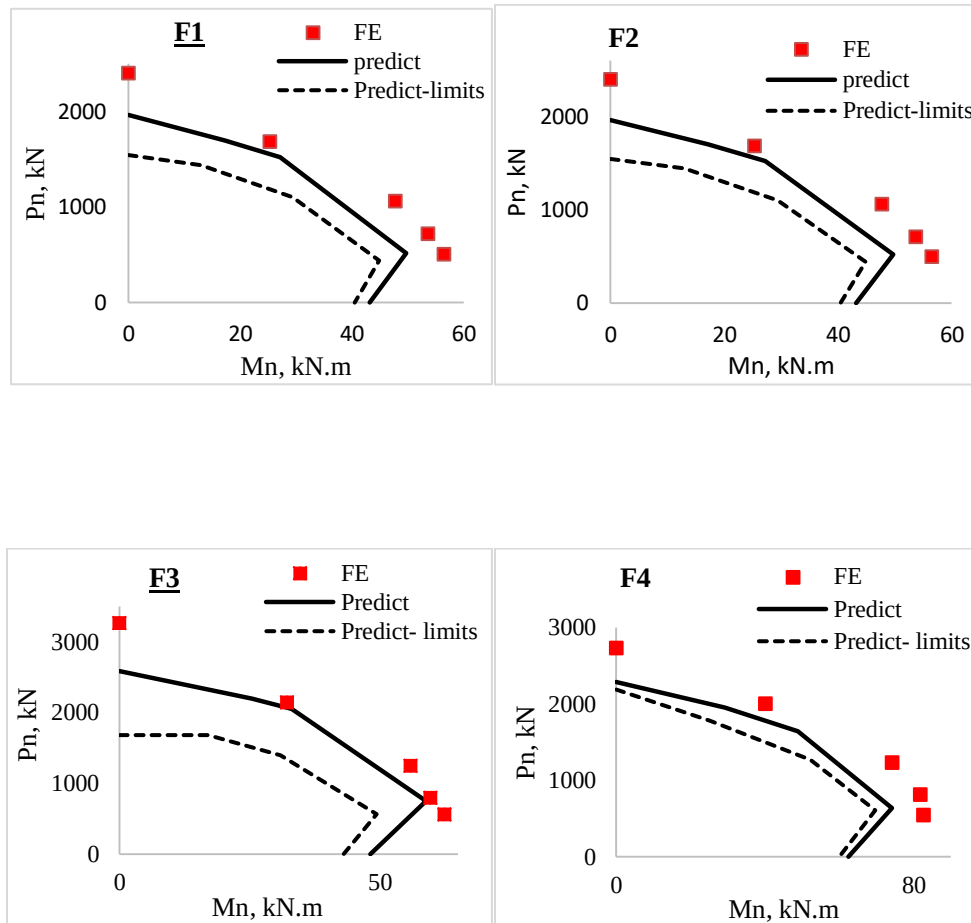


Fig. 6-15 : Comparison between the predicted and experimental results.



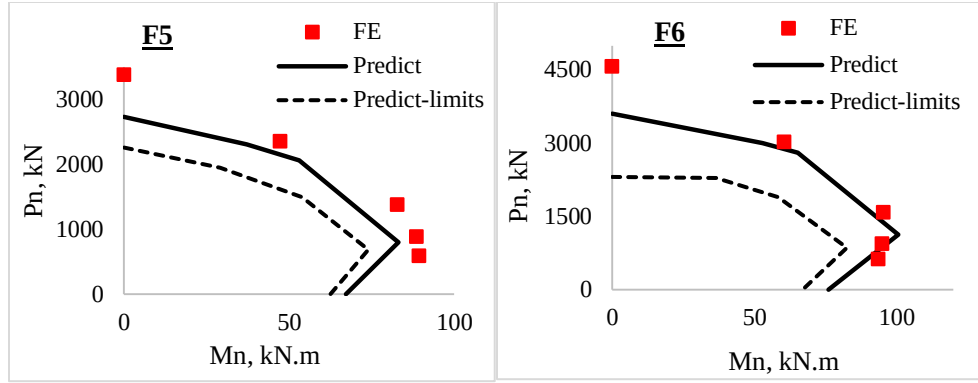


Fig. 6-16: Comparison between the predicted and FE results.

6.6 Concluding Remarks

Experimental, numerical and analytical investigations were presented in this chapter to evaluate the structural behavior of eccentrically loaded circular CFRP-wrapped CFSSTCs. The external CFRP wrapping was utilized as strengthening technique for CFSSTCs. Following conclusions can be drawn within the scope of the research presented in this chapter.

1. The CFRP jacketing is effective in improving the eccentric ultimate load carrying capacity of CFSSTCs.
2. CFRP-confined CFSSTC specimens failed due to the rupture of the CFRP in the compression side, however, the unwrapped specimens failed due to inelastic outward local buckling at the mid-height.
3. The 3D FE model using ABAQUS software to simulate circular CFRP-confined CFSSTCs subjected to eccentric compressive loading was experimentally verified.
4. The P-M predictions provided by the proposed P-M interaction model were conservative predictions compared with the experimental and FE results.

CHAPTER 7

BEHAVIOUR OF BARS-WELDED HSSTCs AND CFSSTCs UNDER AXIAL LOADING

The use of carbon steel bars welded to the inner surface of circular stainless steel tubes will enhance their strength. This chapter presents the results of experimental and numerical investigations into the performance of strengthened circular filled and unfilled stainless steel stub columns. Ten stub columns of 141.3 mm outer diameter and 3.4 mm thickness were tested under axial compression for different numbers and sizes of carbon steel bars. 3D FEMs were developed for strengthened stub columns using ABAQUS software and validated with the experimental results. The validated numerical model was utilized to conduct a parametric study as well as generate extensive data by varying the geometrical and material properties of the stub columns. The test results and FE results were compared with design models, including strengthening strength for ACI, Eurocode 4 and CSM.

7.1 Experimental Investigation

The experimental program was designed and performed to evaluate the structural performance of strengthened HSSTCs and CFSSTCs under monotonic axial

compressive loading. The cross-sections for the strengthened HSSTCs and CFSSTCs are shown in **Fig. (7-1a)** and **Fig.(7-1b)**, respectively. Experiments were conducted using circular hollow stainless steel tube with a cross-sectional size of 141.3×3.4 mm, where the first number refers to the tube outer diameter and the second number represents the tube thickness. Ten stub columns were tested as follows: four strengthened HSSTCs, four strengthened CFSSTCs and one control unstrengthened for each. CFSSTCs were filled with normal concrete. The main strengthening variables considered were the cross-sectional area and the number of carbon steel bars. All of the stub columns were 300 mm in length to insure that the specimens were appropriately short to avoid overall flexural buckling, while the second reason relates to the available height in the hydraulic testing machine. The test specimens and parameters are summarized in **Table 7-1**.

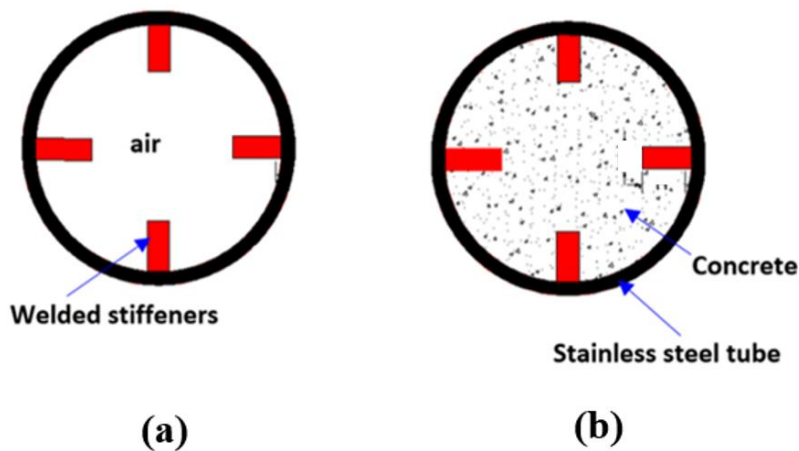


Fig. 7-1: Strengthened cross-sections by carbon steel bars (a) HSSTC (b) CFSSTC

Table 7-1: Numbers and sizes of carbon steel bars for strengthening

Series	Specimen	Number of bars	Cross section dimensions (mm)
Unfilled	H- 0- 0	0	-
	H-4-16	4	16×16
	H-6-16	6	16×16
	H-4-20	4	20×20
	H-6-20	6	20×20
Filled	F- 0- 0	0	-
	F-4-16	4	16×16
	F-6-16	6	16×16
	F-4-20	4	20×20
	F-6-20	6	20×20

Each specimen was given a distinctive label that starts with the letter H or F, followed by two numbers: the first number represents the number of stiffeners and the second number refers to the square cross-sectional dimensions. The H and F letters mean, respectively, hollow and filled tubular columns. For example, the label F-6-20 refers to the CFSSTC reinforced by six stiffeners with cross section dimensions of 20×20 mm.

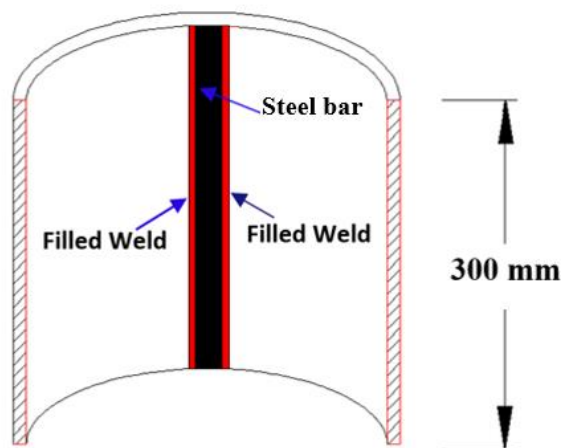
7.1.1 Preparation of Stub Column Specimens

Ten stainless steel circular tubes were cut from a long cold-rolled tube, as displayed in **Fig.7-2a**. The longitudinal carbon steel bars were cut from long rods with the same length as the stainless steel tubes. Subsequently, the bars were fillet welded to

the inner surface of the stainless steel tubes along its entire length, as shown in **Fig.7-2b**. Then, two thick steel plates with 25 mm thickness were welded to both ends of the specimens assigned as unfilled specimens. For the filled specimens, one thick steel plate was welded to the bottom end of the stainless steel tubes as shown in **Fig.7-2c**, while the top end was left without a plate to cast concrete. After tubes were filled with normal concrete, appropriate shaking was done to avoid any concrete segregation, as displayed in **Fig. 7-2d**. To ensure uniform compressive loading on the filled specimens, a thin gypsum layer was applied to the top surface to fill any voids. A minimum gypsum amount was used to reduce its effect on the early stages of loading.



(a)



(b)

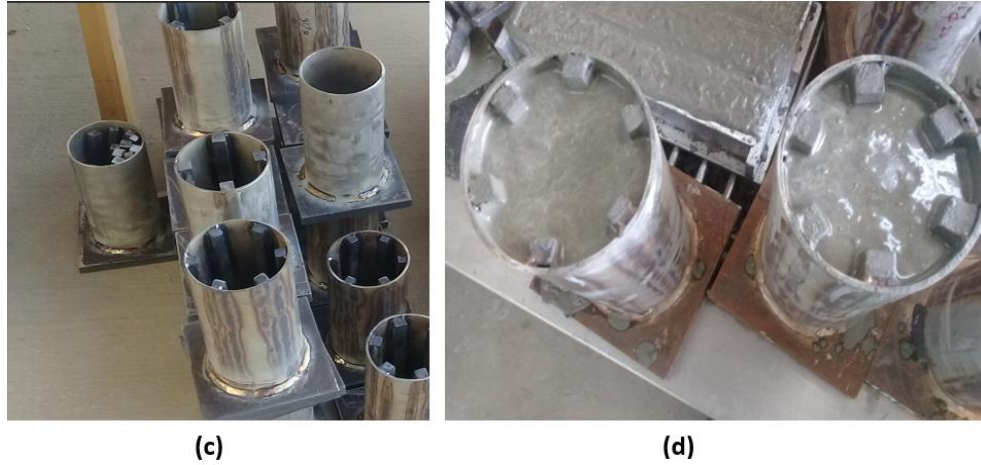


Fig. 7-2: Stub column specimen preparation (a) stainless steel tube cutting (b) welding of carbon steel bars to stainless steel tube (c) stub column specimens with bottom thick plates (d) concrete casting

7.1.2 Instrumentation and Testing

For each specimen, an axial compressive test with monotonic loading was performed to determine the load-deformation response. The test setup as shown in **Fig. 7-3**, comprised of four LVDTs which were located vertically, 90^0 degrees apart from each other to measure the average axial displacement. Two strain gauges were glued longitudinally to all specimens at mid-height. A MATEST 3000 kN hydraulic testing machine was used to apply axial compressive loading with a loading rate of 1 kN/s. The LVDTs, strain gauges and load cell readings were connected to the data logger.

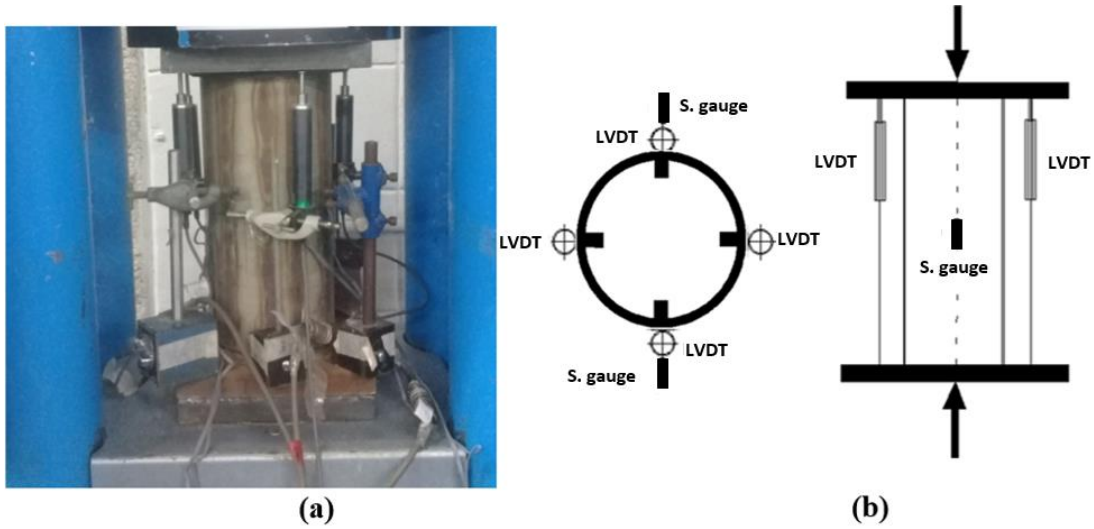


Fig. 7-3: Axially loaded stub column: (a) test Setup (b) schematic drawing for measurements

7.2 Experimental results and discussion

7.2.1 Unfilled Specimens

The axial load vs. axial displacement curves for the unfilled specimens are displayed in **Fig. 7-4**. The axial displacement was taken as the average value of the four LVDTs. By comparing the results of the strengthened specimens with the control-unstrengthened specimen, it is clear that the longitudinal welded bars provided significant improvement in terms of the load carrying capacity, as shown in **Fig. 7-4**. The load carrying capacity and maximum axial shortening of the specimens are summarized in **Table 7-2** as N_{test} and δ_{test} , respectively.

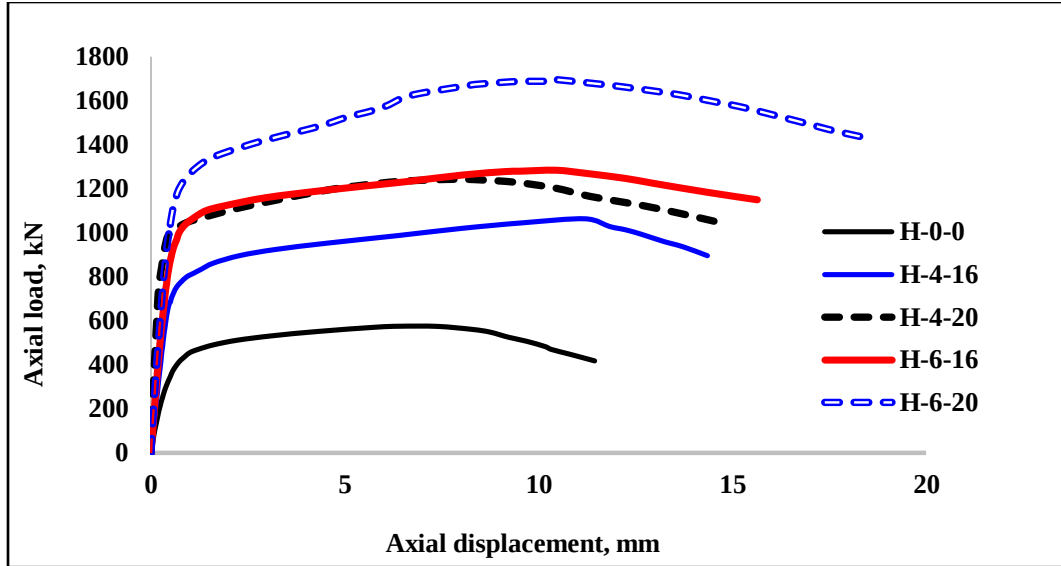


Fig. 7-4: Experimental axial load vs. Axial displacement of unfilled specimens

Table 7-2: Experimental results of load carrying capacity and maximum axial displacement of unfilled specimens

series	Specimen Label	N_{test} , (kN)	δ_{test} , (mm)	SE %
Unfilled	H- 0- 0	576	6.4	-
	H-4-16	1063	11.2	185
	H-6-16	1284	10.4	223
	H-4 -20	1243	7.8	216
	H-6-20	1698	10.3	295

The strength enhancement (SE) is evaluated from **Eq.(7-1)** and listed in **Table 7-2** as the percentage increase relative to the control unstrengthen specimen.

$$SE = \frac{N_u}{N_{u,un}} \times 100 \quad (7-1)$$

where: N_u , and $N_{u,un}$ are the ultimate strength of strengthened and unstrengthened columns, respectively. It is clear that the SE are directly proportional to the cross-sectional area of the bars.

All of the strengthened unfilled specimens failed due to the local buckling of the stainless steel tube associated with the buckling of the carbon steel bars at mid height, as shown in **Fig.7-5**. The control specimen failed due to outward local buckling at the ends, as displayed in **Fig.7-5**.



Fig. 7-5: Failure mode of unfilled specimens

7.2.2 Filled Specimens

The axial load vs. axial displacement curves for all of the filled specimens are displayed in **Fig.7-6**. The vertical strain gauge readings were employed to correct the axial load vs. axial displacement curves in the initial loading stage to obtain true axial displacement values. At the initial loading stage, the effect of the end platen deformation and the gypsum layer deformation resulted in a shift in the axial displacement values. The vertical mid-height strain gauge readings were multiplied by the specimen length to obtain the average axial displacement of the specimen.

The axial load-axial displacement curves shown in **Fig.7-6** represent the true axial load-axial displacement after the correction.

The filled specimens display a ductile behavior and the strength increases with no strictly defined ultimate value. This increase in strength is accompanied by a large plastic deformation. The increase in the strength associated with excess plastic deformation occurs in the stocky specimens, as reported by Lam and Gardener [5]. By comparing the results of the strengthened specimens with the control unstrengthen specimen, it is clear that the longitudinal welded carbon steel bars provided significant improvement in terms of the load carrying capacity. The ultimate load carrying capacity was taken as the load carrying capacity reaching a 5 % average strain equal to 15mm axial displacement as recommended in [5]. The maximum load carrying capacity, $N_{\text{test}(5\%)}$, of the filled specimens and strength enhancement are summarized in **Table 7-3**.

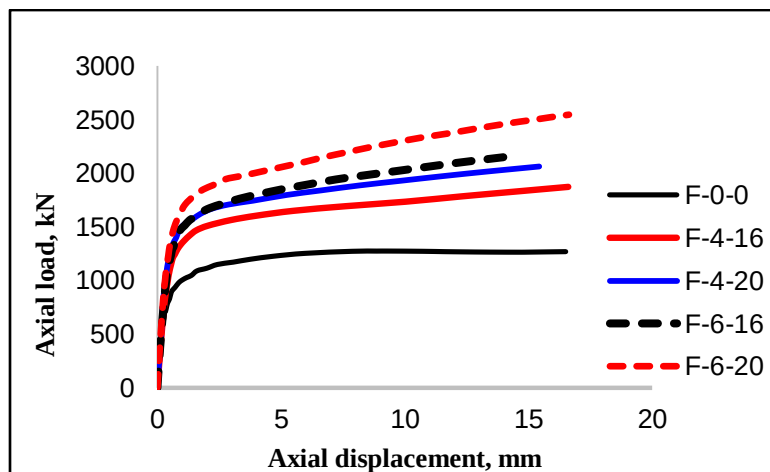


Fig. 7-6: Experimental axial load vs. axial displacement of the filled specimens

Table 7-3: Experimental results of load carrying capacity of filled specimens

series	Specimen Label	$N_{test,(5\%)}$ (kN)	SE %
Filled	F- 0- 0	1263	-
	F-4-16	1839	145
	F-6-16	2176	162
	F-4 -20	2052	172
	F-6 -20	2491	197

Similar to the unfilled specimens, strength enhancement is directly proportional to the cross-sectional area of the bars.

The function of the stainless steel, concrete core and longitudinal welded bars in the filled specimens can be explained as follows. The stainless steel tube has three main functions: contributing to axial load support, preventing outward buckling of the bars, and confining the concrete core. The concrete core carries axial compression load and prevents the inward local buckling of stainless steel tube and bars. The bars contribute to supporting the axial load and stiffens the tube against outward buckling. The failure mode of the strengthened filled specimens was the excess plastic deformation, as illustrated in **Fig.7-7**.



Fig. 7-7: Failure mode of the filled specimens

7.3 Finite Element Modeling

The unfilled and concrete filled stainless steel tubes strengthened by carbon steel bars were modeled using nonlinear finite element (FE) ABAQUS software [45]. The axially loaded stub columns were modeled using shell and solid elements available in the element library of ABAQUS as shown in **Fig.7-8**. C3D8 solid elements were utilized to model the concrete core and longitudinal carbon steel bars. The circular stainless steel tube was modeled using S4R shell elements which have six DOF at each node making them able to provide accurate predictions for buckling modes. Rigid shell plates were modeled to place at the both ends to transfer the axial load uniformly.

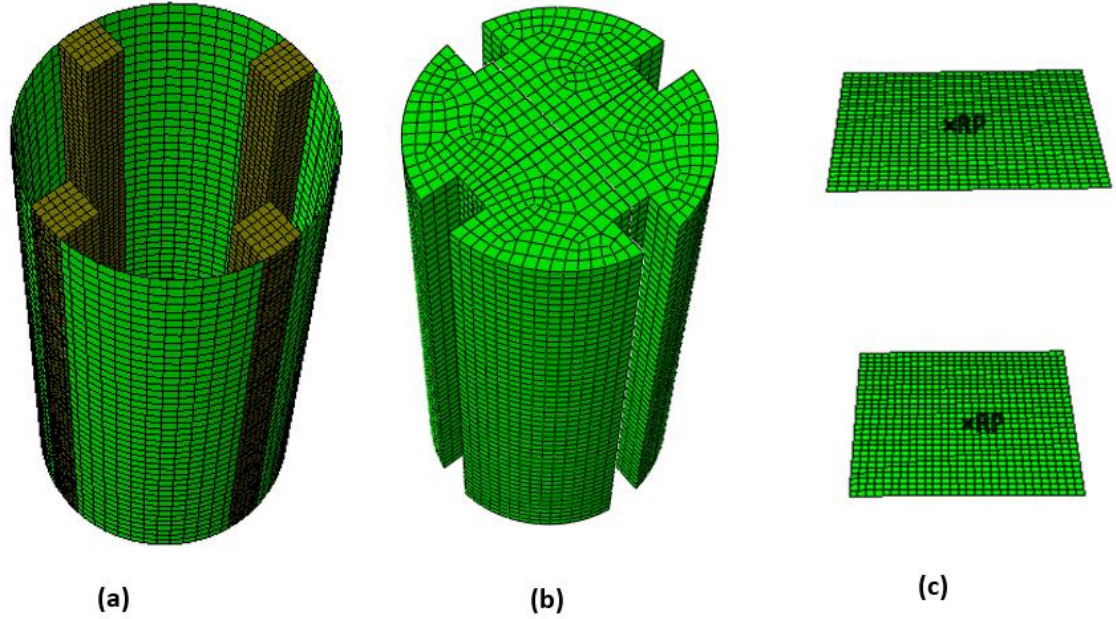


Fig. 7-8: Finite Element modeling (a) stainless steel tube with longitudinal bars elements (c) concrete core elements (c) rigid plates with reference points

Each end of the rigid plate was coupled to a reference point, so that all ends of the DOF were applied to the reference points as point degrees of freedom [57]. At the upper rigid plate, all of the DOFs were restrained except for translation in the longitudinal direction. On the other hand, all of the DOFs at the lower rigid plate were restrained (fully fixed end). Compressive loading was applied as a displacement control to capture the strain softening behavior.

7.3.1 Initial Geometric Imperfections

In this study, the amplitude was taken as ($t/20$, $t/50$ and $t/100$) for the unfilled specimen models to evaluate which amplitude can provide an acceptable match with the test results in terms of the ultimate load carrying capacity, the load-axial

displacement relationship and the failure mode. Once the imperfection level was verified for unfilled specimens then it was applied for the filled specimens.

7.3.2 Materials Modeling

The stainless steel and concrete materials were modeled in ABAQUS as discussed earlier in sections 4.3.4.1 and 5.3.4.1, respectively.

7.3.2.1 Carbon Steel

An elastic-Perfectly plastic model with the Von Mises yield criterion was employed to define the constitutive behavior of the carbon steel material. The engineering stress-strain relationships (σ_{Eng} , ϵ_{Eng}) obtained from tensile testing were converted to a true stress-true strain (σ_{true} , ϵ_{true}) format.

7.3.3 FE Interaction Modeling

The contact between the stainless steel tube and the longitudinal bars was represented using tie interaction where the stainless steel surface was assigned to be the master surface. The interaction modeling between the stainless tube and the concrete infill was specified as a friction interaction in the tangential direction with a friction coefficient of 0.25 as reported in [61], and the interaction in the normal direction was assigned as a hard contact to prevent any penetration between the two surfaces. The contact between the concrete core and the longitudinal bars was similar to the contact between the concrete core and the stainless steel tube. The interaction between the rigid plate surfaces and the steel tube surfaces was specified as a tie contact where the rigid plates were assigned to be the master surfaces as reported in

[57]. The interaction between the rigid plates and the concrete infill surfaces was introduced using a friction interaction in the tangential direction with the friction coefficient of 0.35 as recommended in [30] and hard contact in the normal direction. To carry direct axial compression loads, the contact between the longitudinal bars and the rigid plate surfaces was defined as a hard contact in the normal direction and a friction contact in the tangential direction were assumed.

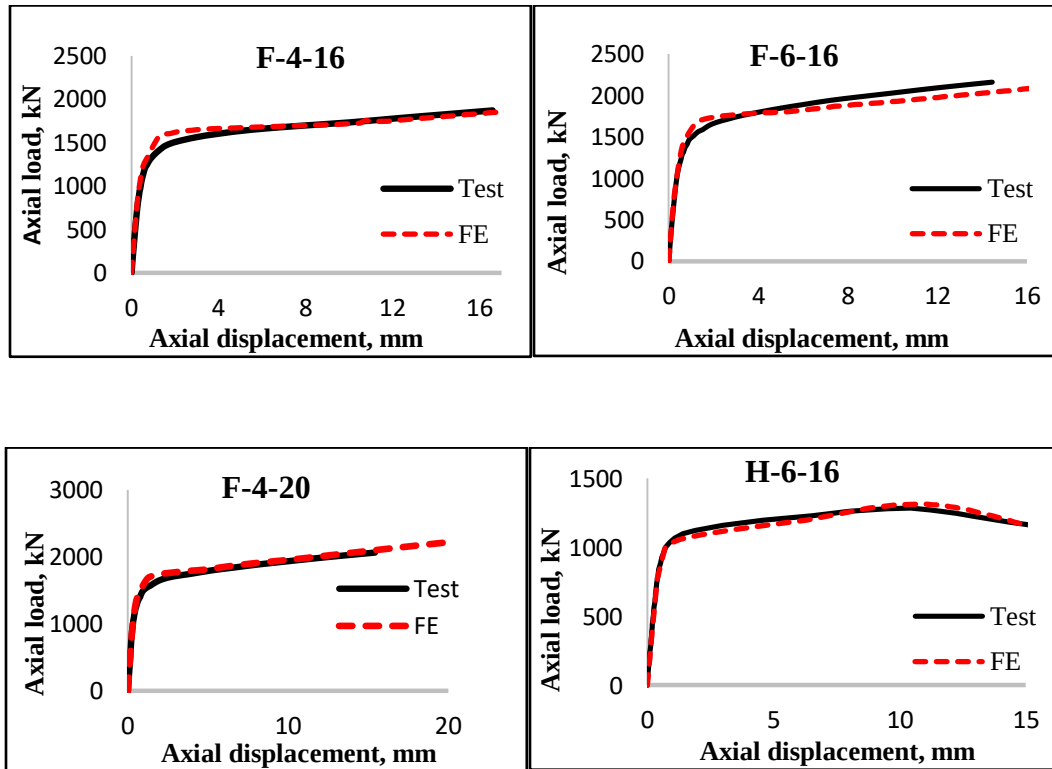
7.3.4 FE Model Validation

The FE results for load carrying capacity, N_{FE} , and maximum axial displacement, δ_{FE} were evaluated for three imperfection amplitudes ($t_s/20$, $t_s/50$ and $t_s/100$). The ratio of FE results to experimental results (N_{test} and δ_{FE}) for unfilled stub columns are given in **Table 7-4**. The imperfection amplitude $t_s/100$ was adopted for the FE model since it revealed the best prediction of test results.

Table 7-4: Experimental and FE results for different levels of imperfection amplitudes

Specimen	N_{FE} / N_{test}			$\delta_{FE} / \delta_{test}$		
	Imperfection Amplitudes			Imperfection Amplitudes		
	t/20	t/50	t/100	t/20	t/50	t/100
H-0- 0	0.94	0.96	1.01	0.74	0.91	0.98
H-4-16	0.93	0.99	1.02	0.89	0.90	0.92
H-6-16	0.96	1.00	1.02	0.89	0.94	1.06
H-4 -20	0.98	0.99	0.98	1.04	1.00	1.00
H-6 -20	0.99	1.01	1.01	0.95	1.05	1.06
Mean	0.96	1.00	1.01	0.90	0.96	1.00
COV	0.03	0.01	0.02	0.12	0.07	0.06

The FE results for the axial load versus axial displacement curves were verified with the experimental results as shown in **Fig.7-9** for some selected specimens. The FE results indicated excellent matching with experimental results. The failure modes observed experimentally and those predicted by the FE model as shown in **Fig. 7-10** reveals very good agreement. Finally, the load carrying capacity from experimental work and FE model are given in **Figs. 11** and **12** for unfilled and filled stub columns, respectively. It is evident that the FE model has predicted well the behavior of the strengthened stub columns up to failure.



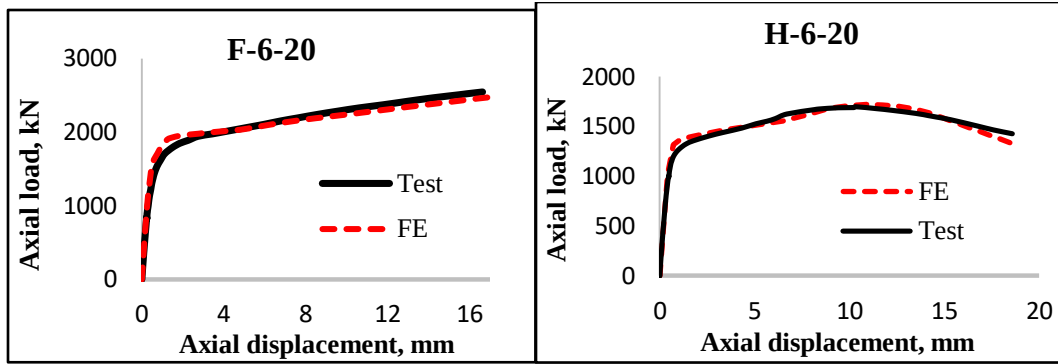


Fig. 7-9: Experimental and numerical axial load vs. axial displacement curves for some selected specimens

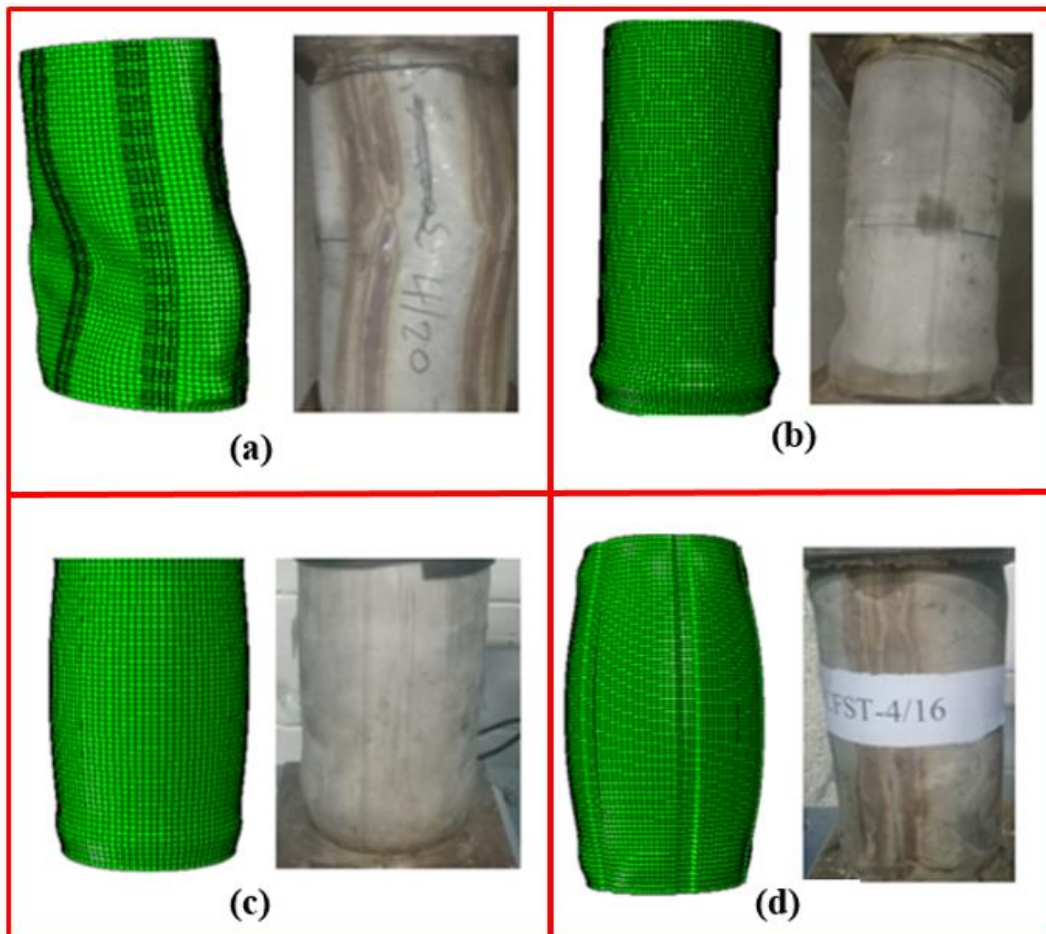


Fig. 7-10: General failure mode of (a) strengthened unfilled specimen (b) control unfilled specimen (c) strengthened filled specimen (d) control filled specimen

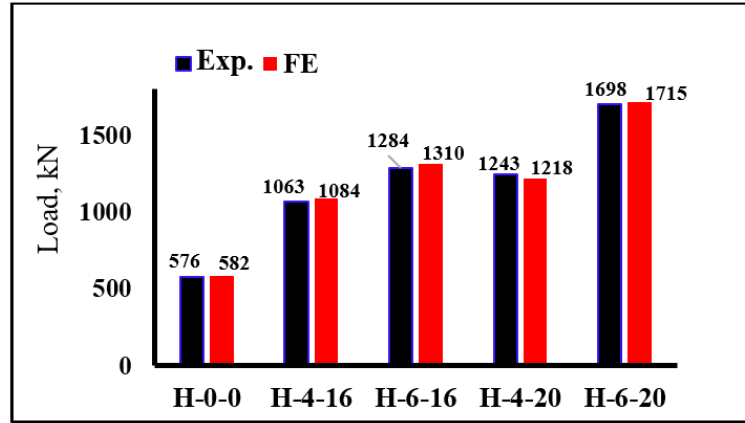


Fig. 7-11: Experimental and FE ultimate load carrying capacity results for unfilled specimens

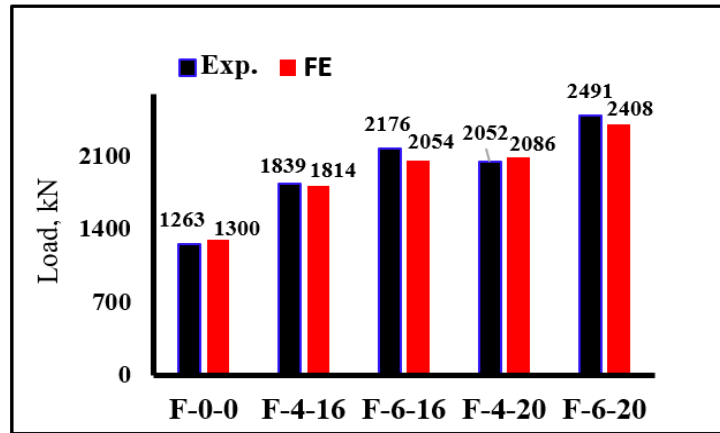


Fig. 7-12: Experimental and FE ultimate load carrying capacity results for filled specimens

7.3.5 FE Parametric Study

A parametric study of the strengthened stainless steel tubes was conducted using the verified FE model to assess: (i) the practical configuration of carbon steel bars, (ii) the thickness reduction of stainless steel tube and (iii) effect of D/t ratio on the load carrying capacity. The same material properties presented in section 2.2 for stainless steel, carbon steel and concrete were used in this parametric study. The FE model

was further utilized to generate additional data for more dimensional and material properties presented in section 7.5 to verify the simplified calculation discussed in section 7.4.

7.3.5.1 Configuration of Carbon Steel Bars

The total area of carbon steel bars required to strengthen a stainless steel tube can be equivalent to a certain number of carbon steel bars. In this parametric study, a stainless steel tube of outer diameter 141.3 mm and 3.4 mm thickness was strengthened with a constant area of carbon steel (A_b) equal to 2073 mm². The equivalent number of carbon steel bars for this constant area were taken as two, three, four and five as shown in **Fig.7-13**. The bars were arranged to maintain symmetry within the stainless steel tube. The axial load versus axial displacement for the unfilled and filled specimens are shown in **Fig.7-14**. The results clearly indicate identical behavior for all equivalent number of carbon steel bars. The same results were experimentally demonstrated for specimens H-4-20 and H-6-16 having same carbon steel area for different equivalent number of carbon steel bars. The experimental results for unfilled and filled specimens are shown **Figs. 4 and 6**, respectively. It can be concluded that equivalent two bars configuration for the required area of carbon steel is the most practical to minimize welding requirements.

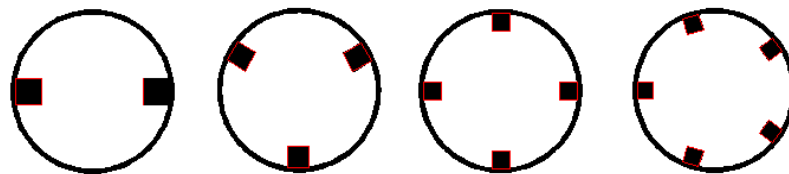


Fig. 7-13: Equivalent Carbon Steel Bars Configuration

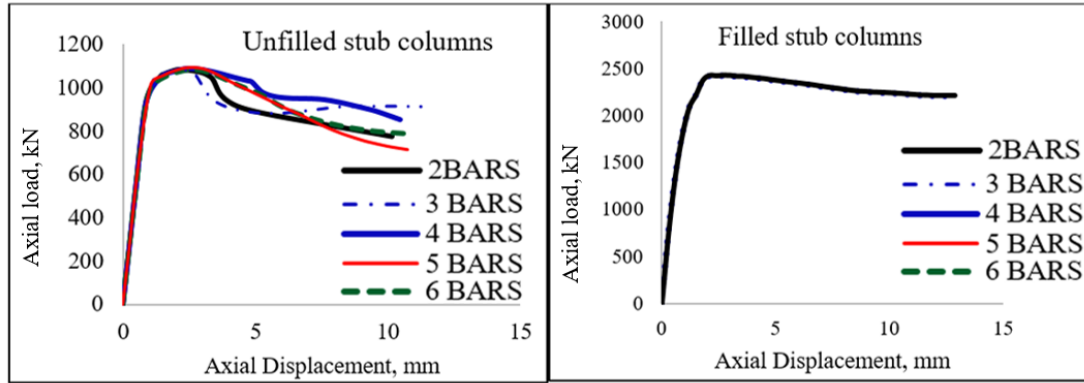


Fig. 7-14: Axial load vs. axial displacement for different configurations of carbon steel bars.

7.3.5.2 Thickness Reduction of Stainless Steel Tube

The cost of the stainless steel material is much higher than that of carbon steel. In this parametric study, it is intended to assess the possible thickness reduction of stainless steel tube that can be compensated by carbon steel bars area (A_b). The initial load carrying capacity of a stainless steel tube without carbon steel area was evaluated. Then for each reduction of the stainless steel thickness (t), carbon steel area A_b was added to maintain the initial load carrying capacity. Three tube diameters were used in this parametric study 100, 141.3 and 170 mm while the initial thickness of 3.4 mm was kept the same for all considered diameters. A plot of D/t_s ratio versus area of carbon steel bars (A_b) is shown in **Fig.7-14** for unfilled and filled stainless steel tube. Each curve represents a certain tube diameter and a constant initial load carrying capacity. The thickness reduction of the stainless steel tube is limited by the elastic buckling for slender sections. The critical stress for elastic local buckling of cylindrical tube under axial loading is given by **Eq.7-2** developed by

Timoshenko [70]. Buchanan et al. [44] reported that the local cross-section slenderness limit of a circular section (λ_c) given by **Eq.7-3** should be greater than 0.3 to define a slender section.

$$\sigma_{cr} = \frac{E_0}{\sqrt{3(1-\nu^2)}} \frac{2t_s}{D} \quad (7-2)$$

$$\lambda_c = \sqrt{\frac{f_y}{\sigma_{cr}}} \quad (7-3)$$

For stainless steel yield stress, f_y in **Eq.7-3** is replaced by $\sigma_{0.2}$. The limiting value of D/t ratio to avoid the elastic local buckling is given by **Eq.7-4** derived by substituting **Eq.7-2** into **Eq.7-3** and taking $\lambda_c=0.3$. The limiting D/t ratio is plotted in **Fig.7-15** for unfilled and filled tubes.

$$\frac{D}{t_s} \leq \frac{0.104 E_0}{\sigma_{0.2} \sqrt{(1-\nu^2)}} \quad (7-4)$$

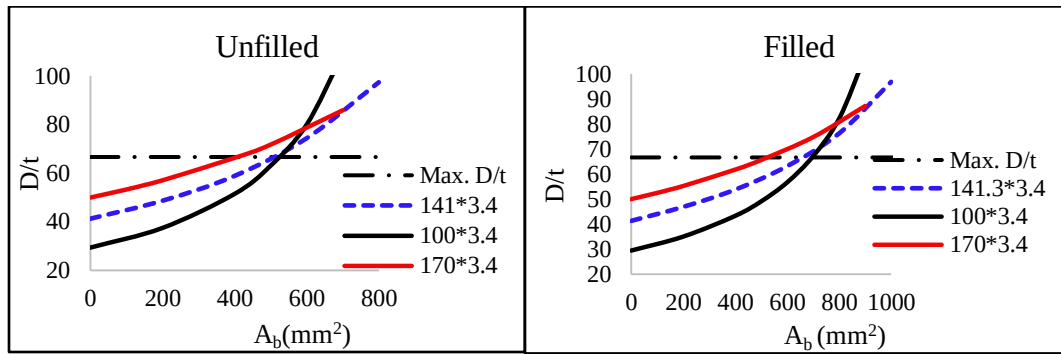


Fig. 7-15: Area of carbon steel bars required to compensate for the thickness reduction of stainless steel tube.

As expected, the thickness reduction of stainless steel tube is larger for a small initial D/t_s ratio since D is kept constant for each curve. The initial value for each D/t_s ratio

at which $A_b=0$ and $t=3.4$ mm are listed in **Table 7-5** with the corresponding area of stainless steel tube (A_s). At the limiting value of $D/t = 67$, the percentage reduction of stainless steel thickness and the corresponding added area of carbon steel bars (A_b) to compensate for this maximum reduction is given in **Table 7-5** for unfilled and filled tubes. The amount of thickness reduction of the stainless steel is inversely proportioned to the initial D/t_s ratio.

Table 7-5: Maximum thickness (t) reduction of stainless steel tube and added area of carbon steel bars (A_b) to maintain the initial load carrying capacity

Initial D/t Ab=0 t = 3.4 mm	Limiting case						
	Stainless steel tube thickness reduction (%)		Unfilled specimens			Filled specimens	
			Ab (mm²)	Ab/As		Ab (mm²)	Ab/As
29.4	56		540	1.17		700	1.52
41.5	38		500	0.54		660	0.72
50.0	25		400	0.30		550	0.41

This clearly confirms that stainless steel area (A_s) can be reduced significantly when compensated by carbon steel bars area (A_b) to maintain the initial load carrying capacity.

7.3.5.3 Effect of D/T Ratio on the Load Carrying Capacity

The effect of D/t ratio on the axial load vs. axial displacement of strengthened stainless steel tube is presented in **Fig.7-16** for unfilled and filled specimens. The stainless steel tube diameter $D= 300$ mm and area of carbon steel bars $A_b= 1536$ mm² are kept unchanged. It is clear that the load carrying capacity is inversely

proportional to D/t_s ratio for both unfilled and filled specimens. The strain hardening response on the axial load vs. axial displacement is noted for $D/t = 30$ for both unfilled and filled specimens.

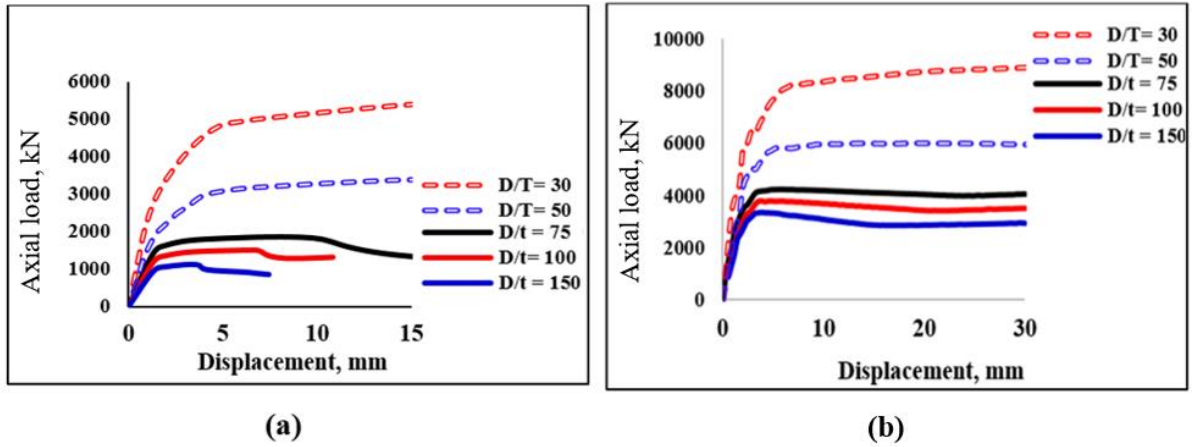


Fig. 7-16: Effect of D/t ratio on the axial load vs. axial displacement response of (a) unfilled specimens (b) filled specimens.

7.4 Simplified Evaluation of the Ultimate Load Carrying Capacity

Design codes adopted from ACI-318 code [71] and Eurocode 4 [72] and CSM [44,53–55], were utilized to evaluate the ultimate load carrying capacity including the strengthening effect of the carbon steel bars. The predicted ultimate load carrying capacity was evaluated for unfilled and filled stainless steel stub columns. The evaluations are based on nominal strength, not taking into consideration strength reductions.

7.4.1 ACI-318 code

The design formula adopted from ACI-318 code for load carrying capacity predictions was developed for concrete-filled carbon steel tubes. The design formula excludes the interaction between the concrete core and steel tube. The axial compressive load carrying capacity ($N_{u, ACI}$) of a concrete-filled carbon steel tube is the strength summation of the carbon steel tube and the concrete core, as given by **Eq.7-5**.

$$N_{u,ACI} = A_s f_y + 0.85 A_c f'_c \quad (7-5)$$

Where: A_c , f'_c , A_s and f_y are, respectively, the cross-section area of the concrete core, the concrete compressive strength, the cross-section area of the steel tube and the yield strength of the carbon steel tube.

The load carrying capacity of concrete-filled stainless steel tubes was evaluated using **Eq.7-5** with changing f_y to 0.2% proof stress of stainless steel ($\sigma_{0.2}$) and adding the contribution of carbon steel bars as given by **Eq. 7-6**.

$$N_{u,ACI} = A_s \sigma_{0.2} + A_b f_b + 0.85 A_c f'_c \quad (7-6)$$

Where A_b and f_b are the cross-section area and yield stress of the carbon steel bars, respectively. For the unfilled columns, the third term in **Eq.7-6** is canceled.

7.4.2 Eurocode 4

The Eurocode 4 evaluation for the load carrying capacity ($N_{u,EC4}$) of circular concrete-filled carbon steel tubes includes the interaction between the steel tube and the concrete core as given by **Eq.7-7**.

$$N_{u,EC} = \eta_s A_s f_y + A_c f'_c \left[1 + \eta_c \left(\frac{t_s}{D} \right) \left(\frac{f_y}{f'_c} \right) \right] \quad (7-7)$$

Where: the two factors η_s and η_c are proposed to take into consideration the interaction between the steel tube and the concrete core. They are defined for the pure compressive loading as follows:

$$\eta_s = 0.25 (3 + 2 \lambda_c) \quad (\text{but } \leq 1.0) \quad (7-8)$$

$$\eta_c = 4.9 - 18.5 \lambda_c + 17 \lambda_c^2 \quad (\text{but } \geq 0) \quad (7-9)$$

where λ_c is the local cross-section slenderness given by **Eq. 7-3**.

Eq.7-3 is modified for stainless steel tube and including the strength contribution of the carbon steel bars as given in **Eq.7-10**.

$$N_{u,EC} = \eta_s A_s \sigma_{0.2} + A_b f_b + A_c f'_c \left[1 + \eta_c \left(\frac{t_s}{D} \right) \left(\frac{\sigma_{0.2}}{f'_c} \right) \right] \quad (7-10)$$

For the unfilled columns, the η_s factor and the third term in **Eq.7-10** are canceled.

7.4.3 Continuous Strength Method

This method was discussed in details in Chapter 4, section.4.5.1

7.5 Results Verification of Simplified Methods

The load carrying capacity results obtained experimentally N_{test} and numerically N_{FE} were compared with those predicted from ACI (N_{ACI}), Eurocode 4 (N_{EC4}) and the CSM (N_{CSM}) for unfilled and filled specimens. As indicated earlier, the FE model was utilized to generate additional results for different levels of D/t ratio, f'_c and $\sigma_{0.2}$ for verifying the simplified methods. The carbon steel bars area (A_b) was kept unchanged at 1536 mm^2 for the generated FE results presented in **Tables 7-6** and **7-7** for unfilled and filled specimens, respectively.

Table 7-6: Comparison of Numerical results with design models for unfilled specimens

label	D (mm)	t_s (mm)	D/ t_s	$\sigma_{0.2}$ (MPa)	N_{FE} (KN)	$\frac{N_{\text{FE}}}{N_{\text{ACI}}}$	$\frac{N_{\text{FE}}}{N_{\text{EC4}}}$	$\frac{N_{\text{FE}}}{N_{\text{CSM}}}$
C1	141.3	7.0	20	320	1961	1.33	1.33	1.12
C2	141.3	4.7	30	320	1292	1.09	1.09	1.04
C3	141.3	2.8	50	320	920	1.00	1.00	0.99
C4	141.3	7.0	20	412	2223	1.27	1.27	1.15
C5	141.3	4.7	30	412	1510	1.11	1.11	1.07
C6	141.3	2.8	50	412	1057	1.02	1.02	1.02
Mean						1.14	1.14	1.07
COV						0.11	0.11	0.05

Table 7-7: Comparison of Numerical results with design models for filled specimens

Specimen	D (mm)	t _s (mm)	D/t _s	f'_c (MPa)	$\sigma_{0.2}$ (MPa)	N _{FE} (KN)	$\frac{N_{FE}}{N_{ACI}}$	$\frac{N_{FE}}{N_{EC4}}$	$\frac{N_{FE}}{N_{CSM}}$	$\frac{N_{FE}}{N_{CSM.Mod}}$
C7	141.3	7.0	20	30	320	2874	1.59	1.33	1.20	1.20
C8	141.3	4.7	30	30	320	2210	1.45	1.25	1.22	1.22
C9	141.3	2.8	50	30	320	1680	1.30	1.18	1.18	1.18
C10	141.3	7.0	20	60	320	3008	1.41	1.18	1.09	1.16
C11	141.3	4.7	30	60	320	2400	1.28	1.11	1.08	1.15
C12	141.3	2.8	50	60	320	1896	1.14	1.02	1.02	1.09
C13	141.3	7.0	20	90	320	3191	1.30	1.09	1.01	1.13
C14	141.3	4.7	30	90	320	2531	1.14	0.98	0.96	1.07
C15	141.3	2.8	50	90	320	2112	1.04	0.92	0.92	1.02
C16	141.3	7.0	20	30	412	2872	1.38	1.16	1.09	1.09
C17	141.3	4.7	30	30	412	2208	1.29	1.13	1.11	1.11
C18	141.3	2.8	50	30	412	1680	1.20	1.10	1.10	1.10
C19	141.3	7.0	20	60	412	3008	1.25	1.06	1.00	1.07
C20	141.3	4.7	30	60	412	2400	1.16	1.02	1.00	1.07
C21	141.3	2.8	50	60	412	1896	1.07	0.97	0.97	1.04
C22	141.3	7.0	20	90	412	3188	1.17	0.99	0.94	1.05
C23	141.3	4.7	30	90	412	2782	1.15	1.00	0.99	1.10
C24	141.3	2.8	50	90	412	2315	1.08	0.97	0.97	1.08
Mean						-	1.24	1.08	1.05	1.11
COV						-	0.11	0.10	0.08	0.05

7.5.1 Unfilled Specimens

Generally, the design codes (ACI, EC4 and CSM) provide conservative predictions for the load carrying capacity compared to numerical and experimental results as shown in **Tables 7-6 and 7-8**, respectively. Generally, CSM predictions are the most

accurate compared to ACI and EC4 since it takes into account the strain hardening behavior of the stainless steel material.

Table 7-8: Comparison of experimental results with design models for unfilled specimens

Specimen	N_{test}, kN	N_{test}/ N_{ACI}	N_{test}/ N_{EC4}	N_{test}/ N_{CSM}
H-4-16	1063	1.28	1.28	1.26
H-6-16	1284	1.28	1.28	1.25
H-4-20	1248	1.21	1.21	1.19
H-6-20	1698	1.30	1.30	1.28
Mean	-	1.27	1.27	1.21
COV	-	0.027	0.027	0.027

7.5.2 Filled Specimens

ACI predictions are more conservative compared with numerical and experimental results as shown, respectively, in **Tables 7-7 and 7-9**. This is mainly due to ignoring the interaction between the stainless steel tube and the concrete core as well as the strain hardening behavior of the stainless steel material. For low strength concrete, CSM predictions are more accurate than EC4 predictions. In general, EC4 and CSM provided unconservative predictions for concrete strength higher than 60 MPa. To overcome this, the following modification can be multiplied by CSM formula for the $f'_c \geq 60$ MPa.

$$\Omega = 1 - \frac{f'_c}{900} \quad (7-11)$$

Where f'_c measured in MPa. The modification shown in **Eq.7-11** was proposed by Lam and Gardner [5] to be used for concrete-filled rectangular or square stainless steel tubes. In this study, the modification factor shown in **Eq.7-11** was used for filled specimens strengthened by carbon steel bars. The modified formula can be modified as follows.

$$N_{csm,mod} = \Omega N_{csm}, \quad (12)$$

Table 7-7 indicates that the modified CSM provides accurate predictions for the strengthened filled specimens.

Table 7-9: Comparison of experimental results with design models for filled specimens

Specimen	N_{test} (kN)	N_{test}/ N_{ACI}	N_{test}/ N_{EC4}	N_{test}/ N_{CSM}
F-4-16	1839	1.37	1.20	1.19
F-6-16	2179	1.43	1.27	1.26
F-4-20	2052	1.35	1.18	1.17
F-6-20	2491	1.40	1.24	1.23
Mean	-	1.39	1.22	1.21
COV	-	0.022	0.029	0.029

7.6 Concluding Remarks

Experimental and numerical investigations were presented in this chapter to evaluate the behavior of axially loaded circular unfilled and concrete filled stainless steel stub columns internally strengthened using welded longitudinal carbon steel bars. The following conclusions can be drawn within the scope of the research presented in this chapter.

1. The use of inner-welded carbon steel bars to be part of the structural supporting system of the axial compressive loading was greatly effective when used as a solution to balance the initial high cost of the stainless steel material.
2. Unfilled strengthened specimens failed by local buckling of the stainless steel tubes associated with the buckling of the carbon steel bars at mid height. The concrete filled strengthened specimens failed by excess plastic deformation.
3. The 3D FE model using ABAQUS software to simulate circular stainless steel tube strengthened with carbon steel bars for unfilled and concrete filled specimens subjected to axial compressive loading was experimentally verified.
4. Stainless steel section can be reduced significantly when compensated by carbon steel bars to maintain the initial load carrying capacity.
5. The load carrying capacity of strengthened stainless steel tube is inversely proportional to D/t ratio.
6. CSM prediction for the load carrying capacity of the strengthened stainless steel tubes is the most accurate relative to ACI and EC4.

CHAPTER 8

CONCLUSIONS AND FUTURE RECOMMENDATIONS

In this chapter, the key conclusions of this research are summarized, whereas more concluding remarks can be found at the end of each chapter. Furthermore, suggestions for future research are included in this chapter.

8.1 Conclusions

Recently, stainless steel material has been used as a construction material, while it was previously only used for special purposes or for decoration due to its advantages over carbon steel which include its aesthetic appearance, high resistance to corrosion, ease of maintenance and high fire resistance. Taking into account the long-term cost, stainless steel material can be selected as a competitive material. In addition, one of the reasons for considering stainless steel as a competitive structural material is its favorable mechanical properties and its high ductility. Stainless steel material exhibits a nonlinear stress- strain relationship with no defined yield point, unlike carbon steel material.

This dissertation focused on investigation of two strengthening approaches, one technique directed to those already designed and constructed hollow stainless steel tubular stub columns (HSSTCs) and concrete-filled stainless steel tubular stub columns (CFSSTCs) using CFRP jacketing and the other technique directed to the new designed HSSTCs and CFSSTCs using longitudinal stiffeners welded to the

inner stainless steel surfaces. This research has three major components: an experimental work, numerical simulations and simplified analytical solutions.

For experimental work, testing of material properties was conducted for the materials used in this research as well as forty-three stub circular columns were tested under monotonic compressive loading to evaluate the effectiveness of the CFRP jacketing and longitudinal stiffening. The experimental stub column specimens were divided into four categories: HSSTCs stiffened using CFRP jackets under axial loading, CFSSTCs confined by CFRP jackets under axial loading, CFSSTCs confined by CFRP jackets under eccentric loading, and HSSTCs and CFSSTCs stiffened using longitudinal welded stiffeners under axial loading.

Parallel with experimental program, three-dimensional finite element modeling was developed and validated against the experimental results using ABAQUS software. Subsequently, the validated FEMs were employed to perform extensive parametric studies; more than 250 FEMs were analyzed to obtain the load carrying capacity results. These parametric study results were employed either to check proposed models or to develop proposed models to predict the load carrying capacities. Based on experimental, numerical and analytical investigations, the following key conclusions can be drawn.

1. Using CFRP jacketing to stiffened HSSTCs is effective in improving the axial compressive behavior in terms of the ultimate strength and the axial shortening capacity. However, when the dominated failure mode is inward inelastic local buckling, the ultimate strength can be improved slightly

compared to the specimens dominated by outward local buckling. However, using partial wrapping to stiffen HSSTCs has not shown any improvement in the load carrying capacity as well as deformation capacity.

2. The CFRP jacketing is highly effective in improving the axial compressive behavior of CFSSTCs, in terms of the ultimate strength and the axial shortening capacity. However, Partial wrapping has not shown noticeable improvement in load carrying capacity. The increase in the CFRP jacket thickness lead to a significant improvement in the ultimate load carrying capacity and the axial shortening capacity.
3. The CFRP jacketing was effective in improving the load carrying capacity of CFSSTCs under eccentric loading. However, as load eccentricity increases, the amount of enhancement decreases. The increase in the CFRP jacket thickness led to increase the improvement in the ultimate load carrying capacity and the axial shortening capacity.
4. Use of longitudinal welded carbon steel stiffeners to carry a part of axial compressive loading was greatly effective to be used as a solution to balance the initial high cost of the stainless steel material.
5. Using finite element modeling, the concrete core should be modeled as a confined concrete to take in account the confining pressure coming from the circular steel tube.
6. The finite element analysis showed that HSSTCs are sensitive to the initial geometric imperfections, whereas CFSSTCs are not sensitive.

7. The continuous strength method (CSM) provided accurate and consistent results because it reflects the actual stress-strain relationship of the stainless steel material.

8.2 Future Recommendations

Based on this research, some suggestions for future research can be drawn:

- 1- Further research is needed to study non-circular stainless steel tubular columns stiffened using CFRP.
- 2- This research can be extended in a future to investigate long columns instead of stub columns
- 3- Further study is required to investigate CFRP-CFSSTCs under bi-axial moments.
- 4- Structural performance of ultra-high performance concrete-filled stainless steel tubular columns under eccentric loading can be investigated in future.
- 5- Ultra-high performance concrete-filled stainless steel tubular long columns can be studied in future.
6. Further research is needed to study the CFSSTCs under bearing stress.
7. Compacted soil-filled circular steel tubes under axial loading is needed to study.
8. Investigation of CFRP-confined CFSSTCs subjected to lateral and axial loads can be a future work.

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