Republic of Iraq
Ministry of Higher Education and Scientific Research
University of Misan/College of Engineering
Civil Engineering Department

INVESTIGATION STUDY OF UPGRADED RC COLUMNS BY STEEL FIBERS CONCRETE JACKET

A THESIS
SUBMITTED TO THE COLLEGE OF ENGINEERING MISAN UNIVERSITY IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER IN CIVIL ENGINEERING
(STRUCTURES)

BY
WATHIQ JASSIM MHUDER
B.Sc. in Civil Engineering, 2001

Supervised by
Assist. Prof. Dr. Samir Mohammed Chassib

June 2020 **Shawal 1441**

سورة البقرة (٢٥٥)

Dedication

- ❖ To whom stood beside me and took care of me over the years, to my parents, family, and close friend.

- ❖ To all those who supported and helped me, made the difficult easy, especially to my supervisor Assist Prof. Dr. Samir M. Chassib

I present my effort to them with all of my respect and appreciation.

ACKNOWLEDGEMENTS

First of all, all my thanks for **Allah** who led me during my way to complete this work.

I would like to express my cordial thanks and deepest gratitude to my supervisor **Assist. Prof. Dr. Samir Mohamed Chassib**, whom I had the honor of being under his supervision, for his advice, help, and encouragement during the course of this study.

I would like to extend my thanks to **Asst. Prof. Dr. Abbas Oda Dawood** Dean of the college of engineering, and **Assist Prof. Dr. Samir Mohamed Chassib**, the Head of Civil Engineering Department.

Special thanks go to **my father, mother, wife, and children** for their great efforts. Also, thanks go to **my brothers**.

Special thanks also go to **Prof. Dr. Ahmed Alsharaa, Mr. Yasser Neama, Assist lect. Ali Wathiq , and Assist lect. Mohammed Mahood,** for their effort in helping me in this thesis.

Wathiq Jassim Mhuder

2020

CERTIFICATION

I certify that the thesis titled "**Investigation Study of Upgraded RC Columns by Steel Fibers Concrete Jacket"** which is being submitted by **Wathiq Jassim Mhuder** and prepared under my supervision at the University of Misan, Department of Civil Engineering, in partial fulfillment of the requirements for the Degree of Master of Science in Civil Engineering (Structures).

Signature:
Assist. Prof. Dr. Samir Mohamed Chassib
Date:

In view of the available recommendations, I forward this thesis for discussion by the examining committee.

Signature:
Assist. Prof. Dr. Samir Mohamed Chassib
(Head of Civil Eng. Department)
Date:

Committee Report

We certify that we have read this thesis titled **" Investigation Study of Upgraded RC Columns by Steel Fibers Concrete Jacket "** which is being submitted by **Wathiq Jassim Mhuder**, and as Examining Committee, examined the student in its contents. In our opinion, the thesis is adequate for award of degree of Master of Science in Civil Engineering.

Signature:
Name: Prof. Dr. Sa'ad F. Resan
(Chairman)
Date: / /

Signature:
Name: Assist. Prof. Dr. Ihab S. Saleh
(Member)
Date: / /

Signature:
Name: Dr. Faten I. Mussa
(Member)
Date: / /

Signature:
Name: Assist. Prof. Dr. Samir Mohamed Chassib
(Supervisor)
Date: / /

Approval of the College of Engineering

Signature:
Name: Assist. Prof. Dr. Abbas Oda Dawood
Dean, College of Engineering
Date: / /

ABSTRACT

One of the major requirements for strengthening or upgrading existing reinforced concrete structures is to increase their column capacities to withstand larger expected loads. There are different techniques to increase existing column capacities; however, such techniques differ in advantages and disadvantages. The main objective of the present study is to investigate the efficiency of confining RC concrete column with steel fibers reinforced concrete jacket. The study consists of two parts, experimental and numerical analysis.

The main variables considered in the experimental study were steel fibers types and ratio, epoxy as bond material between the concrete core and the retrofitting jacket, jacket thickness, types of strengthening (partial and full strengthening), types of confinement (hoop and composite cases) and length. Increase of hooked steel fiber ratio (0, 1, 1.5, and 2%) without epoxy showed enhancement in the maximum load by (4% and 20%) for ratios of (1% and 2%) in comparison with the strengthened column by jacket without steel fiber ratio. In case of using (2%) fibers crushed in earlier stage than column with zero steel fibers ratio. Increase the jacket thicknesses (25, 35, and 45) mm increased the ultimate load capacity by (51%, 128%, and 164%) respectively in comparison with the reference column. Improvement in the stress capacity by use of straight fibers were better than those columns which used hooked fibers. Change of the jacket height from composite to hoop case showed better development in the stress capacity. Use of steel fibers jacket in the circular columns was more significant than square columns, increase steel fibers by (1%, 1.5%, and 2%) showed increase in the stress capacity of the circular columns. Using epoxy showed lower stress capacity by (21% and 24%) for (1% and 2%) hooked steel fibers ratio respectively.

In the second part of the study, the tested columns are analyzed using nonlinear three-dimensional finite element method. ANSYS (15.0.7) program is used to analyze the three-dimensional model. Concrete core and jacket are modeled by using the 8-noded iso-parametric brick elements (SOLID65) and (LINK180) for concrete and steel reinforcement, while the loading steel plate as iso-parametric brick elements (SOLID185) with 8- nodes. Steel fibers inside the concrete is assumed to be smeared throughout the concrete element. Perfect bond between concrete core and jacket is assumed for columns without epoxy but for epoxy, (INTER205) element is used for modeling the epoxy. The adopted finite element models presented a good agreement with the experimental results. After verification, a parametric study is performed. The numerical phase of this investigation consists of 10 short square concrete small- and full-scale columns strengthened with many variables such as jacket thickness, inclined and vertical reinforcement, and types of confinement (hoop and composite cases). hoop SFRC jacketed numerical columns with inclined reinforcement improved the ultimate stress by (35%) and ultimate strain by (68%). Regarding the full-scale columns, strengthened column with composite SFRC jackets (5, 6, and 7) cm thickness showed an enhancement in the load carrying capacity by (177%, 210%, and 245%) respectively with slight increase occurred in the ultimate strain.

CONTENTS

NOMENCLATURES

Symbol	Description	Unit
A	Cross-sectional area	mm^2
E	Modulus of elasticity	MPa
Ec	Modulus of elasticity of concrete	MPa
Es	Modulus of elasticity of reinforcing bars	MPa
E_T	Strain hardening modulus	
F	Function of principal state $(\sigma_{xp}, \sigma_{yp}, \sigma_{zp})$	
f_t	Ultimate uniaxial tensile strength of concrete	MPa
f_r	Modulus of rupture of concrete	MPa
fcb	Ultimate biaxial compressive strength	MPa
f_1	Ultimate compressive strength for a state of biaxial compression superimposed on hydrostatic stress state	MPa
fc'	Compressive strength of concrete cylinder	MPa
fy	Yielding stress of steel reinforcement	MPa
f_2	Ultimate compressive strength for a state of uniaxial compression superimposed on hydrostatic stress state	MPa
$f'c$	Concrete ultimate uniaxial compressive strength	MPa
f_y	Steel yield strength	MPa
f_u	Steel ultimate tensile strength	MPa
f_{cr}, ε_{cr}	Cracking stress and strain	MPa
h	Height of the cross section	mm
I_1	First stress invariant	

J_2	Second deviatoric stress invariant	
Symbol	**Description**	**Unit**
L	length of the column	mm
Le	Effective length	mm
P and V	Any applied force on the structure	kN
P_{cr}	Cracking load	kN
P_u	Ultimate load	kN
u,v,w	Displacement components in x,y and z coordinates	mm
W_{ext} ,W_{int}	External and internal work	
x,y,z	Global coordinate	
α	Haunch angle with horizontal line	Degree
β	Shear transfer coefficient	
B_t , β_c	Opened and closed shear transfer coefficient	
γ,	Shear strain	
ε	Normal strain	mm/mm
ε_o	Strain at ultimate compressive stress $f'c$	
ε_u	Ultimate strain	
ζ , η	Local coordinates	
σ	Normal stress	MPa
σ_h^a	Ambient hydrostatic stress state	MPa
σ_{xp}	Principal stress in the x – direction.	MPa
σ_{yp}	Principal stress in the y – direction.	MPa
σ_{zp}	Principal stress in the z – direction.	MPa
σ_h	Hydrostatic stress	MPa
τ	Shear stress	MPa

ν	Poisson's ratio	
$[A]^{T}$	Transpose of matrix [A]	
Symbol	**Description**	**Unit**
$[A]^{-1}$	Inverse of matrix [A]	
$[B]$	Strain-displacement matrix	
$[D]$	Constitutive matrix	
$[J]$	Jacobian matrix	
$[K]$	Overall stiffness matrix	
$[K]_{e}$	Element stiffness matrix	
$[L]$	Differential operator matrix	
$[N]$	Matrix of shape functions	
$[T]$	Transformation matrix	
$\{a\}$	Nodal displacement vector	
$\{F^{a}\}$	Vector of applied loads	
$\{f\}$	Load vector	
$\{u\}$	Displacement vector	
$\{\varepsilon\}$	Strain vector	
$\{\sigma\}$	Stress vector	

ABBREVIATIONS

Symbol	Description
ACI	American Concrete Institute
ANSYS	(ANalysis SYStem) Computer Program
APDL	Ansys Parametric Design Language
BS	British Standards (BSI: British Standards Institute)
CFRP	Carbon Fiber Reinforced Polymer
EC	Euro Code
FE	Finite Element
FEA	Finite Element Analysis
FEM	Finite Element Method
FRP	Fiber Reinforced Polymer
GFRP	Glass Fiber Reinforced Polymer
HSC	High Strength Concrete
NSC	Normal Strength Concrete
RC	Reinforced Concrete
SF	Steel fibers
SP	Super plasticizer
SFRC	Steel fibers reinforced concrete
UHPSCC	Ultra-high-performance self-compacted concrete
UHPFRC	ultra-high-performance fibers reinforced concrete
UHPC	Ultra-high-performance concrete
FFP	Forta-Ferro Polypropylene fibers

LIST OF FIGURES

LIST OF TABLES

CHAPTER ONE
INTRODUCTION

1.1 General

The Compression member or column is one of the most important structural elements. The column may be defined as columnar constructional member support and transfer axial loads, in addition to the moments. The column's cross-section dimensions are usually less than its length. The column carries the vertical loads coming from the slabs and transferred it to the foundations [1]. Column reinforcement involving two types (longitudinal and transverse steel). The main reinforcement (longitudinal reinforcement) participates to carry the axial and flexural loads. The shear reinforcement (stirrups) resists the shear forces beside the concrete, prohibition, or postponing buckling that occurs in the main rebar due to the compression loads. Also, to confine the concrete to improve the concrete strength capacity [2]. Reinforced concrete columns frequently bear damages due to the overloading and problems of the natural phenomena such as the earthquake, storms, deluge, blazes, in addition to the variety of environmental effects (such as corrosion), and alteration in building use. Another cause is the publication of new design codes that increases the actions, such as the seismic action, prior hook up the infectious design life. The failure in the structural element may happen due to the occurred damages if suitable care is not paid in this regard. The whole building may fail to carry its design load and scourge maybe happened. Crush or failure in the most important building elements like column can cause the total collapse of the whole building as it is the only structural portion that transmits the dead and live loads to the ground. Happening the damages in these members causes losing in the strength and stiffness of the entire structure [3,4].

1.2 R.C Strengthening Techniques

Due to the imperative need for improvement in columns to ensure safety and durability. Many techniques and methods have been discovered for retrofitting the structures such as confining the columns by several techniques. Concrete confinement considered an effective method for improving the load-carrying capacity and ductility of a column because of the transverse confining stress. Subjecting laterally confining pressure to the concrete makes an effect on the concrete properties. These effects occur as follows, compressive strength f'_c, and the coincide strain ε_{cc} will be too much greater than properties in unconfined concrete f'_{co} and ε_{co}. Loading the concrete in uni-axial direction, Poisson's effect motivates transverse strain that leads to radial dilation. Happened transverse strain causes volumetric expansion [4]. Resisting the transverse expansion of the concrete will performs by the jacket. The pressure limiting effect provided by the casing is to stimulate a state of triaxial pressure in concrete that appears superior as appeared in Fig (1-1). The pressure provided by closely spaced circular spirals and vertical column reinforcement can be considered to be uniform around the perimeter of the core. This lateral pressure can be computed from statics as illustrated in Fig. (1-2). While the uniform confining pressure for spirally reinforced columns can be found from the hoop tension, it is difficult to determine the confining pressure exerted by a square or a rectangular hoop. However, an approach analogous to that used for spirally reinforced columns can be employed based on equivalent lateral pressure, starting from material and geometric properties. Passive confinement pressure exerted by a square hoop is dependent on the restraining force developed in the hoop steel or concrete. The jacket can develop high restraining forces at the corners, where transverse legs and low restraining action between the laterally supported comes support it laterally. The restraining force at the corners depends on the force that can be developed in the transverse legs, which, in turn, is related to the area and strength of the hoop steel or concrete [5].

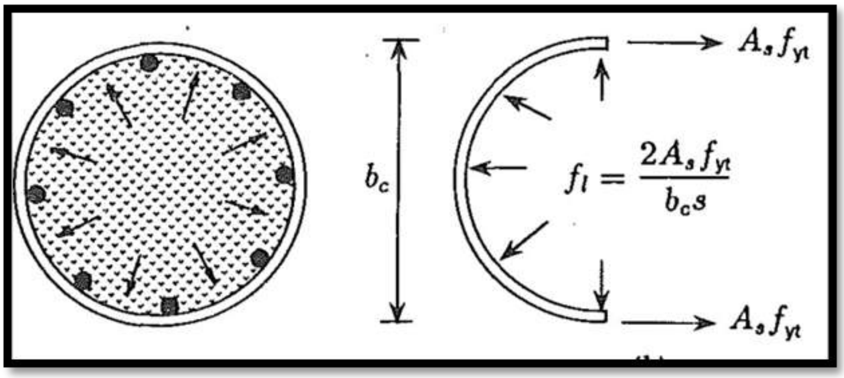


Figure (1-1): Lateral pressure in circular columns of uniform buildup of pressure; and Computation of lateral pressure from hoop tension [5].

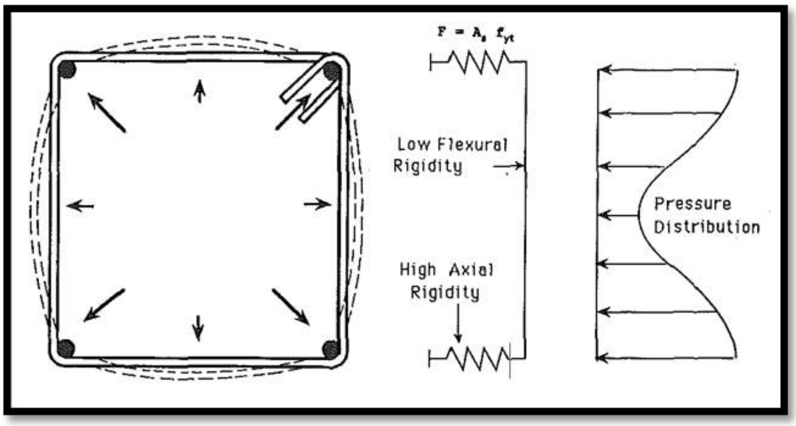


Figure (1-2): Lateral Pressure in Square Columns: (a) Lateral Pressure Buildup in Square Columns [5].

Concerning the shape factor for confinement effectiveness, in structural members, triaxial compression stress states are usually activated by the prevention of lateral expansion. The amount of lateral expansion determines the confinement stress, which is called passive confinement. Fig. (1-3) shows the dilation of the concrete cylinders due to the axial load σ33 strains the confining material in the circumferential direction and activates radial stresses [6].

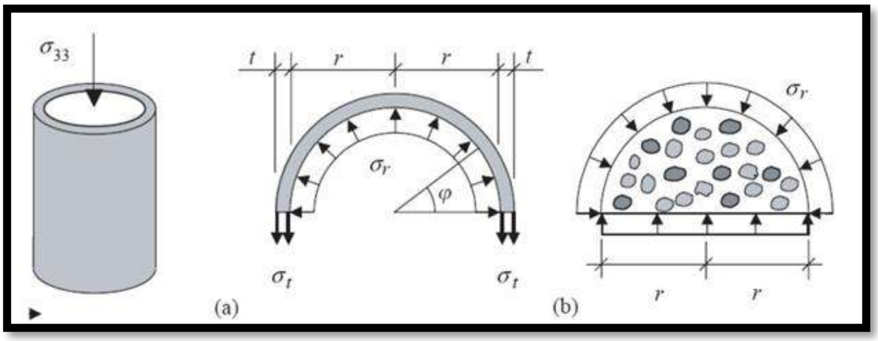


Figure (1-3): Strengthening of the RC column by jacketing [6].

The confining technique used for strengthening the RC columns is called "Jacketing". There are several types of column jacketing as following [7]:

1) Concrete casing (jacketing)

2) Steel casing (jacketing)

3) Composite Materials casing (by carbon fiber reinforced polymer CFRP)

4) Precast Concrete Jacketing

5) External Pre-stressing Jacketing using Steel Strands

6) ferrocement jacketing.

Methods of retrofitting rely on many factors such as the structure and loading type, and structure member's behavior. Regarding the buildings exposed to dynamic loads such as earthquake or wind loads, increasing flexural and shear strength will be more massive. Among the damages to the building, which may include cracking and crushing the columns, rebar buckling, or ties rupture and depending on the type of damage can determine the appropriate strengthening technique for each case [8]. The confinement of concrete columns by these means is passive by nature. The activation of these means depends on the lateral expansion due to axial compressive load. The lateral strain or the dilation of the column increases as the axial strain increases with an increasing amount of compressive load. At the instant when concrete starts to crack due to the axial load carried by the column with strengthening means, the strengthening materials will experience tensile hoop stresses [2]. The utmost public way used for retrofitting is jacketing by steel. In circular columns with a circular cross-section, the passive pressure of confinement is established from hoop tension. However, this mechanism cannot be applied in the rectangular columns, unless re-shaped these columns to have an elliptical or circular shape before a steel jacket is put in place. Due to the too many amounts of used steel, steel jacketing may be relatively costly. Each concrete jacket for the columns can perform by putting RC sleeve around the columns and this technique necessitates putting the reinforcement cage around the column. However, using RC jackets results at a relatively lower cost. Additional retrofitting techniques, that are being investigated and established for RC columns, are FRP wraps and FRP tubes, involving FRP materials. FRP can be used in strengthening systems because of its advantages relating to weight, strength, stiffness, durability, fatigue, impact, and resistance of corrosion. The use of the strengthening techniques can improve the mechanical properties of the member, make it with higher capacity without resorting to the costly techniques such as changing the cross-section. Also, this way can save more construction time as well it is possible to be used during the operation of the structure [2].

1.3 Composite Concrete Concept

Composite structures combine two or more materials in a unit structure to provide tangible benefits and a versatile solution to suite different applications. A composite system reduces the unnecessary and unwanted material properties, such as weight and cost, without sacrificing required capacity. A structure can be considered composite only so far as the various components are connected to act as a single unit. The structural performance depends on the extent to which composite action can be achieved. Composite structures, in general, have a higher stiffness and a higher load bearing capacity when compared with their non-composite counterparts. Hence the composite sections have got smaller section depth. Generally, high-rise buildings, tall structures, towers and blocks require high strength members which the composite column considered a good choice in the building construction. Consist of the ability to give more ground space and accommodating more people in less space. Hence giving us benefits such as a beautiful skyline, important landmarks and optimum land use. Main reason behind these have achieved structure's stability, strength and stiffness and for reached that purpose new innovation came in to introduction. 'Composite construction'. Composite construction governs the non- residential high storey buildings sector. This has been the case for over twenty years. Its recognition is due to it has strength and stiffness that can be achieved, with minimum use of material the reason why composite construction is frequently so good can be expressed in one simple way - concrete is good in compression and steel is good in tension. By combining these two materials together structurally these strengths can be exploited to result in a highly efficient and light weight design. Composite systems also offer benefits in terms of time saving construction. The floor depth decreases that can be achieved using composite construction can also provide major benefits in terms of the costs of services. Regarding the advantages of composite columns [8]:

- Increase the available usable floor space area for given strength.
- It has great fire and corrosion resistance where in the concrete encased columns.
- By using it, has economic advantages over conventionally use structural steel or reinforced concrete.
- Constant outer dimensions of the column over a number of floors so, architecture detailing and construction becomes easy.
- No need to additional reinforcing steel in case of composite concrete filled tubular sections provide. That's why Formwork avoidable for CFST columns. For large constructions, bridges, industrial workshops, etc.

- Drying shrinkage and creep are much smaller of composite members than ordinary conventional reinforced concrete [8].

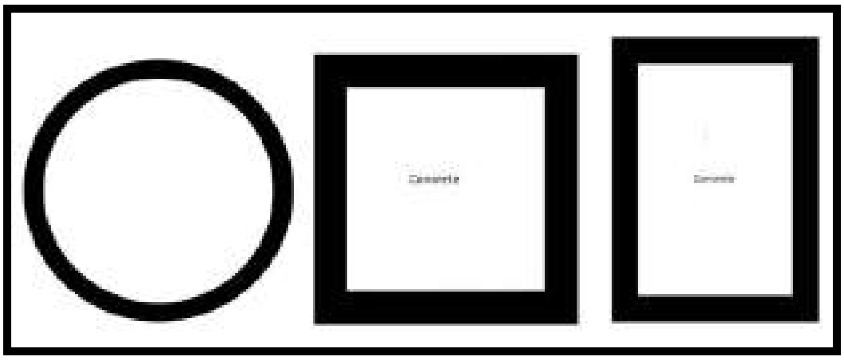


Figure (1-4): Composite columns shapes.

1.4 High-Strength Concrete

High Strength Concrete (HSC) is a modified mix of concrete; it can be getting it by adding water-reducing admixtures with high strength cementitious materials. The use of this type of concrete in structures has gradually increased during the former years leading to a reduction in the size of the structure members. Several studies have demonstrated that the economy of using HSC in low-rise, mid-rise, and high-rise buildings. The increasing use of HSC has raised the concern over the applicability of the current empirical methods, which were developed from experimental data on specimens having compressive strength below 40 MPa. HSC, as a material in compression, may be brittle, and a decrease in ductility which could be a serious drawback for HSC [8]. Research indicates that steel fibers are more effective in increasing both the strength and ductility of HSC than those of NSC. This is due to the improved bond characteristics associated with the use of fibers in HSC as compared to NSC. One of the main advantages of using the HSC is providing performance and economy of the constructed members. Using this type of concrete enhances the load-carrying capacity for the member with the same size and geometry that used with NSC which allows the engineers to construct the tall building and long-span bridges with the same size as that mentioned above [8]. It can also be considered that this type of concrete is a successful alternative to the increase in steel rebar that used in compression members. In addition to the savings cost of the used material, this mix has good speed to gain the strength which allows removing the formwork at an early age. The improvement also included decrease the deflections because of the high young modulus, less creep, and better resistance to the physical and chemical deterioration [9].

1.5 Steel Fiber-Reinforced Concrete

Steel fiber reinforced concrete (SFRC) is modified and composite material mix from cement, fine and coarse aggregate, water, and steel fibers. Due to the Lack of information concerning the performance improvements, durability, and toughness, many researchers started to investigate these mechanical behaviors since the 1960s. Since that time, researchers made a widespread study on SFRC, motivated by formulas had exposed that increasing indication that concrete brittle behavior can be got by adding steel fibers. The addition of steel fibers to the concrete makes the construction of the airport, erosion resistance structures, building against the earthquakes, and explosive resistance structures [10]. The main purposes of using these fibers are:

(a) Upgrade the flexural and tensile strength.
(b) Improve the toughness and impact strength of the structure.
(c) Using these fibers reduce and control the cracks spread and failure mode.
(d) Enhance durability.

The effect of steel fibers on the compressive strength of concrete is variable. Typical stress-strain curves for steel fiber reinforced concrete in compression and presented in Fig. (1-5) [11]. In stress-strain curves for steel fiber reinforced mortars, a substantial increase in the strain at the peak stress can be noted, and the slope of the descending portion is less steep than that of control specimens without fibers. This is indicative of substantially higher toughness, where toughness is a measure of the ability to absorb energy during deformation, and it can be estimated from the area under the stress-strain curves or load-deformation curves [11]. The improved toughness in compression imparted by fibers is useful in preventing sudden and explosive failure under static loading, and in absorbing energy under dynamic loading. Under compression, HSC is significantly brittle. For this purpose, HSC has lacked in ductility. Ductility of RC columns with high strength features can be upgraded by confining the concrete core by material jacketing with high ductility behavior such as the SFRC. The steel fiber concrete mix has an improved ductility because of the elasticity in the steel material. Using of steel fiber in concrete constructions had become a very common and active way to upgrade the overall behavior, in addition to his ability to control the homogeneity of the mixture by spreading it. Furthermore, adding the steel fibers in a concrete mix of column outcomes an increase in its ductility, cohesive between the concrete particles, and stops early cover spalling [9] as shown in Fig. (1-5) [11]. Variation in tensile stress-strain curve of different types of steel fibers seems clear as demonstrated in Fig (1-6).

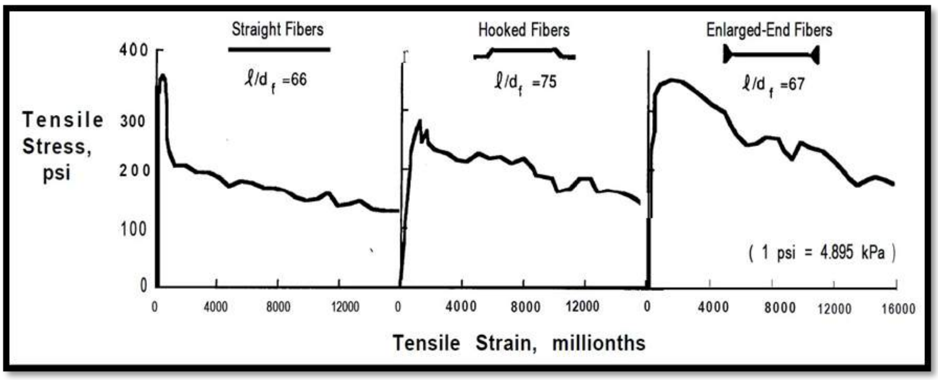


Figure (1-5): Stress-strain curves for steel fiber reinforced mortars in tension [11].

Figure (1-6): Steel fiber reinforced concrete column [11].

1.6 Objectives of Research

The objective of this study is to examine the behavior of fiber RC short columns and to investigate the effect of using steel fibers as a strengthening technique of columns. The main purposes of this work are:

1. To inspect the effects of different variables on the column behavior, such as kind and ratio of steel fiber, the shape of columns and thickness of strengthening.
2. To explore the effect of steel fibers on improving the post-peak behavior of short concrete columns in providing a passive confining pressure to enhance its strength and ductility.
3. To evaluate the adequacy of the proposed models for confined concrete strength.

1.7 Thesis Layout

The study is offered in five chapters, as follows:

[1] **Chapter One**: presented a general introduction for confined column, application, steel fibers effect, and high strength concrete.

[2] **Chapter Two**: presented literature review concerning the experimental and theoretical studies of the confined columns with variable materials.

[3] **Chapter Three**: presented material description, and geometrical details of the columns.

[4] **Chapter Four**: presented the results and discussion of the test results.

[5] **Chapter Five**: summarized the conclusions and recommendations.

CHAPTER TWO
LITERATURE REVIEW

2.1 General

The common retrofit techniques that use for repair and improve the general behavior of the structure members are jacketing by concrete, steel, fiber reinforcement polymer (FRP). Jacketing by concrete can be constructed by enlarging the cross-section with a new layer of concrete and reinforcement. This reinforcement is traditionally provided by hoop or spiral rebar, or welded wire fabric. FRP reinforcement is typically applied in two ways: prefabricated (tubes) jackets or wraps. Steel jackets are constructed by placing a steel sheet or tube with diameter larger than column in slight value around.

In this chapter, a literature review is presented of the research as performed in the area of confinement of concrete columns, with a special emphasis on the commonly used confinement models. The confinement of concrete columns is reviewed in the first part of the chapter, followed by the behavior of in the second part.

2.2 Confinement History

Since the last decades, the use of confinement by full or partial jackets with several materials such as concrete, steel tubes and angles, FRP …etc. According to the obtained researches, experimental and theoretical studies started since 1928 by Richart et al.[12] to 2019 by Zhou et al. [13]. Most of these studies with varied parameters and strengthening method referred to the increase in the mechanical properties of columns. The last studies about strengthening the columns can be showed by sort the studies according to the year and type of the material as follow:

A. Confinement by Reinforced Concrete

In twentieth century, Richart et al. [12] (1928) pioneered studies regarding beneficial effects of lateral confinement on the strength and deformation characteristics of concrete. It was reported that an increase in lateral pressure leads to significant increase in ductility and strength, and reduces internal cracking. Since then, numerous experimental and analytical studies have been conducted on confined concrete.

In 1988, Bett et al. [14] conducted testing of square columns with concrete jackets strengthening under compressive axial loading as well as lateral loading. Three square column test specimens were constructed, retrofitted with a concrete jacket, and then tested. One of the specimens was tested, repaired, and then retested. The other two specimens were strengthened before testing. To imitate the earthquake phenomena, lateral and axial loading used to test the capacity of these specimens. It was also proved that the damaged columns reinforced by concrete jackets appeared the same strength and stiffness to intact counterparts without concrete jackets. While the strengthened columns without damages showed an improvement by many times in the stiffness, strength, and ductility.

In 1994, Rodriguez and Park [15] presented an extensive study on the columns with rectangular shapes with the use of peripheral strengthening and subjected to the axial and lateral loads to see the effect of change in general behavior after the addition of concrete jackets. Concrete jacket is strengthened by using rebar hoops around the column. The results appeared that jackets improved the strength and stiffness by up to threefold. The outcomes illustrated that the damage before the retrofitting process has no considerable effect on the overall performance of the jacketed columns. Overall and concerning the specimens that have jackets with strengthening by hoop reinforcement exhibited enhancement in stiffness, strength, and ductility.

In 2001, Lehman et al. [16] used three approaches to repair and strengthen loaded columns to examine the behavior of columns with concrete jacket. Testing included damaging the columns by subjecting an axial and lateral loading. These damages involved crushing, buckling in concrete and longitudinal rebar, cutting, and separating the spiral rebar. Transverse reinforcement was used to strengthen the RC columns were reinforced then clean the cover by removing the loose concrete due to the exceeded loading. As conclusion, these jackets strengthened the member by increasing both stiffness and strength in comparison with reference columns.

In 2008, Konstantinos et al. [17] presented an experimental investigation about concrete jacket effectiveness on the columns. Three alternative methods of concrete jacketing are investigated and results are compared with results from an original unstrengthen specimen and a monolithic specimen. The unstrengthen column and the original columns of the strengthened specimens were designed to old 1950s Greek Codes. Various construction procedures were carried out in order to evaluate if it is worth performing the procedures when considering the practical difficulties involved. These procedures involved welding the jacket stirrup ends together, placing steel dowels across the interface between the original column and the jacket in combination with welding the jacket stirrup ends together and connecting the longitudinal reinforcement bars of the original column to the longitudinal reinforcement bars of the jacket. Earthquake simulation displacement controlled cyclic loading was used for the testing. The seismic performance of the tested specimens is compared in terms of strength, stiffness and hysteretic response. Even when the jacket was constructed with no treatment at the interface, a significant strength and stiffness increasing had been observed. It was also found that the failure mechanism and the observed crack patterns are influenced by the strengthening method. The separation of the jacket from the original column was obvious in the case when there was no treatment or other connection means performed at the contact interface between the column and the jacket. In addition. It was found that welding the jacket stirrup ends together stopped the longitudinal bars of the jacket from buckling as revealed in Fig. (2-1).

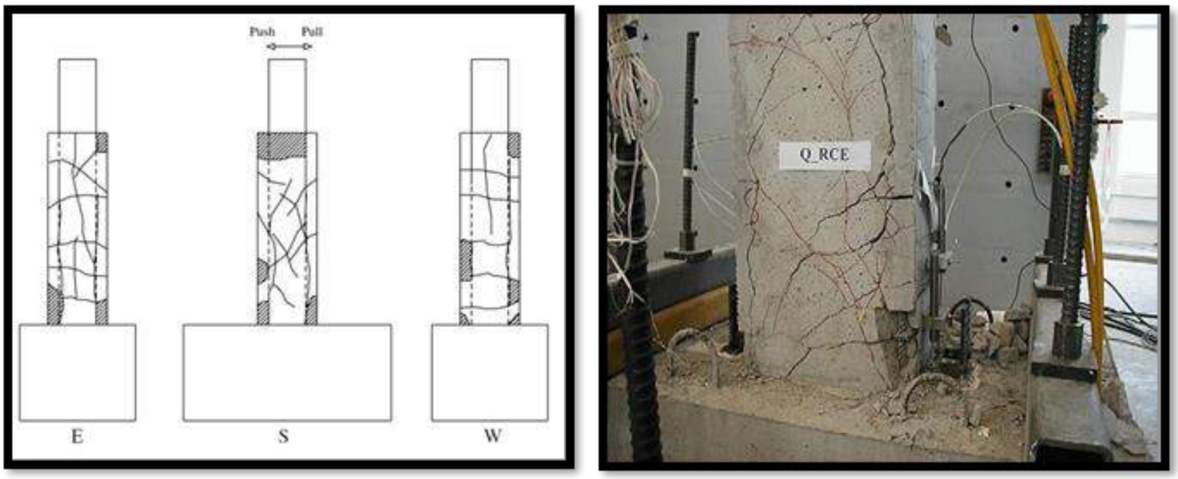


Figure (2-1): Crack patterns for strengthened specimen [17].

In 2014, Heles [18] studied the behavior of square RC column jacketed by ultra-high-performance self-compacted concrete (UHPSCC). Total of 27 identical column cores were made of NSC having similar cross sections of 100×100mm and 300mm high. Three jacketing styles with three jacket thicknesses (25, 30 and 35) mm were used. The first group consisted of nine column cores jacketed by Fibrous UHPSCC without steel reinforcement, group two consisted of nine column cores jacketed by Non-Fibrous UHPSCC with steel reinforcement, and group three consisted of nine column cores jacketed by fibrous UHPSCC with steel reinforcement. A comparative study was exhibited an important increase in the ultimate load carrying capacity higher about 4.4 times than the reference column. The measured longitudinal axial strain of groups one and three jacketed column specimens were higher about 2.1 and 2.3 times that reference columns.

The results also revealed that the failure patterns and crack formation were significantly influenced by both the jacketing thickness and the jacketing style. Adding Forta-Ferro Polypropylene fibers (FFP) of 1.5 % the volume of UHPSCC mix has improved the properties by increasing the maximum longitudinal axial strain of jacketed column specimens. The group three jacketing style have significantly increased the ultimate load carrying capacity and ductility of the jacketed column cores because of the application of both Fibrous and steel reinforced UHPSCC jacket.

In 2016, Koo and Hong [19] made a research about strengthening RC columns with ultra-high-performance concrete (UHPFRC). The dimensions adopted for the four-columns cross section and height were 300×300mm and 1260 mm respectively. One column (R0) left unstrengthen, the other columns were jacketed with UHPFRC in different ways: first one(R3) was strengthened with 30mm jacket; the second one (R5) was strengthened with 50mm jacket; third specimen(R5S) was strengthened with 50mm jacket plus stirrups(D10@150) inside UHPFRC. The length of the fiber, 13mm was chosen to minimize congestion between steel fiber and reinforcing stirrups considering the thickness of jacket and stirrup diameter. From the experimental results, the UHPFRC jacketing method shows high strengthening effect. The result showed that 10% of column thickness jacket brought over 70% shear strength increase and 16.7% thickness brought over 125% strength increase. Additional strength can be achieved by adding transverse reinforcement, if needed.

In 2018, Cho et al. [20] studied strengthening of RC columns by high-performance fiber-reinforced cementitious composite sprayed mortar with strengthening bars. Three specimens of dimensions (600x600x1600) mm were tested. specimen1 was without strengthening, second specimen with transverse reinforcement strengthening only. Third Specimen was with transverse and longitudinal reinforcement strengthening. The outcomes exposed the retrofitting by HPFRC mortar could prevent the failure by shear by reduce the diagonal shear cracks. Also, this retrofitting could reduce the bending cracks amount in the plastic hinge region. The hysteretic damping energy had been enhanced during the cyclic load reversal due to strengthening of the column.

Tayeh et al [21] in 2019 carried out nine reference 300mm long RC column specimens: three specimens had a cross-sectional dimension of 100mm× 100 mm, three specimens had a cross-sectional dimension of 150mm× 150 mm, and three specimens had a cross-sectional dimension of 170mm× 170 mm. Total of 36 identical column cores were cast with similar cross sections of 100mm× 100mm and a height of 300 mm. these cores were damaged by loading them with approximately 90% of their actual ultimate axial load capacities. Then, the columns were repaired and strengthened by applying two jacketing materials, which were 25 and 35mm thick, on all four sides. The first group consisted of 18 column cores jacketed by normal strength concrete with a maximum aggregate size of 4.75mm and steel reinforcement, whereas the second one consisted of 18 column cores jacketed using ultrahigh-performance fiber-reinforced self-compacting concrete with steel reinforcement. The experimental program showed that the first group specimens had ultimate load capacities more than twice those of the unjacketed reference columns and the same axial capacity as the monolithically cast reference columns. The second group specimens showed a significant increase in ultimate load capacity, which was approximately 3 times that of the unjacketed reference column and 1.86 times that of the monolithically cast reference columns. Moreover, using the shear studs was found to be the most effective among the three surface preparation techniques.

In 2019, Xie et al. [22], presented an experimental investigation concerning strengthening the concrete column by UHPC jacket. The studied parameters in this testing program were the thickness of jacket and the shapes of specimens. The influences of these parameters on the compressive behavior of stub concrete columns confined by UHPC jacket were reported and discussed Six groups of specimens with three identical specimens in each group were constructed. 18 specimens in total, were prepared in this testing program. Each specimen consisted of a plain concrete stub column and an externally strengthening UHPC jacket. To study the effects of the shape of the specimens on the compressive behaviors, the first four groups were designed with circular cross section whilst the rest two groups of specimens were designed with square cross section. For studying the influence of the thickness of strengthening UHPC jackets, the first four groups of testing specimens (i.e. C1–C4) were designed with external UHPC jackets in different thickness of 0, 20, 30, and 40 mm. For the square specimens (i.e., S1–S2), the inner square stub concrete column measures 150 mm and 300 mm in width and height, respectively. A gap of 15 mm between the concrete columns and UHPC jacket were reserved at both loading ends of the specimens. The compressive behaviour of stub concrete columns with UHPC jacket was also simulated by 3D nonlinear finite element model (FEM). Test results showed that introducing UHPC jacket on stub concrete column increased its ultimate compressive resistance, ultimate displacement, and ductility. The amplitudes of these improvements were influenced by the thickness of UHPC jacket and shapes of the columns. For cylindrical columns, increasing the thickness of the UHPC jacket from 0 to 20, 30, and 40 mm increased the compressive resistance by 14.9%, 27.5%, and 46.4%, respectively meanwhile, the displacement at ultimate load was also increased by 20.3%, 45.4%, and 68.0%, respectively However, increasing the thickness of UHPC jacket shows no improvement on the initial stiffness of concrete columns. Compared with the cylinder stub columns, UHPC jacket exhibit less significant influences on the compressive behavior of square stub concrete columns. Validations of predictions of the FE and test data indicated that FEM offered reasonable estimations on the compressive behaviours of the concrete stub columns strengthened by UHPC jacket as revealed in Fig. (2-2).

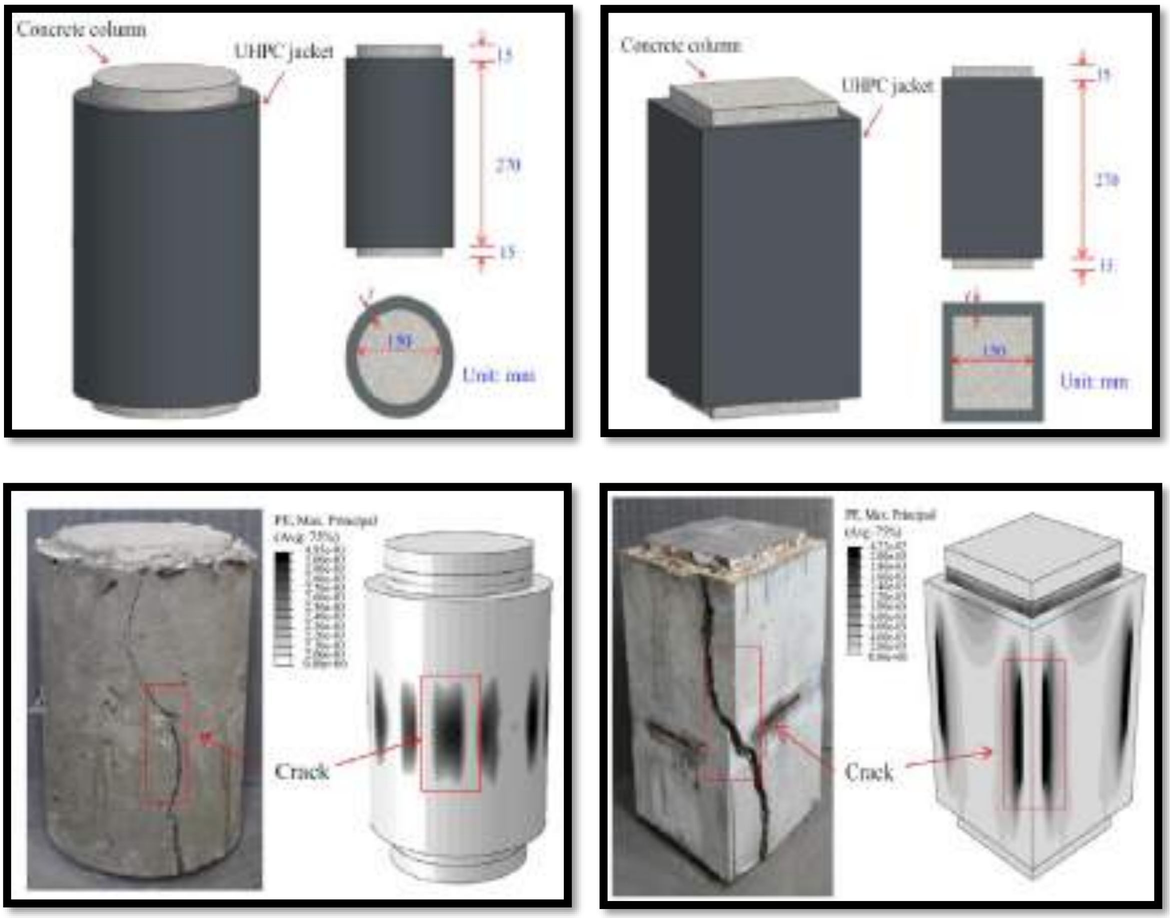


Figure (2-2): Square and circular columns with concrete jackets presented by Xie et al. [22].

Table (2-1): Summary of studies about strengthening by concrete jackets.

Researcher	Strengthening Method	Strength and Ductility	Stiffness
Richart et al. [12]	lateral confinement by concrete jackets	Enhanced	Not reported
Bett et al. [14]	Concrete jackets constructed on damaged column	Have multiplied	Have multiplied
Rodriguez and Park [15]	Concrete jacket is retrofitted by using rebar hoops around the column.	Enhanced	Enhanced
Lehman et al. [16]	Jacketing by concrete after clean the cover by removing the loose concrete	Enhanced	Enhanced
Konstantinos et al. [17]	Welding the jacket stirrup ends together, placing steel dowels across the interface between the original column and the jacket	Enhanced	Enhanced
Heles [18]	jacketing by fibrous UHPSCC with & without steel reinforcement,	Upgraded by 4.4 times	Not reported
Koo and Hong [19]	Confinement with UHPFRC in different jackets thicknesses	Enhanced	Enhanced
Chang-Geun Cho and Byung-Chan Han [20]	Confinement with HPFRC sprayed mortar	Enhanced	Not reported
Tayeh et al [21]	Confining the damaged columns by NSC & UHPFRC with varied thicknesses	Enhanced	Enhanced
Xie et al. [22]	UHPC jackets in different thickness with gap of 15 mm between the concrete columns and UHPC jacket	Enhanced	Enhanced

B. Confinement by Ferrocement Jackets

In, 1990, Mansur and Paramasiva [23] predicted strength of the short columns strengthened by ferrocement jacket subjected to an axial loading with varied distance at the longitudinal center and eccentric. The key parameters were the arrangements, types, and reinforcement fractions volume. The outcomes exhibited that ferrocement box in which strengthening mesh strata were tucked in shape of surrounded cage considered a good way to strengthen the column. Using the welded wire mesh appeared a better performance than the woven mesh with the same amount.

Takiguchi et al. [24] in 2001, investigated the effect of using ferrocement jackets to improve the overall behavior of circular columns regarding to the shear capacity. Four specimens were constructed and divided into two groups. The first group involved two columns were tested as control specimens. The remaining two columns were tested after being strengthened with four and six sheets of wire mesh respectively. The conclusions were drawn from the test results that circular ferrocement jacket was effective in preventing the shear failure that occurred on control specimens, in restoring the flexural strength and enhancing the ductility of concrete columns.

Abdullah and Takiguchi [25] in 2003, presented an experimental study to examine the behavior of RC columns after strengthening by ferrocement jackets. Six specimens with different cross-section shape were analyzed and retrofitted by circular and square ferrocement jackets. This study included several variables such as jacketing schemes and layers number of wire mesh. The results exposed that the ductility can be enhanced by using the confinement for the whole height. Its conclusions exhibited that the use of square ferrocement jackets can be used for the strengthening the columns with small shear strength due to the ability of the jackets to improve the column ductility. In addition, method of the used design, projected by the researchers had a good outcome. Fig. (2-3) explains the details of the tested specimens.

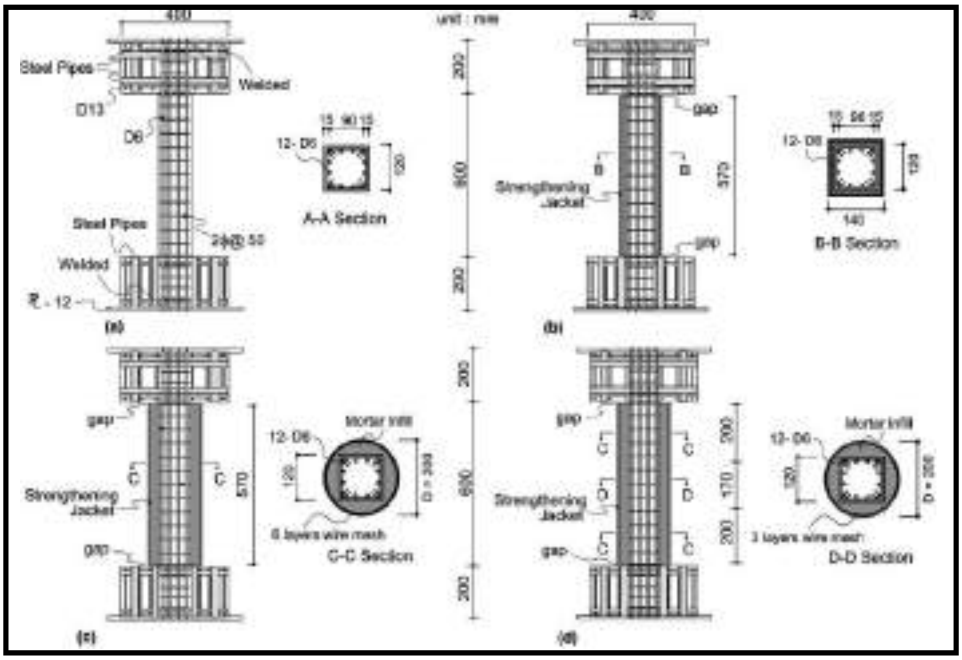
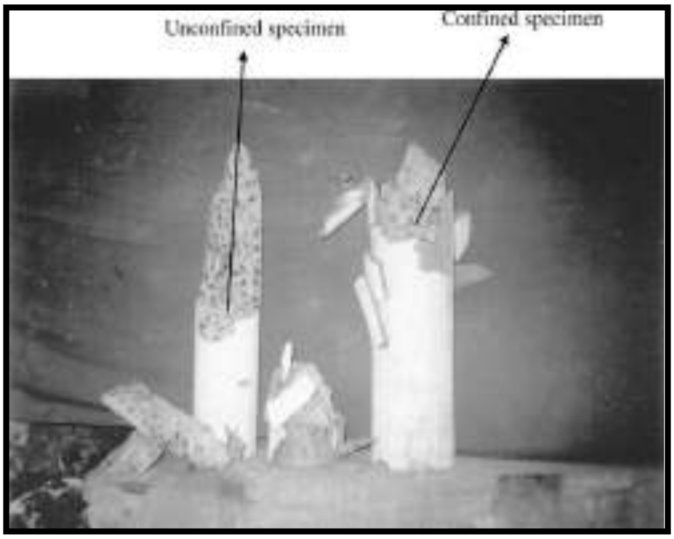


Figure (2-3): Square and circular columns with ferrocement jackets presented by Abdullah and Takiguchi [25].

Kondraivendhan and Pradhan [26] in 2009, devoted a study to put an outer of ferrocement confinement on RC columns. The main test parameter that used in this study was the compressive strength of these columns. Remaining parameters dealt with geometric changes such as shape, using wire mesh with varied numbers of layers, size and ratio of height to diameter (L/D). The test outcomes displayed that the confining feature enhanced the ultimate capacity of these column with significantly changes in the failure mode as appeared in Fig. (2-4).

Figure (2-4): Failure modes of confined and unconfined concrete specimens [26].



Mourad S.M., Shannag M.J. [27] in 2011 performed an investigation to examine the influence of adding the ferrocement jackets on the general behavior of square RC columns. Ten specimens with 1:3 scale square RC column models were fabricated and compressed by axial loads with various percentage of the maximum capacity (0%, 60%, 80%, and 100%) and the construct the ferrocement jacket with putting two layers of welded wire mesh. The program consisted of analyzing specimens with dimensions (150 x 150 x 1000) mm in three segments as following; Stage 1: the reference specimens were constructed without jacketing. Stage two included jacketing the specified specimens by ferrocement and without preloading. While stage three involved strengthening the columns by ferrocement jackets with preloading these models by 60, 80, and 100 % respectively. The outcome exposed that retrofitting by these types of jackets made unloading for these columns with using two wire mesh layers by 33% and 26% by made increase in its maximum strength and stiffness respectively in comparison with the control specimens. In addition, repairing similar RC square columns made a preloading larger than 60% and 80% of its maximum capacity.

Jaafer [28] in 2012, studied the strengthening of short column using ferrocement. A total of forty-eight circular short concrete columns were prepared for the test. The jacket thickness was constant and equal to 23 mm for all strengthened specimens. The strengthened columns were divided into twelve groups (J-J11). Group J and J9 have one specimen only and remaining groups, four specimens were prepared one of them was tested under monotonically increasing axial compressive loading condition and the three others had been tested for cyclic axial load. The main variables considered were the volume fraction, the mortar compressive strength, column size, and column loading type. It was found that the ferrocement jacket gave a sufficient crosswise support to the concrete core with marked increment in the strength and ductility of these specimens as shown in Fig. (2-5).

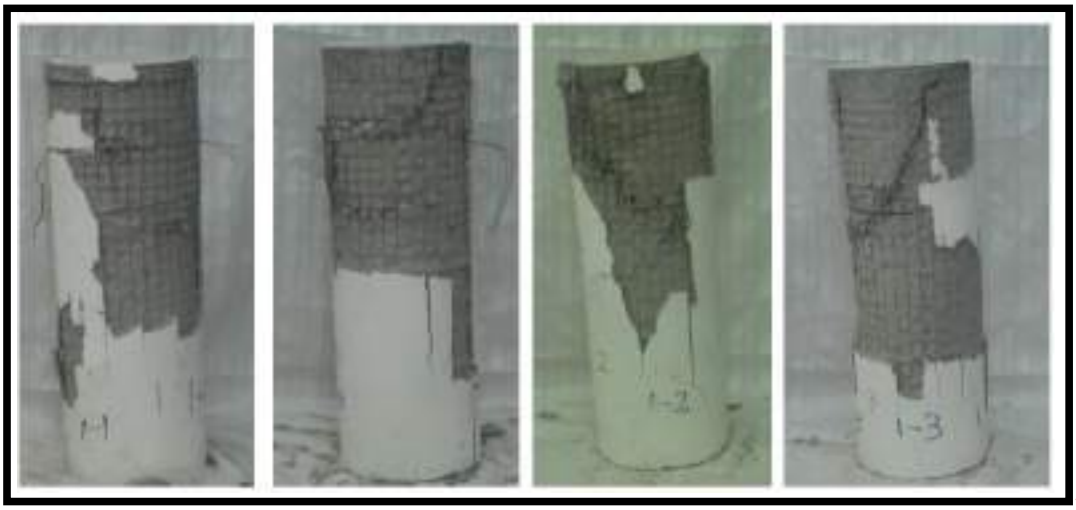


Figure (2-5): Failure mode for the strengthened RC circular columns [28].

Rehabilitation short square columns by using the most important technique for strengthening the damaged members occurred by a study by Soman and Veena [29] in 2015. There specimens had been prepared by using of optimum percentage of mix and then the curing occurred for the specimens for 28 days. The columns consisted of two series with 140 x 140 x 1200 mm; the first one involved square columns were retrofitted by rounding ferrocement jacketing in the corners of the specimen by 20 mm thickness. While the second series of columns were strengthened using conventional jacketing and the third one included damaging the members by subjecting loading to reach the overall failure. Loading the remaining square columns to 40, 60, and 80 % from the overall capacity. The outcomes revealed that fully damaged models restored the full strength and stiffness that it has before loading when the rehabilitation made by jacketing. Also, the two techniques for strengthening had different effect on the entire behavior as follows; the energy absorption was improved due to the use of jackets because the confinements delay the failure which cause more deformation in the concrete core.

Sabu and Binu in 2017 [30] made a search about strengthening of RC cylindrical columns using micro concrete jacketing to increases the seismic resistance, strength, stiffness, ductile behavior and lateral load capability of the building without any demolition. The control columns to be jacketed later are cast in molds of 150 x 600 mm. Tied wire mesh is used as additional reinforcement in the jacketed region. Control columns are loaded up to 80 % ultimate load. Comparing with the control specimen, there is a 12.5 % strength increment for conventional concrete and 86.6% increase for micro concrete jacket. Among the two types of jacketing, micro concrete jacketing has a more ultimate load, the control specimens failed suddenly while jacketed specimens failed gradually. The load-deflection diagrams were plotted and compared for various jacketed specimens.

In 2019, study on strengthening of RC columns using ferrocement wrapping has been presented by Balamuralikrishnan et al. [31]. This paper examines the performance of the ferrocement wrapping in RC columns experimentally with numerical simulation using ANSYS19. Totally sixteen number of RC column of size 150 mm × 150 mm in cross section and 450 mm in length were cast and tested in laboratory. Twelve specimen were strengthened by wrapping technique which six columns were strengthened by full wrapping technique and six columns of strip wrapping technique. The remaining four columns are control columns in virgin condition to compare with the strengthened columns. Concerning the volume fraction of each specimen, the number of pre-woven mesh layers were single layer, double layer and three layers. Finite element analysis using ANSYS19 adopted to compare the experimental data with the numerical simulation. The results were analyzed and observed that the ferrocement has increased the confinement and strength of the RC columns. The retrofitted columns with full wrapping and strip wrapping at the optimum volume fractions showed decreased deflection with respect to control column at the same load level by 50 to 60% and 30% respectively. The cracks in the strengthened RC column with ferrocement full wrapping and strip wrapping occurred in crushing mode of failure and none of the columns exhibits brittle or premature failure as revealed in Fig. (2-6).

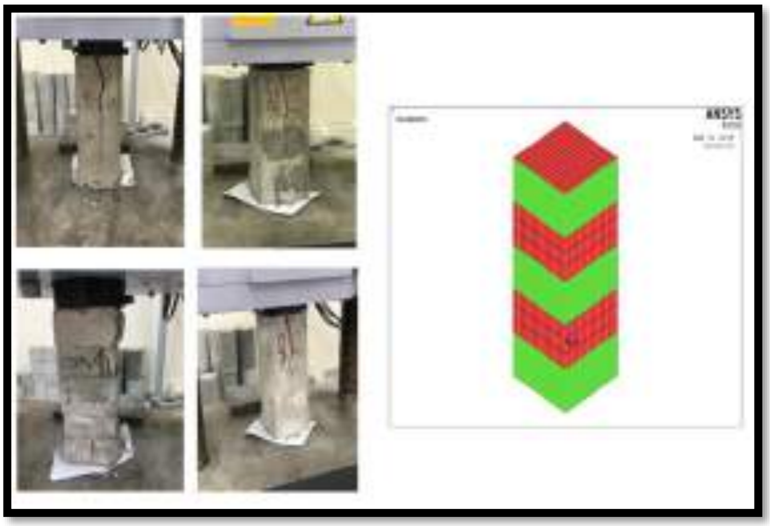


Figure (2-6): Failure modes of confined and unconfined concrete specimens [26].

Table (2-2): Summary of studies about strengthening by ferrocement jackets.

Researcher	Strengthening Method	Strength & Ductility	Stiffness
Mansur and Paramasiva [23]	Confinement with ferrocement box sections	Enhanced	Not reported
Takiguchi et al. [24]	Confinement with ferrocement jackets with wire mesh around circular columns	Enhanced	Not reported
Abdullah and Takiguchi [25]	Confinement with ferrocement jackets with variable wire mesh layers	Enhanced	Enhanced
Kondraivendhan and Pradhan [26]	Ferrocement Jacketing with varied numbers of layers, size and ratio of height to diameter	Enhanced	Enhanced
Mourad, and Shannag [27]	Confinement with ferrocement jackets with putting two layers of welded wire mesh	Enhanced	Enhanced
Jaafer [28]	Confinement with ferrocement jackets with wire mesh around circular columns	Enhanced	Not reported
Soman and Veena [29]	rounding ferrocement jacketing in the corners of the specimen by 20 mm thickness	Enhanced	Enhanced
Nayana V Sabu [30]	Using of Micro concrete Jacketing with tied wire mesh	Enhanced	Enhanced
Balamuralikrishnan et al. [25]	Retrofitting of RC column using ferrocement full and strip wrapping	Enhanced	Not reported

C. Confinement by Steel Jacketing

In 1998, Schneider [32] offered a study with both experimental and theoretical to examine the behavior of short concrete-filled steel tube columns. A Group of 14 models was constructed and investigated the effect of the steel jacket with variable thickness on the maximum strength capacity. Concerning the geometry of the used jacket, depth over thickness were ranged between (17- 50), while length over depth was (4-5). Regarding the theoretical side, nonlinear finite element method was used to represent these models for doing the verification to prove the accuracy of the gotten results. The test outcomes revealed that these circular jackets offered a gaining post-yield strength, ductility, and stiffness, with a value higher than that found in most other square or rectangular cross sections.

Thirteen specimens casted to examine the mechanical behavior of circular composite columns. This research presented in 2002 by the Johansson and Gylltoft, [33] when they carried out a steel concrete composite column with dimensions 159 x 650 mm. To inspect the influence of loading conditions, three loading stages were used. Concerning the analytical side of this research, nonlinear finite-element models were carried out to prove the accuracy the experimental results and to check the how the bond can affect the general behavior of these columns. This study exhibited that behavior was greatly affected by the used method of applying loads. In addition, when the loads applied on the concrete and steel jacket together in the same time, the effect of the bond strength on the load transformation of behavior was impalpable. The buckling type at the failure mode varied because it depends on the applying load modality. The researchers concluded the if the load applied on the whole column or only on the concrete core it will cause a loss in its stability which occurred due to the combination of crushing and local buckling as shown in Fig. (2-7).

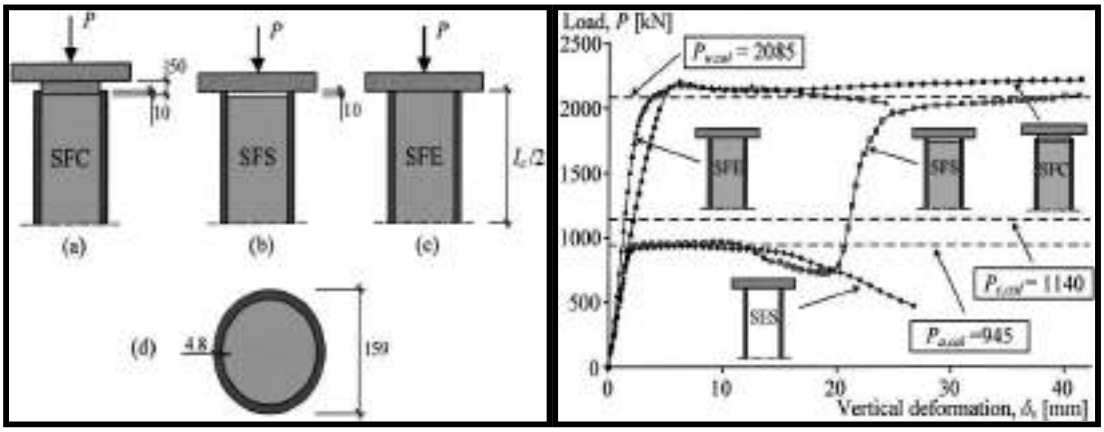


Figure (2-7): The specimens that presented by Johansson and Gylltoft [33] with using variable cases of loading with its results.

In 2005, Lam and Wong [34] performed a group of tests for short composite concrete-steel columns to inspect the influence of these jackets on the entire behavior. The specimens were prepared by casting of NSC and HSC and the using the steel jackets to enclose these columns. Eight columns with dimension of 100 x 100 mm were constructed with using three concrete compressive strength starting from the values of normal strength reaching the high strength (30, 60, and 100 MPa) and two thickness values for the steel jackets (2 and 5 mm). The results demonstrated that the standard instructions for design of these strengthened columns offered a sensible forecast of the compressive strength. Also, Eurocode (EC) 4 displayed that it offered to provide an over speculation for the compressive strength for composite columns of concrete with high strength property. Whereas the ACI- 318 and Australian codes give lower speculation for the compressive strength for columns filled with normal strength property.

In 2006, a research was published by Yang and Han [35] about investigation the behavior of circular and square columns with normal concrete and recycled aggregate concrete retrofitted by steel jackets. Total of 30 columns divided into two series strengthened by steel jackets. The first one included 24 columns with recycled aggregate concrete while the second series consisted of six columns with normal concrete. The main variables that used in this research were the jacket shape of steel, concrete type, and load eccentricity ratio which ranged from (0 to 0.53). The test outcomes exhibited that both types failed to resist the overall buckling. A comparison was performed between the code of practices of American, British Japanese, and Eurocode for the forecast maximum strengths. Concerning the analytical side, the results between the analytical and experimental offered a good matching.

In 2014, Abdel-Hay [36], examined the influence of steel jackets on the maximum capacity of RC columns experimentally by casting columns and theoretically by modeling these columns by ANSYS program. Seven specimens with dimensions 200 x 200 x 1500 mm were carried out, one of them was reference column while the remaining six were the parametric specimens. The main variable in this research were partial retrofitting by steel jackets (using 4 angles at the column corners), using outer ties with varied spacing, and using steel plates with different thickness connected to the column by welding and bolts. The results exposed several points as follows; the failure was brittle by crushing the concrete core, increasing external supporting plated and reducing the spacing of ties improved the strain behavior and ductility. The results illustrated that it is not recommended to use steel plate with thickness less than 3 mm for retrofitting the columns. Also, the occurred failure happened outside the retrofitted portion as revealed in Fig. (2-8).

Figure (2-8). Failure modes of confined and unconfined concrete specimens.

In 2016, Mosheer [37] studied the failure mode and the behavior of strengthened RC columns by partial jacketing. Thirteen RC columns were built. The cross sections of columns were 150 ×150 × 1700 mm. These columns were separated into two sets, the first one was strengthening columns by external steel collars, and the second assembly was repairing damaged columns by external steel collars as displayed in Fig. (2-9). An external steel angle (L33×33×2) mm, (L40×40×3) mm, and (L40×40×4) mm was selected to made collars. The result showed that the confinement with external steel collars techniques in reinforced concrete columns can improve strength and ductility of strengthening and repairing columns and enhancing concrete capacity duo to increases the lateral pressure on the member. Confinement with external steel collars increased the axial deflection by about (18.3 – 60%) of reference column, and decreased the lateral deflection of a reinforced concrete column by about (11.5 – 82.5%) of reference column. The confinement techniques provided increasing in ultimate axial load reach to (47.5 –96.7%) for repairing column.

Figure (2-9): Strengthened Columns by Mosheer [37].

In 2019, Zhou et al. [13] have implemented a wide experimental and theoretical research. This study involved an analysis of seismic performance and bearing capacity of circular RC columns strengthened by wrapped steel plates. formula for calculating the bearing capacity of circular RC columns strengthened with externally wrapped steel plates was derived, with reference to the code for design of strengthening concrete structure in order to investigate of steel plate on the entire behavior of these columns. The experimental side included testing of 45 cm diameter and 420 cm length reinforced with 6 Ø22 and Ø10@10 cm for longitudinal and transverse reinforcement respectively as exhibited in Fig. (2-10). As for the theoretical side, it included a verification process with practical models, as well as theoretically with the proposed equations, as it showed good results. The results showed that the bearing capacity of the normal section of the RC column after strengthening was about 80% higher than that before strengthening, and the results of FEM software were in accord with the calculation results of theoretical formula to some degree. Under horizontal low cyclic reversed loading, the columns after strengthening had both plumper hysteresis curves and higher ductility factors and equivalent viscous damping coefficient than those before strengthening, indicating the energy dissipation capacity, plastic deformation capacity, and seismic performance of the RC columns after strengthening were all obviously improved.

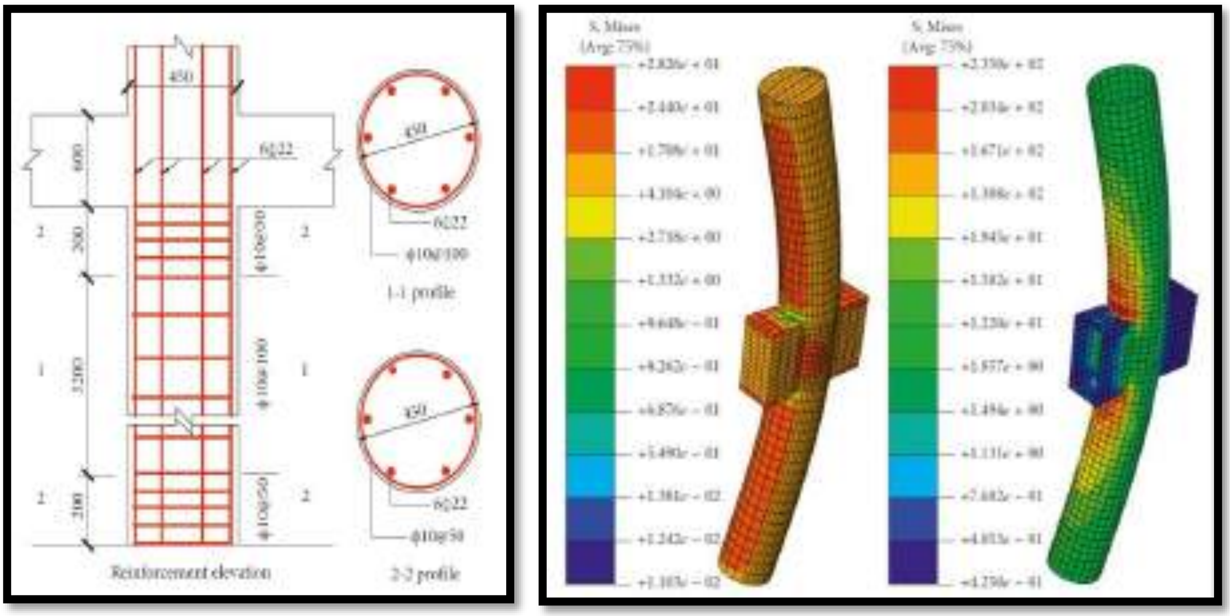


Figure (2-10): Experimental and theoretical specimens of Zhou et al. [13].

Table (2-3): Summary of studies about strengthening by steel jackets.

Researcher	Strengthening Method	Strength & Ductility	Stiffness
Schneider [38]	Confinement with steel tube with variable thickness	Enhanced	Enhanced
Johansson and Gylltoft, [40]	Confinement with steel jacket with variable cases of loading	Enhanced	Enhanced
Lam and Wong [42]	Jacketing the NSC & HSC column with steel jackets with variable thicknesses.	Enhanced	Not Reported
Yang and Han [44]	Confining the NSC and recycled aggregate concrete	Enhanced	Not Reported
Abdel-Hay, A., S. [55]	partial retrofitting by steel jackets (using 4 angles at the column corners), using outer ties with varied spacing, and using steel plates with different thickness connected to the column by welding and bolts.	Enhanced	Enhanced
Mosheer [36]	Partial strengthening columns by external steel collars (steel angles)	Enhanced	Enhanced
Zhou et al. [25]	strengthening with externally wrapped steel plates	Enhanced	Not Reported

D. Confinements by Fiber Reinforcement Polymers (FRP)

In 1997, Pico [38] analyzed of total of 9 (152.4mm x 152.4mm x 304.8mm) of square columns jacketed by FRP tubes. The used parameter was the cross section in order to inspect its effect on the general behavior. It should be noted that the connection between the FRP and concrete core was direct without and bonding material. The outcomes referred to an increment in the strength and the correlation between the FRP thickness and gaining strength was direct.

In 1999, resting of FRP circular tubes filled by concrete had been carried out by Saafi et al. [39]. The variables included usage of two types of circular tube (carbon and glass tubes). The outcomes referred to that using of glass retrofitting tubes gave gaining in the strength by (51-137) % additional strength capacity while the carbon tube involved increment by (57-177) % additional strength in comparison with reference specimens.

In 2000, Shahawy et al. [40] presented a testing of concrete cylinders with 6 inches diameter and 12 inches height to investigate the wrapping of FRP influence. Two mixes of concrete were used (NSC and HSC) and five layers (from 1 to 5 layers) were used separately as appeared in Fig. (2-11). Concerning the theoretical side and to prove the validity of the gotten outcomes, a nonlinear finite element model was used to represent these cylinders and the obtained results were compared with the experimental ones which appeared a good matching. The results appeared that presence of adhesive bond between the two materials (concrete and wrap) had impalpable influence on the behavior of confinement behavior. The similar model of this confinement was possible to apply on the glass and carbon fibers, as long as the model had integration between the concrete dilation tendency and the stiffness function of used jackets.

Figure (2-11): Strengthened cylinders by Shahawy et al. [40].

In 2003, Li et al. [41] presented a study on the strengthened concrete columns behavior by FRP jackets. Eight RC column were constructed with dimensions 152.4 x 609.6 mm. This research involves two sides, the first one included representing these columns by ANSYS to perform the parametric study. Another side is to check the validly the obtained results of the theoretical analysis by an experimental concrete specimen with using steel rebar #4 as longitudinal reinforcement and #3 as confinement rebar. The results exhibited that effect of the FRP thickness was impalpable on the strength, stiffness, and ductility. The influence of fiber orientation on the mechanical properties was united with the interfacial bonding influence, presence of fibers in axial directions had more influence than those in hoop direction.

In 2005 Li et al. [42] tested twenty-seven 101.6 x 304.8 mm composite columns with FRP tube. The composite columns were divided into three equal groups. The first set prepared by using of normal concrete (compressive strength 35 MPa). The second one included using of concrete with grade of 50 MPa. While the third series, concrete with 80 MPa compressive strength concrete. FRP tubes with thickness of 5 mm was utilized. It is founded that the overall behavior of these analyzed columns relied on the concrete strength. The influence of tubes of FRP decreases with increasing in the grade of compressive strength.

In 2006, Li [43] used the compression loading to test 24 jacketed concrete columns by FRP jacket. The main parameters were using about 6 orientation for fiber, four mixes of concrete (NSC and HSC), in addition to two thicknesses for the FRP. From the gotten results, it was exposed that the confined cylinder behaves like the unconfined ones but the FRP jacketing provided additional strength an increment in the ductility. These types of jackets could not confine the core of concrete unless the concrete is damaged (damaging by cracking or crushing) because of increment in transverse Poisson's ratio. There is an inverse relationship between the increment in confinement efficiency and the confinement ratio increases.

In 2008, Chakrabarti et al. [44] performed a theoretically study by using of nonlinear FEM to analyze plain and RC column jacketed by FRP sheets. In order to check the validity of the used procedure and to check the accuracy in the analysis results, a verification was carried out for experimental specimens which exhibited a good matching in the obtained results. This theoretical study included using several parameters such as FRP wall thickness and fiber orientation in order to compute the amount of change and the impact on the overall behavior. According to the gotten analysis results, additional strength, increment in ductility, stress redistributions were occurred due to usage of FRP sheets as demonstrated in Fig. (2-12).

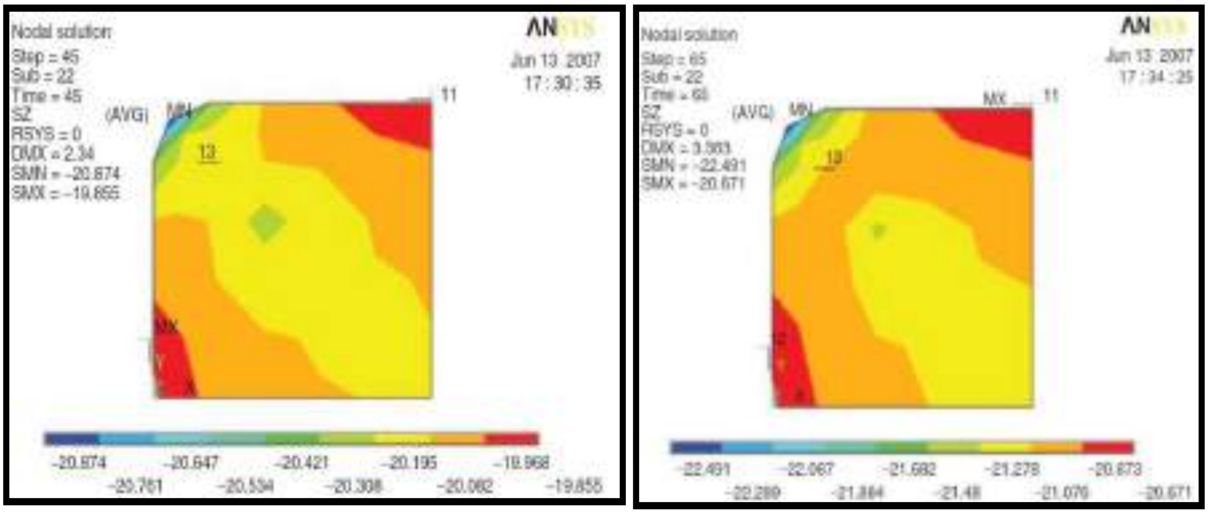


Figure (2-12): Axial stresses in concrete at various stages of the analysis carried out by Chakrabarti et al. [44].

According to the implemented research about the investigation about the strength and ductility of concrete column jacketed by CFRP sheets by Sadeghian et al. [45] in 2009, forty columns were carried out with dimensions 150 × 300 mm. Several parameters such as sheets thickness and fiber orientation were chosen. CFRP sheets thickness were selected by increasing the number of layers (1, 2, 3, and 4 layers), changing of fiber orientation from (0°, 90°, and ±45°). The outcomes confirmed that insignificant improvement in the strength capacity, overall stiffness, with gaining more ductility due to the adding of CFRP sheets.

In 2014, Tiwari et al. [46] studied the effect of GFRP jackets on the RC circular columns. Total of five specimens of dimensions (150 x 300) were utilized. GFRP sheets thickness were selected by increasing the number of layers (2, 4, 6, and 8 layers). By comparison with reference specimens, the results exposed that control specimens had failure by crushing in the concrete with an amount of cracks started from the loading plane to the column base while for the jacketed column by GFRP, the column failure happened in the GFRP composite fracture at the specimen because of the concentration of stresses in these districts. In addition to an expected increment in the strength capacity.

In 2016, Al-Nimry and Ghanem [47] investigated the influence of FRP confinement of heat-damaged circular RC columns. 15 circular RC column specimens were tested under axial compression. The effects of heating duration, stiffness and thickness of the FRP wrapping sheets were examined. Two specimen groups, six each, were subjected to elevated temperatures of 500 C for 2 and 3 h, respectively. Eight of the heat-damaged specimens were wrapped with unidirectional carbon and glass FRP sheets. Test results confirmed that elevated temperatures adversely affect the axial load resistance and stiffness of the columns while increasing their ductility and toughness. Full wrapping with FRP sheets increased the axial load capacity and toughness of the damaged columns. A single layer of the carbon sheets managed to restore the original axial resistance of the columns heated for 2 h yet, two layers were needed to restore the axial resistance of columns heated for 3 h. Glass FRP sheets were found to be less effective; using two layers of glass sheets managed to restore the axial load carrying capacity of columns heated for 2 h only. Confining the heat-damaged columns with FRP circumferential wraps failed in recovering the original axial stiffness of the columns.

In 2018, Fossetti et al. [48] offered a numerical study concerning the FRP-confined concrete columns. Nonlinear FE analysis was carried out using ATENA 3D software to analyze these types of columns. A verification was carried out to ascertain the accuracy and validity of FE procedure. The results of the validation showed a well matching between the FE and the experimental tests. To the numerical database, three series of nonlinear FE analyses were carried out, for three different values of t_f (0.117, 0.125, and 0.187mm) and varying r from 30 to 90 mm. The proposed model is capable of taking into account the mechanical and geometrical properties of the reinforcement and the corner radius ratio of the cross section. It is shown that with this model, the definition of the strength increase does not require definition of the lateral confinement pressure. Comparison with results of some available confinement models shows that the proposed model is capable of capturing experimental results coming from different experimental campaigns, while most of the models often were able to reproduce experimental results from the experimental campaign.

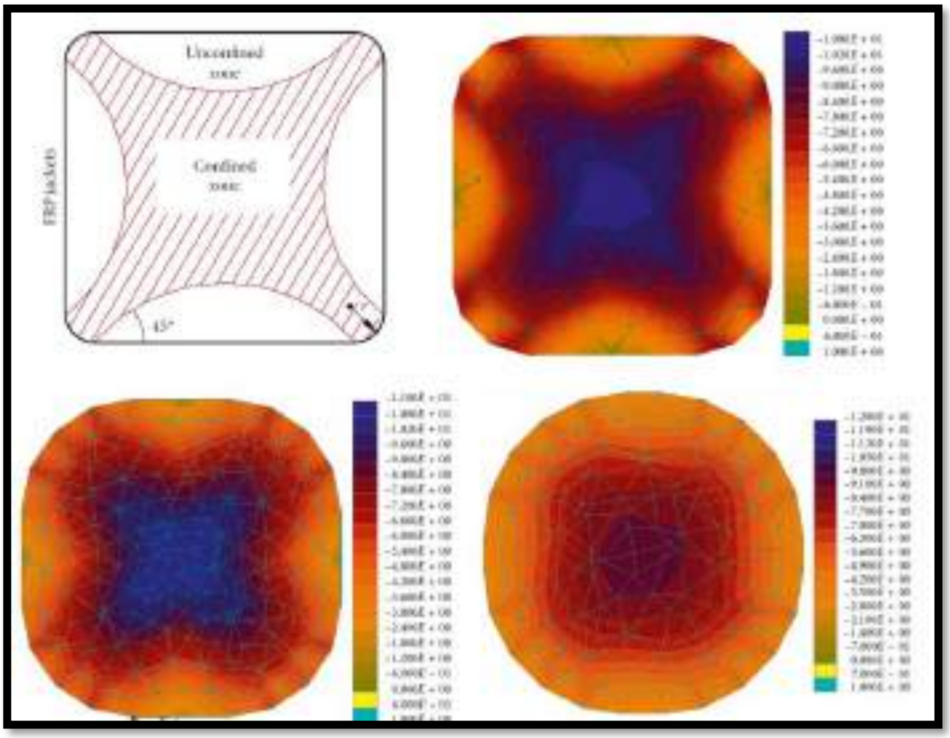


Figure [2-13]: Stress distribution in MPa at peak loads in square columns with different corner radii [48].

Numerical modeling of circular, square and rectangular concrete columns wrapped with FRP under concentric and eccentric load was presented by Oliveira and Carrazedo [49] in 2019. The numerical modeling was successfully performed with the experimental data considering axial load, axial strain and transverse strain. 36 different short columns with circular, rectangular and square cross sections were tested. Other variables such as radius of the rounded corners, concrete strength, number of FRP layers and the load eccentricity were analyzed. The outcomes showed that the distribution of compressive stresses in the cross section of the column indicates that for centered load, circular cross sections have uniform distribution and for square and rectangular sections the effective confined concrete was defined by parabolas and concentrates next to the rounded corners as revealed in Fig. (2-14). For eccentric load, the effective confined region moves to the most confined edge, thus, this does not reduce the gain for square and rectangular columns, but was unfavorable for circular columns.

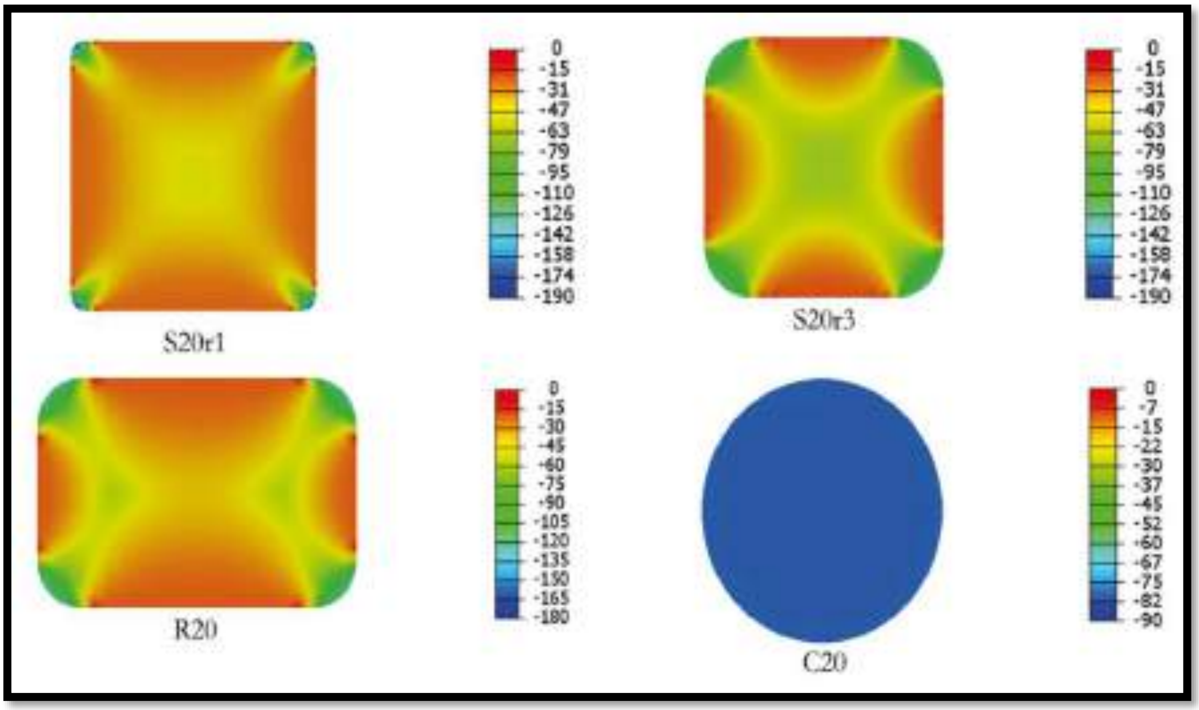


Figure (2-14): Axial compression stress distribution in columns confined with FRP obtained by numerical modeling [49].

Table (2-4): Summary of studies about strengthening by FRP jackets.

Researcher	Strengthening Method	Strength &Ductility	Stiffness
Pico [38]	Confinement with FRP tube with variable thickness	Enhanced	Enhanced
Saafi et al. [39].	Usage of two types of circular FRP tube (carbon and glass tubes).	Enhanced	Not reported
Shahawy et al. [40]	Confinement with FRP tube with variable FRP layers with existence of adhesive material	Enhanced	Not reported
Li et al. [41]	Confinement with FRP sheets with variable thickness	Enhanced	Enhanced
Li et al. [42]	Confining NSC and HSC columns with FRP with thickness 5 mm	Enhanced	Not reported
Li [43]	using about 6 orientation for fiber, four mixes of concrete (NSC and HSC), in addition to two thicknesses for the FRP	Enhanced	Enhanced
Chakrabarti et al. [44]	Jacketing plain and RC column with FRP with variable FRP wall thickness and fiber orientation	Enhanced	Not Reported
Sadeghian et al. [45]	Confinement with CFRP sheets with variable thicknesses and fiber orientation	Enhanced	Enhanced
Manish Kumar Tiwari [46]	Confinement with GFRP sheets with variable thicknesses	Enhanced	Enhanced
Al-Nimry and Ghanem [47]	confinement of heat-damaged circular RC columns under effect of heating duration, stiffness and thickness of the FRP wrapping sheets	Enhanced	Enhanced
Fossetti et al [48]	strengthening with externally wrapped FRP sheets	Enhanced	Not Reported
Oliveira and Carrazedo [49]	Confining the columns by FRP with variables radius of the rounded corners, concrete strength, number of FRP layers and the load eccentricity	Enhanced	Not Reported

2.3 General Summary and Conclusion

From the previous literature review, the following points may be noted:

- This chapter has reviewed many experimental and theoretical studies concerning the behavior of concrete columns confined with different techniques. It is apparent that a few studies concerning of applications of Steel fiber for retrofitting RC columns in comparison with the other techniques of confinement.
- Strengthening the RC column by steel fiber reinforced concrete are one of the performances improving technique that increases the ultimate capacity and improved other characteristics of RC columns.
- This thesis will investigate the behavior of the RC columns strengthened by steel fiber reinforced concrete using variables are not available in the literature review in former researches.
- The research submitted by Xie et al. [22] was chosen as a reference and the results are compared with it with taking into account some differences in the variables.

CHAPTER THREE
EXPERIMENTAL PROGRAM

3.1 General

The experimental work was carried out in the construction materials laboratory in the college of engineering at the University of Misan. The experimental and numerical program included casting and testing of 66 columns subjected to axial compression.

Details of the studied specimens, construction methodology, material properties, test set-up, instrumentation, and test procedure are presented in this chapter.

The studied parameters in this work thesis is explained in Table (3-1). The main objectives of the experimental and numerical analysis were chosen in order to strengthen the structural elements and find a solution to a realistic and real problem and considering that the columns are the most sensitive structural elements affected by loads and change the use of origin and some other problems.

Repairing Column requires practical and realistic solutions that will perpetuate and strengthen the column. Strengthening columns included restore the loss in the strength of the column.

Table (3-1): Used parameters.

Sq.	Parameters	Aim of the parameter
1	Two types and three steel fibers ratio	To explore the influence of use of straight and hooked steel with variable ratios on the confined column behavior.
2	Thickness of strengthening layer for both square and circular columns	Check the effect of confinement thickness on the maximum capacity.
3	Type of confinement (composite or hoop)	To study the effect of the jacket length on the confined column behavior
4	Types of strengthening (partial and full)	To investigate the strengthening directions and its effectiveness towards strengthening
5	Use of epoxy as bond.	To investigate the effect of epoxy as bond material between the core & jacket and its effectiveness towards stress distribution.
6	Inclined and vertical reinforcement in the steel fiber reinforced jacket	Investigate of effect of rebar orientation on the ultimate stress of the confined column.
7	Strengthening full scale column numerically	Examine the adopted strengthening techniques on the real concrete columns.

3.2 Flow Chart of the Research

The experimental program involved of two stages. The first stage contained selection of materials and evaluation of the physical and chemical properties used in this study. Subsequently, this stage also performs trail mixes to adopt the optimum mix weight proportions and to select the proper mineral and chemical additives type and dosage.

After that, the selected materials are then mixed with the adopted optimum mix proportions and using a suitable mixing procedure. Finally, the casted samples and column specimens cured for the required ages (7 and 28 days).

The second stage involved testing the column specimens . The overall experimental investigation is shown in the flowchart as given in Figure (3-1). This chapter describes the experimental program, while the obtained results from the mentioned tests will be discussed in chapter four.

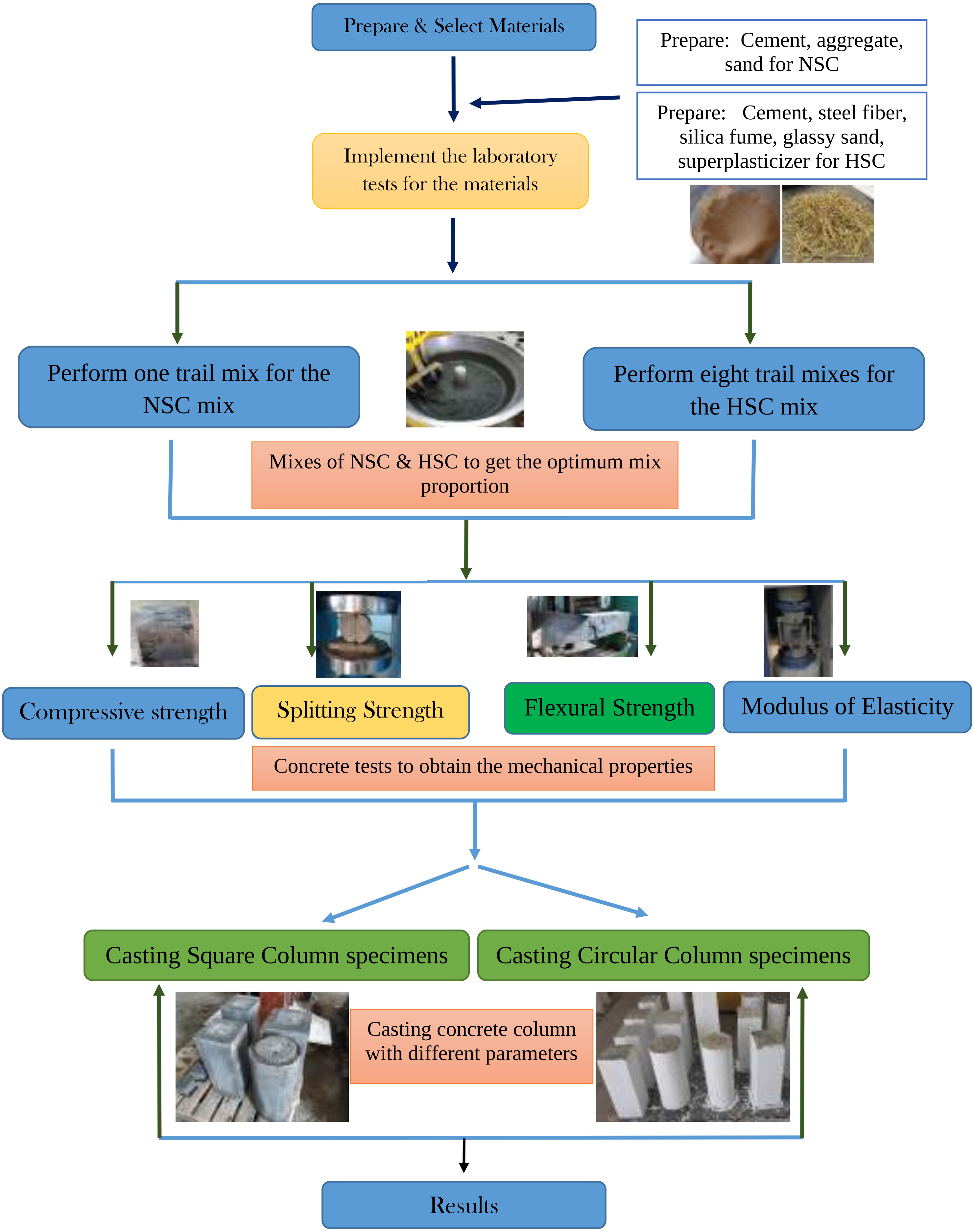


Figure (3-1): Flow chart of the research plan.

3.3 Materials Used to Fabricate the Specimens

Specimen preparation is a crucial issue and cannot be tolerated. Without due attention to specimen preparation details, any testing program is potentially destined to expected failure. The selection of samples in terms of the quantity, type, and quality of the material was done according to an extensive study and numerous experiences before the concrete column casting process was involved. The materials used in this investigation were commercially available materials, which include cement, natural gravel, natural silica sand, glassy sand, silica fume, water, steel fibers, and superplasticizer. The concrete admixtures used for increasing the workability of fresh concrete, changing the settling time, increasing the strength and durability of hardened concrete are applied either by adding to the concrete mixing water or by mixing directly with the newly prepared low-slump fresh concrete. The amount of admixture may necessitate being increased or decreased. The usage rates of chemical admixtures may vary according to aggregate properties, concrete class, water/cement ratio, and ambient temperature. In this respect, the compatibility of admixtures with other materials determined by pre-tests done in laboratories [50].

3.3.1 Cement

Iraqi cement designed as ordinary Portland cement from Cresta factory was used throughout this investigation which the material testing performed in the technical institute in Misan. The whole quantity required was brought to the laboratory and stored in a dry place. The physical properties of cement used throughout this work are presented in Table (3-2). The setting time test is conducted according to ASTM C191 [51]. The compressive strength test is accomplished according to ASTM C109 [52]. The chemical compositions of it are presented in Table (3-3). Results indicate that the cement conforms to the Iraqi standard No. 5/1984 [53].

Table (3-2): Physical properties of ordinary Portland cement.

Physical Properties	Test Results	Limit of Iraqi specification
Fineness using blaine air permeability apparatus (m2/kg)	382	≥230
Setting Time using Vicats Method		
Initial (hrs: min)	2:00	≥45 min
Final (hrs: min)	3:45	≤10 hrs
Soundness using autoclave method	0.22	<0.8
Compressive strength of morter		
3 day (Mpa)	21.2	≥15
7day (Mpa)	27.8	≥23
28day (Mpa)	34.7	-

Table (3-3): Chemical composition of cement.

Compound Composite	Chemical Composite	% by Weight	Limits of Iraqi specification No.5/1984
Lime	Cao	63.96	-
Silica	Sio2	21.32	-
Alumina	Al2o3	4.58	-
Iron oxide	Fe2o3	3.25	-
Magnesia	Mgo	2.44	≤5%
Sulfate	So3	2.32	≤2.8%
Loss on ignition	L.O.I	3.61	≤4%
Insoluble residue	I.R	1.17	≤1.5%
Lime saturation factor	L.S.F	0.75	(0.66-1.02)%
Tricalcium silicate	C3S	50.69	-
Dicalcium silicate	C2S	18.28	-
Tricalcium aluminate	C3A	8.14	-
Tetracalcium allumminoferrite	C4AF	9.89	-

3.3.2 Silica Fume

Silica fume is an ultrafine powder collected as a by-product of the silicon and ferrosilicon alloy production and consists of spherical particles with an average particle diameter of 0.15 μm. The main field of application is as pozzolanic material for high-performance concrete. Silica fume was added to portland cement concrete to improve its properties such as its compressive strength, bond strength, and abrasion resistance. These improvements stem from both the mechanical improvements resulting from the addition of very fine powder to the cement paste mix as well as from the pozzolanic reactions between the silica fume and free calcium hydroxide in the paste. Addition of silica fume also reduces the permeability of concrete to chloride ions, which protects the reinforcing steel of concrete from corrosion. Especially in chloride-rich environments such as coastal regions and those of humid continental roadways and runways (because of the use of deicing salts) and saltwater bridges [54]. The silica fume used in this study with a chloride content of less than 0.1%. It is a grey powder of density ranging between 600 kg/m^3 as shown in Fig. (3-2). Tables (3-4) and (3-5) shows the chemical and physical test results [50].

Table (3-4): Physical properties of silica fume [50].

Requirement	Analysis %	Limit of specification requirement ASTM C 1240
Sio2	88.21	>85
Moisture content	0.72	<3
L.O.I	4.32	<6
Percent Retained on 45µm (No.325) Sieve, Max	8	<10
Accelerated Pozzolanic Strength Index with Portland cement at 7 days, Min. percent of control	129.1	>105
Specific Surface, Min, cm2/g	210000	>150000

Table (3-5): Chemical properties of silica fume [50].

Oxide composition	Oxide content %
Al_2o_3	1.6
Fe_2o_3	1.11
Na_2o	0.3
K_2o	1.9
Cao	1.8
Mgo	1.9
So_3	0.25

Figure (3-2): Silica fume material.

3.3.3 Aggregate

1. Fine aggregate (Sand):

Natural silica sand brought from Al-Zubair area was used as a fine aggregate. The sieve analysis test was conducted according to ASTM C-136 [55]. Table (3-3) shows the grading of sand used for normal concrete. Results indicate that the sand conforms to Iraqi specification No. 45/1984 [56]. For HSC mixture, this mix was produced by using glassy sand from DCP strong coat HB60 as shown in Fig. (3-3), and Table (3-6) with a maximum size of (0.75-1.5) mm. The grading, physical and chemical properties were conformed to the Iraqi Standard specification No. 45/1984 [56]. The product datasheet is given in appendix B.

Figure (3-3): Glass sand sample.

Table (3-6): Sieve analysis of sand (for NSC) [56].

Sieve size (mm)	Percent Passing	
	Sand	Iraqi Specification No. 45/1984
4.75	100	90-100
2.36	93	85-100
1.18	77	75-100
0.60	48	60-79
0.30	22	12-40
0.15	3	0-10
Sulfate content	0.09%	<0.1%

2. Coarse aggregate (Gravel):

In this study, crushed natural gravel was used in concrete mix with grading satisfied to the limits of Iraqi standard No.45/1984 [56] for graded gravel with a maximum size of 12 mm. The sieve analysis test was conducted according to ASTM C136 [55] as shown below in Table (3-7).

Table (3-7): Grading of coarse aggregate

No.	Sieve Size (mm)	Passing by Weight %	Limits of IOS No. 45/1984
1	12	99	90-100
2	10	84	50-85
3	5	7	0-10

3.3.4 Water

Water is considered one of the most important elements that affect the performance of concrete in general, as it is the main factor controlling the workability of concrete and controlling, its strength, durability, and other mechanical properties of concrete. Through the process of wetting the surface of cement and aggregate granules, it is possible to easily mix and place concrete materials, in short ensure its workability. The used water in the production of concrete in this study comply with Iraqi standards and as revealed in Fig. (3-4).

Figure (3-4): Adding potable water to the concrete mixture.

3.3.5 Superplasticizer

An important component of most modern concrete mixes, water reducers improve the workability of wet concrete while decreasing the amount of water used in the mix. Among water reducers, superplasticizers, also known as high-range water reducers, represent one of the fastest-growing chemical additives in the cement and concrete additives market. The superplasticizer used was highly effective in the production of free-flowing concrete to enhance the early and ultimate strength of concrete. Hyperplast PC 260 which conforms to ASTM C494-99 [57] type was used for the present study. Different dosages of superplasticizer were used for finding the target strength of the mixes. Fig. (3-5) illustrates the working mechanism of superplasticizer. Cement particles are dispersed by repulsive force generated by negatively charged superplasticizers and the entrapped water would be released. Therefore, the flow characteristics of concrete are improved. Analyses that presented by studies indicate that addition of superplasticizer does not alter the types of hydration products, but improves the degree of crystallinity and results in highly amorphous hydrates [58].

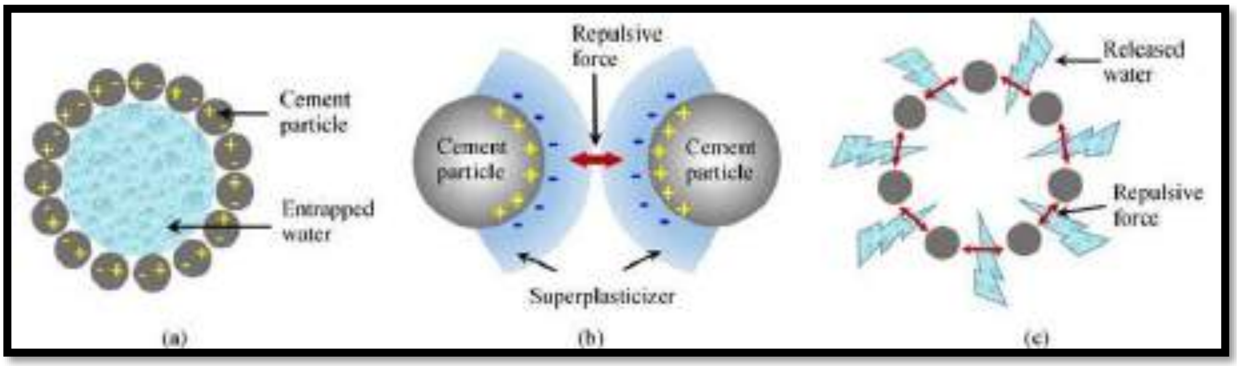


Figure (3-5): Action of superplasticizer on cement particles.
(a) Flocculated cement particles;
(b) dispersing cement particles by repulsive force generated by negatively charged superplasticizer;
(c) releasing of entrapped water [58].

3.3.6 Steel Fibers

Steel fibers are used in many kinds of concrete due to its ability to improve the mechanical properties of concrete. Benefits of steel fiber reinforced concrete are [59]:

❖ Increases tensile strength and toughness.

- ❖ Resistance to impact loads.
- ❖ Resistance to freezing and thawing.
- ❖ Shrinkage reduction.

Two types of steel fibers are used in this study (hooked and straight steel fiber). Properties of steel fiber are shown in Table (3-8) while Fig. (3-6), shows fibers that used in this study. The properties of the fibers have an effect on the mechanical properties of the concrete. The hooked fibers will have smaller occupation in the concrete than straight ones due to the fibers density.

Table (3-8): Properties of used steel fibers.

Type of steel fiber	Length (mm)	Diameter (mm)	Aspect Ratio (l/d)	Density kg/m3	Tensile strength N/mm2
hooked	30	0.55	55	7860	1345
straight	15	0.2	75	7800	2850

Figure (3-6): Hooked ends and straight steel fibers.

3.3.7 Steel Reinforcement

Two sizes of steel reinforcing bars are used in the tested columns, deformed steel bars of size Ø8 mm are used as longitudinal reinforcement, and deformed steel bars of size (Ø6) mm are used as closed stirrups. Tensile test of steel reinforcement is carried out on three specimens, prepared from each type of the steel reinforcing bars which are used in the tested columns to determine their tensile properties according to the requirements of ASTM A615[60] as revealed in Fig. (3-7). The results in Table (3-9) conform to the limitation of the Specification ASTM A615 [60]. The tensile test is performed using the testing machine (1000 kN). The bars are tested to determine the yield stress, ultimate stress, and elongation. Steel reinforcement used in this study is made in Turkey.

Table (3-9) Properties of the used steel bars.

Nominal diameter (mm)	Actual diameter (mm)	Yield stress (MPa)	Ultimate strength (MPa)	Elongation %
6	5.92	464	502	9
8	7.82	432	540	25

Figure (3-7): Steel reinforcement test.

3.3.8 Epoxy

The most interested point in the field of this new suggested composite structures is in adhesive bonded connections between concrete core and jacket. The use of epoxy allows many problems related to the traditional connecting system to be solved [61]. Using the epoxy (Sikadur-32 LP) as illustrated in Fig. (3-8) as a parameter to investigate the bond purpose between the concrete column and the strengthening jacket. Sikadur®-32 Normal is a moisture tolerant, structural, two part bonding agent, based on a combination of epoxy resins and special fillers, designed for use at temperatures between +10 °C and +30 °C. Sikadur®-32 Normal has the following advantages [62]:

- Easy to mix and apply and suitable for dry and damp concrete surfaces
- Very good adhesion to most construction materials
- High bond strength, high initial and ultimate mechanical strength
- Hardens without shrinkage
- Impermeable to liquids and water vapour with good chemical resistance

Concerning the technical properties of this structural material, such as Tensile Modulus of Elasticity, Flexural Strength ...etc. They are shown below in Table (3-10).

Table (3-10) Properties of epoxy [62].

Technical properties	Information
Modulus of Elasticity in Compression	3 250 N/mm2
Flexural E-Modulus	3 600 N/mm2
Tensile Modulus of Elasticity	4 000 N/mm2
Elongation at Break	1.0 ± 0.1 %
Shrinkage	Hardens without shrinkage.
Coefficient of Thermal Expansion	8.2 × 10−5 1/K

Figure (3-8): Using Sikadur®-32 epoxy for the bond purpose between the concrete column and the strengthening.

3.4 Mix design and mixing procedure for NSC and SFRC

A horizontal rotary mixer with (0.03m^3) capacity was used for the concrete mixing. Concrete mixing implemented by several stages which included firstly adding the cement and silica fume, then the addition of sand is implemented. Then, the water and superplasticizer added respectively to the mix.

To avoid the conglomerate in the mix, the steel fiber added gradually in small amounts, to produce the concrete with uniform material consistency and good workability. For concrete mixes with a 2.0% volume of fibers, extra time was required for mixing. Mixing process, higher speed mixing should be taken into consideration to ensure the proper homogeneity and distribution of all particles and steel fibers within the SFRC mix. The freshly mix steel fiber-reinforced concrete was placed in two equal layers into a cylinder mold to cast a standard 100x200 mm cylindrical concrete specimen for a splitting tensile test,100x100x100mm cube mold for a compressive strength test and into a 100x100x500 mm prism mold for a flexure strength test. All design mixes are presented in Tables (3-11) and (3-12).

After one day, the specimen was removed from the mold and cured in water for 28 days. And then a strength test was performed. One mix design has been carried out for normal concrete as illustrated in Table (3-12) and eight mixes design has been carried out for SFRC which included different using a different ratio of steel fibers and addition. After that we were select the best one mix from these eight mixes which was the (mix 8) as illustrated in Table (3-11).

Table (3-11): Proportions of constituent materials in SFRC mixes.

Material /(kg/m3)	Mix 1	Mix 2	Mix 3	Mix 4	Mix 5	Mix 6	Mix 7	Mix 8
Cement	900	900	800	800	900	800	900	800
Sand	1000	1000	1000	1000	1000	1000	1000	1000
Silica fume	100	100	200	200	100	200	100	200
w/c	0.25	0.22	0.22	0.2	0.2	0.22	0.22	0.22
Super PS	2%	2%	3%	3%	3%	2%	3%	2%
Steel fiber%	1%	1.5%	1%	1.5%	1.5%	1%	2%	2%
fcu_1 (MPa)	49.2	50.16	44.12	54.3	76.4	71.01	75.42	81.44
fcu_2 (MPa)	61.56	71.56	67	78	80	80.72	78.3	91
ft (MPa)	5.5	5.6	4.74	6.16	6.33	4.38	8.36	8.13
fr (MPa)	9.7	12.52	12.1	13.43	13.14	11.74	13.95	12.65

* fcu_1 = compressive strength at seven days.
** fcu_2 = compressive strength at twenty-eight days.

Table (3-12): Proportions of constituent materials in NC mix.

cement	Sand	Gravel	W/C	Slump (mm)	*ft* (MPa)	fcu_1 (MPa)	fcu_2 (MPa)
1	1.8	2.4	0.48	70	3.1	37	44.46

* fcu_1 = compressive strength after seven days.
** fcu_2 = compressive strength after twenty-eight days.

The following steps were carried out in the mixing procedure:

1) Superplasticizer PC-260 were added to the potable water.
2) Mixing the cement with the silica fume for 3 minutes.
3) Glassy sand was added to the dry mix and mixed for 3 minutes.
4) 80 to 85% of water was added with SP to the mix to be mixed for 5-6 minutes.
5) The remaining water with SP would be added to the mixture for 3-5 minutes until be like a paste.
6) Continuing the mixing operation for 5-8 minutes.
7) Steel fibers were sprayed to the mix progressively during mixing operation for 10 minutes.
8) Continuing the mixing operation to get a homogeneous and thick paste mixture.
9) The rated total time to finish the mixing operation took 25-30 minutes.

3.5 Experimental Tests

The tests were carried out for both fresh and hardened concrete. The slumps were measured for normal strength concrete mix. The compressive strength, flexural strength and splitting tensile strength tests were carried out for hardened SFRC at 28 days age. Six (100x100x100) mm cubes and three (100x100x500) mm prisms and three (100x200) mm cylinders were used. For NC, six (150x150x150) mm cubes and three (100x200) mm cylinders were tested only at

28 days age. The workability for normal strength concrete was measured immediately after mixing in accordance with test method ASTM C-143 [63]. The slump test for this concrete was 70 mm.

3.5.2 Tests of Hardened Concrete

3.5.2.1 Compressive Strength Test

Six cubes of (150x150x150) mm for NC and six (100x100x100) mm for SFRC were cast to measure the compressive strength as shown in Fig. (3-9). After casting and de-modeling, the specimens were cured by submerging in a water basin for the test at 28 days age for NC and SFRC. The specimens were tested under compression using (ELE) universal testing machine with a capacity of (2000 kN) as shown in Fig. (3-9). Table (3-13) and (3-14) gives the results of the compressive strength for each mix. The micro-cracks increased due to the presence of steel fibers. This behavior was noticed because of the steel fiber preventing the propagation of the micro-cracks and excessive deterioration in the bond between adjacent segments.

Figure (3-9): Compressive strength test with failure pattern of the specimen subjected to axial compressive stresses.

Table (3-13): Compressive strength test of normal concrete mixes at 28 days.

Group No.	NO. of specimens	*f'c* - Cube (MPa)	*f'c* - Cylinder (MPa)
S1	4	44.3	35.44
S2	4	45.5	36.4
S3	4	43.1	34.48
S4	4	47.3	37.84
S5	4	46.3	37.04
S6	4	45.1	36.08
S7	4	46	36.8
S8	4	47.3	37.84
S9	4	46.7	37.36
S10	3	48	38.4
C11	3	43.7	34.96
C12	3	42.3	33.84
C13	3	45.4	36.32
C14	4	48.7	38.96
C15	4	44.5	35.6

S: specimens for square specimens.

C: specimens for circular specimens.

***f'c:* Compressive strength.**

Table (3-14): Compressive Strength Test of high strength concrete mixes.

Group No.	NO. of specimens	SF	SF Ratio %	*f'c* - Cube (MPa)	*f'c* - Cylinder (MPa)
G1	2	hooked	1%	77.5	62
G2	2	hooked	1%	74.8	59.84
G3	2	hooked	1.5%	86.4	69.12
G4	2	hooked	1.5%	80.6	64.48
G5	4	hooked	2%	92.5	74
G6	4	hooked	2%	90.7	72.56
G7	4	hooked	2%	93.4	74.72
G8	4	hooked	2%	88.6	70.88
G9	4	hooked	2%	97.2	77.76
G10	1	straight	1%	91.3	73.04
G11	1	straight	1.5%	102.2	81.76
G12	4	straight	2%	120.3	96.24
G13	3	straight	2%	123.7	98.96
G14	3	straight	2%	116.8	93.44

3.5.2.2 Tensile Strength

Splitting tensile test was carried out on three standards (100*200) mm cylinders of SFRC and three standard cylinders of NC. The test procedure was according to ASTM C496 [57]. The indirect tensile strength of SFRC and NC were measured by subjecting a line load within the cylinder specimen. Fig. (3-10) shows the oriented tensile stresses induced in the specimens which were led to the splitting of the cylinder specimen. An (ELE) universal testing machine with a capacity of (2000 KN) was used. Fig. (3-10) shows the pattern of failure was depicted in and the test set up for the splitting tensile strength.

$$f_t = \frac{2P}{\pi D l} \qquad \text{...... (3.1)}$$

Where:

f_t = tensile strength (MPa), P_u = ultimate failure load (N),

D = diameter of cylinder specimen (mm),

L = length of cylinder specimen (mm).

For high and normal strength concrete, three-cylinder specimens were tested at 28 days.

Figure (3-10): Split cylinder test with Failure pattern of cylinder specimen subjected to splitting stresses.

3.5.2.3 Flexural Strength

The flexural strength tests were carried out by using three (100 x 100 x 500 mm) simple support prisms loaded by third points. Flexural strengths were determined by using three prisms at 28 days age and three prisms were tested of NC at 28 days. Flexural strength tests were carried out on specimens in accordance with ASTM C78 [64]. Fig (3-11) shows the point loads, the same machine was used for the test by using two solid bars with a steel plate at the top.

Figure (3-11): Flexural prism test for SFRC.

The test results of the flexural strength. Modulus of rupture can be determined using the following equation.

$$f_r = \frac{3PL}{2bd^2} \qquad 3.3$$

Where: f_r = flexural strength (MPa), P = ultimate failure load (N), L = Span length (mm), b = width of prism section (mm), and d = depth of prism section (mm).

3.6 Mould Preparation

Four circular steel moulds were used in this work. The dimensions of these moulds are (170x500mm) was used for core concrete, (220x500mm), (240 x 500mm), and (260 x 500mm) were used for strengthened columns. They were cleaned with a scraper and a steel hairbrush and were lubricated to ensure an easy demoulding. Forms for the square column specimens were prepared using 18 mm plywood sheets cut and assembled very carefully to ensure, accurate vertical sides and to provide 90 degrees corners with plywood formed the bottom, as shown Figure (3-12). The dimension of square column is (150x150x500) mm and (150x150x750) mm which used for core concrete, (200x200x500) mm, (220x220x500) mm, (200x200x750) and (240x240x500) mm which used for strengthening of columns.

Figure (3-12): Moulds of the specimens.

3.7 Columns details

3.7.1 Reference Columns Details (Series A)

This work included an extensive study involved casting and testing 66 concrete columns. A total of 56 short concrete columns (39 square columns while the circulars were 17 specimens) were designed and fabricated experimentally.

Regarding the experimental work, these columns carried out with and without strengthening, 44 of them are strengthened with concrete jackets reinforced with steel fiber while the remaining are without strengthening jacket.

Reference columns are fabricated with dimensions of (150x150x500) mm and (150x150 x 750) mm for the square columns. While (170x500) mm for the circular ones. Using steel reinforcement by 6 Ǿ8 mm for the main reinforcement and Ǿ6 mm @ 100 mm as transverse reinforcement for both circular and square columns as revealed in Figs. (3-13) and (3-14). Used concrete is RC with compressive strength (44) MPa. These columns are non-strengthen with NSC represent the reference columns. Reference square and circular columns illustrated in Tables (3-15) and (3-16) with variable cross section dimensions which designed and fabricated for comparison with strengthened columns with the same dimensions.

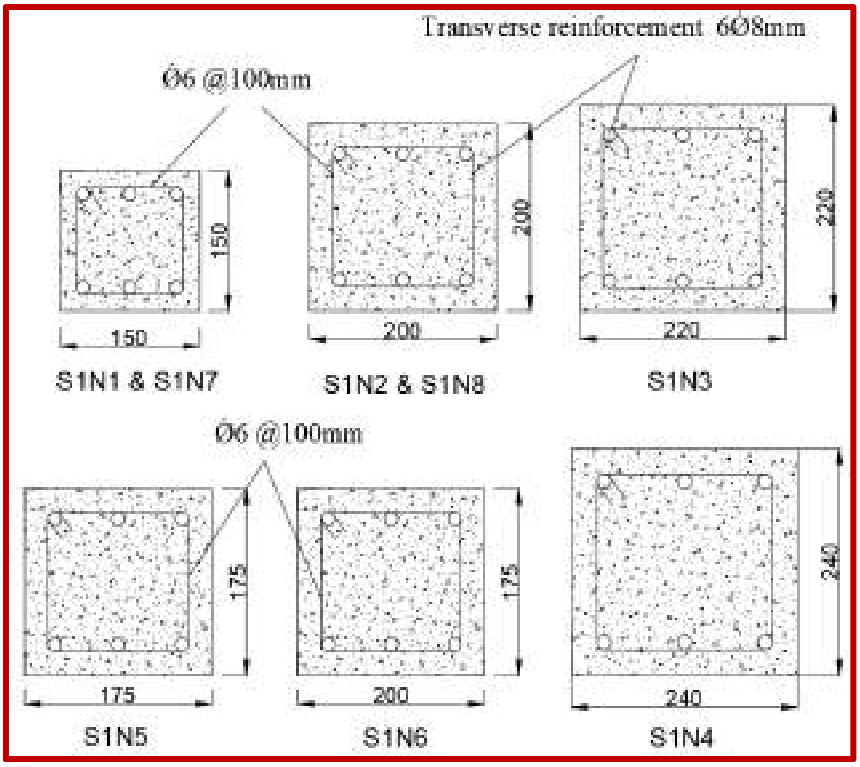


Figure (3-13): Geometric details of series A columns (all dimension in mm).

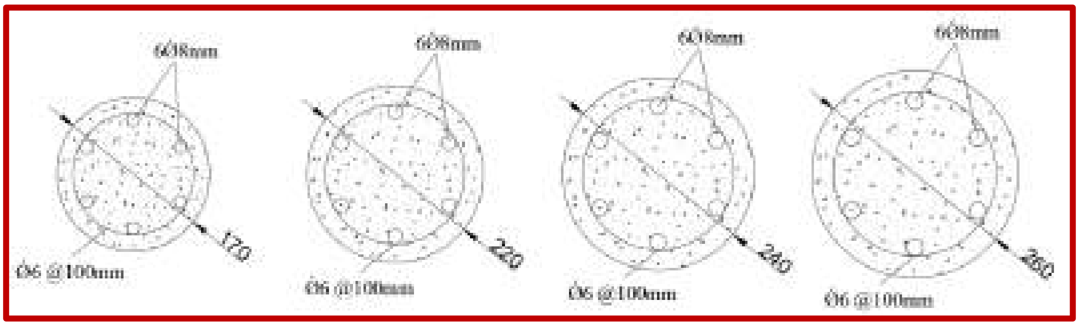


Figure (3-14): Reference circular beams (all dimension in mm).

Table (3-15): Details of control square columns (group 1).

Column ID	Dimensions (mm*mm)	Height (mm)
S1N1	150*150	500
S1N2	200*200	500
S1N3	220*220	500
S1N4	240*240	500
S1N5	200*175	500
S1N6	175*175	500
S1N7	150*150	750
S1N8	200*200	750

Table (3-16): Details of the control circular columns of group 2

Column ID	Dimensions (mm)	Parameter
C2N1	170*500	Reference Column
C2N2	220*500	Reference Column
C2N3	240*500	Reference Column
C2N4	260*500	Reference Column

3.7.2 Parametric study

A parametric study is implemented on the concrete columns to investigate the effect of these variables on the ultimate strength, load, displacement response, longitudinal and lateral strain, ductility index, energy absorption crack pattern, and failure mode. The parametric study divided into several series starting form series B to series E where each series have many parameters as follows;

3.7.2.1 Series B

Regarding this series, series B is the first paramtric series which included 14 column specimens strengthened with SFRC jacket. This series consists of three groups (S3, S4, and S5) which have many parameters. Third Group (S3) included square columns with two parameters (steel fibers ratio and use of epoxy), while the fourth and fifth group (S4 & S5) included the thickness parameters beside use of types of strengthening (hoop and composite) in addition to use of epoxy as demonstrated in Fig. (3-15) and Table (3-17). Term of composite and hoop refers to the jacket height which composite word mean the height of the jacket is the same of concrete core while hoop is the jacket height is less than the concrete core by 1.5 cm.

Table (3-17): Details of square columns of series B.

Series B						
ID	**Dimension (mm)**	**Jacket thickness (mm)**	**SF ratio %**	**Epoxy**	**SF type**	**Jacketing**
S3n0h	200*200*500	25	0%	without	---	composite
S3n1h	200*200*500	25	1%	without	hooked	composite
S3n1.5h	200*200*500	25	1.5%	without	hooked	composite
S3n2h	200*200*500	25	2%	without	hooked	composite
S3E0h	200*200*500	25	0%	with	---	composite
S3E1h	200*200*500	25	1%	with	hooked	composite
S3E1.5h	200*200*500	25	1.5%	with	hooked	composite
S3E2h	200*200*500	25	2%	with	hooked	composite
S4n2c	220*220*500	35	2	without	hooked	composite
S4E2c	220*220*500	35	2	with	hooked	composite
S4n2H	220*220*500	35	2	without	hooked	Hoop
S5n2h	240*240*500	45	2	without	hooked	composite
S5E2h	240*240*500	45	2	with	hooked	composite
S5n2s	240*240*500	45	2	without	straight	composite

S3E0h; S: square column, 3: third group, E: epoxy, 0: SF ratio, h: hooked fibers

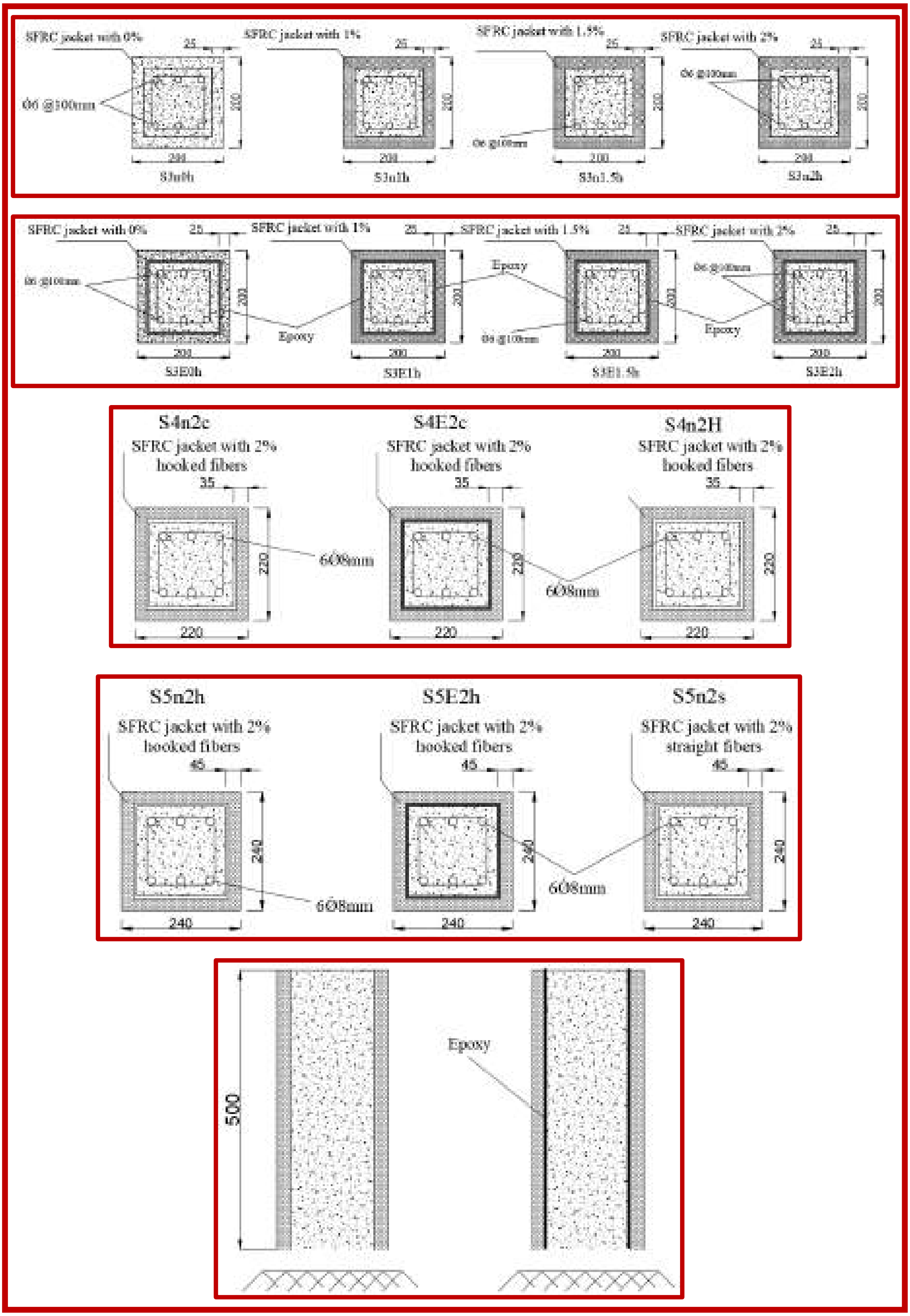


Figure (3-15): Geometrical details of the series B columns (all dimension in mm).

3.7.2.2 Series C

New variables were used in this series which included use of straight fibers with incremental ratios, types of confinement (hoop and composite), and strengthening types (two or three sides strengthening) beside the use of bond material (epoxy). A total of 12 column specimens divided into two groups (group 6 have 8 columns and group 7 have 4 columns as illustrated in Fig. (3-16) and Table (3-18). This group consist of columns with a constant jacket thickness equal to 25 mm.

Table (3-18): Details of square columns of series C.

Series C						
ID	**Dimension (mm)**	**Jacket thickness (mm)**	**SF ratio %**	**Epoxy**	**SF type**	**Jacketing**
S6E1s	200*200*500	25	1	with	straight	composite
S6E1.5s	200*200*500	25	1.5	with	straight	composite
S6E2s	200*200*500	25	2	with	straight	composite
S6n2s	200*200*500	25	2	without	straight	composite
S6noH	200*200*500	25	0	without	---	Hoop
S6n2hH	200*200*500	25	2	without	hooked	Hoop
S6n2sH	200*200*500	25	2	without	straight	Hoop
S6E2m	200*200*500	25	2	with	Both	composite
S7E2hp3	175*200*500	25	2	with	hooked	Three side composites
S7E2sp3	175*200*500	25	2	with	straight	Three side composites
S7E2hp2	175*175*500	25	2	with	hooked	Two side composites
S7E2sp2	175*175*500	25	2	with	straight	Two side composites

S6E1s; S: square column, 6: sixth group, E: epoxy, 1: SF ratio, s: straight fibers.

S6n2sH; S: square column, 6: sixth group, n: no epoxy, 2: SF ratio, s: straight fibers, H: hoop strengthening case.

S7E2hp3; S: square column, 7: seventh group, E: epoxy, 2: SF ratio, h: hooked fibers, 3: three sides strengthening

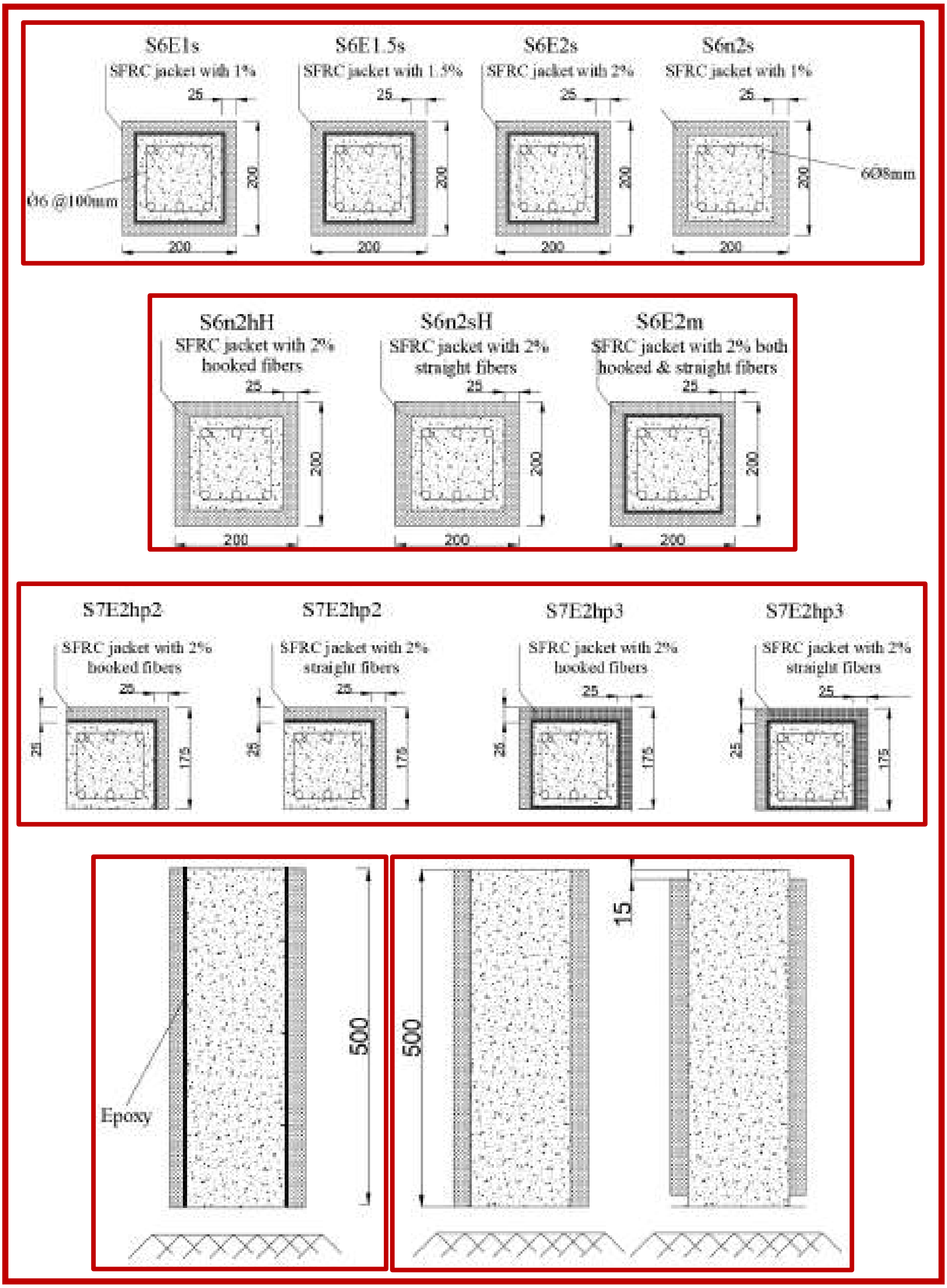


Figure (3-16): Geometrical details of the series C columns (all dimension in mm).

3.7.2.3 Series D

Concerning the circular columns, the same parameters that used in square columns are used in circular ones. The parameters divided in this series into three groups (S8, S9, and S10), which each group have a parameter. Group eight and ten have only four columns with jacket thickness and epoxy parameters otherwise the group nine which have nine columns with several parameters such as (steel fibers types, steel fibers ratio, bond effect, and types of confinement).

Using variable strengthening thicknesses (25, 35, and 45 mm), two types of steel fibers with different ratios (1, 1.5, and 2%), confinement types (composite and hoop), in addition to the use of epoxy material as a bond material between the concrete core and strengthening jacket as revealed in Tables (3-19) and Figs (3-17).

Table (3-19): Details of circular columns of series D.

Series D						
ID	**Dimension (mm)**	**Jacket thickness (mm)**	**SF ratio %**	**Epoxy**	**SF type**	**Jacketing**
C8n2h	260*500	45	2	without	hooked	composite
C8n2s	260*500	45	2	without	straight	composite
C9n1h	220*500	25	1	without	hooked	composite
C9n1.5h	220*500	25	1.5	without	hooked	composite
C9n2h	220*500	25	2	without	hooked	composite
C9E1h	220*500	25	1	with	hooked	composite
C9E1.5h	220*500	25	1.5	with	hooked	composite
C9E2h	220*500	25	2	with	hooked	composite
C9E2s	220*500	25	2	with	straight	composite
C9n2hH	220*500	25	2	without	hooked	Hoop
C9n2sH	220*500	25	2	without	striaght	Hoop
C10n2h	240*500	35	2	without	hooked	composite
C10E2h	240*500	35	2	with	hooked	composite

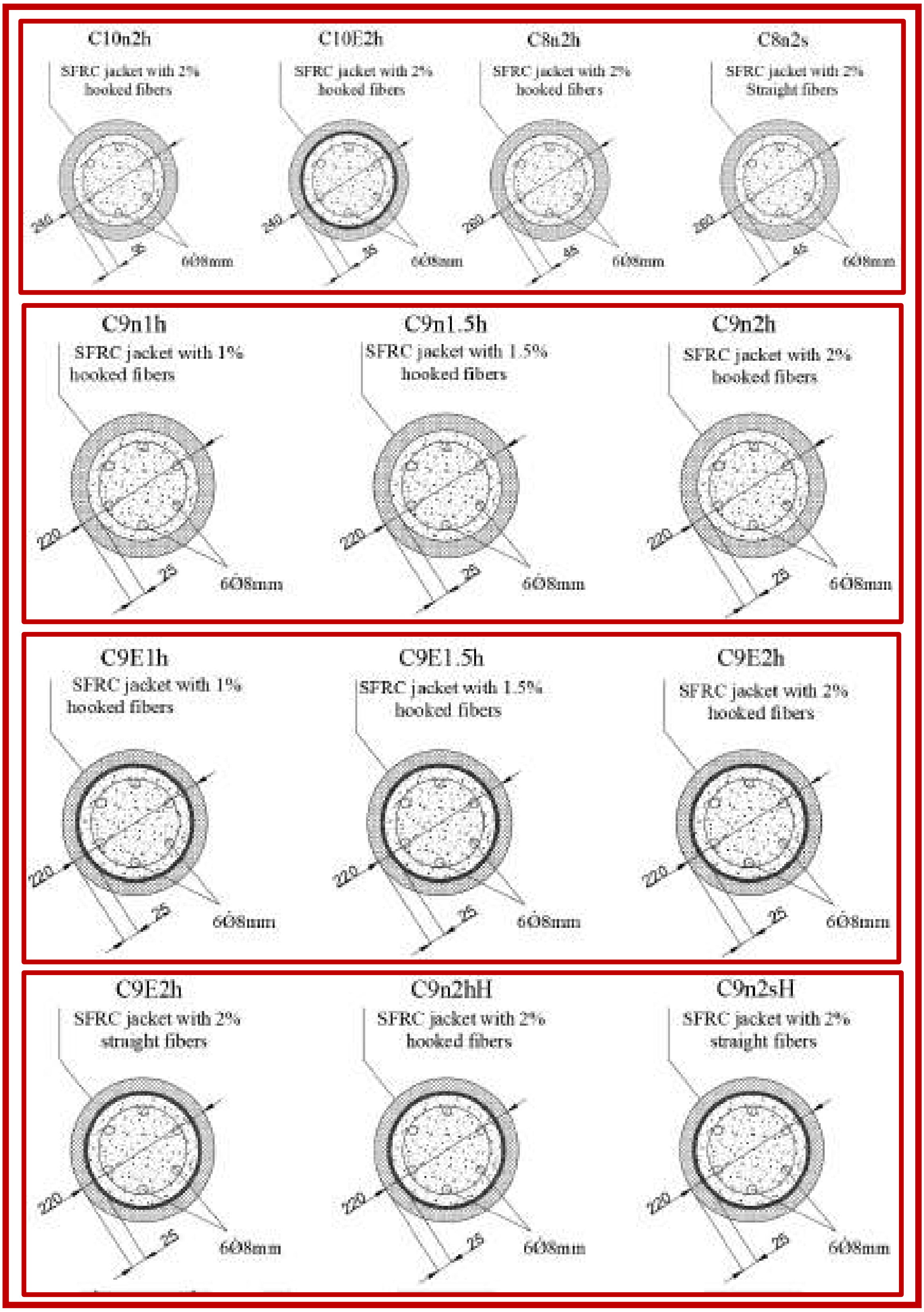


Figure (3-17): Geometrical details of the series D columns (all dimension in mm).

3.7.2.4 Series E

Last experimental series included five columns with length of 750 mm. used parameters were use of hooked and straight fibers in hoop and composite strengthening with use of epoxy as bond material. Jacket thickness was constant in this series (25 mm) as appeared in Fig. (5-18) and Table (3-20).

Table (3-20): Details of square columns of series E.

Series E						
ID	**Dimension (mm)**	**Jacket thickness (mm)**	**SF ratio %**	**Epoxy**	**SF type**	**Jacketing**
S11n2hc	200*200*750	25	2	without	hooked	composite
S11E2hc	200*200*750	25	2	with	hooked	composite
S11n2sc	200*200*750	25	2	without	straight	composite
S11E2sc	200*200*750	25	2	with	straight	composite
S11n2hH	200*200*750	25	2	without	hooked	Hoop

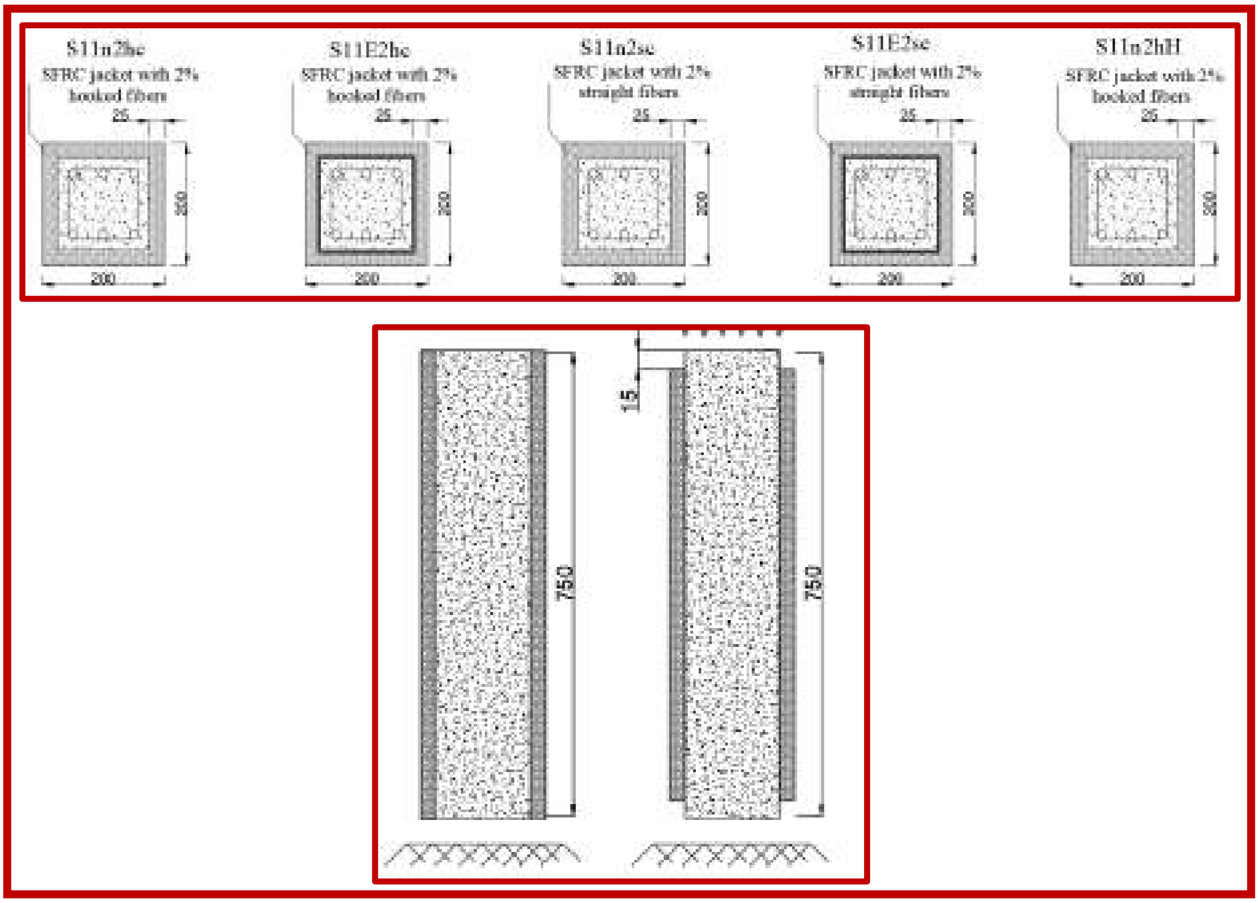


Figure (3-18): Geometrical details of the series E columns (all dimension in mm).

3.8 Casting Procedure

An 80 kg mixer was used to mix the concrete for columns. The formwork were treated with oil before putting the reinforcement cage or casting control specimens. The steel bars were placed inside the forms and held in place by 1 cm square stock to ensure that the proper cover was maintained. Before mixing, all the quantities were weighted and backed in a clean metal container. Casting the concrete performed by plywood forms for the square forms and steel forms for the circular specimens as revealed in Fig. (3-19). After performing the mixing and casting, the formworks removed after 24 hours and treatment the specimens by water for 28 days. Then the strengthening can be used after this period.

Figure (3-19): fabrication of the experimental columns.

3.9 Instrumentation

3.9.1 Strain Gauge of concrete

The strain in concrete columns were measured by using the foil strain gauge of (120 Ώ) resistance from TML Japan. The strain gauges were installed by the advised adhesive (CN-E) preceded by preparing the contact surface, then applying the coating material (W-1) and waterproofing material for embedded strain gauges as recommended by manufacturer company.

At the mid-length of the columns**,** a strain gauge was fielded in order to measure the lateral strain. The type of strain gauge that used to this purpose is PFL-30-11-3L wire type with (120Ώ) resistance as shown in Fig.(3-20).

Figure (3-20): Strain gauge of concrete.

The strain gauge was bonded using CN-E cyanoacrylate adhesive to the previously treated surface of the column by cleaning.

LVDT (Linear variable differential transformer) was adopted instrument to measure the deflection in the tested column inside the test device as shown in Fig. (3-21).

Figure (3-21): test device with specimen.

3.9.2 Strain and deflection Indicator

The test program of this research includes measuring the strain and deflection of specimens by using LVDT that connected to data logger by using external supply convert voltage. The euros data logger DT8 smart model was model number DTBSG was used an exterior supply convert voltage and data logger as shown in Fig. (3-22). The operating software of this device provides a facility to control the delay time (No. of reading per sec) to be varying from record/sec.

Figure (3-22): Photograph for data logger used in this study.

3.9.3 Installation of Strain Gauge

Surface preparation is done by cleaning an area uniformly and finely with abrasive paper. Then, cleaning the abraded area with a cloth soaked in a small quantity of acetone. Bonding: The adhesive series CN-E, is a single component room temperature curing adhesive for strain gauges. After marking the position of strain gauge installation with a pencil, one drop of CN-E adhesive was applied to the back of the strain gauge base, and the adhesive was spreading uniformly over the entire back of the gauge. Then, the gauge was aligned to the guide mark, the polyethylene sheet was placed over the strain gauge, and constant pressure was applied with the thumb.

3.10 Test Procedure

All columns specimens were tested by the ALFA Testing Machine with maximum compressive capacity 5000 kN as shown in Fig. (3-23). This equipment has many features [65] such as:

1- Automatically determines the load rate.
2- Auto-stop function which permit to the test to stop automatically.
3- Fully automatic mode or manual mode in which the user gets the ability to adjust the load rate and period manually are available.
4- Real time graph indication and Max. load and stress.
5- Test can also be performed through computer via ALFA’s state-of-the-art software.
6- with the ability to save/recall and report the test results.

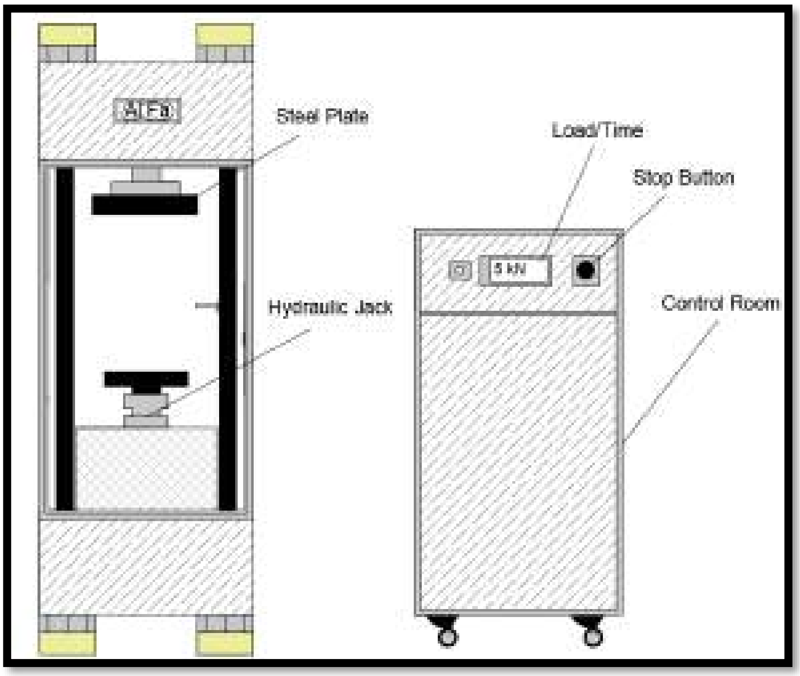


Figure (3-23): Test device.

Concerning the test procedure, the test of the specimens was carried out at the age of 28 days after strengthening casting. All specimens were cleaned, and painted with white paint before testing, in order to clarify the propagation of cracks. The single concentrated load was applied through a steel load plate moving plate used to achieve uniform contact. The test stage involved palcing the specimen on the testing machine and adjusted so that the centerline, supports, line loads and LVDT were fixed in their correct locations, then all the instruments that needed to complete the testing concocted as shown in Fig. (3-24). The load was applied with a small increment about (8 kN) with average of the load and set up the data logger to take the measurement each one second. The readings of deflection, slip, uplift, strains, and load was reported at every increment. The load was increased gradually up to failure.

Figure (3-24): Setup of typical tested column.

CHAPTER FOUR
RESULTS & DISCUSSION

4.1 General

This chapter describes the results of the experimental work conducted on 56 concrete columns strengthened with SFRC jackets and tested under axial monotonic loads. Concrete columns behavior and failure modes of strengthened columns are discussed. The behavior of the strengthened columns is compared with that of the control specimens and another recent studies. A study is carried out to explore the affect the various parameters which are expected to affect the behavior of such columns.

The three-dimensional finite element model described in the previous chapter was used to analyze the tested columns in order to examine the ability of the model to predict the failure of the columns and to obtain more information about the stresses, strains, crack pattern, failure mode, and stress distribution developed in the columns. With the purpose of checking the validation and the accuracy of the finite element models, a verification process was performed. In this section, the SFRC jacketed columns are investigated via experimental and theoretical study by FEM. Stress-strain curves, first crack load, ultimate load, axial and lateral displacement, cracks pattern and failure mode, and stress distribution are considered to explore the performance of these columns.

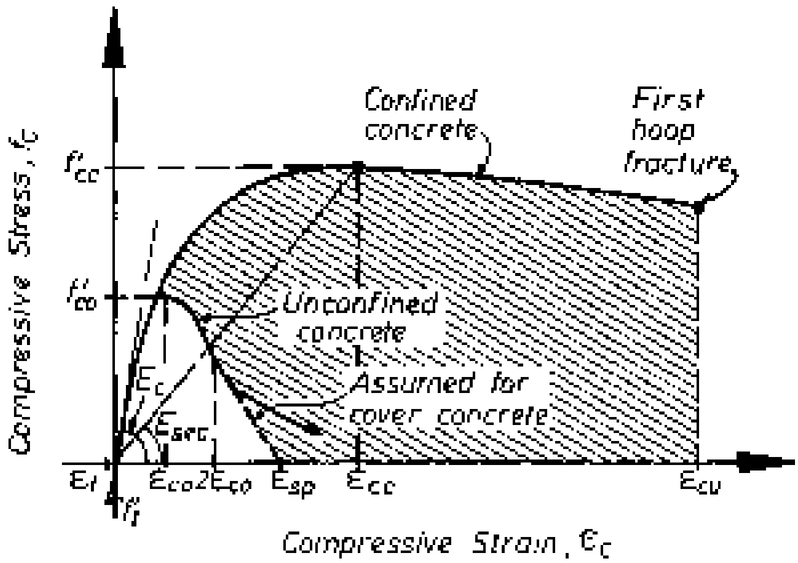
4.2 Monotonically Increasing Axial Load Tests

Monotonically increasing axial load tests were conducted to obtain the ultimate strength of specimens and to determine the distribution of stresses and strains at the failure point. Axial load tests were conducted on all series of specimens. The maximum capacity and the strain at the maximum stress were determined for each specimen tested under monotonic load.

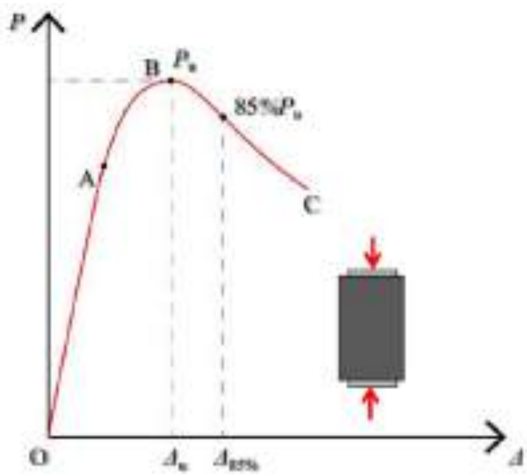
4.3 Concrete Behavior under Compressive Force

In order to evaluate the beneficial effect of confinement on the carrying capacity of concrete column strengthened with jacket, Mander et al. (1984) [4] have proposed a unified stress-strain approach for confined concrete. This proposed approach is applicable to both circular and rectangular shaped transverse reinforcement. The obtained stress-strain curve is illustrated in Fig. (5-1 a & b) and compared with stress-strain curve of the equation that suggested by Popovics (1973) [66] which showed a good agreement and similarity in the general behavior. The figure clearly indicates that the capacity of the strengthened column significantly exceeds the stress capacity of the unconfined concrete columns by many times. After that, the concrete jacket was activated and the axial load-axial strain curve of confined concrete showed nonlinear behavior until the rupture. From that stage the concrete is under triaxial compression due to restraint by the concrete jacket, which is under vertical compression and transverse tension. Due to the confinement of the concrete, the triaxial stress state causes a reduction of the axial compressive strength of the concrete while there is an increase in the longitudinal compressive strength of the concrete core. The interaction between the concrete jacket and the concrete core works beneficially and leads to an ultimate load exceeding the uniaxial compression loads of concrete core. Fig. (5-1 b) shows the general compressive P-Δ curves of stub concrete column confined by SFRC jackets that presented by study of Xie et al. [22] which revealed a similar mechanism of the behavior of this study. The load versus axial shortening curves showed that there are three working stages, i.e., elastic stage, nonlinear stage, and recession stage. At the first elastic stage, the compressive resistance of specimen increases almost linearly with the applied displacement loading as shown in Fig.

(5-1 a). In the second stage, the specimens start to perform nonlinearly due to the cracking of the jackets and nonlinear compressive behavior of the center concrete column. At the end of nonlinear stage, all the specimens achieve their ultimate resistance. During the recession stage, the compressive strength of the specimens decreases rapidly as the applied loading increases. It can be also found that the concrete column with larger thickness of confining jackets exhibits more ductile behavior during the recession stage.



(a)



(b)

Figure (4-1): (a) Mander's stress-strain relationship of confined columns [6]
(b) General behavior of confined column by Xie et al. [22].

4.4 Test Results

4.4.1 Series A

4.4.1.1 Group One

This series that includes group one besides the group two represent the key for this study which considered the reference for the other series. Group one consists of eight column specimens with different cross section dimensions for the comparison with the strengthened column that have the same dimensions.

A. Stress-Strain Relationship

First group involved testing of eight columns with variable dimensions, these columns tested to the failure point under static loads. The results exhibited that the obtained strengths results ranged between (895 – 1357) kN with vertical displacement ranged between (3.64-5.31) mm as shown in Fig. (5-2) and Table (5-1). The variance in the ultimate strength capacity was due to the changing in the cross-section dimension. Expressing the maximum strength of the columns analyzed not only by using the failure load values but also by using several calculations. These calculations represent the true behavior of the columns in its various aspects, including the energy absorption, the ductility index, and the longitudinal and transverse strains that were calculated.

The stress-strain relationship in Fig. (5-2) shows that the jacketed columns behaves linearly until it reaches about average value 67% of their ultimate strength. In this thesis, energy absorption values which equals to the area under the curve of load and its corresponding displacement. Table (5-1) lists the calculated energy absorption (Tn) for each specimen and the ductility of jacketed concrete columns.

The ductility may be evaluated by the ductility index (*DI*). In this thesis, the ductility calculated by using the load-displacement curves [22]. The definition of *DI* is the ratio of the displacement at 85% of the ultimate load (Δu) during the recession stage, to the displacement at the rupture load (Δm), as shown in Eq. 4.1. Where Δu is the ultimate displacement when the post-peak remaining capacity of the column has dropped 85% of the peak load [22].

$$DI = \frac{\Delta u}{\Delta m} \qquad \ldots(4.1)$$

Table (4-1): Test results of reference concrete columns of group one.

ID	P_{cr} (kN)	P_{cc} (kN)	Δ_V (mm)	Δ_L (mm)	ε_{cc}	$P_{cc}/P_{Ref.1}$	$\varepsilon_{cc}/\varepsilon_{Ref.1}$	DI	Tn (kN)
S1N1	902	992.3	3.69	0.721	0.00738	1	1	1.03	1932
S1N2	828	1142.6	3.88	0.002	0.00776	1.15	1.05	1.20	1998.7
S1N3	995	1195.8	5,62	3.282	0.01124	1.21	1.52	1.16	3045
S1N4	815	1356.8	4.27	1.823	0.00854	1.36	1.15	1.32	2510
S1N5	707	1177.5	4.45	1.057	0.0089	1.19	1.2	1.10	1992
S1N6	582	895	3.64	0.657	0.00728	0.91	0.98	.112	1995
S1N7	252	1009.7	3.79	0.002	0.00505	1	1	1.11	1435
S1N8	950	1153	5.31	0.224	0.00708	1.14	1.44	1.09	1841

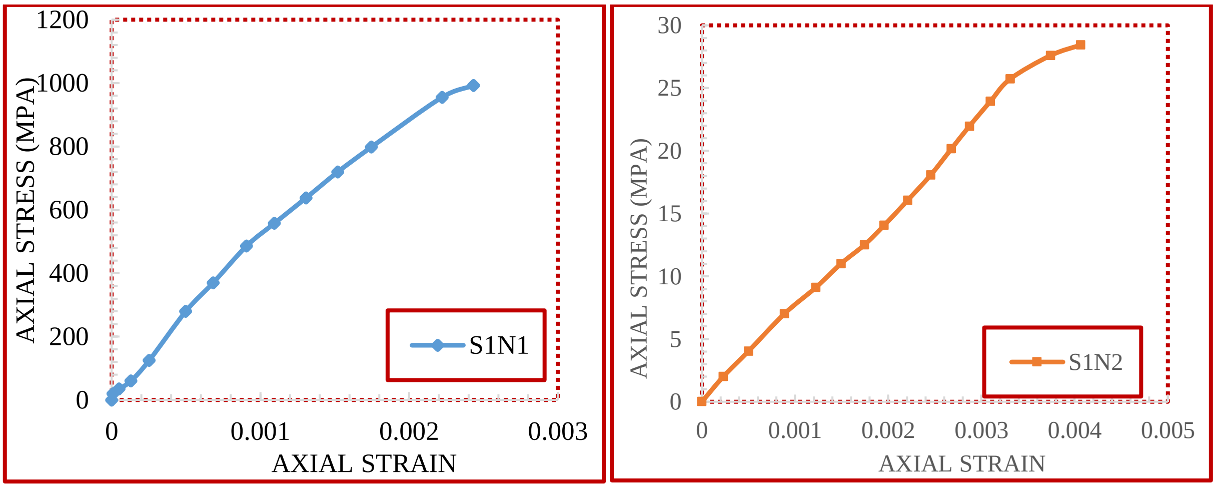


Figure (4-2): Stress-strain relationship of reference columns of group one.

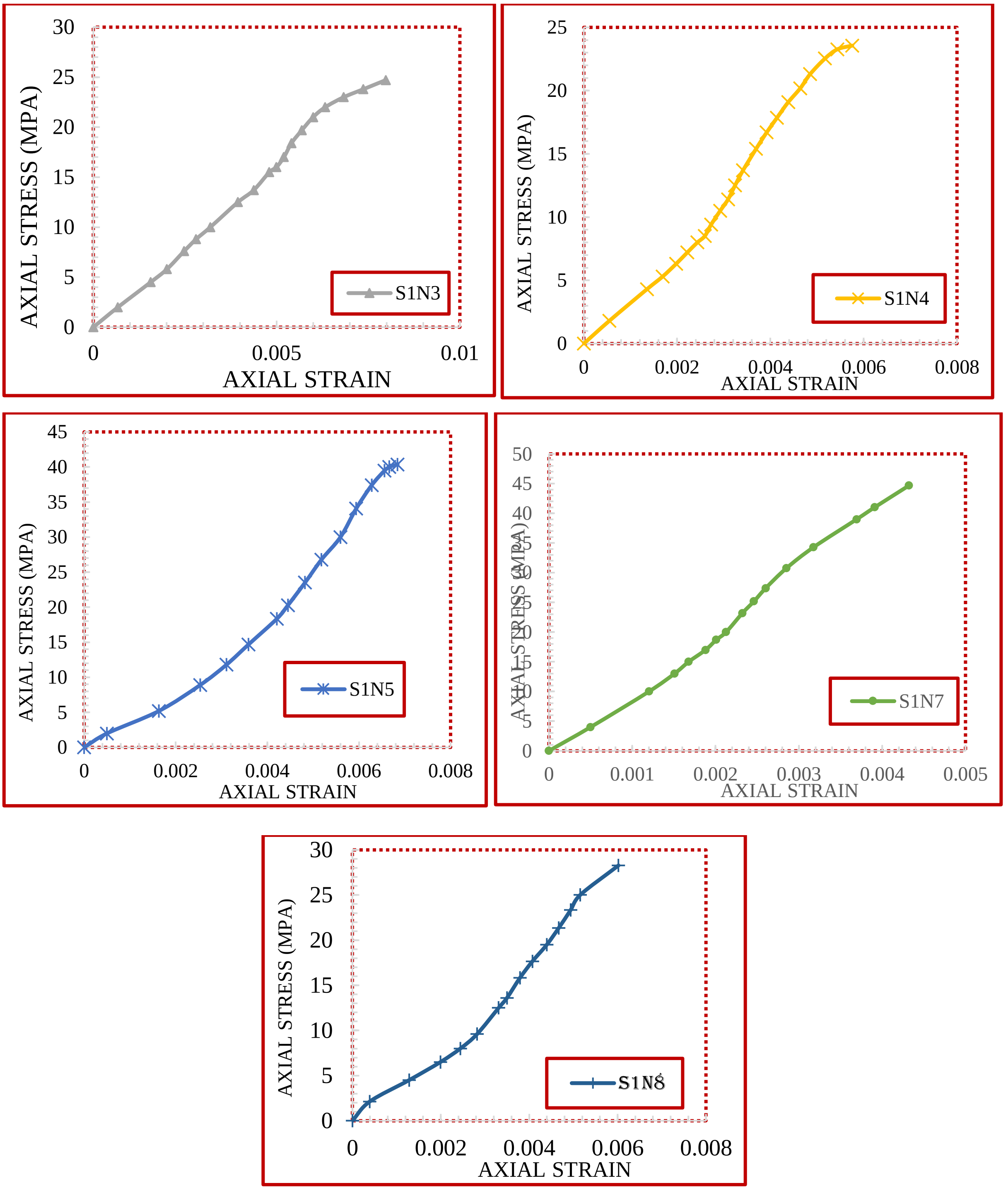


Figure (4-2): Cont.

B. Failure Mode and Stress Distribution

Figs. (4-3) shows the failure mode in each column in group one which are observed during the tests. For the unconfined specimens (i.e., S1N1 to S1N8), the stub columns failed in a brittle manner with the spalling and crushing of concrete.

Splitting tensile cracks were observed in all the specimens as shown in Fig. (5-3). The cracks initiated at 60–70% of ultimate strength. Vertical shear cracks through the height and the cross section were finally developed as the specimen achieved their ultimate resistances which these cracks started from the corner towards the bottom surface of the column. The jacket will suffer from excessive lateral expansion due to unstable propagation of the internal micro-cracks, which causes the strain softening behavior and eventually the concrete mass loses its integrity and fails in splitting manner. The complete collapse of the column usually occurred suddenly at strains between 0.005 and 0.011.

(a) S1N1. **(b)S1N2.** **(c) S1N3.**

(d) S1N4. **(e)S1N5.** **(f) S1N6.**

Figure (4-3): Crack pattern and failure mode of G1 specimens.

(g) S1N7. **(h)S1N8.**

Figure (4-3): Cont.

4.4.1.2 Group Two

This group consists of four circular column specimens with different cross section dimensions introduced so as to compare with the circular strengthened column that have the same dimensions.

A. Stress-Strain Relationship

This group involved testing of four columns with variable dimensions, these columns tested to the failure point under static loads which showed variable strengths ranged between (614.8 – 1313.7) kN with shortening ranged between (2.37-3.718) mm as shown in Table (4-2).

The variance in the maximum strength capacity is due to the changing in the cross-section dimension. The load-displacement response in Fig. (4-4) shows that the stub columns behave linearly until it reaches about 65% of their ultimate resistances. An increase in the maximum strength of the circular columns occurred as a result of the change in the cross section is what actually happened.

The load carrying capacity has been increased by (47%, 60.6%, and 114%) for the increment in the cross-section diameters from 220, 240 and 260 mm respectively which showed maximum strength in the greater diameter (260) mm by (114%) when compared with column (C2N1). Ultimate displacement besides the cracking loads also increased when the diameter of cross-section increased. However, the deformation exhibited a different change in the values in terms of increase as it depended on the amount of stress and its distribution relative to the cross section affected by several factors, the most prominent of which are the cross-section dimensions and the distances between the steel reinforcement ... etc.

Table (4-2): Test results of reference concrete columns of group two.

ID	P_{cr} (kN)	P_{cc} (kN)	Δ_V (mm)	Δ_L (mm)	ε_{cc}	$P_{cc}/P_{Ref.1}$	$\varepsilon_{cc}/\varepsilon_{Ref.1}$	DI	Tn (kN)
C2N1	359	614.8	2.429	2.297	0.00486	1	1	1.12	1012
C2N2	493	902.6	2.370	1.276	0.00474	1.46812	0.97571	1.04	1032
C2N3	725	986.2	3.718	4.83	0.00744	1.604099	1.530671	1.06	2210
C2N4	971	1313.7	3.255	0.545	0.00651	2.136792	1.340058	1.09	1890

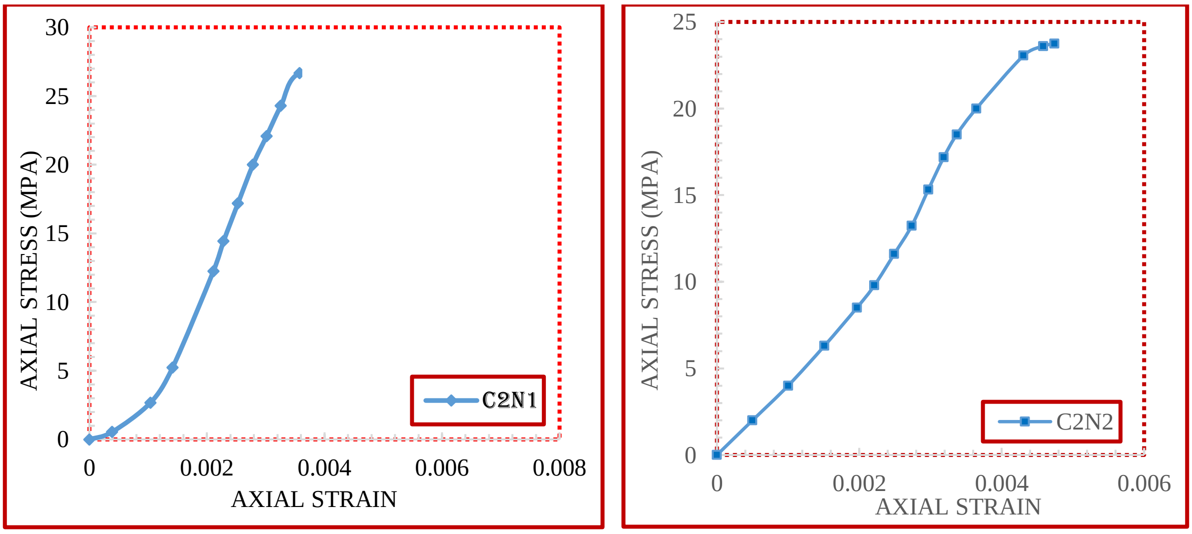


Figure (4-4): Stress-strain relationship of reference columns (group two).

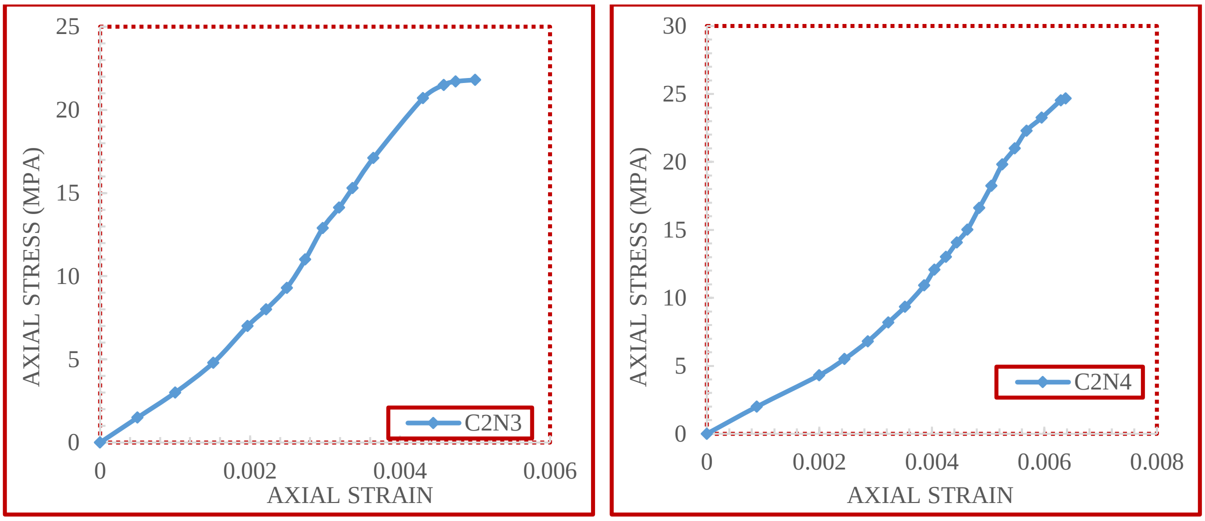


Figure (4-4):Cont.

B. Failure Mode and Stress Distribution

Fig. (4-5) shows the representative failure modes in each column in group two observed from the tests. For the unconfined specimens (i.e., C2N1 to C2N4), the stub columns failed in a brittle manner with the spalling and crushing of concrete. Splitting tensile cracks were observed in all the specimens. The first crack started at the top of the specimen and the number of cracks started increasing gradually on all the sides of the specimens. The cracks grew at approximately mid height at top of the specimen. The continuous loading of the circular concrete columns caused an increment in the hoop strains and disintegration between the concrete particles, which led to the exfoliation of the outer surface, which is located as a cover and separated from the core of the column. The ratio of the presence of cracks in relation to the length of the column has been distributed as follows, small cracks have large occupation in comparison with the large cracks.

(a) C2N1. (b) C2N2.

(c) C2N3. (d) C2N4.

Figure (4-5): Crack pattern and failure mode of group two.

4.4.1.3 Series B

This series consists of 14 square column specimens with the same cross section dimensions strengthened by high strength concrete jacket reinforced by hooked steel fibers with test results. Using constant jacket thickness (25) mm with using epoxy as a bond material for first eight columns are considered to study the effect of increasing of the steel fibers ratio on the general behavior.

To check the jacket thickness effect, groups four and five included use of 35- and 45-mm thickness for the strengthening jacket. Also in these two groups, thickness for three columns involved using epoxy and two kinds of steel fibers. Discussion of results in this series are based on a comparison between the specimens that have variable jacket thickness and compared them with the reference columns and using other parameters such as epoxy and height of jacket to explore their effect on the general behavior of strengthened columns. The test results are presented in the Table (4-3). Regarding the strengthening by jacket, the load capacity of the group four columns ranged between the (1146-3050) kN, maximum axial displacement (2.361-4.65) mm, and lateral displacement (0.002-4.729) mm.

Table (4-3): Test results of reference concrete columns of series B models.

ID	P_{cr} (kN)	P_{cc} (kN)	Δ_V (mm)	Δ_L (mm)	ε_{cc}	$\frac{Pcc}{P_{Ref.1}}$	$\frac{Pcc}{P_{Ref.2}}$	$\frac{\varepsilon cc}{\varepsilon_{Ref.X.}}$	$\frac{\varepsilon cc}{\varepsilon_{Ref.X.}}$	DI	Tn (kN)
S3n0h	870	1638	2.406	1.816	0.00481	1.65	1.433	0.652	0.62	1.18	2380
S3n1h	1380	1698.8	3.009	0.667	0.00601	1.712	1.486	0.815	0.775	1.21	2311
S3n1.5h	831	1971.6	3.954	4.729	0.00790	1.987	1.725	1.071	1.019	1.13	2897
S3n2h	784	1501	2.851	2.112	0.00570	1.517	1.313	0.773	0.735	1.20	2488
S3E0h	1305	1961.8	3.425	1.166	0.0068	1.977	1.717	0.928	0.882	1.33	4558
S3E1h	1270	1727.5	4.65	0.002	0.0093	1.747	1.512	1.26	1.198	1.12	3644
S3E1.5h	1775	2176.8	2.561	0.502	0.00512	2.197	1.905	0.69	0.66	1.14	2980
S3E2h	736	1901.4	3.001	1.071	0.00600	1.917	1.664	0.813	0.773	1.10	2450
S4n2c	1361	2264.3	2.361	0.947	0.00472	2.28	1.893	0.639	0.42	1.41	2706
S4E2c	2304	2997.9	4.402	0.092	0.00880	3.021	2.5	1.193	0.783	1.28	2632
S4n2H	917	1146.4	3.067	0.002	0.00613	1.15	-	0.831	-	1.2	2280
S5n2h	1855	2620	3.668	0.085	0.00733	2.64	1.931	0.994	0.86	1.25	2511
S5E2h	2112	3002.2	4.075	1.344	0.00815	3.025	2.212	1.104	0.95	1.33	3422
S5n2s	2203	3050	4.055	1.254	0.00811	3.07	2.248	1.099	0.95	1.37	3696

A. Effect of Steel Fibers

Table (4-3) reveal the tested strengthened columns results with the stiffness, the ductility index, and the longitudinal and transverse strains that were calculated. The first four columns included varied steel fiber ratio (0, 1, 1.5, and 2%) without epoxy which showed maximum enhancement in the ultimate load by (20%) for the column (S3n1.5h) in comparison with the strengthened column by jacket with zero steel fiber ratio. The fourth column in this group which strengthened by 2% steel fibers is crushed in earlier level than occurred in specimen (S3n0h) by less than 9%. This is due to the large amount of steel fibers which concentrated the forces on concrete and caused concrete crushing before yielding of steel fibers. Enhancement in the maximum stress occurred from (41) MPa for the column (S3n0h) to (42.5 MPa and 49 MPa) for columns (S3n1h and S3n1.5h) respectively. Specimen (S3n2h) showed drop in the ultimate stress to 37.5 MPa as shown in Fig. (5-6). The ductility of these columns enhanced according to the ductility index which improved by (25% and 50%) respectively.

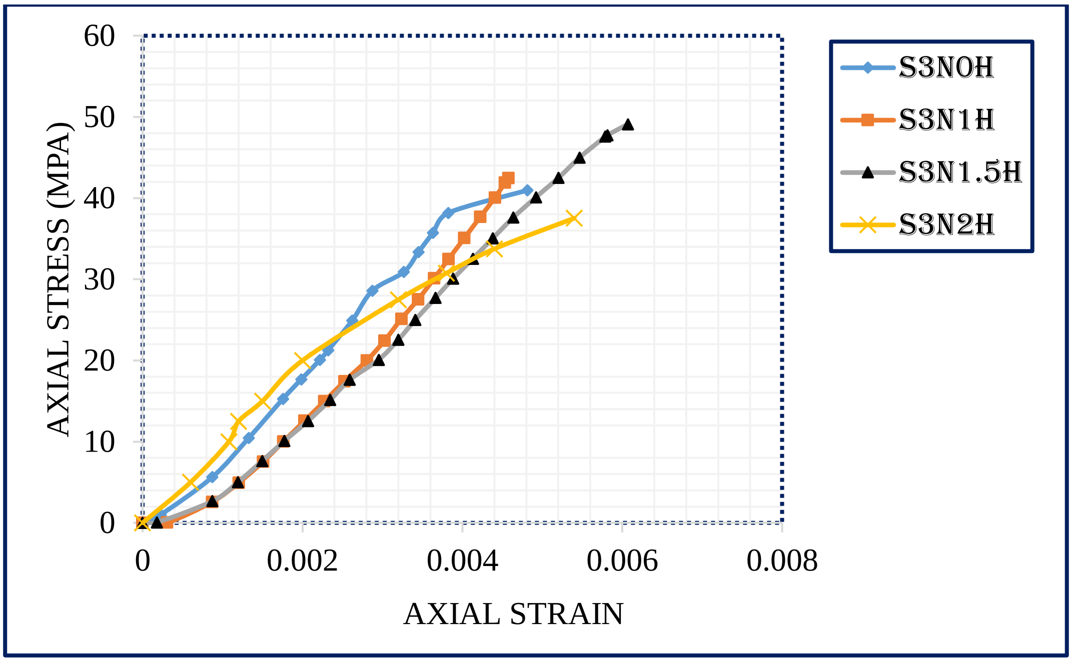


Figure (4-6): Stress-strain relationship clarifying the effect of increasing SF ratio.

Concerning the column that strengthened by SFRC jacket with using epoxy, these columns behaves in a different behavior towards the increasing in the steel fiber ratio as shown in Fig. (4-7). The increase in steel fibers ratio to (1.5%) produced an increase in the strength by (11%) with drop in axial displacement by slight value equal to (5%) only. Specimens (S3E1h and S3E2h) revealed dropping in the strength by (12% and 3%) with increase in strain of 60%. Another comparison can be made to investigate the influence of the epoxy on in the increase in the steel fiber ratio. Presence of epoxy enhanced the maximum strength capacity by (2%, 10%, and 27%) for the transition of steel fiber ratio from (1 to 2%) respectively but with less ductility as shown in Figs. (4-8).

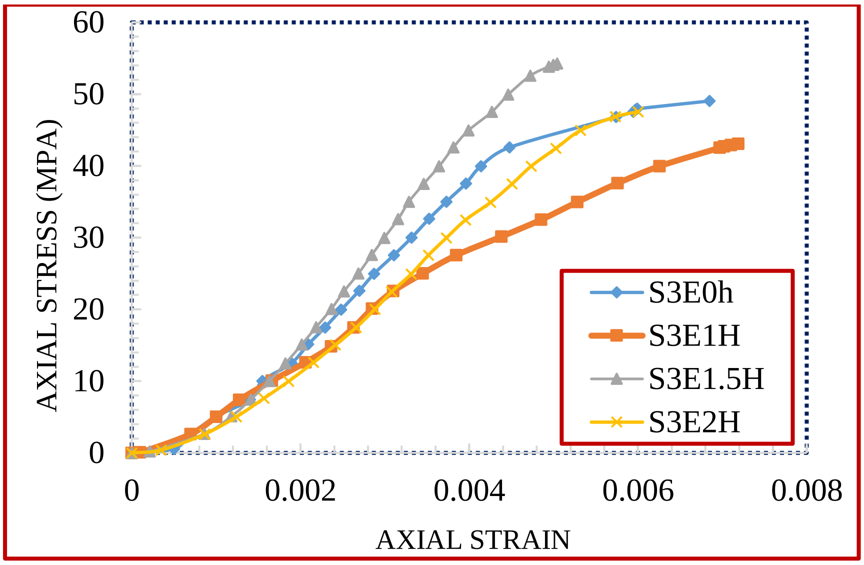


Figure (4-7): Stress-strain relationship clarifying the effect of increasing SF ratio with epoxy.

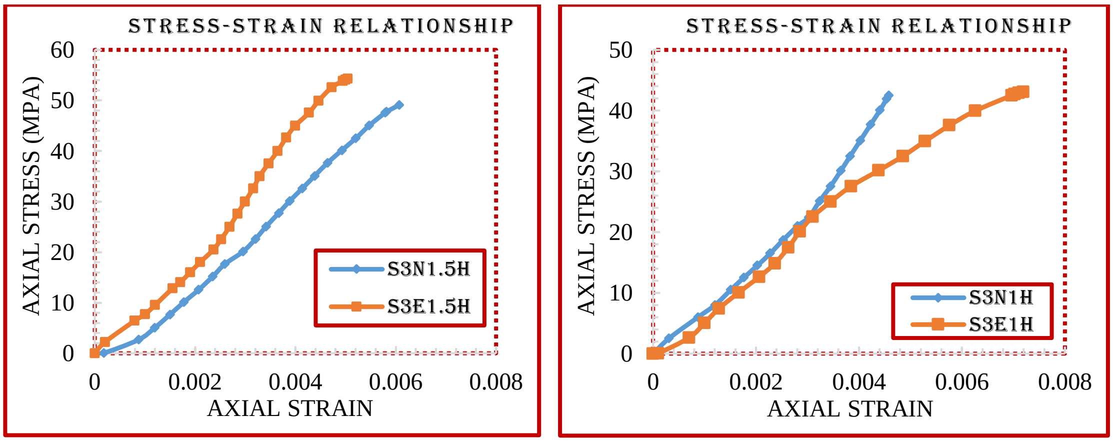


Figure (4-8): Stress-strain relationship clarifying the effect of epoxy.

Ductility index of specimens of this group enhanced with increasing of the steel fiber ratio and strengthening thickness. The enhancement comes with transition of steel fibers between zero to (2%) by average improvement by (9.25%) for the strengthened columns with and without epoxy as revealed in Figs (5-9).

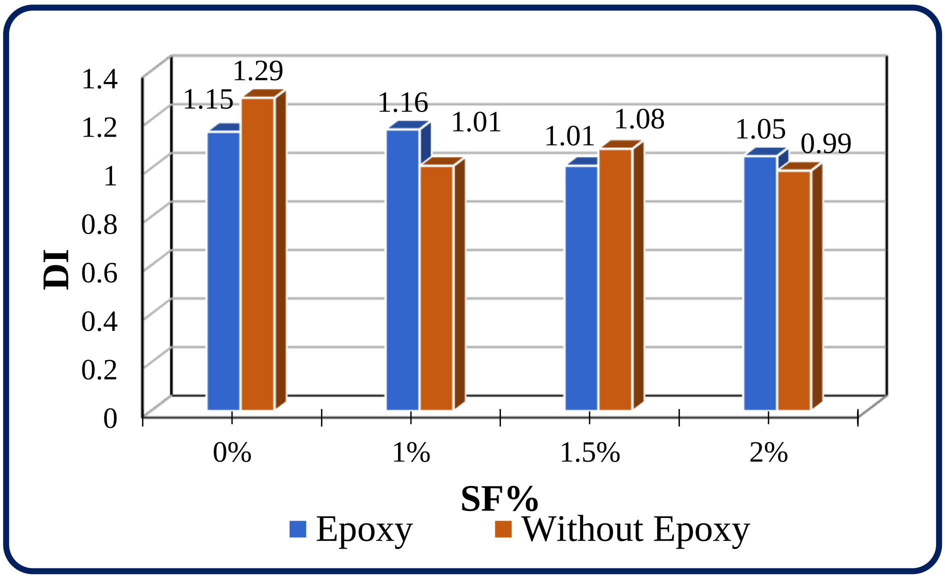


Figure (4-9): Histogram describes: the effect of epoxy and SF% on the ductility index.

B. Effect of Confinement Thicknesses

In this section, the effect of jacket thickness is investigated. To study the influence of this parameter, three thicknesses were carried out by using three values (25, 35, and 45 mm). A comparison is carried out with reference columns by compare this strengthening column with the non-strengthened columns of the original dimension and another comparison with another reference column with the same cross-section area of the strengthened column. It should be noted that all of these comparisons are made with columns that had hooked steel fibers ratio of 2% and without epoxy. In general, using the strengthening enhanced the ultimate strength capacity but with variable enhancement ratio according to the thickness value of the jacket. Addition of (25, 35, and 45) mm jacket enhanced the ultimate load capacity by (51%, 128%, and 164%) respectively in comparison with the reference column (S1N1) that have cross-section area (150 x 150) mm as revealed in Fig. (4-10). The portability of the concrete column to carry stress, and as expected, will be increased by using jackets and the strength capacity increases by increasing the thickness of the jacket, but their values may differ because they depend on the cross-sectional area. The improvement in stresses in case of transition from 25 mm to 45 mm is presented below. Another comparison that can be carried out between the strengthened column and reference column have the same area of strengthened cross-section. The comparison between columns (S3n2h and S1N2), columns (S4n2c and S1N3), and columns (S5n2h and S1N4) for the same cross-section area shown an upgrading in the stress by (31%, 89%, and 93%) respectively (Figs. (4-11 to 13)).

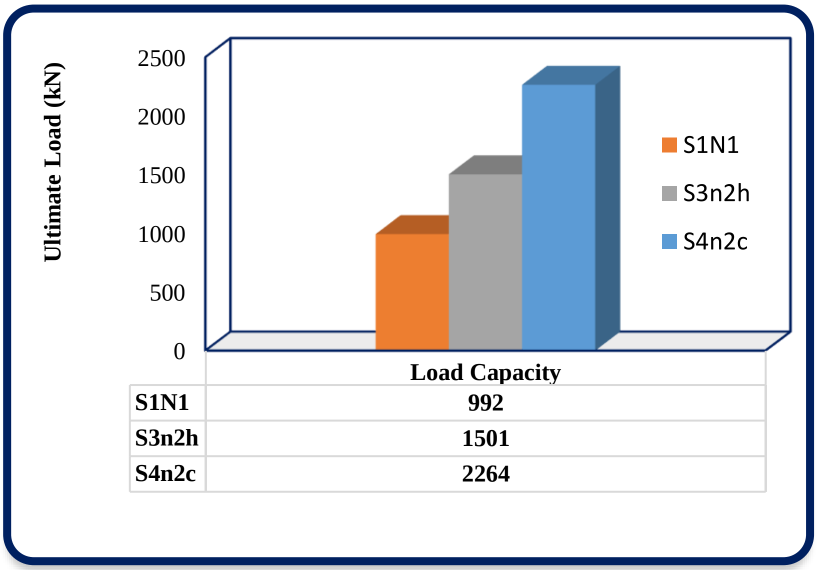


Figure (4-10): Stress-strain curve show the effect of strengthening thickness.

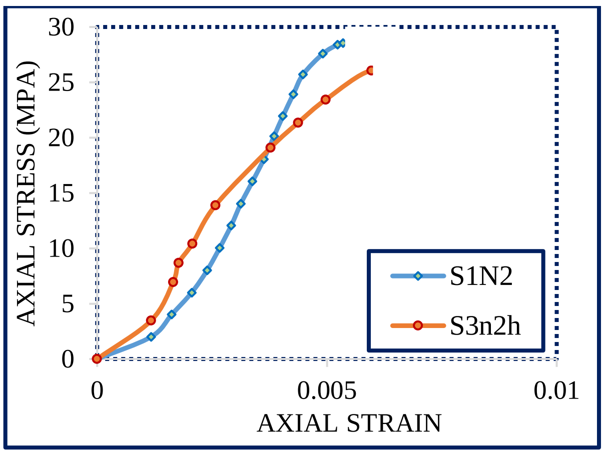


Figure (4-11): Stress-strain curve show the effect of strengthening thickness.

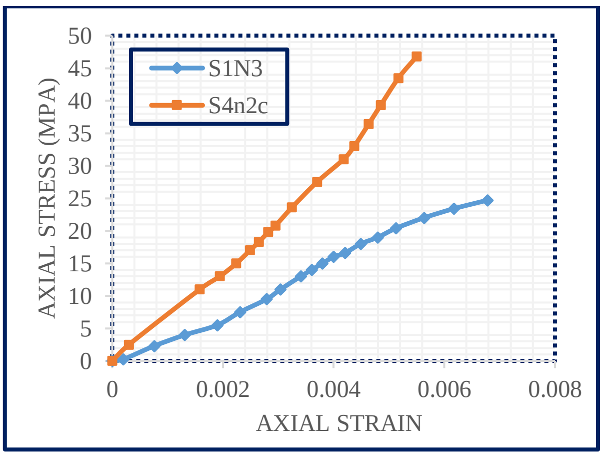
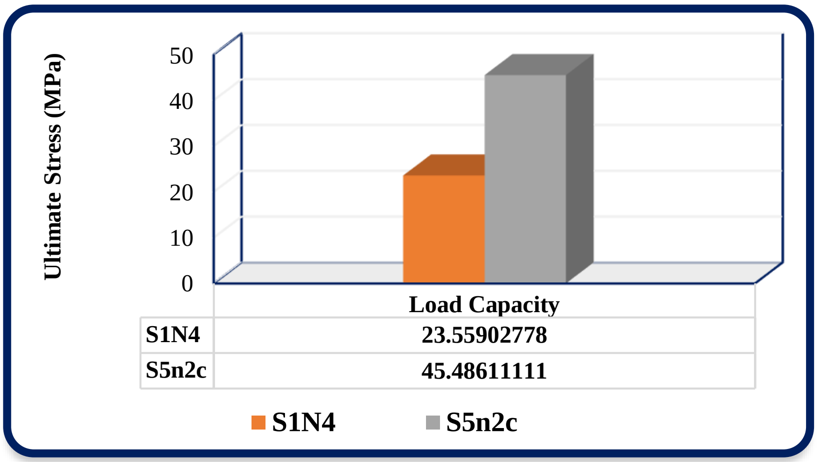


Figure (4-12): Stress-strain response show the effect of strengthening thickness.



Figures (4-13): Bar chart show the effect of strengthening thickness.

Using 35 and 45 mm SFRC composite jacket with and without epoxy showed different behavior. As example, Specimen (S4E2c) with (35) mm SFRC composite jacket (full height jacket) showed stresses equal to (62) MPa approximately while the specimen without epoxy (S4n2c) showed (47) MPa approximately as shown in Fig. (4-14). This means that epoxy material enhanced the strength capacity by (32%) and the axial strain by (68%) after it showed changing in the axial strain from 0.0067 for column without epoxy to 0.0112 for the one that have epoxy. The comparison between the two models with the variable height jacket showed a fundamental difference in general behavior, stress distribution, and even failure pattern. Using of hoop jacket showed higher strength capacity by (9%) than occurred in the column with composite jacket as revealed in Fig. (4-15). As expected, the strains values in hoop jacket case was higher than composite one by (28%) because the applied load was on the core only, this permits to the pressure transfer from the hydraulic jack into the column then to the hoop jacket unlike the composite case which permits the jacket to be in attach with applied load by hydraulic jack where the force is divided into both core and jacket.

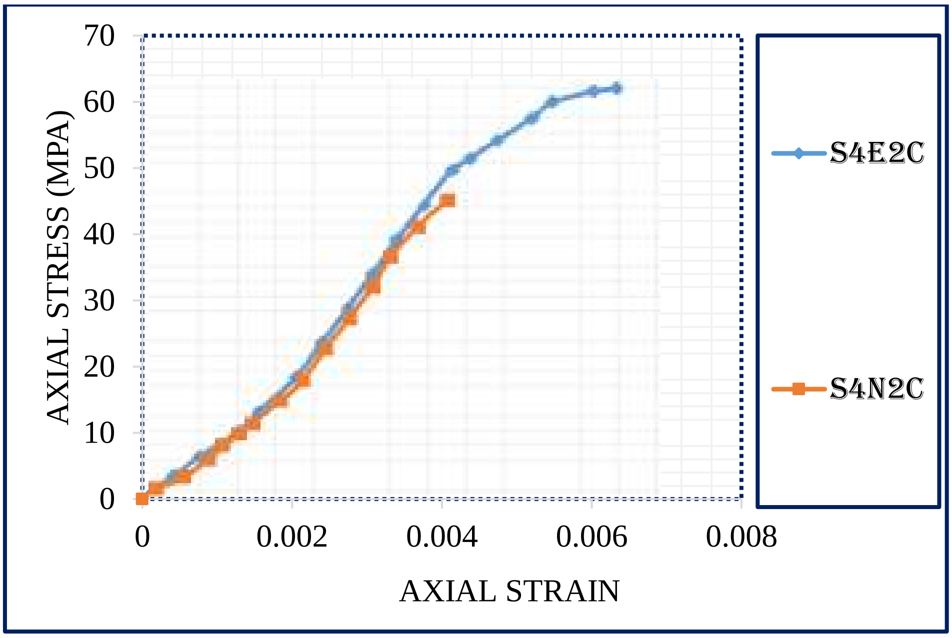


Figure (4-14): Stress-strain curve shows the effect of epoxy.

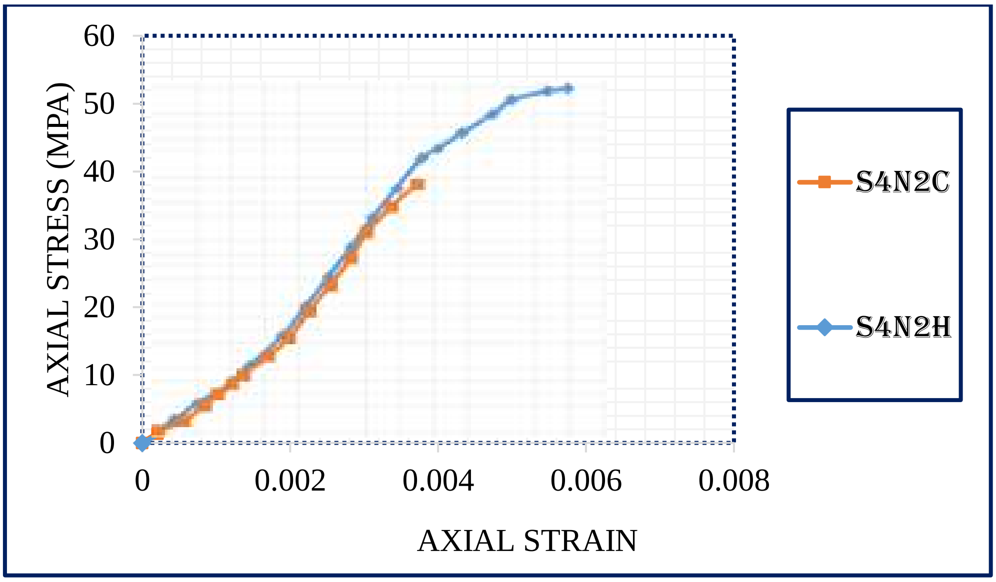


Figure (4-15): Stress-strain curve shows the effect of jacket height.

Ductility index has been enhanced which showed average enhancement is (19%) for strengthened columns with variable strengthening thicknesses as presented in Fig. (4-16).

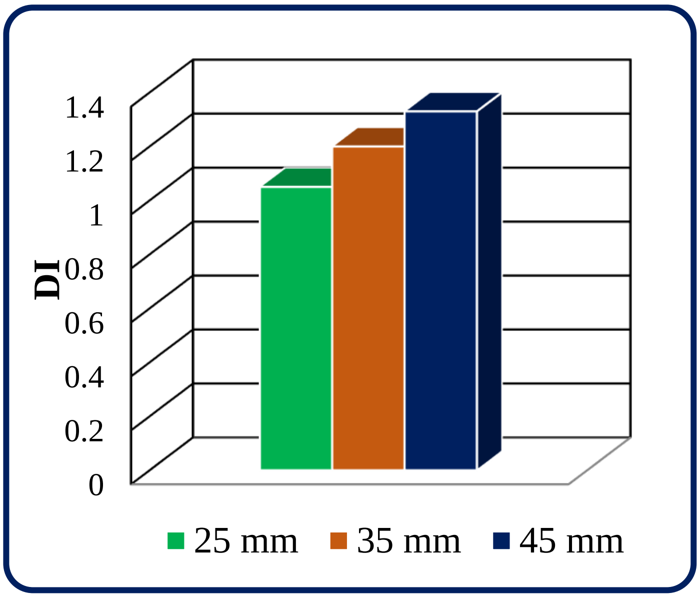


Figure (4-16): Histogram describes the effect of strengthening layer jacket thickness on the ductility index.

C. Failure Mode

Fig. (4-17) shows the representative failure modes of each column in series B observed from the tests. Failure in this series came maybe different with previous group due to the variation in the used parameters. For the confined specimens with zero steel fiber ratio with and without epoxy (i.e., S3n0h and S3E0h) presented a brittle failure and break has occurred in the jacket. The stub confined columns failed in jackets with the spalling and crushing of concrete. Jacketed columns with steel fiber ratio of (1%) (S3n1h and S3E1h) showed crack propagation more than confined column with plain concrete but with smaller size cracks. Steel fibers inside the concrete developed a cohesion between the concrete particles. Columns with steel fiber ratio (1.5% and 2%) showed high deformation in the top and bottom surface due to high cohesion forces that provided by steel fibers. In general, the first crack started at the top of the specimen and the number of cracks started increasing gradually on all the sides of the specimens. The cracks were widened at approximately mid height from top or bottom of the specimen and ultimately the specimen reached the failure. The continuous loading of the concrete columns caused the increment in the hoop strains and disintegration between the concrete particles, which led to the spalling of the outer surface. which is located as a cover and separated from the core of the column.

Column S4n2c that included 35 mm SFRC composite jacket, the failure occurred in the jacket with inclined cracks which started in upper surface corner and extended towards the corner of the bottom surface of the column. This column included two major cracks extended along the column besides the small cracks in all column surfaces. Column (S4E2c) which included parameters of the column specimen (S4n2c) but with using epoxy which showed stress concentration in the top column surface. The breakdown of the top surface of the column means that the cohesion strength between the parts of the column is strong and this indicates the strength of the epoxy by bonding the jacket with the concrete core is active. The remining column that strengthened with SFRC hoop jacket had the maximum strain in comparison with composite jacket. The failure occurred in this column by vertical large crack. The cracks were generated from the center of the side surface of the column, causing a split in the jacket.

(a) S3n0h. **(b)S3n1h.**

Figure (5-17): Crack pattern and failure mode of series B specimens.

(c) S3n1.5h.

(d)S3n2c.

(e) S3E0h.

(f)S3E1h.

(g) S3E1.5h.

(h)S3E2h.

Figure (4-17): Cont.

(i) S4n2c.

(j)S4E2c.

(k) S4n2H.

(l)S5n2h.

(m) S5E2H.

Figure (4-17): Cont.

4.4.3 Series C

Regarding the remaining square column's groups, series C included the the remaining square columns in groups six and seven which studied the parameters such as jacket height, steel fiber type and ratio, and partial strengthening effect. Table (4-4) shows the test results of this series.

Table (4-4): Test results of reference concrete columns of series C columns.

ID	P_{cr} (kN)	P_{cc} (kN)	Δ_V (mm)	Δ_L (mm)	ε_{cc}	$\frac{P_{cc}}{P_{Ref.1}}$	$\frac{P_{cc}}{P_{Ref.2}}$	$\frac{\varepsilon_{cc}}{\varepsilon_{Ref.X.}}$	$\frac{\varepsilon_{cc}}{\varepsilon_{Ref.X.}}$	DI	Tn (kN)
S6E1s	2248	2403	6.114	0.426	0.012228	2.42	2.1	1.65	1.575	1.21	6432
S6E1.5s	2033	2203	6.050	0.794	0.0121	2.22	1.93	1.64	1.56	1.11	5132
S6E2s	1800	2206.8	5.347	0.503	0.010694	2.22	1.93	1.45	1.38	1.21	1779
S6n2s	1568	2090	5.821	0.644	0.011642	2.1	1.83	1.57	1.5	1.53	1688
S6noH	655	946.8	3.514	0.149	0.007028	0.95	-	0.952		1.34	1584
S6n2hH	671	1277.4	3.063	0.171	0.006126	1.28	-	0.83		1.42	1570
S6n2sH	1265	1408	4.373	0.002	0.008746	1.42	-	1.185		1.21	2721
S6E2m	2204	2292.3	4.440	0.45	0.00888	2.31	2.01	1.203	1.14	1.38	3896
S7E2hp3	813	1866	6.120	0.002	0.01224	1.88	1.58	1.658	1.375	1.35	2063
S7E2sp3	1500	1941.4	5.364	1.391	0.010728	1.95	1.65	1.45	1.20	1.46	3422
S7E2hp2	622	1287.5	6.647	0.002	0.013294	1.28	1.44	1.8	1.826	1.33	3380
S7E2sp2	827	1210.4	5.207	0.002	0.010414	1.22	1.35	1.41	1.43	1.52	2389

A. Effect of Steel Fibers

In this study, one of the main variables that are important and directly affecting the performance of the concrete column is use of the steel fibers. Increase of straight steel fiber ratio (1%, 1.5%, and 2%) in presence of epoxy as bond material exposed different strength capacities that showed increments by (22%, 12%, and 12%) respectively in comparison with column (S3E0h) as demonstrated in Fig. (4-18). The obtained ductility in the strengthened column with jacket of (1%) steel fibers ratio is better than other ratios which got strain by (0.0119). An increase in steel fibers does not mean a permanent increase in ultimate load capacity, therefore, the unnecessary increase in the amount of steel fibers causes more cost without large benefit. The best ratio for straight steel fibers in this study is (1%) as demonstrated in Fig. (4-19). The effect of steel fibers in the square columns can be checked by compare the results of this series with the group three of series two. The obtained enhancement in the strength capacity by use of straight fiber were better than those columns which used hooked fibers. Enhancement in the strength capacity with using of straight steel fibers is more than hooked by (39.5% and 17%) for the steel fibers ratio of (1% and 2%) respectively as revealed in Fig. (4-20 a & b). Best enhancement in the stress capacity were by use of 1% for straight steel fibers and 1.5% for hooked fibers. Both types of steel fibers (hooked & straight fibers) were used together (column S6E2m) which presented higher load capacity by (16%) in comparison with columns with hooked steel fibers (S3E2h) and (4%) with columns with straight steel fibers (S6E2s) as shown in Fig. (4-20 c). It should be noted that use of both types of steel fibers was less cost of straight fibers, so it can be considered is the best choice of strengthening because it provides higher strength with less cost than presented by other techniques.

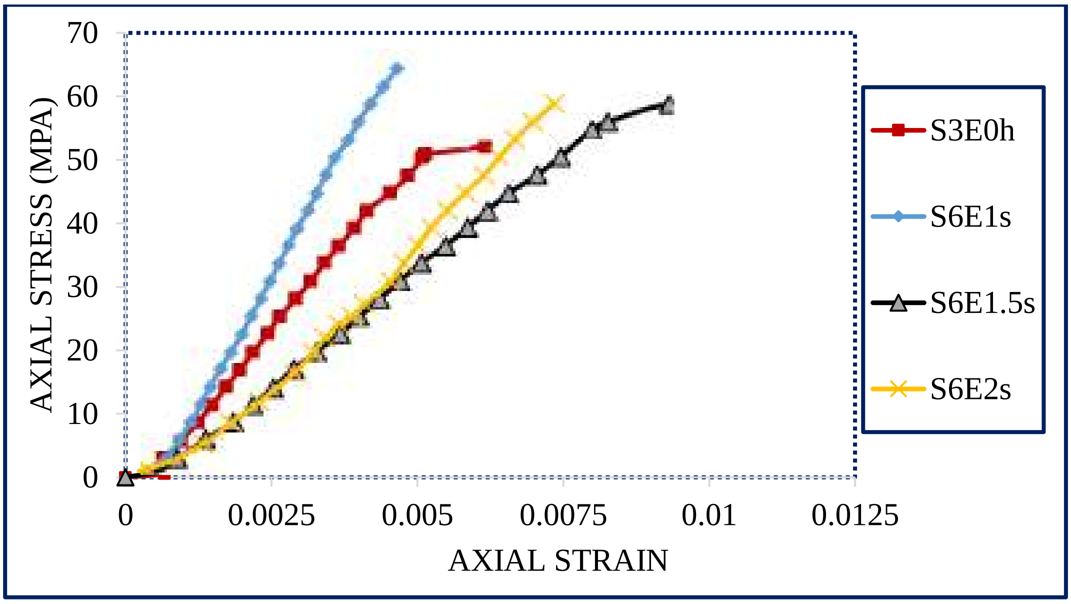


Figure (4-18): Stress capacity of series C columns with increment SF ratio.

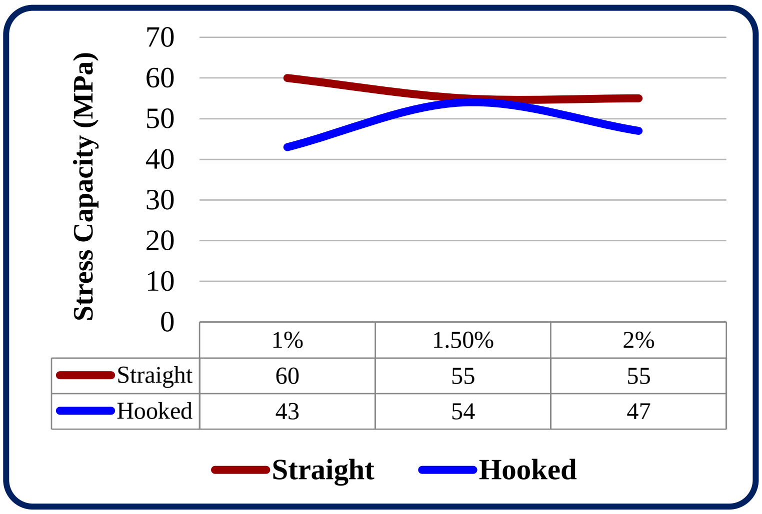
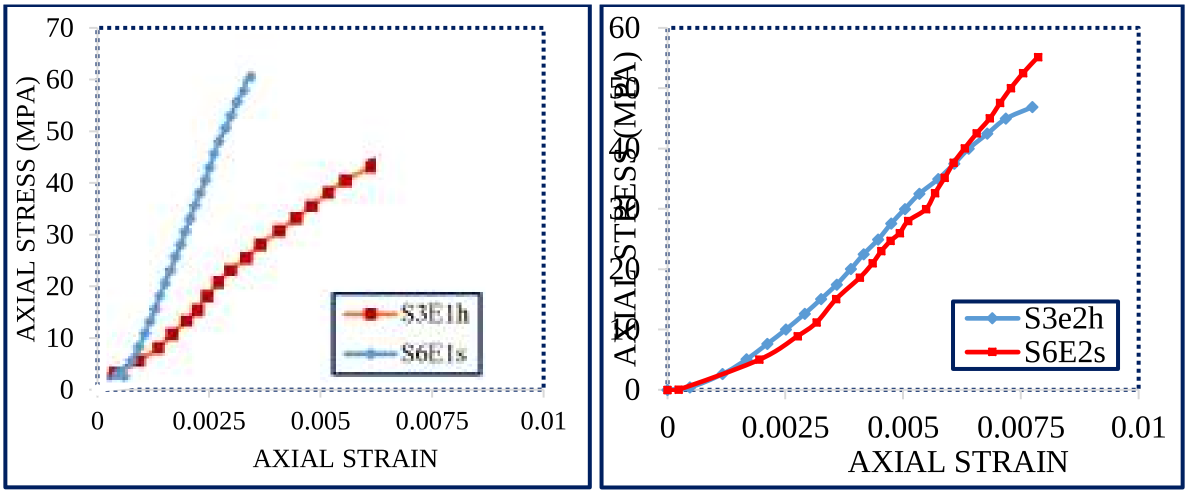
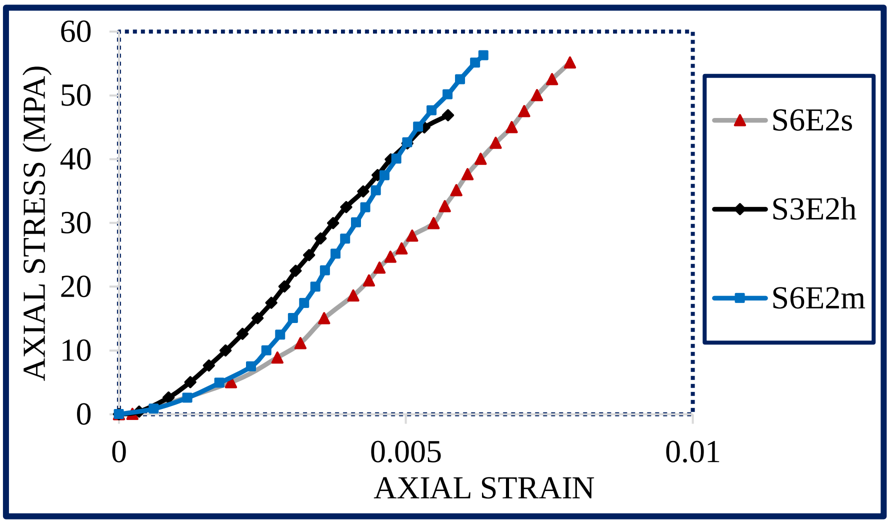


Figure (4-19): Optimum SF ratio for hooked and straight fibers.



(a) Columns with (1% SF). **(b) Columns with (2% SF).**



(c) **Columns with different types of steel fibers.**

Figure (4-20): Stress-strain curve of displayed the effect of steel fibers type effect.

B. Effect of Jacket Height

The effect of jacket height was significant on the overall behavior of the concrete column, the stress distribution mechanism and the failure mechanism. Change of the jacket height from composite to hoop case showed better enhancement in the strength capacity. In comparison between the columns (S3n0h and S3n2h) which included use of composite jacket case which showed decreasing by strength capacity less than (9%). Otherwise the columns (S6noH and S6n2hH) which showed enhancement in the strength capacities by (35%). Also, use straight fibers of ratio (2%) in hoop case presented increasing in the strength capacity more than those of hooked fibers by (10%) in comparison with column (S6n0h) as revealed in Fig. (4-21). It should be noted that hoop strengthening increased the strength capacity by (56%) in comparison with the refence column (S1N1) as shown in Fig. (4-22).

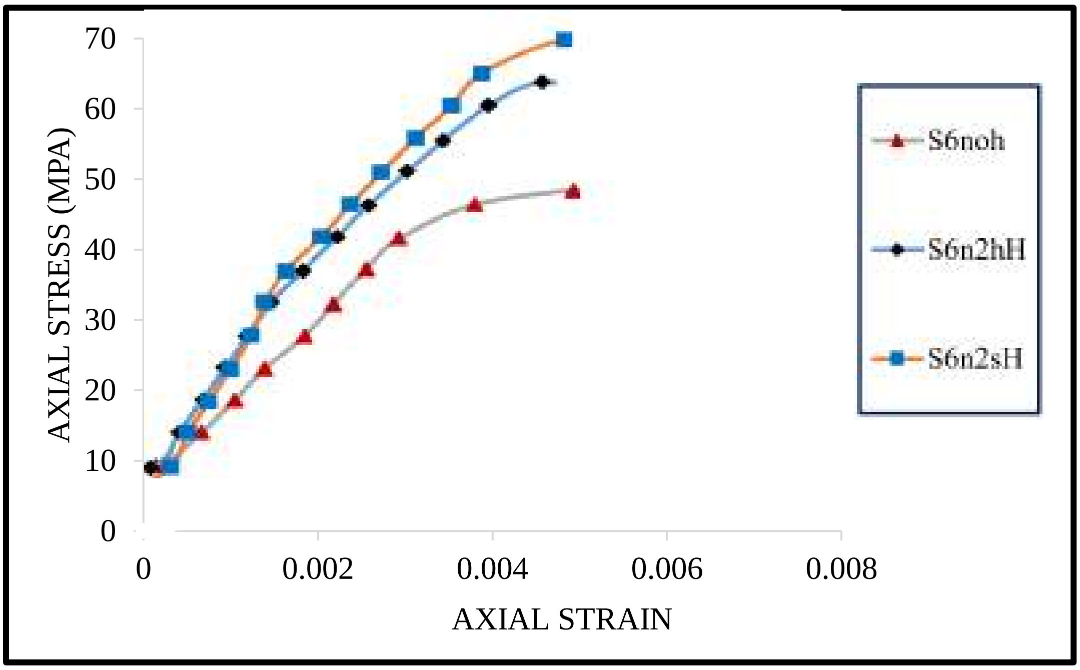


Figure (4-21): Stress-strain response, the effect of jacket height.

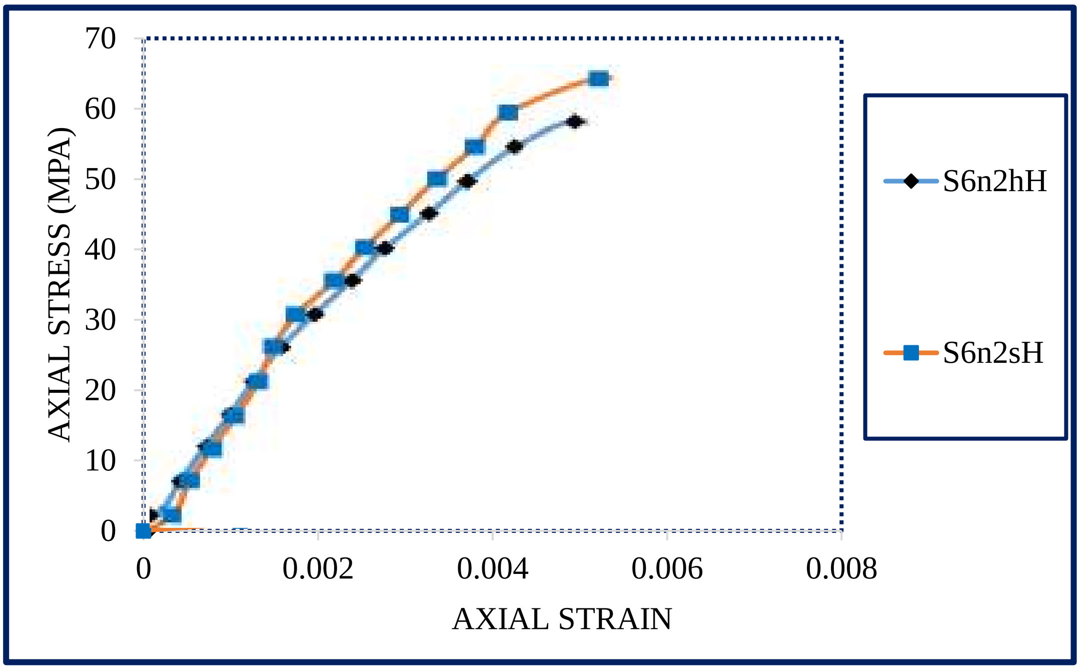


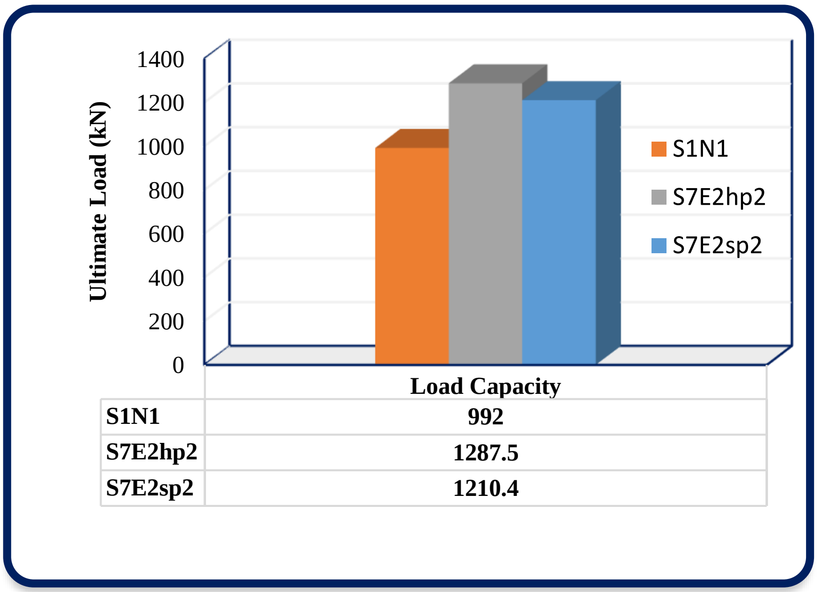
Figure (4-22): Stress-strain response, the effect of steel fiber type.

C. Effect of Partial strengthening Jacket

Other techniques used to strengthen the RC columns by strengthening the column by jacketing into two and three sides and keep the remaining side without strengthening.

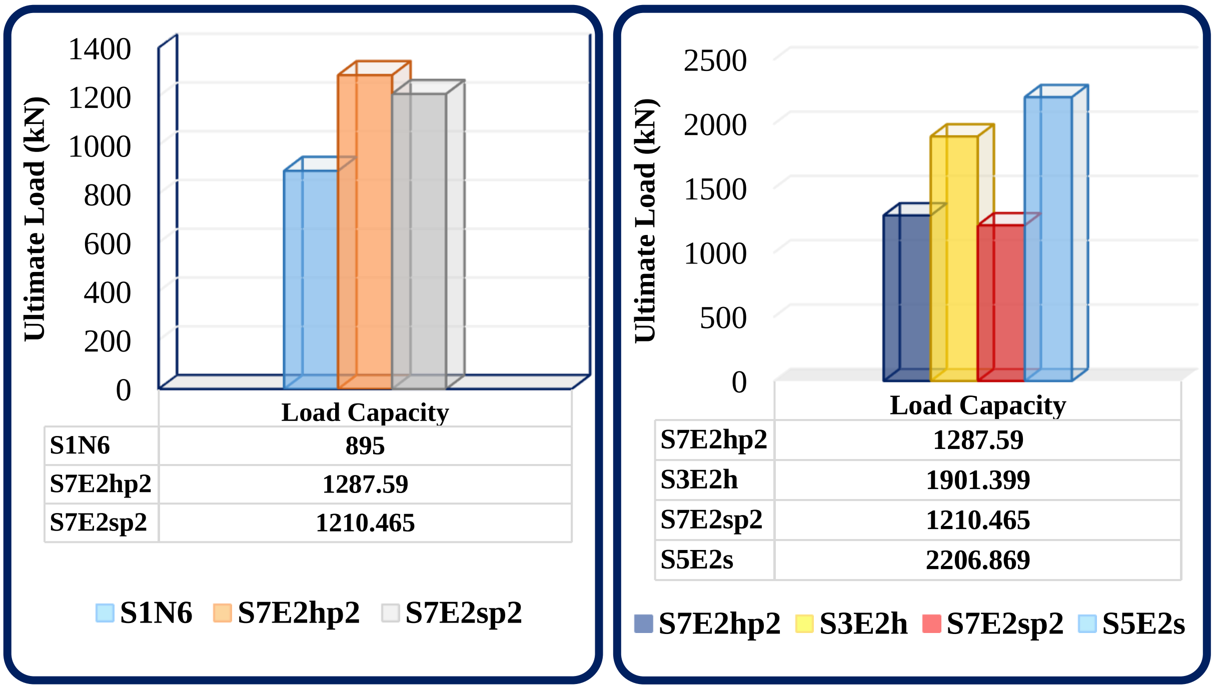
The stress distribution, failure mode also been changed besides the general behavior due to the variation in the cross-section area. Starting from the columns (S7E2hp2 and S7E2sp2) that strengthened by two sides jacket in adjacent directions with straight and hooked steel fibers (each column has one type) which showed increasing in the ultimate load capacity by (22% and 30%) respectively in comparison with the un strengthened column (S1N1) as revealed in Fig. (4-23). These two sides strengthening improved the maximum ultimate strength capacity by (35% and 44%) for the two columns respectively in comparison with the reference column (S1N6) that have the same cross-section area as shown in Fig. (4-24).

While when comparing the two sides strengthening with four sides strengthening, it found that strengthening by four sides had better enhancement in the ultimate load capacity than two sides by (48% and 82%) for the two types of steel fibers (hooked and straight) respectively as revealed in Fig. (4-24).



	Load Capacity
S1N1	992
S7E2hp2	1287.5
S7E2sp2	1210.4

Figure (4-23): Stress capacity of series C columns with increment SF ratio.



	Load Capacity
S1N6	895
S7E2hp2	1287.59
S7E2sp2	1210.465

	Load Capacity
S7E2hp2	1287.59
S3E2h	1901.399
S7E2sp2	1210.465
S5E2s	2206.869

Figure (4-24): Effect of partial strengthening on the ultimate load capacity.

Concerning three sides strengthening, two strengthened columns showed good increment in the ultimate load capacity for the hooked and straight fibers by (88% and 96%) respectively in comparison with the reference un strengthen column (S1N1) as revealed in Fig. (4-25). When comparing these three sides strengthened column with the refence column that have the cross-section area (S1N5), it is found that ultimate stress capacity enhanced by (58% and 65%) for both hooked and straight fibers as demonstrated in Fig. (4-26). In general, three sides strengthening are better than two sides strengthening in the enhancement of the ultimate load capacity.. The ultimate strength enhancement comes as follows; three sides strengthening is stronger than two strengthening of two sides by (60%). While the four sides strengthening is stronger than three sides by (14%) as demonstrated in Figs. (4-27) and (4-28).

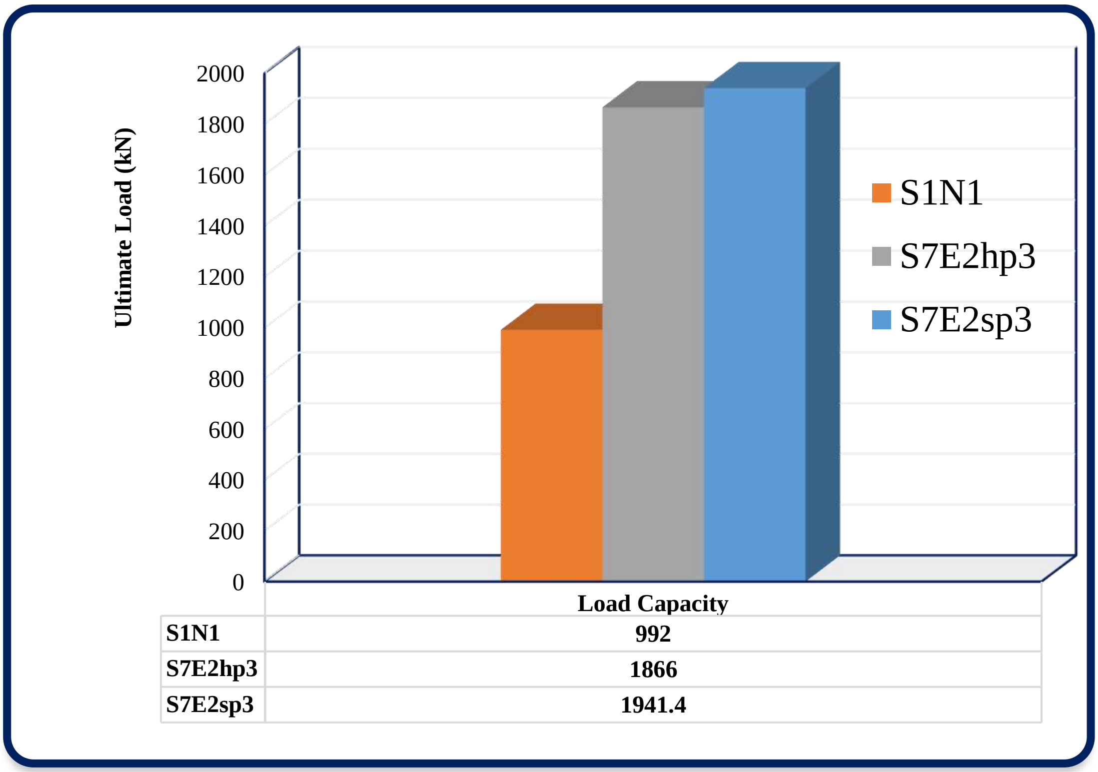


Figure (4-25): Effect of partial strengthening in three directions on the ultimate load capacity.

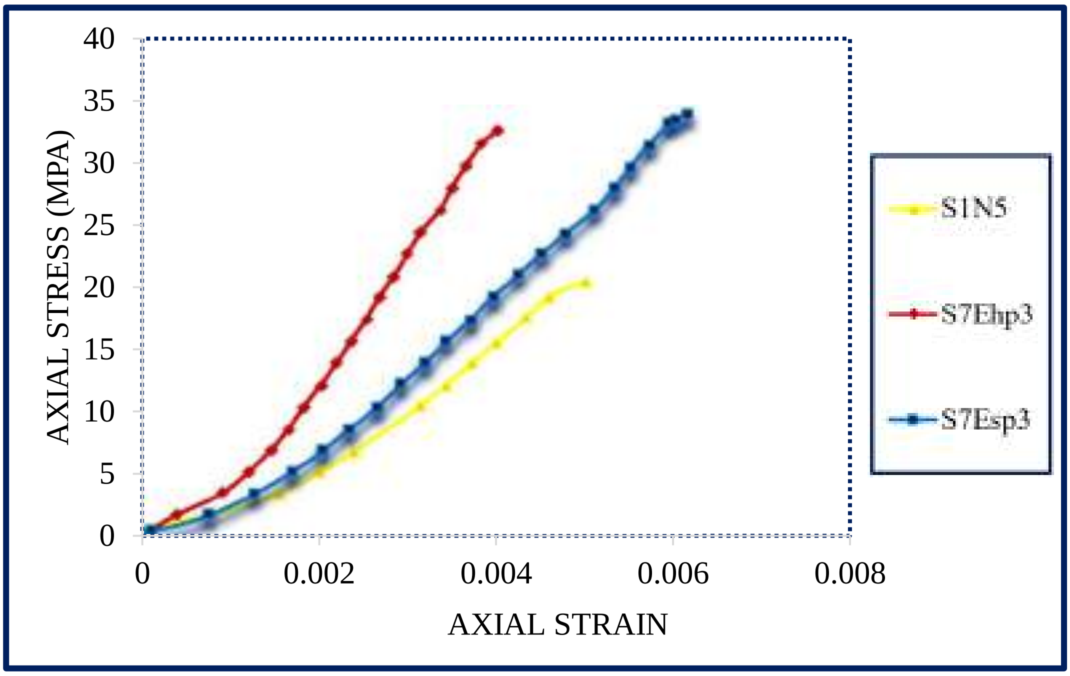
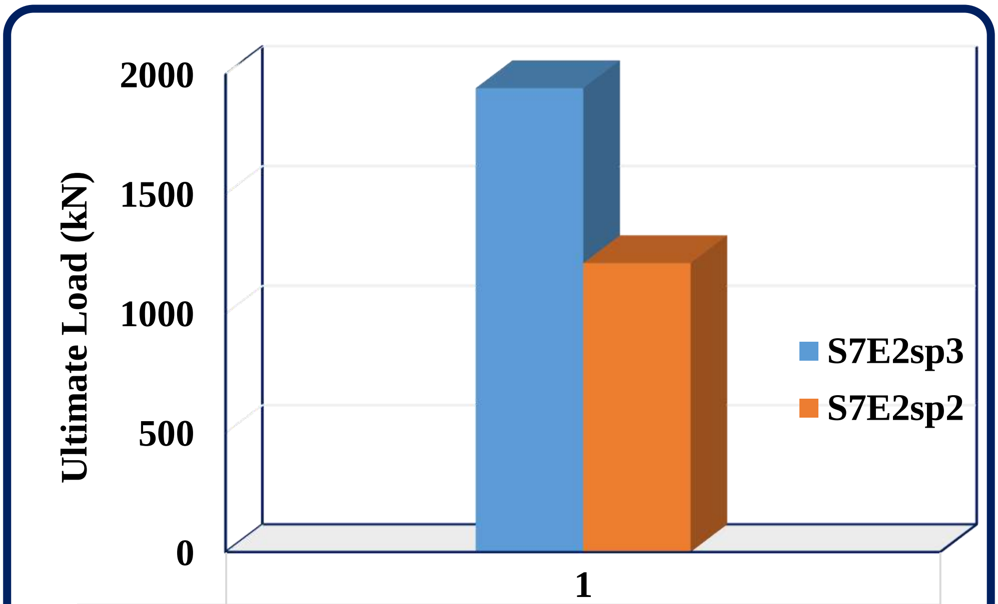


Figure (4-26): Effect of steel fibers types in partial strengthening.



	1
S7E2sp3	1941.4
S7E2sp2	1210.4

Figure (4-27): Comparison between two and three sides partial strengthening.

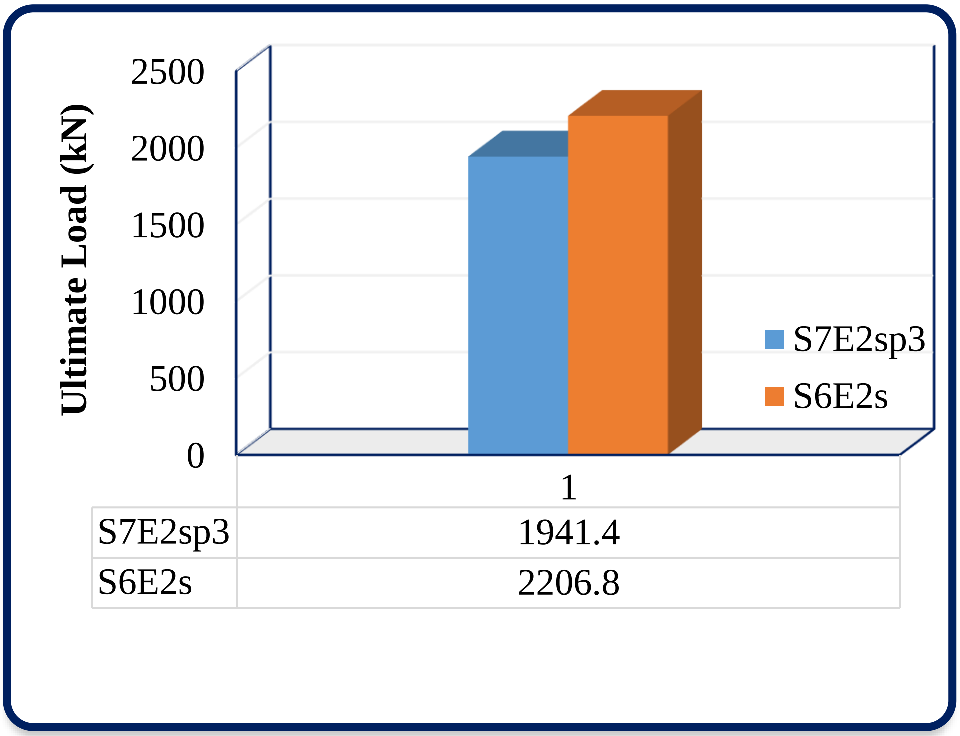


Figure (4-28): Comparison between two and three sides partial strengthening.

D. Failure Mode

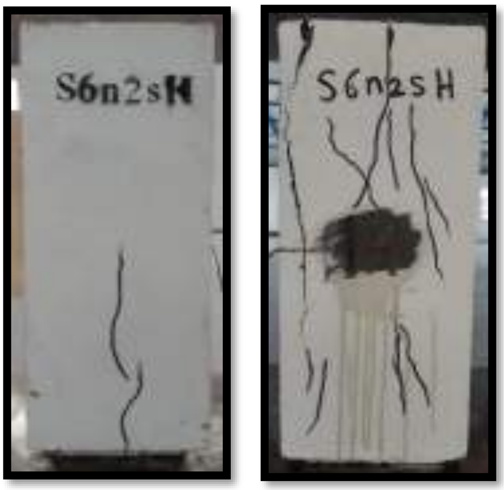
Regarding the failure mode of series three, Figs. (4-29) shows the representative failure modes of these columns. Failure in this series came maybe different with previous group due to the variation in the used parameters. Column (S6E1S to S6E2s) that included use of incremental ratio of straight steel fibers, the failure occurred in the jacket with inclined cracks started in upper surface corner and extended towards the bottom surface of the column. The increase in steel fibers ratio made the cracked headed to the corner as if it has strengthened the cohesion between the concrete particles in the middle cross section length to comeback and focus on the corners. As for the non-reinforced hoop model (S6n0H), the loads caused the jacket to split in two parts and its failure a brittle failure. Presence of steel fibers by their two types transferred the failure mode from sudden split to high deformation mode due to the offered ductility by steel fibers. Use of two and three direction strengthening caused brittle failure and torn in SFRC jacket as revealed in column (S7E2sp3).

(a) S6E1s. **(b) S6E1.5s**

(c) S6E2s. **(d)S6n2s**

(e) S6n0H **(f) S6n2hH**

Figure (4-25): Crack pattern and failure mode of series C columns.



(a) S6n2sH.

(b)S6E2m.

(c) S7E2hp2.

(d)S7E2sp2.

(e) S7E2hp3.

(f)S7E2Sp3.

Figure (4-29): Cont.

4.4.4 Series D

This series consist of circular columns (group 8 to 10) strengthened with variable jacket thickness, epoxy, and steel fibers ratio and type. All test results are presented in Table (4-5).

Table (4-5): Test results of circular concrete columns of series D.

ID	P_{cr} (kN)	P_{cc} (kN)	Δ_V (mm)	Δ_L (mm)	ε_{cc}	$\frac{Pcc}{P_{Ref.1}}$	$\frac{Pcc}{P_{Ref.2}}$	$\frac{\varepsilon cc}{\varepsilon_{Ref.1}}$	$\frac{\varepsilon cc}{\varepsilon_{Ref.X}}$	DI	Tn (kN)
C8n2h	950	1670	3.705	0.268	0.00741	2.716	1.271	1.525	1.138	1.22	3078
C8n2s	729	2670	4.344	0.795	0.00869	4.343	2.032	1.788	1.335	1.43	3652
C9n1h	990	1805.7	4.381	1.506	0.00876	2.937	2.001	1.803	1.849	1.64	2210
C9n1.5h	876	1049.2	3.505	1.554	0.00701	1.707	1.162	1.442	1.479	1.72	2965
C9n2h	954	1390.5	4.097	1.809	0.00819	2.262	1.541	1.686	1.729	1.44	2588
C9E1h	584	1436	5.224	0.065	0.01045	2.336	1.591	2.150	2.204	1.42	2004
C9E1.5h	807	1427	5.786	0.851	0.01157	2.321	1.581	2.381	2.441	1.32	2198
C9E2h	660	1057.6	3.198	0.401	0.00640	1.720	1.172	1.316	1.349	1.29	2157
C9E2s	490	1842	4.623	0.098	0.00925	2.996	2.041	1.902	1.951	1.52	2789
C9n2hH	660	1047	3.510	0.223	0.00702	1.71	-	1.444	-	1.52	1458
C9n2sH	489	1041	3.508	0.148	0.00702	1.695	-	1.444	-	1.61	2100
C10n2h	1753	2103	3.667	1.231	0.00733	3.421	2.132	1.509	0.986	1.62	3489
C10E2h	1804	2203.5	3.458	0.987	0.00692	3.584	2.234	1.423	0.930	1.12	3322

A. Effect of Steel Fibers with Epoxy in Circular Columns Jacket

To study the effect of steel fiber ratio in circular columns, three values were used (1%, 1.5%, and 2%) which showed increasing in the stress capacity of the circular columns. The increase in the steel fibers ratio enhanced the ultimate load capacity by (194%, 71%, and 126%) respectively in comparison with the control circular unstrengthen column (C1N1) as demonstrated in Fig. (4-30). When comparing the strengthened column with control column that have the same cross section area (C2N2), it is found that the increase in the steel fibers ratio enhanced the ultimate stress capacity by (100%, 16%, and 54%) respectively as revealed in Fig. (4-31). Concerning the effect of epoxy, using epoxy appeared a different behavior in comparison with square columns. Use of epoxy showed lower stress capacity by (21% and 24%) for (1% and 2%) jacket of hooked steel fibers ratio respectively. While use of epoxy for column with (steel fiber ratio 1.5%), enhancement occurred in the stress capacity by (76%) approximately (Fig. 4-32). To check the effect of steel fibers type, straight fibers was used in 2% ratio in column (C9E2s) which showed higher strength capacity than those in hooked fibers by (74%) as demonstrated in Fig. (4-33).

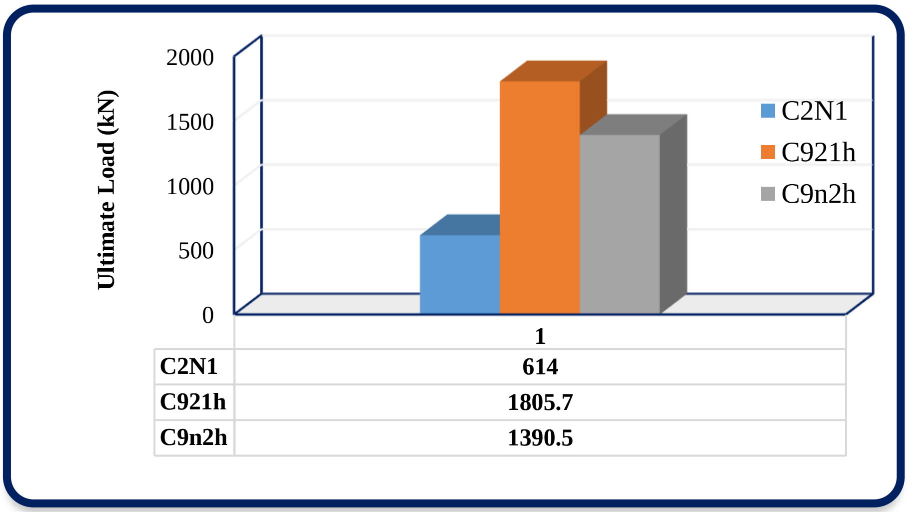


Figure (4-30): Effect of steel fibers ratio on the circular columns.

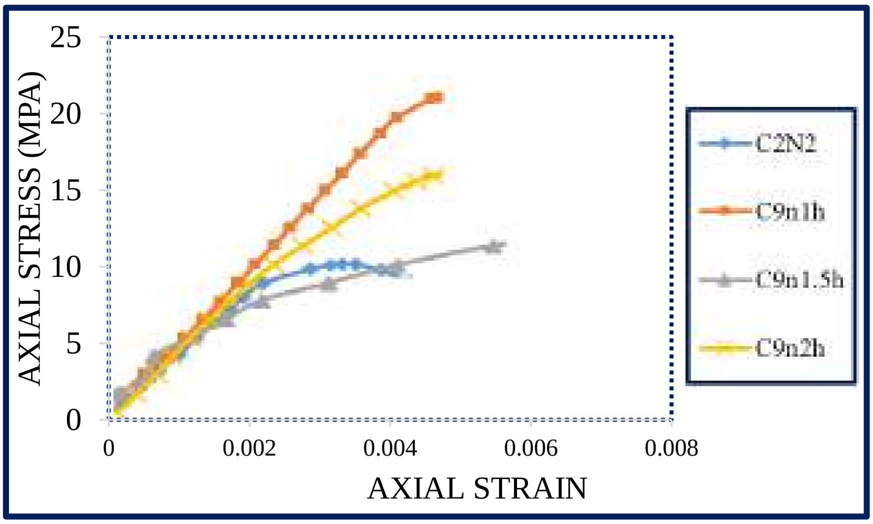
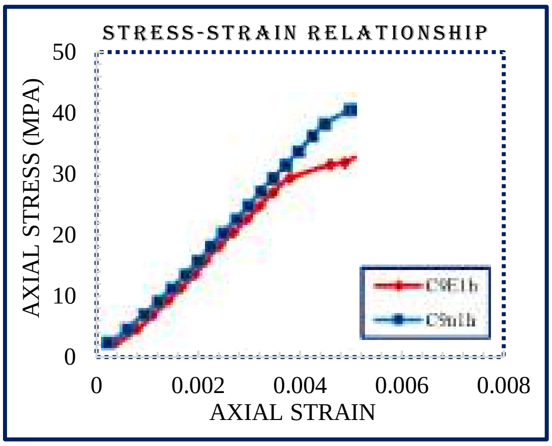
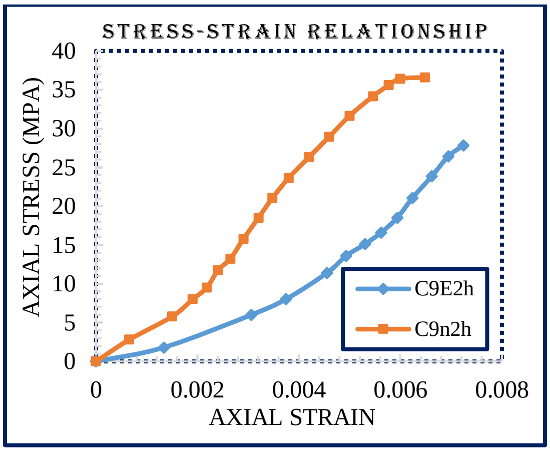


Figure (4-31): Effect of steel fibers ratio on the circular columns.





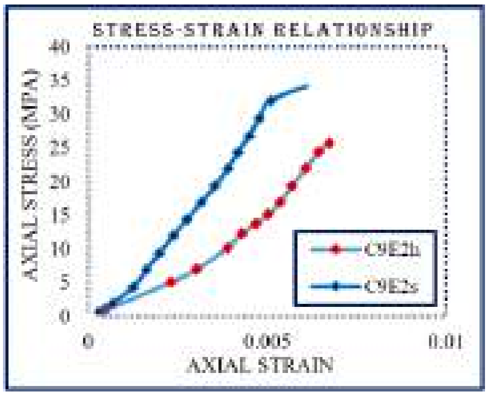


Figure (4-32): Effect of Epoxy and steel fiber type on the circular columns stress-strain.

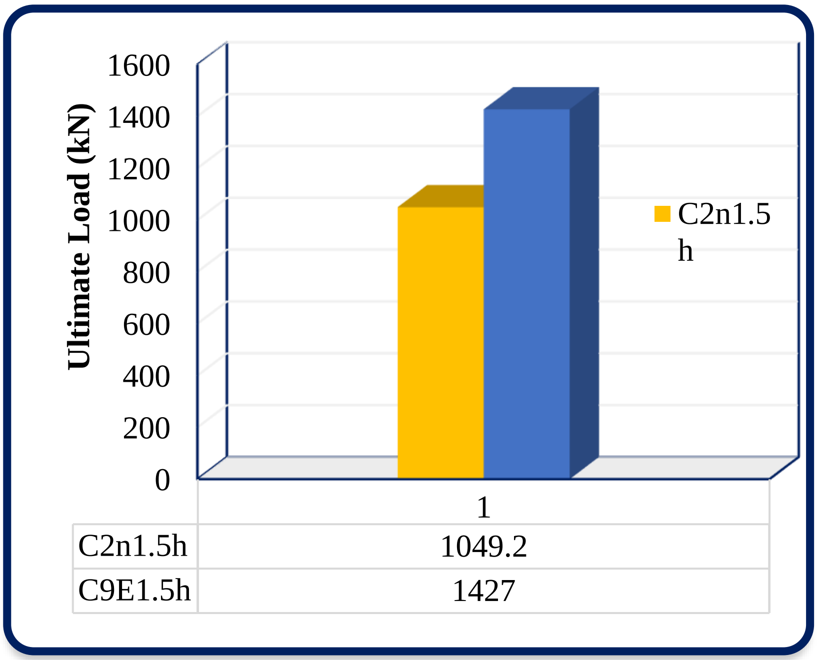


Figure (4-33): Effect of epoxy on the circular columns ultimate stress capacity.

B. Effect of Jacket Height

Figure (4-34) present the stress-strain relationship and the effect of jacket height effect. The effect of jacket height was significant on the overall behavior of the concrete column, the stress distribution mechanism, and the failure mechanism. Hoop jacket in the circular columns enhanced the ultimate stress capacity by (70%) for jacket with the two ends hooked steel fibers (C9n2hH) and (69%) for the small straight fibers case (C9n2sH) in comparison with the reference column (C2N1). Using two types of steel fibers didn't affect the ultimate stress capacity which showed converged enhancement values. But in terms of general behavior, the initial crack load in straight fiber occurred in earlier time than those in hooked fibers. Strengthening by hoop jacket with straight fibers cracked in (47%) of the ultimate load. While of hooked case was cracked in (63%) of the ultimate load capacity. The ductility enhanced by using steel fibers in comparison with the reference column. The enhancement due to straight steel fibers is better than hooked fibers which offered maximum strain by (46%).

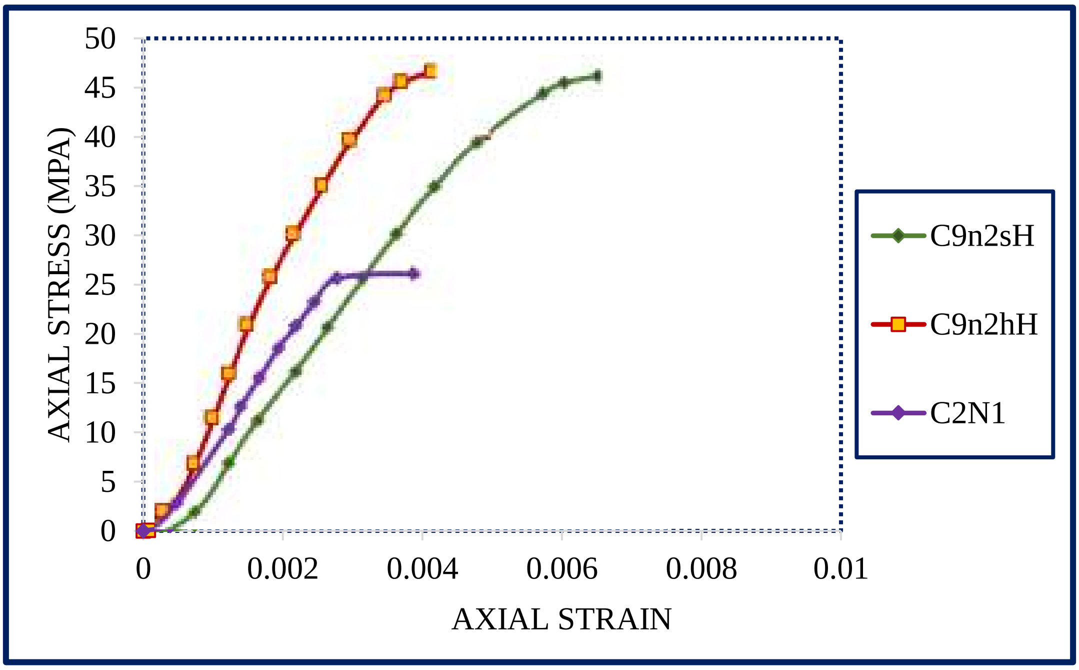
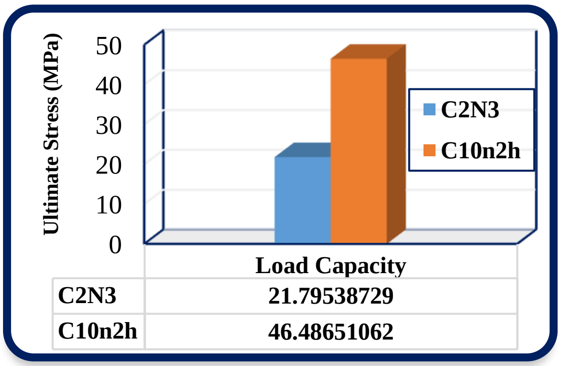


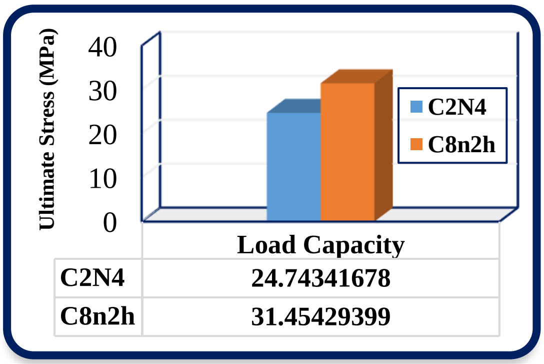
Figure (4-34): Effect of hoop jacket on the circular columns stress-strain response.

C. Effect of Jacket Thickness

Use of variable jacket thicknesses in circular column affected the general behavior, initial cracking load, ductility, ultimate load capacity and failure mode. Strengthened columns by SFRC jacket with thicknesses (25, 35, and 45) showed enhancement in the stress carrying capacity by (126%, 242%, and 171%) respectively for the jacket with hooked fibers as revealed in Table (4-36). Initial cracking load doesn't affect during the transition from the (25 mm) to (35 mm) jacket which showed changing from (71%) to (70%) of the ultimate load capacity. Use of (35 mm) jacket increased the stress capacity in comparison with the column have the same cross-section area (C2N3) by (132%) and initial cracking stress by (102%) as revealed in Fig. (5-35 a). In the same way, column (C8n2h) is higher strength capacity than the control column with the same cross-section area (C2N4) by (27%) as revealed in Fig. (4-35 b). Increasing the thickness of the casing does not mean a permanent increase in the stress capacity, which will in fact be limited to a certain extent. Best jacket thickness in this study was (35 mm) which provided higher enhancement. Presence of epoxy offered a slight increment in ultimate stress capacity by (5%) approximately in comparison between the columns (C10n2h and C10E2h). Initial cracking load also increased from (70% to 77.5%) as demonstrated in Fig. (4-37). Straight fibers were better than hooked fibers by (60%) in the strength capacity as shown in Fig. (4-38).



(a)



(b)

Figure (4-35): Effect of jacket thickness on the circular columns stress.

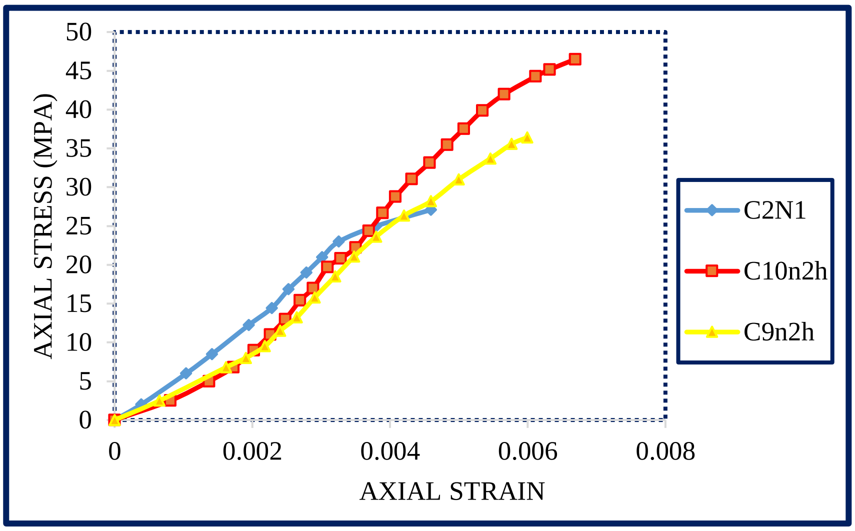


Figure (4-36): Effect of jacket thickness on the circular columns stress-strain.

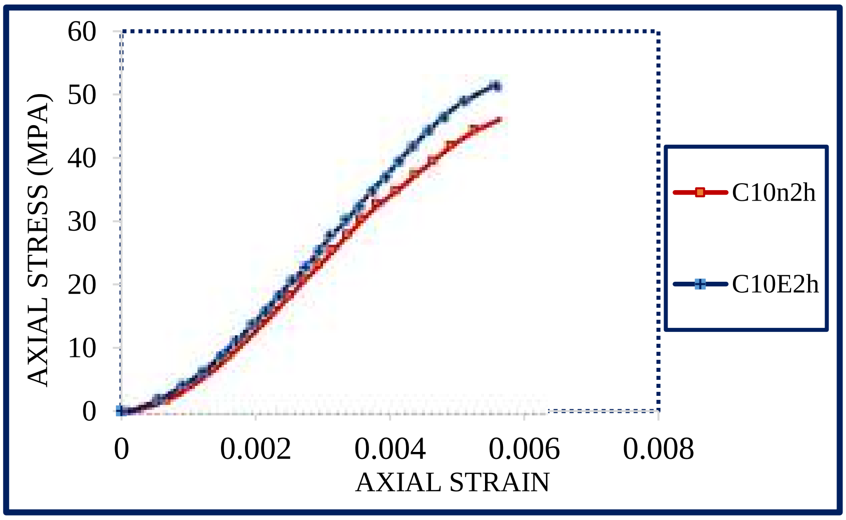


Figure (4-37): Effect of epoxy on the circular columns stress-strain.

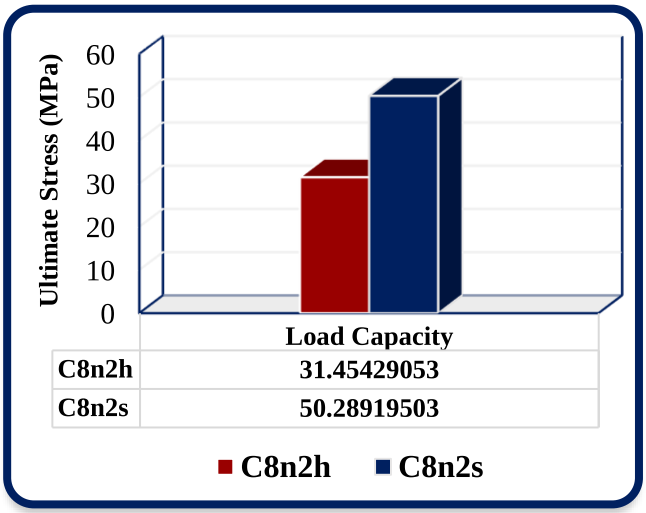


Figure (4-38): Effect of steel fibers types for circular columns with (45) mm thickness.

D. Failure Mode

Regarding the failure mode of circular columns, Figs. 12 shows the representative failure modes of series D columns.

Failure in this group came maybe different with square columns due to the different in the geometry. Increasing steel fiber ratio with and without epoxy caused more deformation in the concrete columns.

The failure occurred in the jackets when the process started from small cracks started in the top of the column and then grew and extended along the RC columns. Increasing thickness of jacket enhanced the crack propagation along the columns. Hoop jackets failed in brittle failure which the loads applied directly on the concrete core, so the stains developed directly in the hoop jackets in the mid-length. Dichotomy occurred in the hoop jackets in both columns (C9n2hH and C9n2hH).

(a) C8n2s. **(b)C9n1h** **(c) C9n2h.**

Figure (4-34): Crack pattern and failure mode of series D columns.

(d) C9E1h. (e)C9E1.5h. (f) C9E2h. (g) C9n2hH.

(h) C9n2sH. (i)C10n2h. (j) C10E2h.

Figure (4-39): Cont.

4.4.5 Series E

This series consist of five square columns strengthened with constant jacket thickness, with and without epoxy, hoop and composite jacket, and two types of steel fibers type. All test results are presented in Table (4-6).

Table (4-6): Test results of square concrete columns of series E.

ID	P_{cr} (kN)	P_{cc} (kN)	Δ_V (mm)	Δ_L (mm)	ε_{cc}	$\frac{P_{cc}}{P_{Ref.1}}$	$\frac{P_{cc}}{P_{Ref.2}}$	$\frac{\varepsilon_{cc}}{\varepsilon_{Ref.1}}$	$\frac{\varepsilon_{cc}}{\varepsilon_{Ref.X}}$	DI	Tn (kN)
S11n2hc	1703	2082.5	5.149	0.802	0.00687	2.062	1.806	0.906	0.646	1.53	3754
S11E2hc	750	1894.6	3.988	0.180	0.00532	1.876	1.643	0.701	0.501	1.60	2621
S11n2sc	2156	2336.7	4.891	0.490	0.00652	2.314	2.027	0.860	0.614	1.21	3927
S11E2sc	1552	2022.4	4.576	0.108	0.00610	2.003	1.754	0.805	0.575	1.32	3400
S11n2hH	1045	1168.9	3.684	0.002	0.00491	1.158	-	0.648	-	1.18	1605

A. Effect of Column Length

Increase the length from 500 mm to 750 mm for square columns affected the general behavior of the column. Use of composite jacket increased the load carrying capacity by (106%) for jacket with hooked fibers and (131%) for jacket with straight fibers in comparison with un strengthen reference column (S1N7) as demonstrated in Fig. (4-40). In comparison with the control column that have the same cross section area (S1N8), these two composite jackets enhanced the ultimate strength capacity by (80% and 102%) for the hooked and straight fibers respectively as demonstrated in Fig. (4-41). Effect of epoxy was cleared in the columns (S11E2hc and S11E2sc), which presented an enhancement in the strength capacity by (9%) in comparison with the strengthened column without epoxy (S11n2hc) as shown in Fig. (4-42 a). Meanwhile the column (S11E2sc), showed an enhancement in the strength capacity by (14%) in comparison with column with straight fibers without epoxy as demonstrated in Fig. (4-42 b). Hoop jacket case was carried out in the long column case which appeared improvement in the stress capacity was (15%) only in comparison with control column (S1N7) as revealed in Fig. (4-43). To check the effect of the length on the ultimate load carrying capacity, a comparison is carried out with the square columns with length of (500) mm have the same parameters. Use of composite jacket for (500) mm length column (S3n2h) enhanced the ultimate load capacity by (51%) while for 750 mm length column (S11n2hc) it enhanced the load carrying capacity by (106%). The full effect of this parameters will be explained in the next section of this chapter.

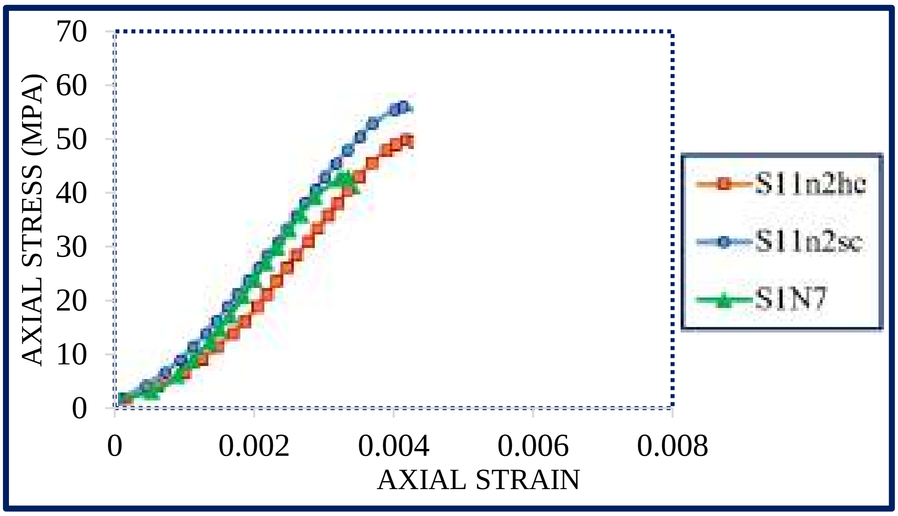


Figure (4-40): Stress-strain relationship clarifying the effect of SF type on series E columns.

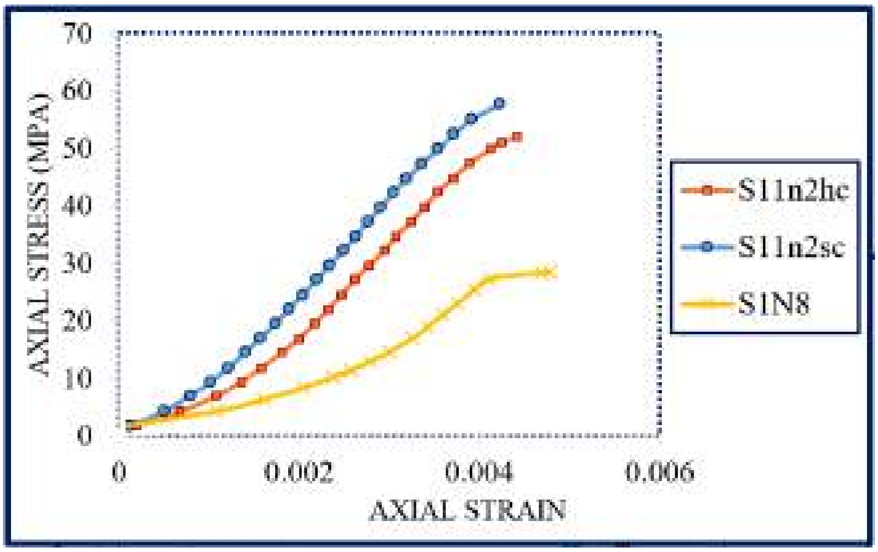
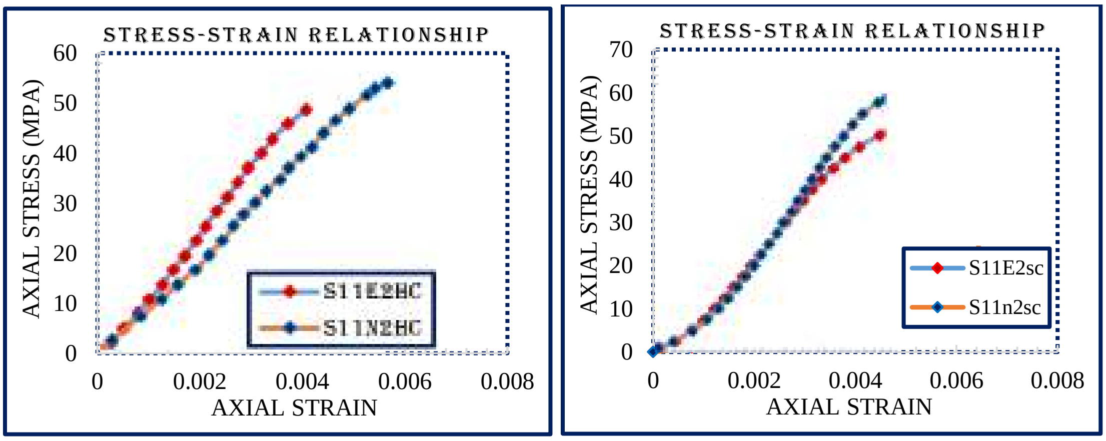


Figure (4-41): Stress-strain relationship clarifying the effect of SF type on series E columns.



(a) **(b)**

Figure (4-42): Stress-strain relationships clarifying the effect of epoxy on series E columns.

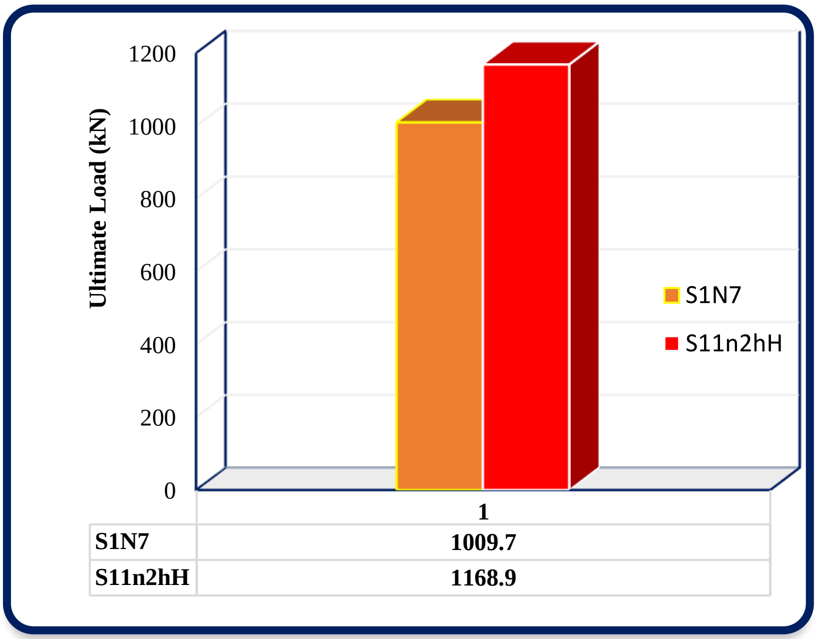


Figure (4-43): Stress diagram clarifying the effect of hoop jacket on series E columns.

B. Failure Mode

The failure mode of columns with higher length under effect the same parameters was different in comparison with the shorter ones. Steel fibers types affected the crack pattern of long columns which showed more deformation in the column with straight fibers than those in hooked ones. Existence of epoxy decrease the crack propagation in both straight and hooked fibers strengthening column. Hoop jackets case (S11n2hH) failed in brittle failure which the loads applied directly on the concrete core, so the stains developed directly in the hoop jackets in the mid-length. Dichotomy occurred in the hoop jackets in column (S11n2hH) as revealed in Fig. (4-44).

(a) S11n2hc. **(b) S11n2sc**

(c) S11E2hc. **(d) S11E2sc.**

Figure (4-44): Crack pattern and failure mode of series E columns.

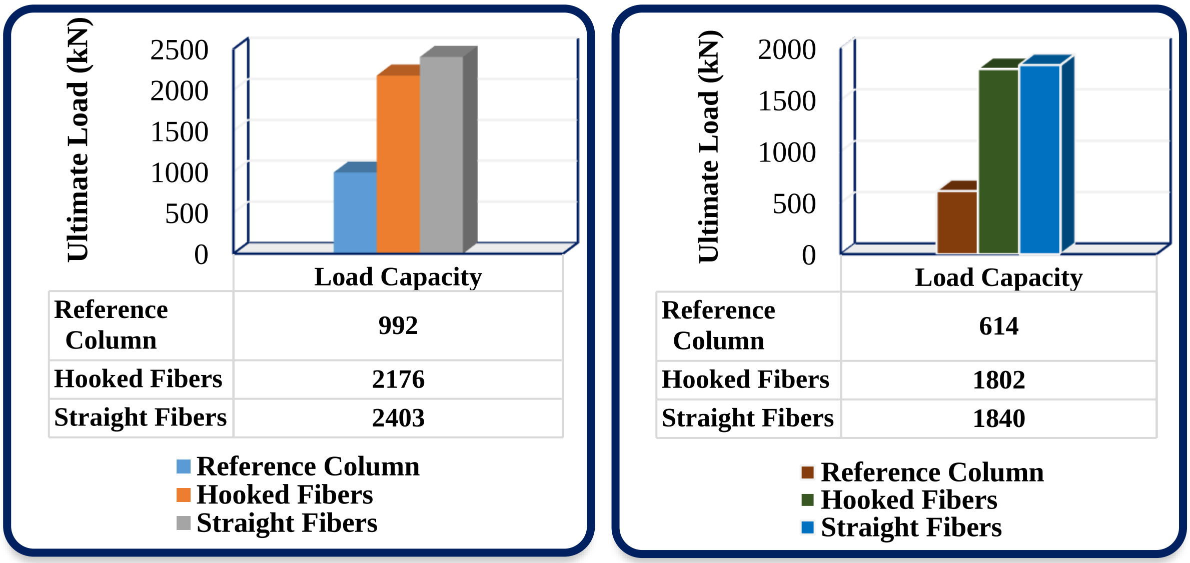
Figure (4-44): Cont.

4.4.6 Effect of Used Parameters on the Square and Circular Columns

A general comparison between the parametric columns for both square and circular cross-section. The comparison includes the effect of used parameters on the ultimate stress and load capacity, ductility, crack pattern, and failure mode.

A. Effect of steel fibers

The main comparison series in this section are series B and series D. The influence of steel fibers seemed clear on the ultimate capacity for both square and circular columns but with variable increment ratio. Increasing the ratio of fibers (0% to 2%) showed maximum increase in the ultimate load carrying capacity by (119%) for the hooked fibers and (142%) for the straight fibers. Concerning circular columns, maximum increase occurred for the same increment ratio of steel fibers by (194%) for hooked fibers and (200%) for straight ones as shown in Fig. (4-45). Ductility of the circular columns is enhanced significantly in both columns which demonstrated maximum value (1.5) for the square columns and (1.6) for circular ones as illustrated in Fig. (4-45). Crack propagation is increased with increasing the steel fibers ratio which showed more deformation in both square and circular columns and small cracks spread along the columns.



(a) Square Column. **(b) Circular Column**

Figure (4-45): Bar chart clarifying the comparison between the square and circular columns with hooked or straight fibers jacket.

B. Effect of Epoxy

Presence of epoxy enhanced the ultimate strength capacity by of (32%) for the column (S4E2c) which is exhibited maximum value of enhancing in square columns. It should be noted that all square column used epoxy gained enhancement in the ultimate strength capacity. Meanwhile, the epoxy existence in circular didn't affect the ultimate capacity in some specimens while in other decreased the ultimate strength capacity in comparison with model that without epoxy except the column (C9E1h) which showed enhancement by (36%). So, it can be said in this study, that the epoxy in the square columns is better than those in circular ones. Regarding the ductility, addition of epoxy enhanced the ductility of the column where the ductility index changed by (45%).

C. Effect of Jacket Thickness

Increasing the jacket thickness lead to an increase in the ultimate load carrying capacity for the both square and circular columns but with variable range. Circular strengthening jacket thicknesses (25, 35, and 45 mm) with straight fibers enhanced the ultimate average stress capacity by (264%) and maximum (334%).

While the square columns, the average enhancement was (165%) and maximum (202%). It should be noted that enhancement due to the increasing the jacket thicknesses in circular columns was higher than those in square columns.

D. Effect of Jacket Height

Use of hoop jackets were increased the ultimate strength capacity. The increment value in was circular columns are better than square ones because the mechanism of stress distribution over cross section. Maximum enhancement in the hoop jacket were (56%) and (%70) in square and circular columns respectively. During the loads, the circular shape suffered from the tension in all direction in equal stresses approximately. The uniform distribution in the stresses let the column carry more loads otherwise the square column which suffers from stresses concentrated in some points. Failure mode and crack pattern was different in mode.

4.5 Analysis the Results According to the Fibers Properties

The reason for using fibers in both categories of composite is to enhance the properties of an inherently weak, brittle and crack-prone cementitious matrix. Depending on fiber type and fiber content, this enhancement may include in varying degrees improvements in tensile or flexural strength, ductility, toughness or energy absorption capability, impact resistance, fatigue resistance, resistance to cracking, permeability, and durability. However, amid the myriad of benefits claimed in available literature on the subject, particularly some of the promotional literature produced by fiber manufacturers, the user should recognize that the amount of fibers present is a major factor influencing the extent and degree of property enhancement. Fibers affect composite properties in both the freshly mixed and hardened states, often in opposite senses. For example, increasing the fiber content naturally tends to improve the degree of enhancement of many properties in the hardened state, but also decreases mixture fluidity in the freshly mixed state, until at some maximum fiber content the manufacturing process is no longer capable of producing uniform fiber distribution in a mixture that can be properly consolidated. This means that the potential enhancement of properties in the hardened state cannot be fully achieved either because of nonuniform fiber distribution or incomplete consolidation, or both.

In this research, two kinds of fibers were used with different aspect ratio, ratio of fibers, mechanical properties, and fibers orientation, which affect the behavior of the tested columns. Fig. (4-46) present the distribution of both types of steel fibers in the strengthening jacket. Hooked end fibers were distributed in parallel to the tension direction and perpendicular to the hoop pressure in order to provide a good confinement to the column and increase the cohesion force between the jacket particles. Best activeness for this case was in the hoop case of strengthening because the strengthening jacket subjected to pressure hoop only from the concrete core which need to material inside the concrete to increase the confinement capacity. Making the distribution in the direction of the tension offers the greatest possible reinforcing effectiveness in terms of maximizing improvements in strength and toughness at each fiber content. Effect of hooked fibers with two direction orientation showed large cracks parallel to the direction of the applied load in both hoop and composite cases.

Straight fibers which have the smaller confining capacity was active in the composite case which need less confinement than hoop. Distribution of the small straight fibers was randomly with orientation in all directions as appeared in Fig. (4-46). These fibers affected the ultimate stress capacity more than hooked fibers due to the high strength of the straight fibers which equal to 2850 MPa (larger than hooked ones by two times approximately). Regarding the crack pattern, straight fibers let the stresses distributed in wide range due to the high density of these fibers. Small and medium cracks distributed along the member causes higher deformation capacity. The breaking stresses are significantly higher for the composites with randomly oriented shorter fibers, reflecting the increased reinforcing effectiveness attributable to shorter fiber length with consequently high tensile strength, and probably to a lesser extent three dimensions random fiber orientation. Important similarities are that as the fiber volume fraction increases there is an improvement in the cracking stress at what is termed the first crack, identified by the sharp decrease in the slope of the stress-strain curve associated with the loss of composite stiffness caused by the onset of cracking in the matrix.

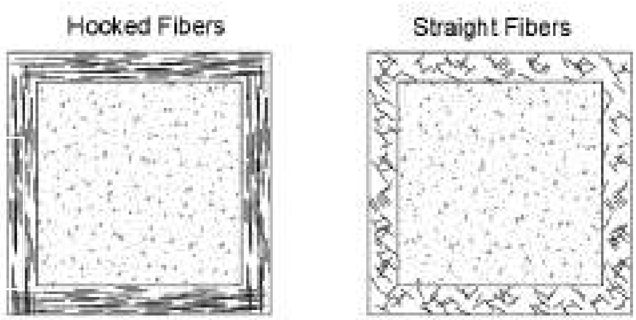


Figure (4-46): Distribution and orientation of the steel fibers.

4.6 Energy absorption

Energy absorption or toughness (Tn) is an indication of the ability of the concrete member to absorb and disperse the energy resulting from the loading and is affected by several factors, including the nature of the materials used and their properties. Energy absorption can be calculated by the area under load-deflection curves which showed a significant effect of the strengthening jacket thickness, steel fibers ratio, etc [112].

Regarding the square columns, increase the steel fibers ratio enhanced the energy absorption of the column average enhancement (8%) for increase the ratio of hooked fibers (1%, 1.5%, and 2%) respectively in comparison with column with zero ratio of steel fibers (S3n0h) as shown in Table (4-3) to (4-7). Effect of epoxy was positive when it is increasing the toughness average enhancement (21.3%) in comparison with SFRC jacketed columns which is without epoxy. Increase of straight steel fibers was more effect on the toughness than hooked ones which enhanced the toughness by (78%) in comparison with column with zero ratio of steel fibers (S3n0h). Increase thickness of SFRC jackets from 25 mm to 45 mm with hooked fibers has small effect which demonstrated an average enhancement of (5%) only. It should be noted that straight fibers are better than hooked ones in toughness enhancement which appeared improvement in the toughness of hoop case strengthening by (36.7%) for column (S6n2sH) in comparison with reference column (S1N1) and better than the same case of strengthening but with hooked fibers by (49.5%). Partial strengthening with two and three sides of jacket strengthened with straight fibers improved the toughness by (26% and 78.8%) respectively. Maximum enhancement occurred in mix of hooked and straight fibers in specimens (S6E2m), (4093 kN.m) which is greater than that of column (S6E2s), (117%) and column (S3E2h) (61.4%).

Concerning the circular columns, increase hooked fibers ratio gave an average improvement of (50.8%) in comparison with the control column (C2N1). While using of epoxy with the same ratios of hooked fibers showed an average enhancement of (48.7%). Using of straight fibers instead of hooked fibers gave enhancement (38%) greater than hooked fibers. Thickness increment from 25 mm to 45 mm increased the toughness by average value (56.7%) which higher than enhancement due to the change the jacket thickness of square columns. Hoop case of strengthening gave enhancement of 106% when compared with the control column (C2N1). Average improvement of (121%) occurred due to the strengthening of square columns of length (75 cm).

4.7 General Behavior and Comparison with Previous Studies

Confinement of concrete columns is very critical because columns are likely to face significant overloading due to seismic and other actions. It is important to reinforce and confine concrete columns in such a way that columns can carry sustained load after the spalling of concrete cover and do not fail suddenly without adequate warning. Despite the availability of a large number of experimental investigations, only a limited number of extensive studies have been developed which include a large dataset.

In general, presence of SFRC jacket enhanced the ultimate stress, load carrying capacity, axial and lateral strain, ductility, and energy absorption. FRP tube/sheet confinement concrete makes a restrain for crack and delays the crack development but not prevents internal cracks. The maximum confined compression strength depends not only on material properties of the concrete columns (*fcu*, *E*, *ft*, *etc*), but also the strengthening jacket properties.

In this thesis, beside the general assumption of fracture mechanics and presented failure mechanism of confined concrete column, effect of SFRC jacket can be described by many additional assumption of confined concrete as the follows: when the radial strain in the SFRC jacket is greater than the maximum allowable tensile strain of concrete, internal cracks propagate in the concrete core. The confinement concrete columns delay formation a new crack, and not prevent it. The confinement of concrete column restrains development cracks. The SFRC jacket restrains deformation with a force called resistance force of jacket. In addition, the tensile stress in the uncracked part is helping in the restraining with a force called resistance force of concrete. The horizontal applying tensile stress due to vertical stress causes tensile deformation of SFRC jacket.

The addition of confinement around reinforced concrete cross sections changes the concrete's stress-strain behavior from relatively weak and brittle to strong and ductile due to the existence of steel fibers which increase the bond between the concrete particles and increased the ductility. The added strength and ductility found in confined concrete is due to the development of a triaxial stress field and the containment of the concrete after cracking. Steel fibers have a great effect of the general behavior and crack pattern of the confined columns, the bond between concrete particles still work and active until the failure of the column. As revealed in Fig. (4-47 & 48), steel fibers distribution and orientation have a significant effect on the failure mode. Open a confinement of the concrete showed many notes, most important of them was the deformation. Most of deformation is distributed in the concrete jacket with inclined cracks extended along the concrete core.

Figure (4-47): crack pattern and failure mode of typical confined column.

Figure (4-48): crack pattern and failure mode of typical confined column.

In comparison with the recent studies, this thesis approved the obtained results with a good agreement with recent studies which showed similar concepts. In case of stress strain curve of confined and unconfined columns, the obtained results were similar to that obtained by Mander and et al. [6] which used an energy balance approach along with effectiveness coefficients to unify the confinement model for both circular and rectangular transverse reinforcement at various spacing. Mander's model was based on the equation suggested by Popovics in 1973 [66] and has gained wide acceptance in the Civil Engineering. Comparison of this thesis results was converged to the results of Xie et al. [22] which included the enhancement in the stresses, strain, ductility. Also these models showed the same crack pattern and failure mode with this thesis results in case of hoop strengthening as revealed in Fig. (4-49 & 50).

Figure (4-49): crack pattern and failure mode of typical confined column.

Figure (4-50): crack pattern and failure mode of typical confined column.

4.8 Stress versus Hoop Strain

The stress versus hoop strain of the stub concrete columns or hoop strain of the SFRC jacket is presented in Fig. (4-51). It can be seen that the hoop strain in the SFRC increased linearly with increasing of axial loading in the initial stage. After the reaction forces exceed 50% ultimate resistance, the stress versus hoop strain curves start to behave nonlinearly. This hoop strain equals to the elastic limit strain of the SFRC obtained from the direct tensile tests. This implies the

SFRC jacket behave nonlinearly. It is because the confining SFRC jackets are subject to circumferential tension during the axial loading process. The axial stress-hoop strain curve of stub concrete columns shows lower strain with that of the SFRC jacket. And there is also an approximate linear relationship existing in all curves after that a turning point can be observed. This is because the expansion of inner stub columns activates the hoop strain of SFRC jacket. It can be observed that the hoop strain of stub concrete columns is lower than that of the SFRC jacket and the difference is more obvious.

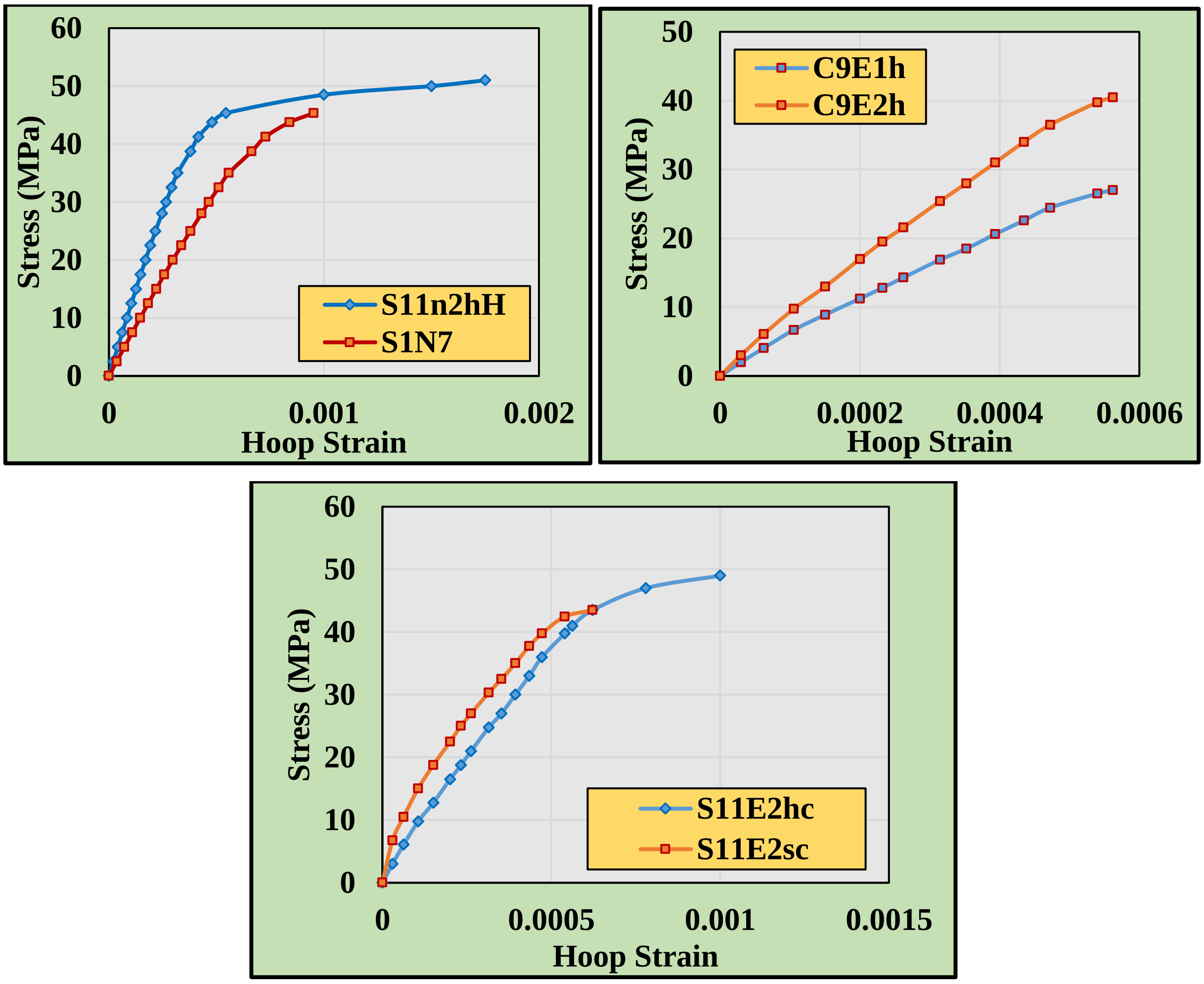


Figure (4-51): Stress-hoop strain relationship.

4.9 Finite Element Analysis

In this chapter, a comparison between the experimental and theoretical studies is presented to implement a new parameters that afford more costs and need to more equipment. FEA by ANSYS 15.0.7 containing study of: elements type, material properties, real constant and convergence study.

The analyzed column specimens were 16 specimens for the verification purpose and 10 columns with new parameters.

The parameters included use of full scale of RC confined square column strengthened by SFRC jacket with variable thicknesses (50 mm, 60 mm, and 70 mm) with hoop and composite cases. In addition, to use (45 mm) hoop jacket and use of other techniques for strengthening of short columns (150 x 150) mm. From the finite element analysis of each of these tested columns, the predicted ultimate stress carrying capacity versus axial strain response were obtained and compared with the corresponding experimental results. In addition to present the stress intensity, principal stresses, stress and strain distribution.

4.9.1 Verification Analysis

To ensure that the procedure used in the analysis process is by ANSYS software, 16 column specimens modeled and tested. The theoretical results showed a good matching between the experimental and numerical outcomes as shown in Table (4-7) and Fig. (4-52).

Table (4-7): Verification results include the failure load and displacement.

ID	P_{EXP} (kN)	P_{Ansys} (kN)	**P_{ANSYS}/ $P_{EXP.}$**
S1N1	992	991	99.9%
S1N2	1142.6	1180	96.8%
S3n1h	1698.8	1636	96.3%
S3n1.5h	1971.6	1813	92.0%
S3n2h	1501	1468	97.8%
S4n2c	2264.3	2076	91.7%
S4n2H	1146.4	998	87.1%
S5E2h	3002.2	2933	97.7%
S6E1s	2403	2006	83.5%
S6E1.5s	2203	1863	84.6%
S6E2s	2206.8	2322	95%
S6n2hH	1277.4	1184	92.7%
S7E2hp2	1287.5	1345	95.7%
S7E2hp3	1866	1905	98.0%
S11n2hc	2082.5	2271	91.7%
S11n2sc	2336.7	2279	97.5%

4.9.2 Load-Deformation Response

Regarding the tested columns, hydraulic jack subjecting a force over the cross-section of the column. For ANSYS-15, the same thing was used, which the forces were applied on the cross-section of the column and the displacement measured from the same location that used experimentally.

Figure (4-52) shows the load-deformation response obtained by finite element analyses and compared with the experimental results which showed a good agreement between the two outcomes. Generally, ANSYS APDL has an ideal condition that isn't available absolutely in the experimental program in addition to the properties of the used element in FEA.

The nonlinear behavior of this concrete is simulated, along with its reduced capability to resist large displacements in compression. In general, modeling the concrete in ANSYS considered a complex issue due to the response of the concrete elements in the FE software's. Reinforced concrete is a difficult material to model with the finite element method. Wide-ranging studies have simulated the behavior of reinforced concrete with finite element software but very few have simulated the plastic region in which concrete cracks reduce its strength. The simulation with a damage-based model of concrete (Solid65) did not allow large displacements and resulted in an unstable model as presented by Montava et al. [25].

In the linear range, the numerical load-deformation response is stiffer than the experimental curve. The failure load from the F.E model is higher than the ultimate load from the experimental one sometimes vice versa. The maximum displacement of the numerical model is less than the experimental beam by (2-18) % for the most columns. Ultimate loads in the numerical columns had difference in the agreement by (0.1-16.5) %. In general, these results considered acceptable and good which also showed the logical stresses distribution as revealed in Fig. (4-53). Stress intensity appeared in the corner of the square columns of unconfined but for confined columns stress concentrated at the jacket with different ratios as shown in Fig. (4-53).

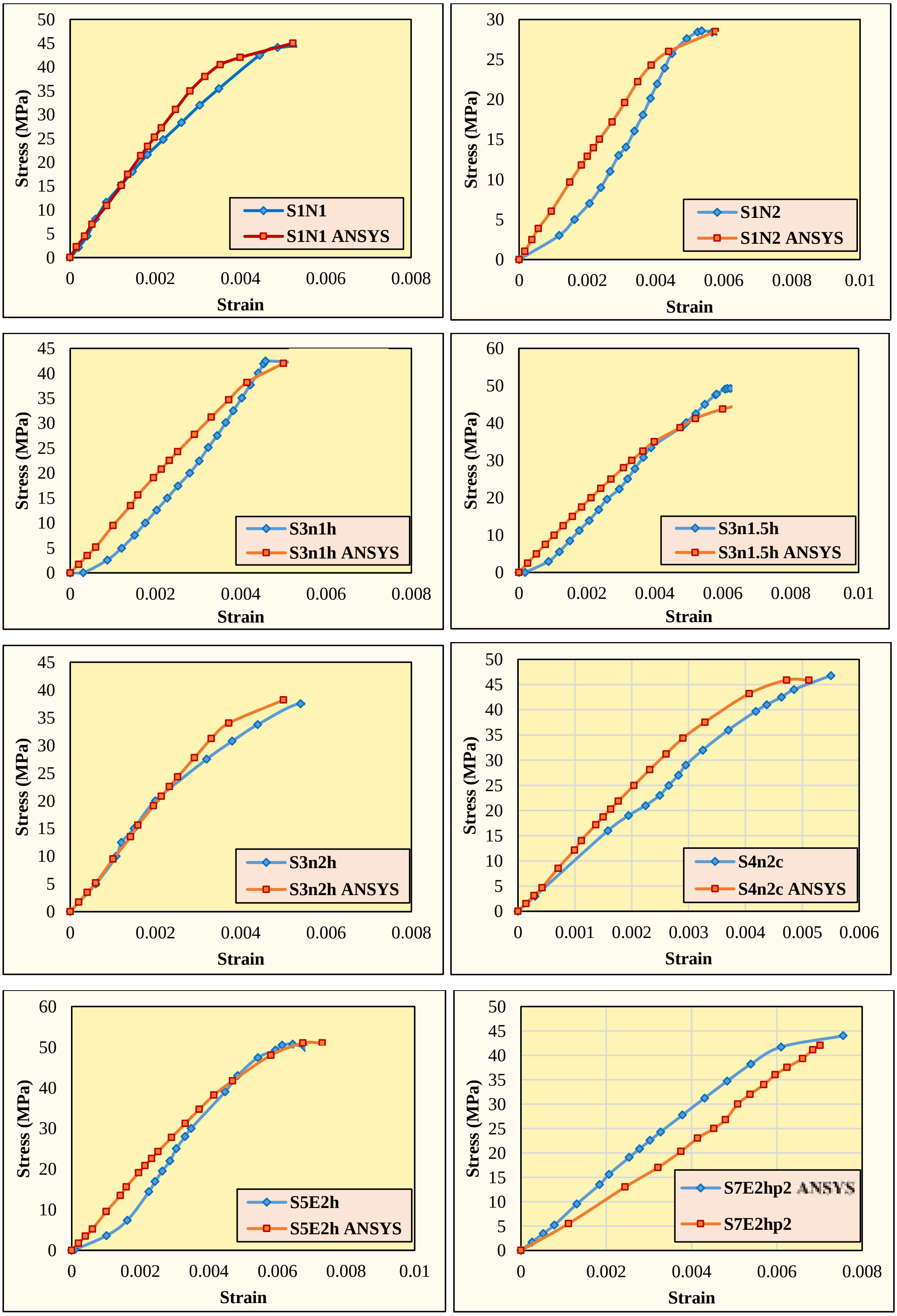
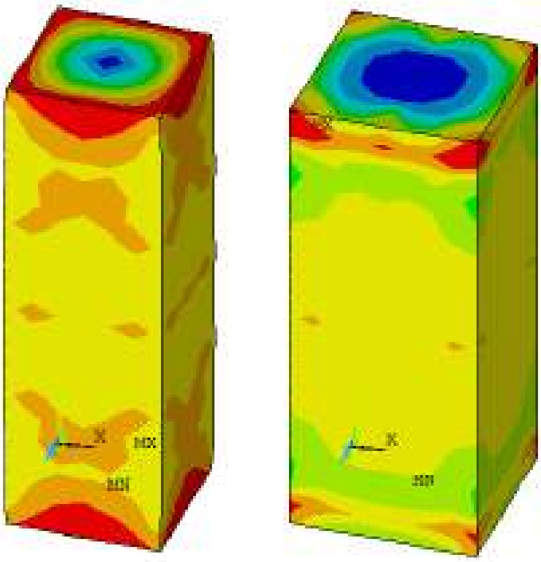
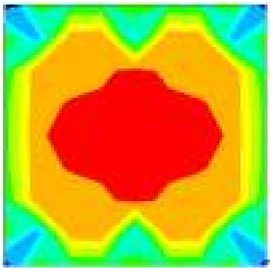


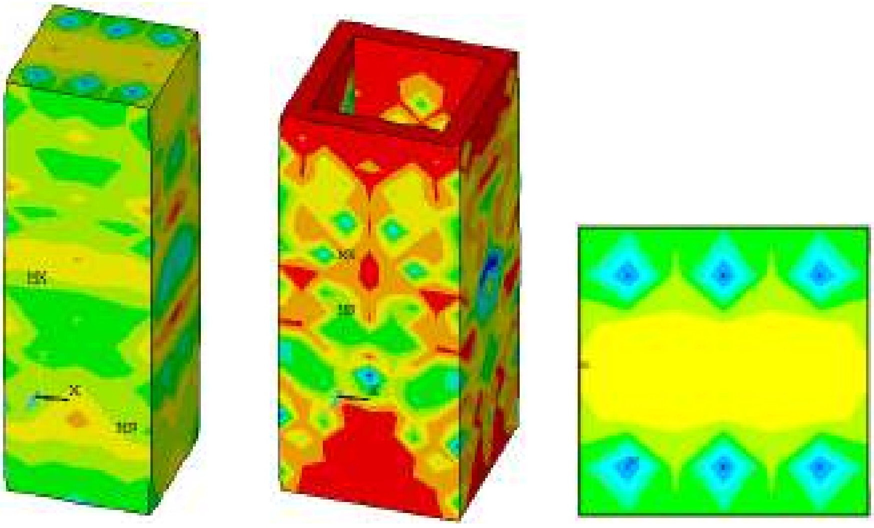
Figure (4-52): Stress-strain comparison between the numerical and experimental columns.



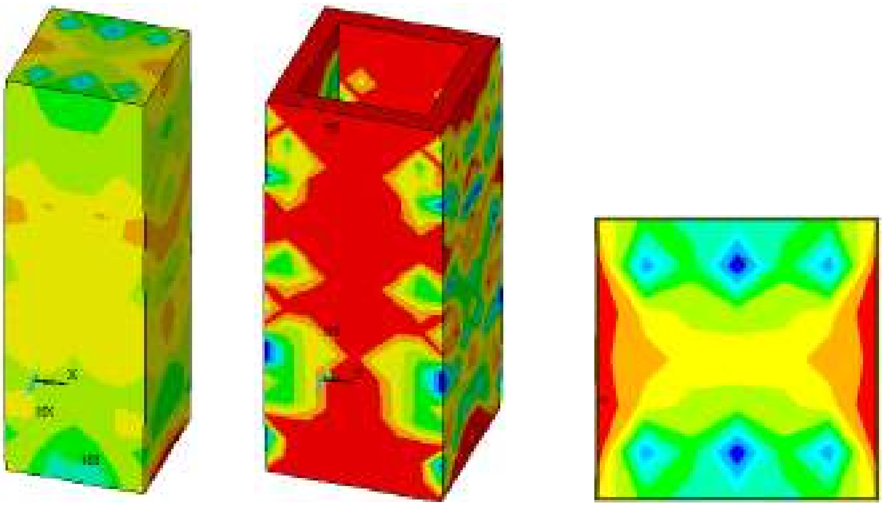
Color	Concentration degree
Red	**Higher concentration**
Orange	**High concentration**
Yellow	**Medium concentration**
Green	**Light concentration**
Blue	**Lighter concentration**



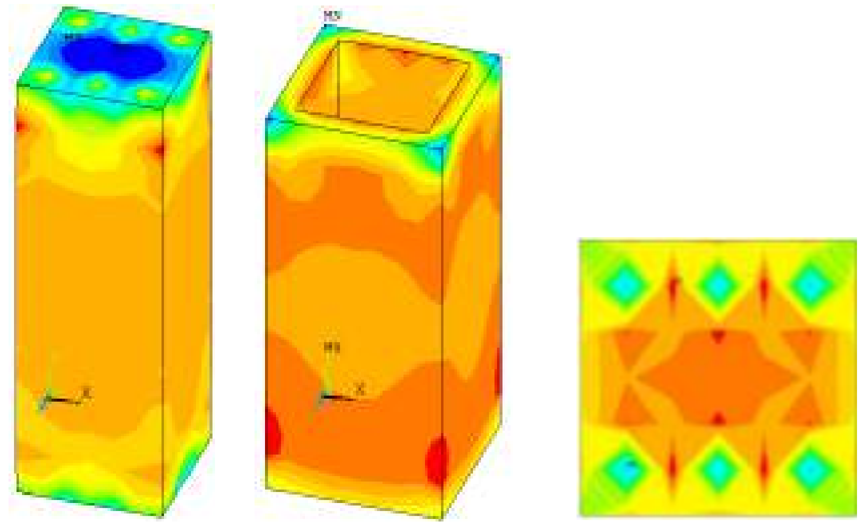
(a) S1N1. **(b) S1N2**



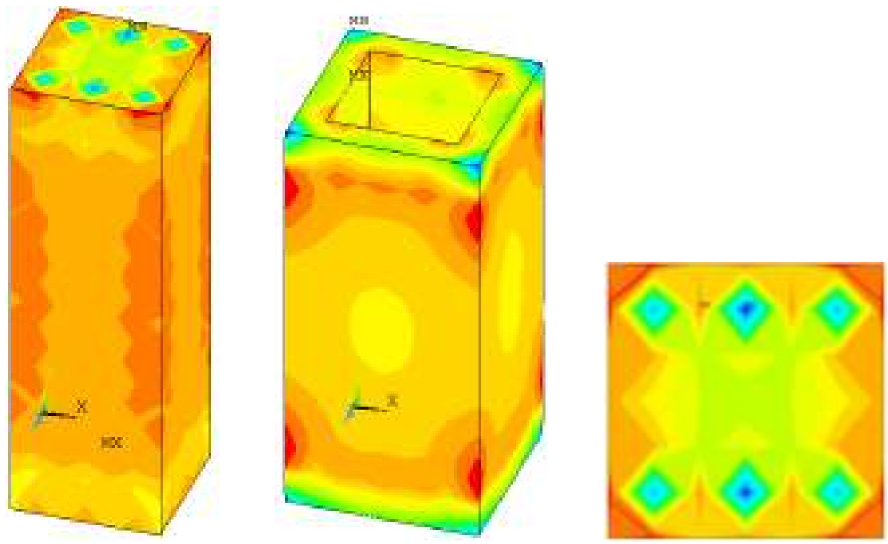
(c) S3n1h model.



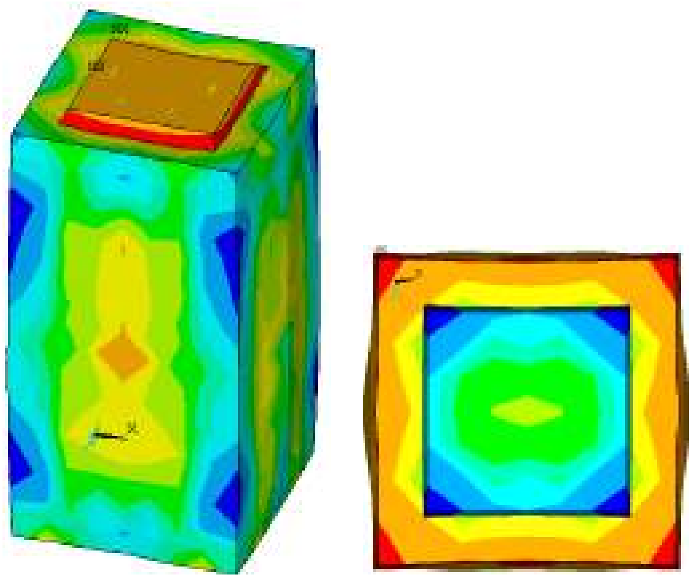
(d) S3n1.5h model.



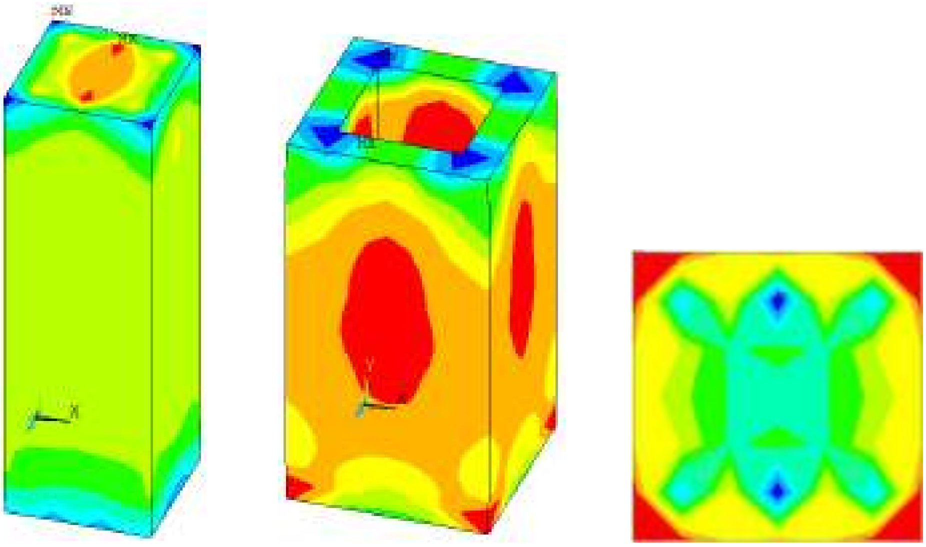
(e) S3n2h model.



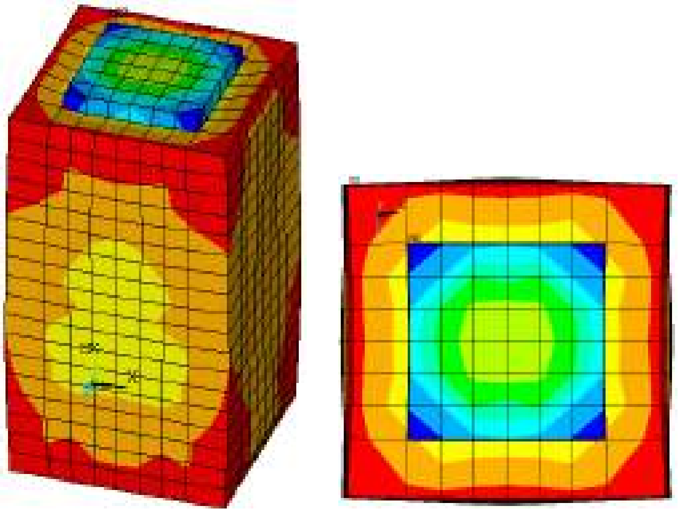
(f) S4n2c model.



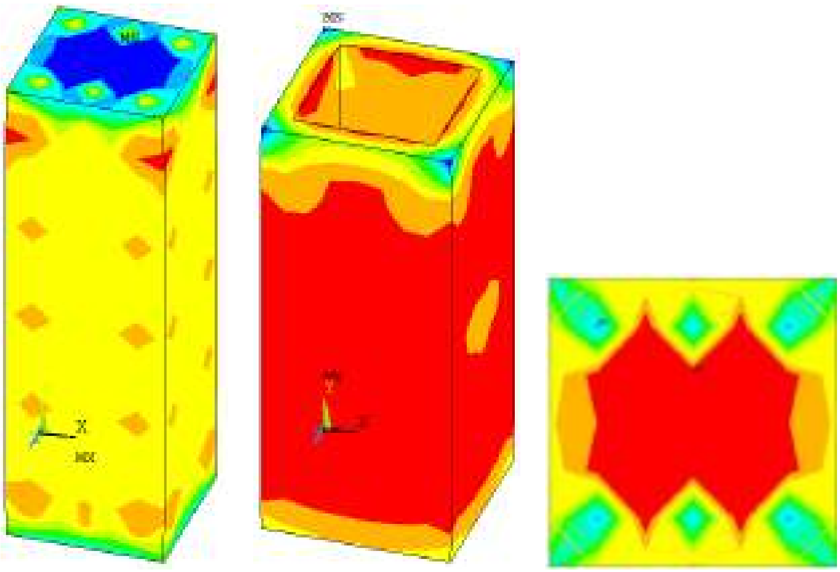
(g) S4n2hH model.



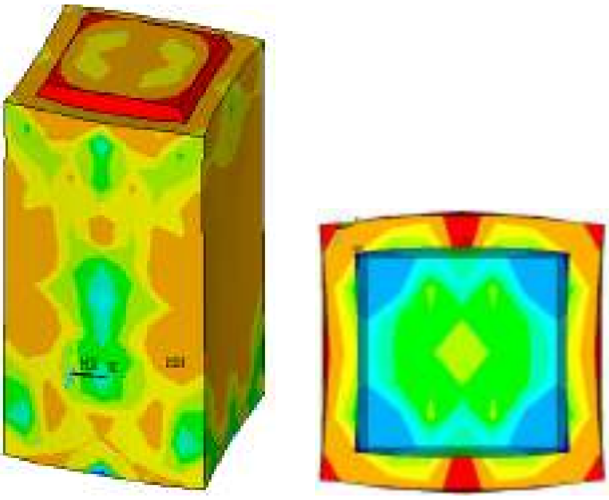
(h) S5n2h model.



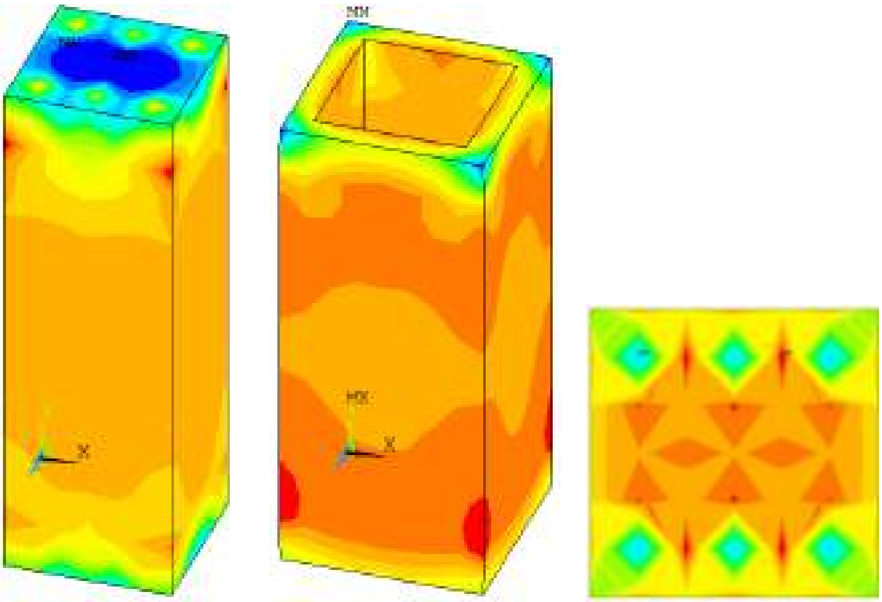
(i) S5E2H model.



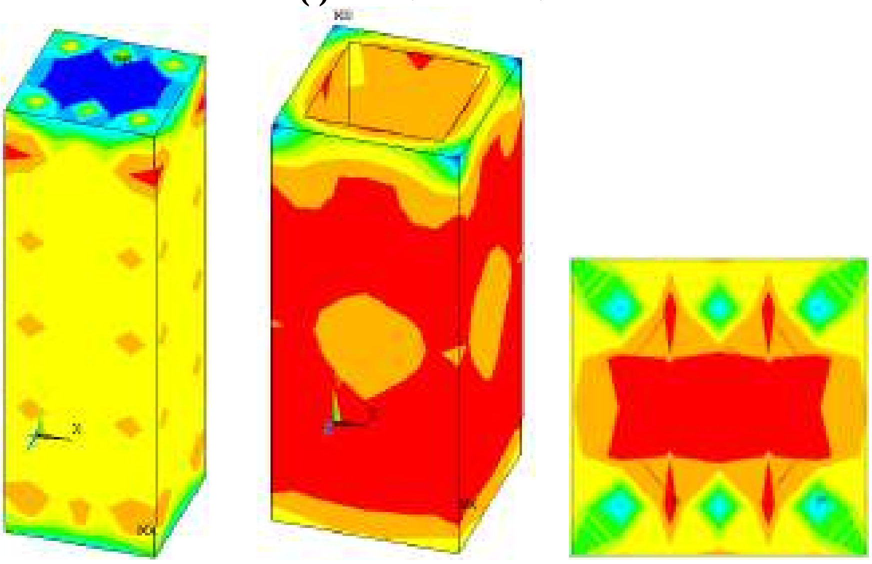
(j) S6E1s model.



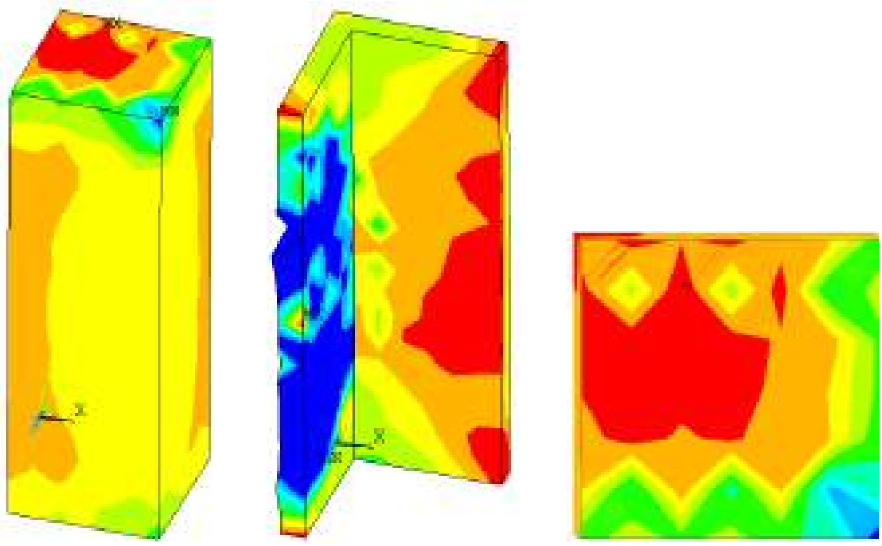
(k) S6n2hH model.



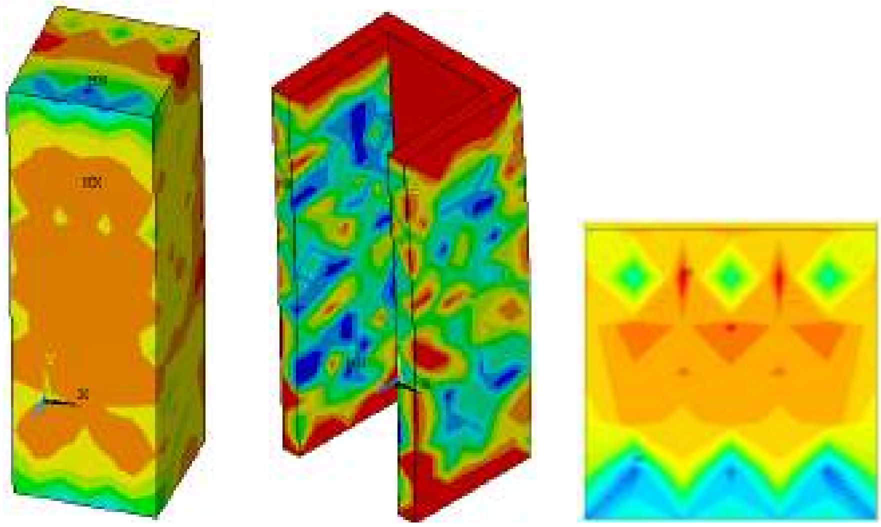
(l) S6E1.5s model.



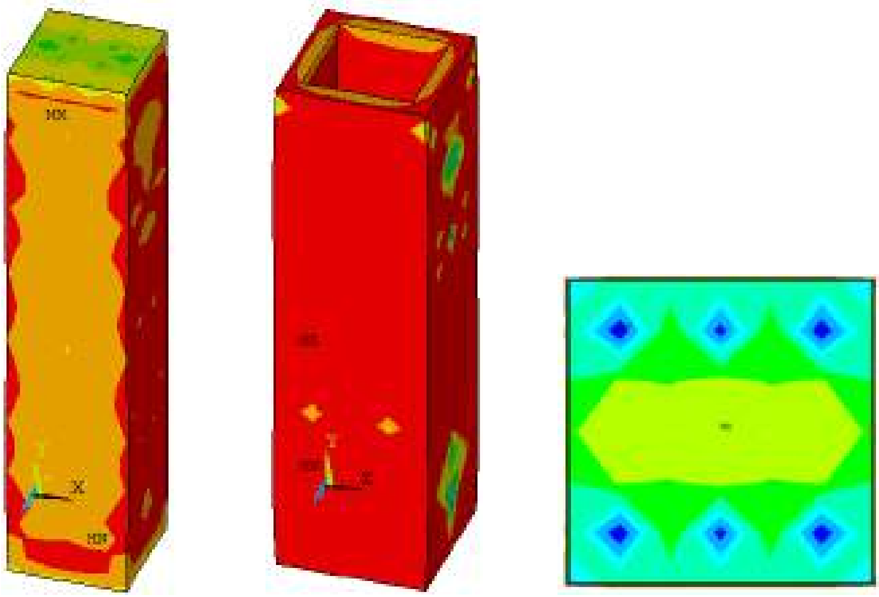
(m) S6E2s model.



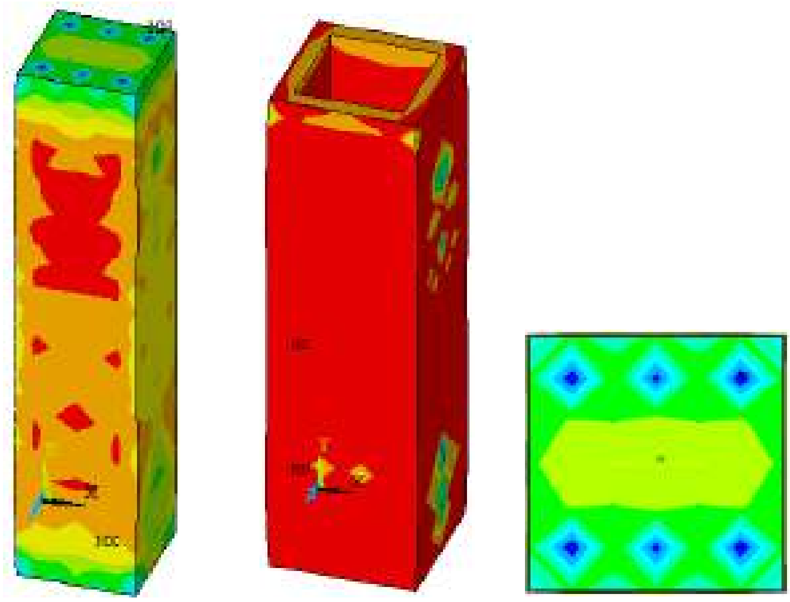
(n) S7E2hp2 model.



(o) S7E2hp3 model.



(r) S11n2hc model.



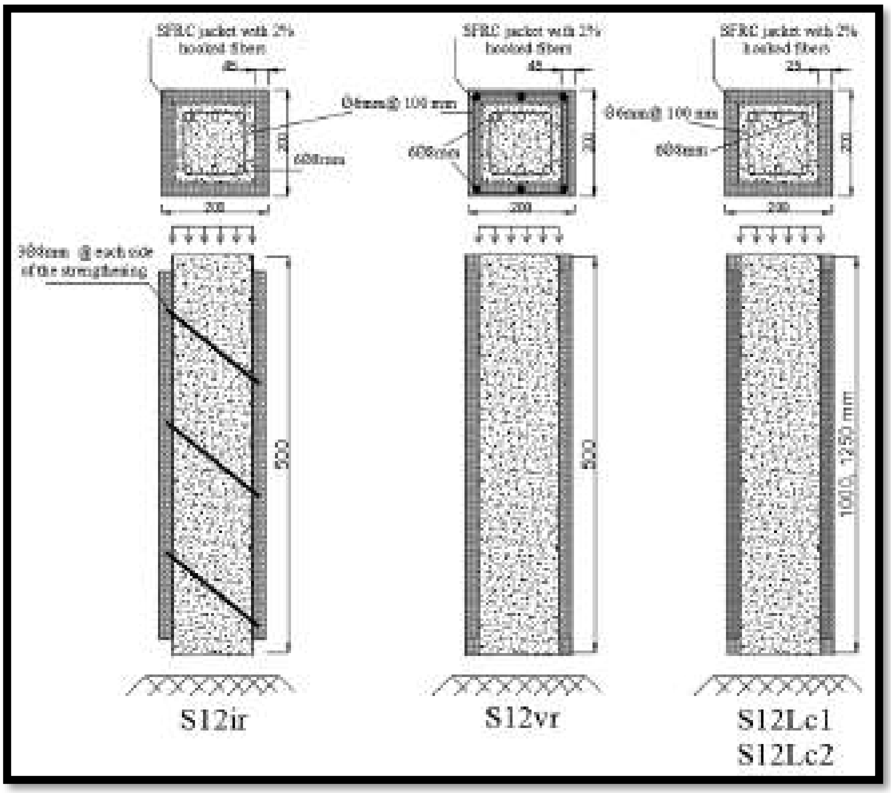
(p) S11n2sc model.

Figure (4-53): Stress distribution of the strengthened columns along the concrete core, concrete jacket, and over cross-section area.

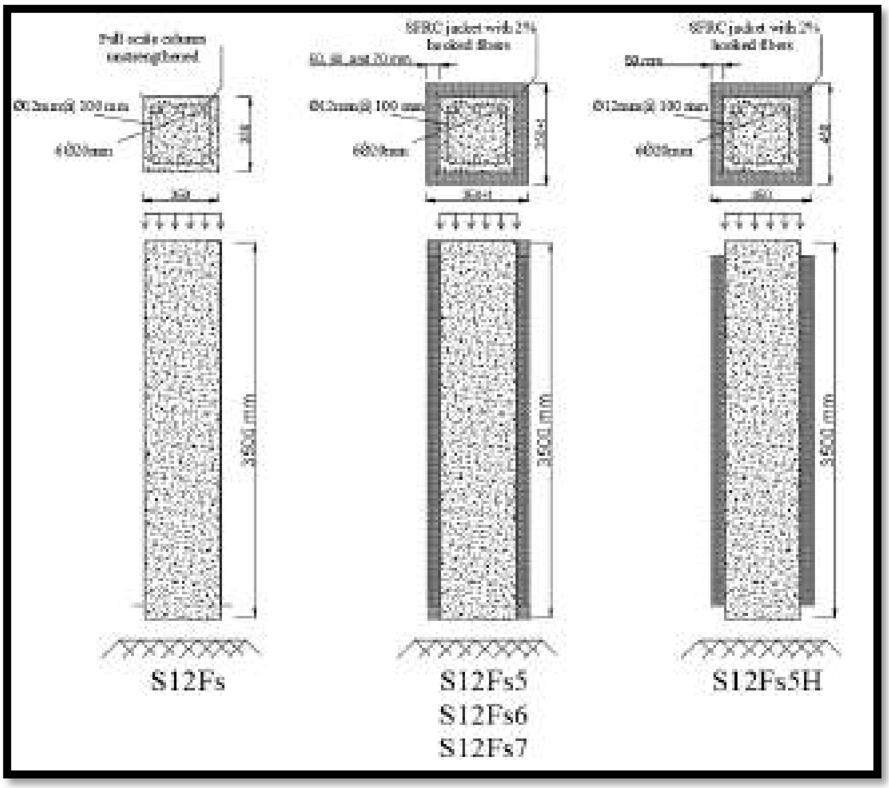
4.10 Parametric Study

A parametric study included new parameters was done to investigate the effect of new strengthening methods. The theoretical program involved of two groups; the first group is confining of five small scale of columns with dimensions of (150x150x500) strengthened with several techniques such as reinforced jacket by vertical and inclined steel rebar in addition to the length parameter (increase the length to 1 m and 1.25 m) as shown in Fig. (4-54 a).

The second group included analyzing of full-scale column (350 x 350 x 3500) mm by confined by composite jacket with varied thickness (5, 6, and 7 cm) and use of hoop confinement with (5 cm) thickness as shown in Table (4-8) and Fig. (4-54 b). All of columns have the same properties used in experimental work (compressive strength, steel rebar properties. etc.)



(a) Group one columns.



(b) Group two columns.

Figure (4-54): Geometric details of considred specimen in the numerical parametric study.

Table (4-8): properties of considered specimens in the numerical parametric study.

Numerical Series						
ID	**Dimension (mm)**	**Jacket thickness (mm)**	**SF ratio %**	**Epoxy**	**SF type**	**Jacketing**
S12n2h	150x150x500	45	2%	without	hooked	Hoop
S12ir	150x150x500	45	2%	without	hooked	Hoop
S12vr	150x150x500	45	2%	without	hooked	Hoop
S12Lc1	200x200x1000	25	2%	without	hooked	composite
S12Lc2	200x200x1250	25	2%	without	hooked	composite
S12Fs	350x350x3500	-	-	-	-	-
S12Fs5	450x450x3500	50	2%	without	hooked	composite
S12Fs6	470x470x3500	60	2%	without	hooked	composite
S12Fs7	490x490x3500	70	2%	without	hooked	composite
S12Fs5H	350x350x3500	50	2%	without	hooked	Hoop

4.10.1 Test Results

Table (4-9) and Fig. (4-55) presents the test results of the analysis. Concerning the columns with dimensions (150x150x500) mm, column with (45) mm thickness with steel fibers of (2%) which not implemented in the experimental work.

Column (S12n2h) showed ultimate strength by (44.4) MPa and strain by (0.0051). hoop SFRC jacketed column with inclined reinforcement (S12ir) enhanced the ultimate strength by (35%) and ultimate strain by (68%) when compared with the column (S12n2h).

Column with reinforced jacket by both steel fibers and steel reinforcement (S12vr) showed enhancement in the ultimate axial stress less than column (S12ir) by (13.5%) and (62.4%) in the ultimate axial strain.

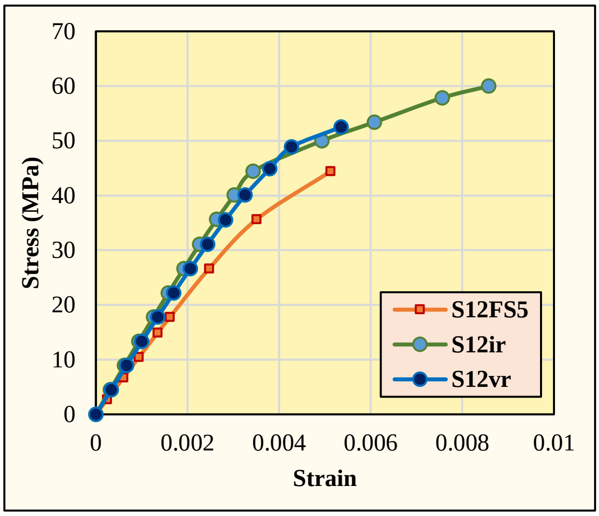
The effect of the column length was significant on the ultimate stress, increase the length of the strengthened column from (1000 mm) to (1250 mm) decrease the ultimate strength from (48.65) MPa to (40.4) MPa which approximately by (17%).

Regarding the full-scale columns, confining the column by (5) cm RC jacket improved the ultimate strength capacity of (67.6%) when compared with the unconfined column (S12FS).

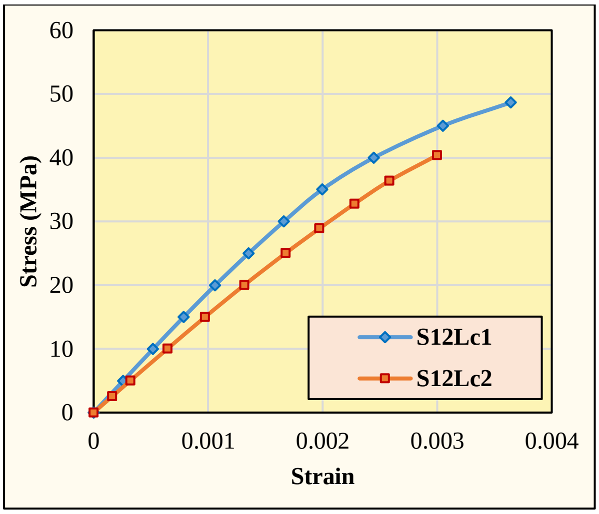
Strengthened column with composite SFRC jackets (5, 6, and 7) cm thickness showed an enhancement in the load carrying capacity by (177%, 210%, and 245%) respectively with slight increase occurred in the ultimate strain. Compare the hoop jacket with the composite one with the same jacket thickness showed that the hoop jacket carried strength more than composite one by (67%) approximately with less strain by more than (25%).

Table (4-9): Test results of reference concrete columns of group one.

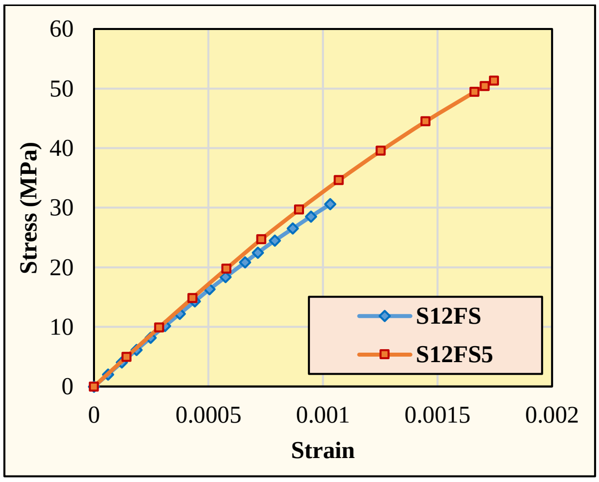
ID	P_{cr} (kN)	P_{cc} (kN)	Δv (mm)	ε_{cc}	$P_{cc}/P_{Ref.1}$	$\varepsilon_{cc}/\varepsilon_{Ref.1.}$
S12n2h	674.4	999.5	2.563	0.005127	-	-
S12ir	854.1	1349.5	4.287	0.008575	1.35	1.67
S12vr	815.235	1181.5	2.677	0.005355	1.18	1.04
S12Lc1	1401	1946	3.641	0.00364	-	-
S12Lc2	1131.13	1615.9	3.751	0.003001	-	-
S12Fs	2061.8	3748.8	3.613	0.001032	-	-
S12Fs5	7120	10390	6.117	0.001748	2.77	1.69
S12Fs6	7220	11653	6.24	0.001783	3.11	1.72
S12Fs7	7326	12938	6.257	0.001788	3.45	1.73
S12Fs5H	4009	7184	4.576	0.001307	1.91	1.27



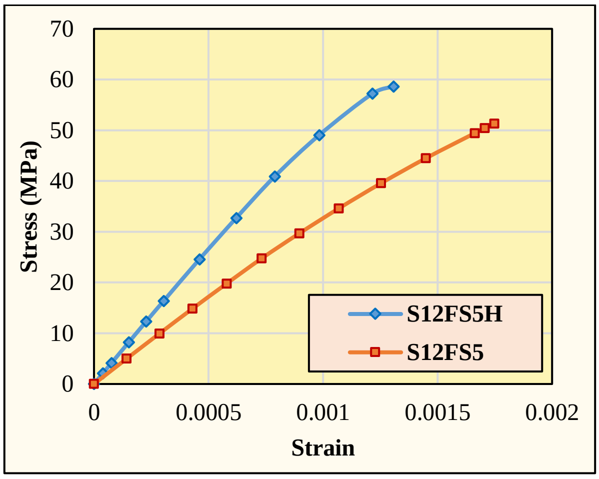
(a) short column comparison.



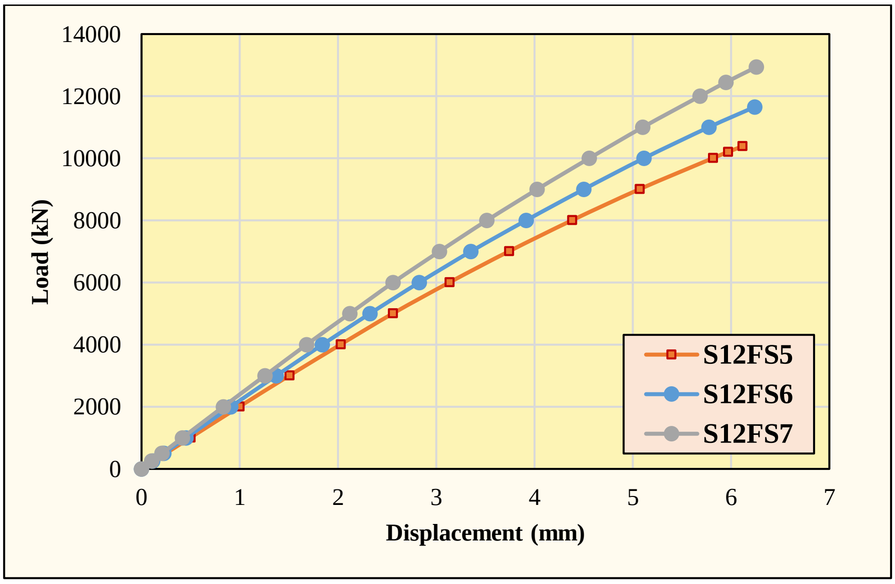
(b) length parameter comparison



(c) Effect of confinement type.



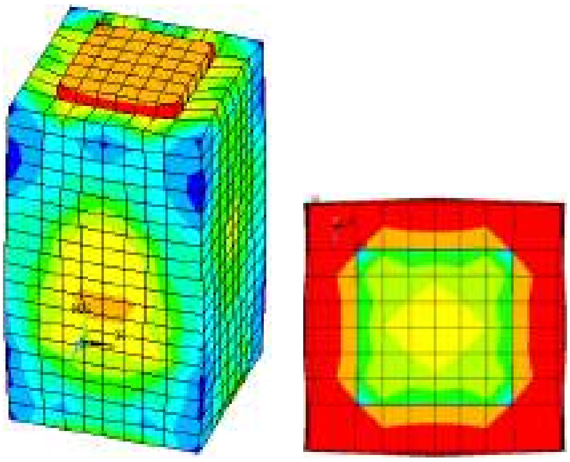
(d) Hoop and composite comparison.



(e) Effect of jacket thickness.

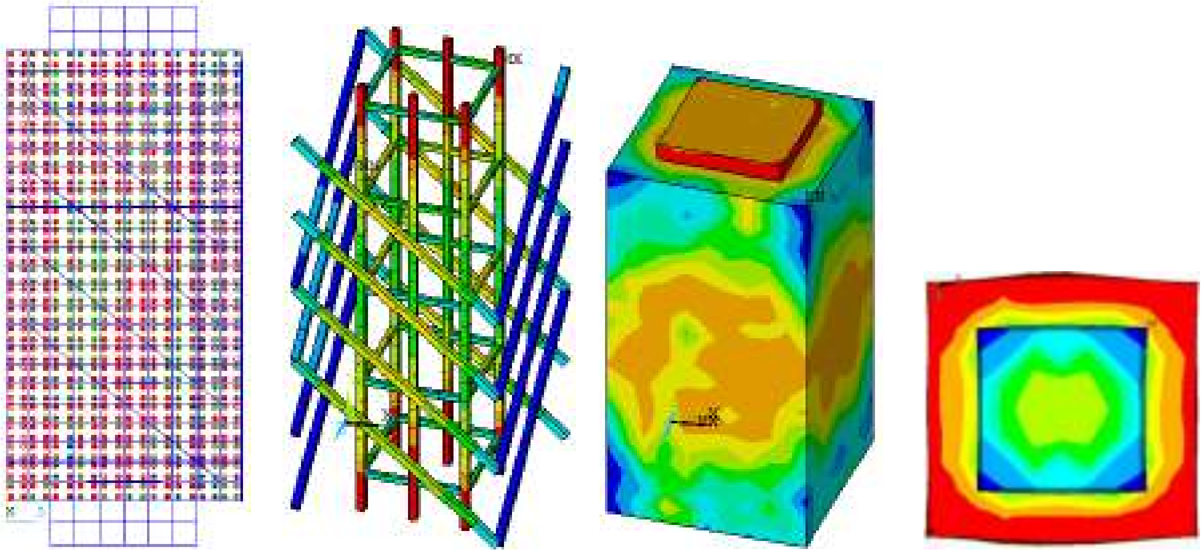
Figure (4-55): Analysis Results of the numerical analysis models.

The crack pattern, damage, and stress distribution along the member and over cross-section are presented in Fig. (4-56). Stress intensity ranged from high stress to the lower according to the colors (red-orange-yellow-green-blue). All columns with composite and hoop strengthening jacket showed crushing failure in the concrete jacket. Small columns with inclined and vertical reinforcement showed more crack propagation which considered a good index to the high ductility. Increase the length decrease the crack propagation. Full scale strengthened columns showed high crack spread and less stress concentration with increase in the jacket thickness.

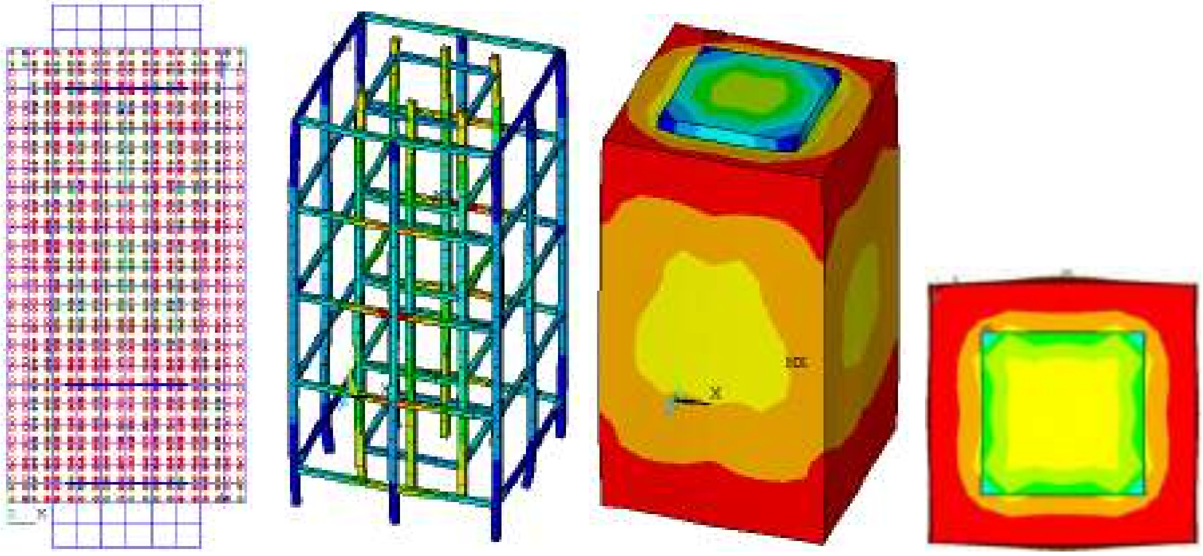


Color	Concentration degree
Red	Higher concentration
Orange	High concentration
Yellow	Medium concentration
Green	Light concentration
Blue	Lighter concentration

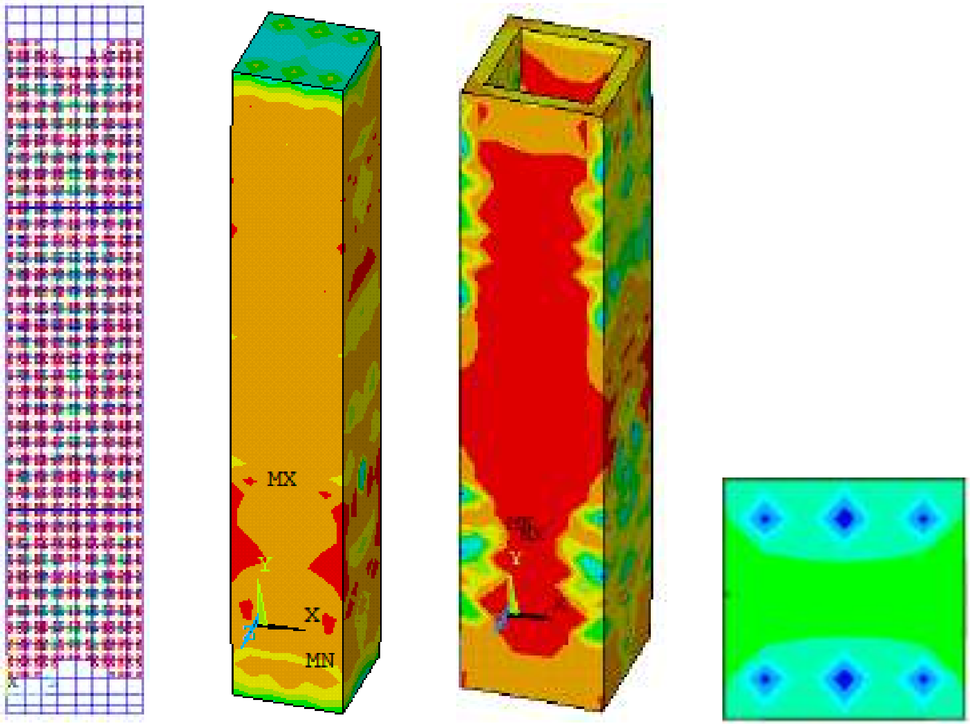
(a) S12n2h model.



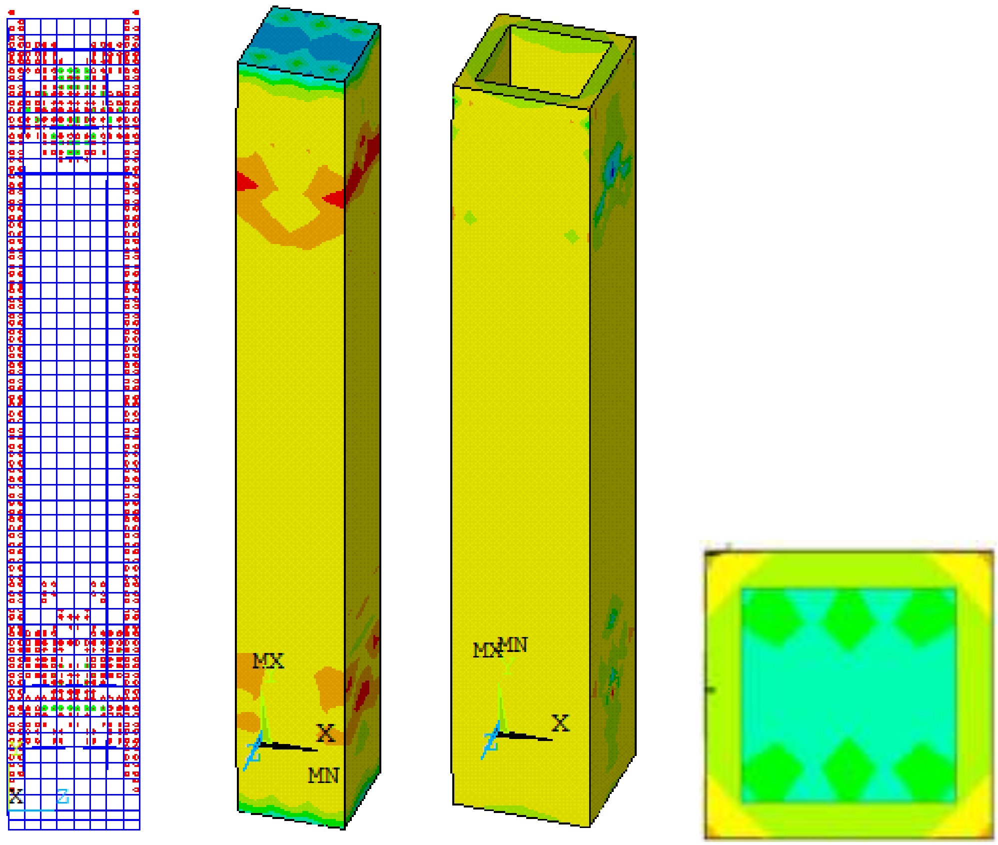
(b) S12ir model.



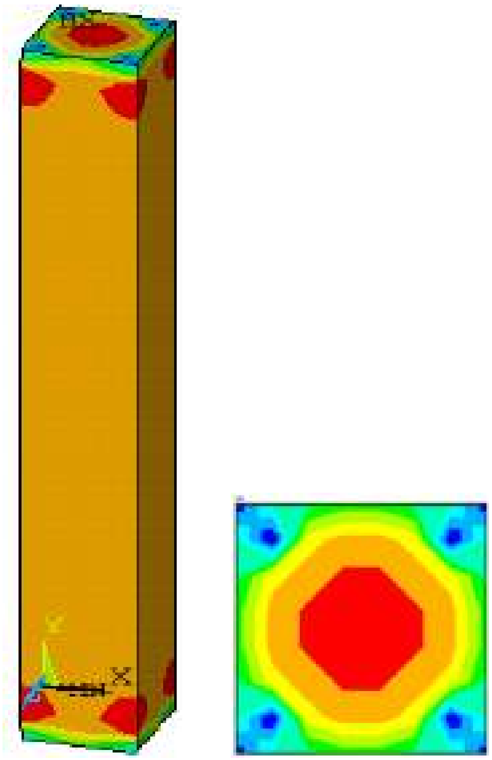
(c) S12vr model.



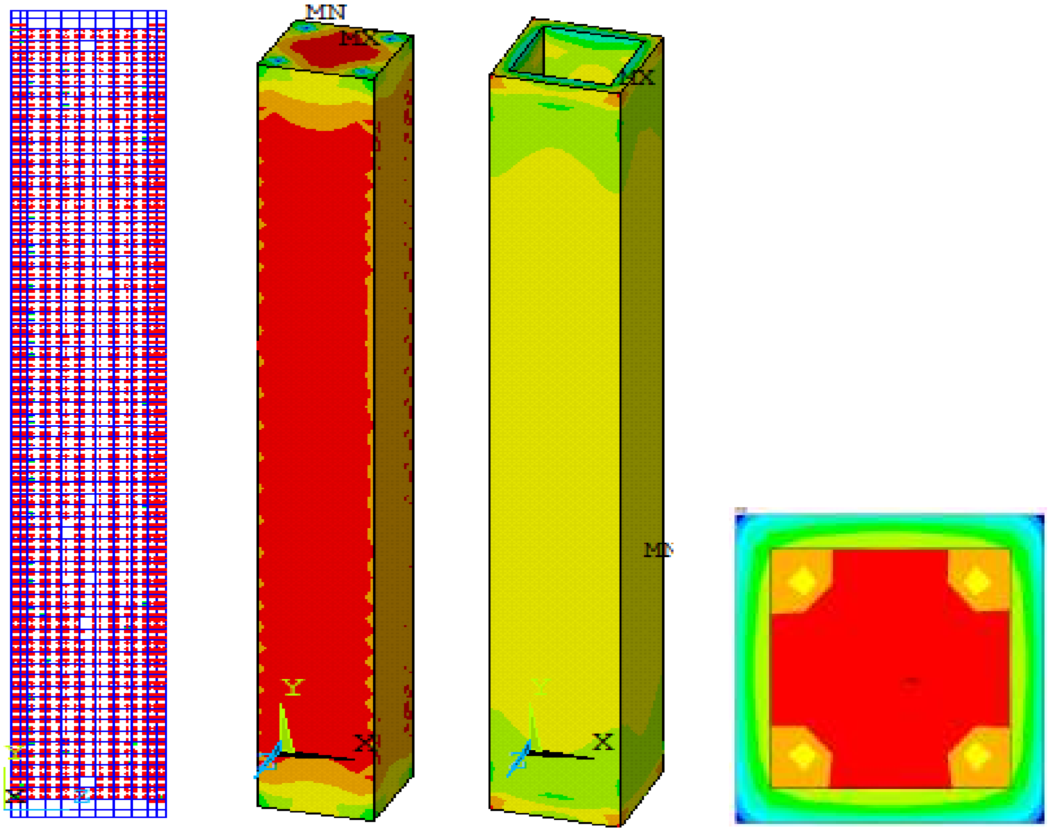
(d) S12Lc1 model.



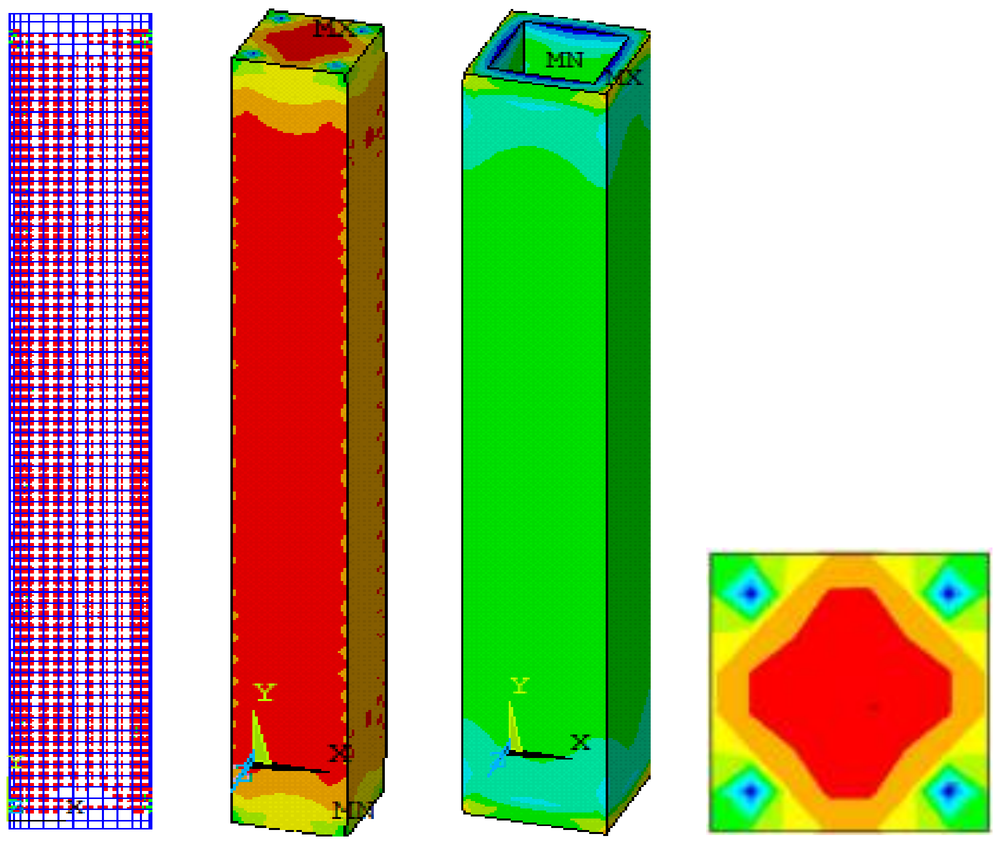
(e) S12Lc2 model.



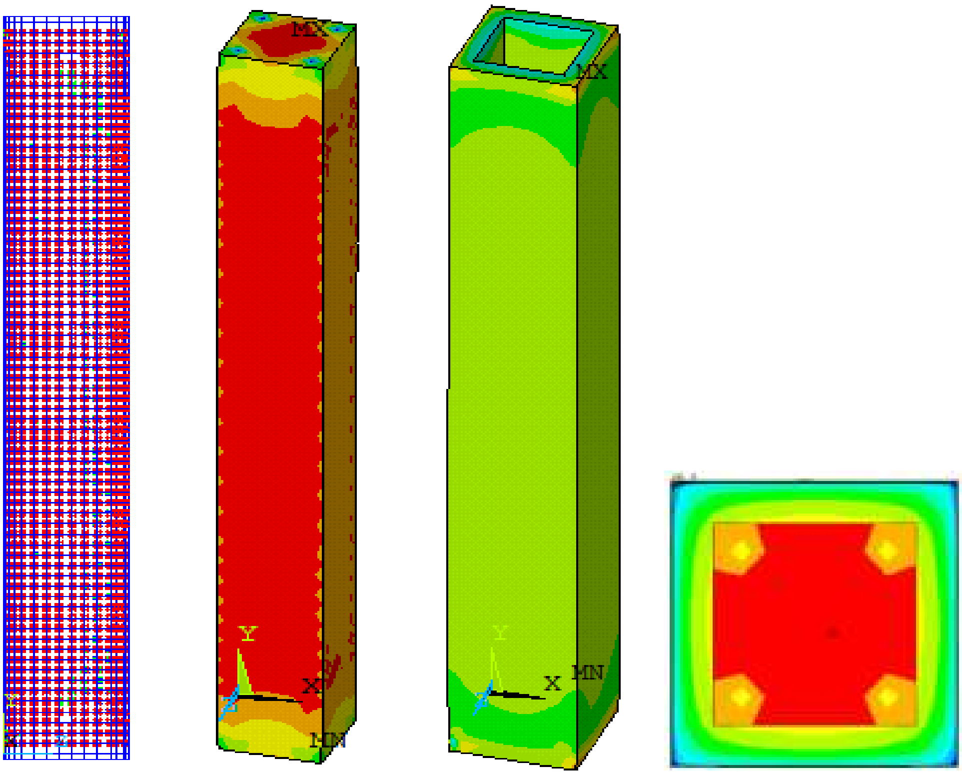
(f) S12FS model.



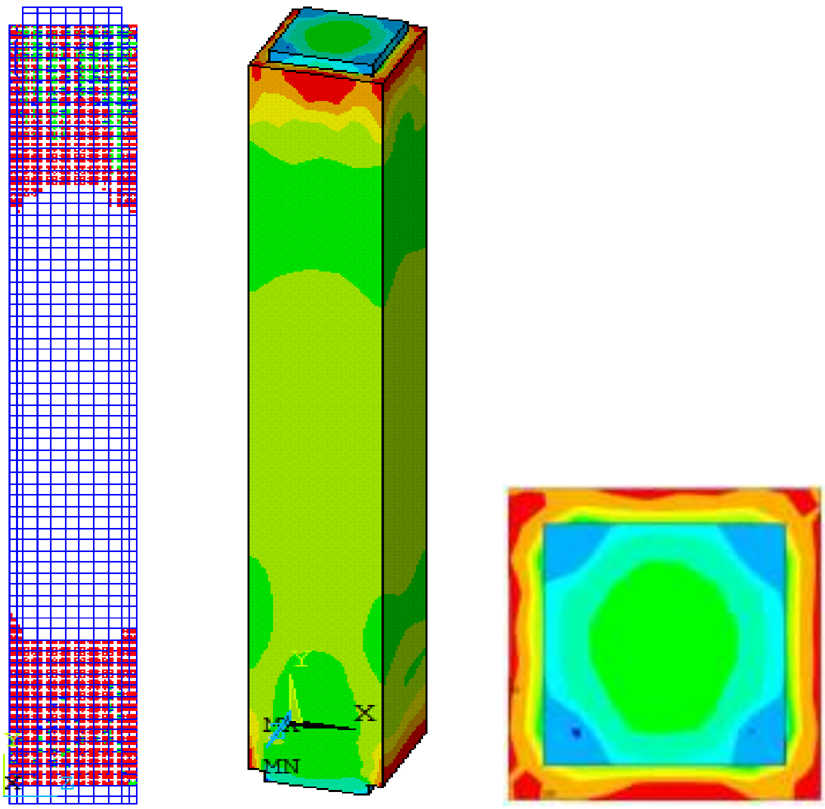
(g) S12FS5 model.



(h) S12FS6 model.



(i) S12FS7 model.



(j) S12FS5H model.

Figure (4-56): Crack pattern and stress distribution of the considered specimens in the numerical analysis.

CHAPTER FIVE
CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The experimental and numerical study is focused on the behavior of the strengthened column by steel fiber reinforced concrete under monotonic loading. Based on the results obtained experimentally and FEM for the stub columns, many factors have significant effect on the behavior of column at failure, and these effects can be summarized as follows:

1) Increase of hooked steel fiber ratio without epoxy which showed enhancement in the maximum load in comparison with the strengthened column by jacket with zero steel fiber ratio but use of (2%) fibers crushed in earlier stage than column with zero steel fibers ratio.

2) Presence of epoxy enhanced the maximum stress capacity but with less ductility.

3) Increase the jacket thicknesses (25, 35, and 45) mm jacket enhanced the ultimate load capacity by (51%, 128%, and 164%) respectively in comparison with the reference column.

4) The enhancement in the stress capacity by use of straight fiber were better than those columns which used hooked fibers.

5) Best enhancement in the stress capacity were by use of 1% for straight steel fibers and 1.5% for hooked fibers.

6) Both types of steel fibers (hooked & straight fibers) were used together which presented higher load capacity in comparison with columns with hooked steel fibers and with straight steel fibers strengthened columns.

7) Change of the jacket height from composite to hoop case showed better enhancement in the stress capacity. Also, use straight fibers for ratio of (2%) in hoop case presented increase in the stress capacity more than happened in the hooked ones by (10%) in comparison with column (S6n0h).

8) Using two sides strengthening showed increase in the ultimate load capacity in comparison with the un strengthen column. These two sides strengthening improved the maximum ultimate stress capacity in comparison with the reference column that have the same cross-section area. While when comparing the two sides strengthening with four sides strengthening, it found that strengthening by four sides had better enhancement in the ultimate load capacity than two sides.

9) Three sides strengthening showed good increment in the ultimate load capacity for the hooked and straight fibers respectively in comparison with the reference un strengthen column . When comparing these three sides strengthened column with the refence column that have the cross-section area , it is found that ultimate stress capacity enhanced for both hooked and straight fibers. Also, the three sides provide less enhancement in comparison with the four sides strengthening. The ultimate strength enhancement comes as follows; three sides strengthening is stronger than two strengthening of two sides by (60%). While the four sides strengthening is stronger than three sides by (14%)

10) Use of steel fibers jacket in the circular columns was more significant than square columns, increase steel fibers showed increase in the stress capacity of the circular columns. The increase in the steel fibers ratio

enhanced the ultimate load capacity in comparison with the control circular unstrengthen column. Use of epoxy showed lower stress capacity for hooked steel fibers ratio respectively.

11) Strengthened columns by SFRC jacket with thicknesses (25, 35, and 45) mm showed enhancement in the stress carrying capacity by (126%, 242%, and 171%) respectively for the jacket with hooked fibers. Increasing the thickness of the casing does not mean a permanent increase in the stress capacity, which will in fact be limited to a certain extent. Best jacket thickness in this study was (35 mm) which provided higher enhancement.

12) Use of hoop jacket case in the circular columns enhanced the ultimate stress capacity by (70%) for jacket with the two ends hooked steel fibers and (69%) for the small straight fibers case in comparison with the reference column.

13) Increase the length from 500 mm to 750 mm for square columns affected the general behavior of the column. Use of composite jacket increased the load carrying capacity by (106%) for jacket with hooked fibers and (131%) for jacket with straight fibers in comparison with un strengthen reference column. Hoop jacket case was carried out in the long column case which appeared improvement in the stress capacity was (15%) only in comparison with control column.

14) hoop SFRC jacketed numerical columns with inclined reinforcement (S12ir) enhanced the ultimate stress by (35%) and ultimate strain by (68%).

15) Regarding the full scale columns, strengthened column with composite SFRC jackets (5, 6, and 7) cm thickness showed an enhancement in the load carrying capacity by (177%, 210%, and 245%) respectively with slight increase occurred in the ultimate strain. Compare the hoop jacket with the composite one with the same jacket thickness showed that the

hoop jacket carried stress more than composite one by (67%) approximately with less strain by more than (25%).

16) Concerning the failure mode of square columns, the failure occurred in the jacket with inclined cracks started in upper surface corner and extended towards the bottom surface of the column. The increase in steel fibers ratio made the cracked headed to the corner as if it has strengthened the cohesion between the concrete particles in the middle cross section length to comeback and focus on the corners. Hoop jacketed columns failed by brittle failure which caused splitting the jacket into two parts.

17) Failure in circular columns came different with square columns due to the different in the geometry. Increasing steel fiber ration with and without epoxy caused more deformation capacity in the concrete columns. The failure occurred in the jackets when the process started from small cracks started in the top of the column and then grew extended along the RC columns.

5.2 Recommendation for Future Works

Extra investigation to understand the basic behavior of RC columns is required. The following suggestions are recommended:

1) Studying the seismic behavior of the jacketed columns.
2) Investigation of stub columns strengthened by ultra-high strength concrete.
3) Study of the behavior of SFRC jacketed column under cyclic loading (experimentally and numerically).
4) Experimental and numerical study of retrofitting damaged columns by SFRC jacket.

References

[1] Bridge, R.Q., “Design of Composite Columns-Steel, Concrete or Composite Approach”, ASCE, International Conference on Composite Construction in Steel and Concrete, 2008.

[2] Jaafer, A. A.,” Behavior of Short Concrete Columns Strengthened with Ferrocement" Ph.D. Thesis, University of Basrah, 2012.

[3] Campione, G., Colajanni, P., La Mendola, L., and Spinella, N., “Ductility of Reinforced Concrete Members Externally Wrapped with Fiber Reinforced Polymer Sheets”, Journal of Composites for Construction, Vol. 11, No .3, pp. 279-290, 2007.

[4] Mander, J. B., Priestley, M.J.N., and Park, R “Observed Stress-Strain Behavior of Confined Concrete”, Journal of Structural Engineering, ASCE, 114(8), pp. 1827-1849, 1988.

[5] Saatcioglu, M., and Razvi, S. R., “Strength and Ductility of Confined Concrete”, Journal of Structural Engineering, Vol. 118, No. 6, June, 1992.

[6] Lignola, P. G., “RC Hollow Members Confined with FRP: Experimental Behavior and Numerical Modeling”, Ph.d Thesis, University of Napoli, 2019.

[7] Meda A., Plizzari G.A., and Rinaldi Z., “Strengthening of RC Existing Columns with High Performance Fiber Reinforced Concrete Jacket, University of Bergamo, Bergamo, Italy, 2009.

[8] Kaiss Sarsam and Zaid M. Al-Azzawi,'' Shear Capacity of High-Strength Fiber Reinforced Concrete Beam-Column Joints’’Eng. & Tech. Journal, Vol.28, No.6, 2010

[9] Al-Shamma, Y. M.,’’ Strength and Ductility of Confined Steel Fiber Reinforced Concrete Short Columns’’ Master thesis, Al-Mustansiriya University, 2011.

[10] Muslim K. Al-kannoon ,'' Studying of Compressive, Tensile and Flexural Strength of Concrete by Using Steel Fibers'' Journal of Babylon University/Engineering Sciences, Vol.24, 2016.

[11] Shah, S. P.; Stroeven, P.; Dalhuisen, D.; Van Stekelenburg, P., "Complete Stress-Strain Curves for Steel Fibre Reinforced Concrete in Uniaxial Tension and Compression", Construction Press, Lancaster, p.p. 399–408, 1978.

[12] Richart, F.E., Bradtzaeg, A., and Brown, R. L., "A study of the failure of concrete under combined compressive stresses", Bulletin No. 185, Engineering experimental station University of Illinois, Urbana, pp. 104, 1928.

[13] Zhou, Ch., Li, X., Wang, D., and Xia, S., "Analysis of Bearing Capacity and Seismic Performance of Circular RC Columns Strengthened with Externally Wrapped Steel Plates", Advances in Civil Engineering Article, 2019.

[14] Bett, B.J., Klingner, R.E., & Jirsa, J.O.,"Lateral Load Response of Strengthened and Repaired Reinforced Concrete Columns", ACI Structural Journal, Vol. 85, No.5, pp.499-508, 1988.

[15] Rodriguez, M., and Park, R.," Seismic Load Tests on Reinforced Concrete Columns Strengthened by Jacketing", ACI Structural Journal, Vol. 91, No.2, pp.150-159, 1994.

[16] Lehman, D.E., Gookin, S.E., Nacamuli, A.M. and Moehle, J.P.," Repair of Earthquake-Damaged Bridge Columns", ACI Structural Journal, Vol. 98, No.2, 2001, pp. 233-242.

[17] Konstantinos, G., Vandoros, Stephanos, E., and Dritsos, "Concrete jacket construction detail effectiveness when strengthening RC columns", Construction and Building Materials, Vol.22, p.p.264–276, 2008.

[18] Helles, Z. H, "Strengthening OF Square Reinforced Concrete Columns with Fibrous Ultra High Performance Self-Compacting Concrete Jacketing", Master thesis, The Islamic University Gaza, 2014.

[19] Koo, I.Y., and Hong S.G. 2016. “Strengthening RC columns with ultra-high-performance concrete”. The 2016 Structures Congress. Jeju Island, Korea, August, 2016.

[20] Cho, C. G., Han, B. C., Lim, S. C., and Morii, N., “Strengthening of Reinforced Concrete Columns by High-Performance Fiber-Reinforced Cementitious Composite (HPFRC) Sprayed Mortar with Strengthening Bars”, Composite Structures 202, 2018.

[21] Tayeh, B. A., Naja, M. A, Shihada, S., “Repairing and Strengthening of Damaged RC Columns Using Thin Concrete Jacketing”, Advances in Civil Engineering, 2019.

[22] J. Xie, Q. Fu, J. Yan, Compressive behavior of stub concrete column strengthened with ultra-high-performance concrete jacket, Construction and Building Materials, Vol.10, p.p. 643–658, 2019.

[23] Mansur, M. and Paramasiva, P., "Ferrocement Short Column under Axial and Eccentric Compression," ACI Structural Journal, Vol. 87, No. 5, September-October, pp. 523-529, 1990.

[24] Takiguchi, K., Abdullah and Fujita, S.," On Strengthening and Repair of Shear Failure Type RC Columns with Circular Ferrocement Jacket”, Journal of Structural Construction Engineering, Architectural Institute of Japan, No. 541, March, pp.145-153, 2001.

[25] Abdullah and Takiguchi, K.," An Investigation into the Behavior and Strength of Reinforced Concrete Columns Strengthened with Ferrocement Jackets", Cement and Concrete Composites, 25, pp. 233-242, 2003.

[26] Kondraivendhan, B. and Pradhan, B.,"Effect of Ferrocement Confinement on Behavior of Concrete", Construction and Building Materials, 23, pp. 1218-1222, 2009.

[27] Mourad, S. S., and Shannag, M. J., “Repair and strengthening of reinforced concrete square columns using ferrocement jackets”, Cement and concrete composite, Vol.2, 2011.

[28] Jaafer, A. A., “Behavior of Short Concrete Columns Strengthened with Ferrocement”, Ph.d Thesis, University of Basrah, 2012.

[29] Soman, M., and Veena M.,” Repair and Rehabilitation of RC Short Square Columns using Improved Ferrocement Jacketing “, International Journal of earth science and engineering ISSN 0974-5904, Vol. 08, No. 02, 2015.

[30] Sabu, N. V., and Binu, P., “Strengthening of RC cylindrical columns using Micro concrete Jacketing”, International Journal of Innovative Research in Science, Engineering and Technology, Vol.6, Issue 5, 2017.

[31] Balamuralikrishnan R., Al Madhani M., and Al Madhani R., “Study on Retrofitting of RC Column Using Ferrocement Full and Strip Wrapping”, Civil Engineering Journal, Vol.5, No.11, 2019.

[32] Schneider, S.P., "Axially Loaded Concrete-Filled Steel Tubes", Journal of Structural Engineering, Vol. 124, No. 10, pp. 1125-1138, 1998.

[33] Johansson, M., and Gylltoft, K., "Mechanical Behavior of Circular Steel-Concrete Composite Stub Columns", Journal of Structural Engineering, Vol. 128, No. 8, pp. 1073-1081, 2002.

[34] Lam, D., and Wong, K.K.Y., "Axial Capacity of Concrete Filled Stainless Steel Columns", ASCE Journal of Structures, pp. 1107- 1120, 2005.

[35] Yang, Y.F., and Han, L.H., "Experimental Behavior of Recycled Aggregate Concrete Filled Steel Tubular Columns", Journal of Constructional Steel Research, 62, pp. 1310-1324, 2006.

[36] Abdel-Hay, A. S., and Fawzy, Y. A., “Behavior of partially defected R.C columns strengthened using steel jackets”, Housing and Building National Research Center, Vol.11, p.p. 194-200, 2015.

[37] Mosheer, K. A., “Strengthening and Rehabilitation of Reinforcement Concrete Square Columns Confined with External Steel Collars”, Kufa Journal of Engineering Vol. 7, No. 1, P.P.129-142, 2016.

[38] Pico, O., "Confinement Effectiveness of square FRP Tubes in Hybrid Columns", Master thesis, University of Central Florida, Orlando, FL., 1997.

[39] Saafi, M., Toutanji, H.A., and Li, Z., "Behavior of Concrete Columns Confined with Fiber Reinforced Polymer Tubes", ACI Materials Journal, Vol. 96, No. 4, pp. 500-509, 1999.

[40] Shahawy, M., Mirmiran, A.and Beitelman, T.," Tests and Modeling of Carbon-Wrapped Concrete Columns", Composites: Part B 31, pp. 471-480, 2000.

[41] Li, G., Kidane, S.,Pang, S., Helms, J.E. Stubblefield, M. A.," Investigation into FRP Repaired RC columns", Composite Structures, 62, pp. 83-89, 2003.

[42] Li, G., Torres, S., Alaywan, W., and Abadie, C., "Experimental Study of FRP Tube-Encased Concrete Columns", Journal of Composite Materials, Vol. 39, No. 13, pp. 1131-1145, 2005.

[43] Li, G., “Experimental Study of FRP Confined Concrete Cylinders", Engineering Structures, 28, pp. 1001-1008, 2006.

[44] Chakrabarti, A., Chandra, A. and Bharagava, P.," Finite Element Analysis of Concrete Columns Confined with FRP Sheets", Journal of Reinforced Plastics and Composites, Vol. 27, No. 12, pp. 1349- 1373, 2008.

[45] Sadeghian, P., Rahai, A.R. and Ehsani, M.R., "Strength and Ductility of Unreinforced Concrete Columns Confined with CFRP Composites under Uniaxial Loading", Concrete Repair, Rehabilitation and Retrofitting II– Alexander et al (eds), Taylor & Francis Group, London, pp. 1277- 1283, 2009.

[46] Tiwari, M. K., Chandak, R., and Yadav, R. K., “Strengthening of Reinforced Concrete Circular Columns Using Glass Fibre Reinforced Polymers”, Journal

of Engineering Research and Applications, Vol. 4, Issue 4(Version 4), pp.50-54, 2014.

[47] Al-Nimry, H., and Ghanem, A., “FRP Confinement of Heat-Damaged Circular RC Columns”, International Journal of Concrete Structures and Materials, Vol. 11, 2017.

[48] Fossetti, M., Basone, F., Arenzo, G. D., and Macaluso, G., “FRP-Confined Concrete Columns: A New Procedure for Evaluating the Performance of Square and Circular Sections”, Advances in Civil Engineering, p.p. 1-15, 2018.

[49] Oliveira, D. S., and Carrazedo, R., “Numerical Modeling of Circular, Square and Rectangular Concrete Columns Wrapped with FRP under Concentric and Eccentric Load”, Ibracon Structures and Materials Journal, Vol.12, No.3, p.p.518-550, 2019.

[50] Hanash, Z. F., “Behavior of Reinforced Concrete Columns Buried into Different Types of Aggressive Soils”, Master Thesis, University of Kerbala, 2017.

[51] American society of testing and materials (ASTM), ASTM C191, West Conshohocken, PA., 2002.

[52] American society of testing and materials (ASTM), Standard test method for compressive strength of hydraulic cement mortars (using 50-mm cube specimens)”, ASTM C-109, West Conshohocken, PA.,2002.

[53] Iraqi Standard No. 5 for the aggregation of natural resources, 1984.

[54] Panesar, D. K., “Developments in the Formulation and Reinforcement of Concrete (Second Edition), Woodhead Publishing Series in Civil and Structural Engineering, 2019.

[55] American society of testing and materials (ASTM), ASTM C-136, West Conshohocken, PA., 2001.

[56] Iraqi Standard No. (45) for the aggregation of natural resources, 1984.

[57] Standard specification for chemical admixtures for concrete, ASTM C494, Developed by Subcommittee: C09.23 Book of Standards Volume: 04.02.

[58] Guowei, M. A., and Wang, L., "A critical review of preparation design and workability measurement of concrete material for largescale 3D printing", Front. Struct. Civ. Eng., 2017.

[59] Dahake, A., "Effect of Steel Fibers on Strength of Concrete", J. Eng. Sci. Manage. Educ., Vol.9, p.p.45-51, 2016.

[60] Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement, ASTM A615 Developed by Subcommittee: A01.05.

[61] Resan, S. F., "Structural Behavior of Simply Supported Ferrocement-aluminum Composite Beams" Ph.d Thesis, University of Basrah, 2012.

[62] Epoxy manual Product Data Sheet Sikadur®-32 Normal.

[63] Standard Test Method for Slump of Hydraulic-Cement Concrete Active Standard ASTM C143 / C143M | Developed by Subcommittee: C09.60.

[64] Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading) Active Standard ASTM C78 / C78M | Developed by Subcommittee: C09.61

[65] Instruction manual for compression tester model: b-001.

[66] Popovics, S., “A numerical approach to the complete stress-strain curves for concrete”, Cement and Concrete Research Vol.5, p.p. 583-599, 1973.

APPENDIX -A-

FINITE ELEMENT MODELING

A.1 Finite Element Modeling

In this section, modeling of one specimen is performed. This specimen is confined column. In building of any FE model, it is necessary to define the element types, element real constants and material properties. A mesh is generated by defining nodes and connecting them to define the elements. After completing this step, the solution process used to define the analysis type and analysis options, applying loads and boundary conditions.

A.1.1 Used Element

For the confined column, used element in the modeling of the concrete, steel reinforcement, steel plate, and steel fibers is shown below in Table (A-1).

Table (A-1): Used element of a model in ANSYS.

Material Type	ANSYS Element
Concrete (Core)	SOLID65
Concrete (Confinement)	SOLID65 (Contain)
Steel reinforcement	LINK180
Steel plate	SOLID185

A.1.2 Real Constants

Another entity for proper modeling is the real constants. It is worth to mention that many real constants may be input for an individual element. All used real constants are tabulated below in Table (A-2).

Table (A-2): Real constant of the used element in ANSYS.

Real constant set	Element type	Constant			
1 (Concrete)	SOLID65		Real constant for rebar 1	Real constant for rebar 2	Real constant for rebar 3
		Material number	0	0	0
		Volume ratio	0	0	0
		Orientation angle	0	0	0
		Orientation angle	0	0	0
2 (Ø8 rebar)	LINK180	Cross-section area (mm2)		50	
2 (Ø6 rebar)	LINK180	Cross-section area (mm2)		28	
4 (Strengthening by Concrete filled with steek fiber)	SOLID65		Real constant for rebar 1	Real constant for rebar 2	Real constant for rebar 3
		Material number	5	5	5
		Volume ratio	0.01	0.01	0.01
		Orientation angle	0	90	45
		Orientation angle	90	0	45

A.1.3 Material Properties

All parameters for the material models were illustrated in Tables. (A-3) to A-7). ANSYS element needs some properties for proper entities which are termed "linear isotropic, linear orthotropic, multilinear isotropic, and bilinear material" for different materials. Many of terms need to define. EX means the concrete modulus of elasticity (Ec). PRXY is the Poisson ratio (μ) which was assumed to be 0.2 for concrete and 0.3 for steel. The stress-strain curve of the concrete that obtained from the equation is used in the modeling of the multilinear isotropic.

Table (A-3): Material properties for concrete Core and confinement jacket.

Linear Isotropic	
EX	25153 MPa
PRXY	0.2
Linear Isotropic	
EX	33965 MPa
PRXY	0.2

Multilinear Isotropic		
Point 1	0.0001	3.113719
Point 2	0.00025	7.733404
Point 3	0.0005	15.1139
Point 4	0.00075	21.84028
Point 5	0.001	27.69965
Point 6	0.00125	32.58086
Point 7	0.0015	36.4663
Point 8	0.0035	44

Multilinear Isotropic		
Point 1	0.000804	27.3
Point 2	0.001304	44.39021
Point 3	0.001604	54.41793
Point 4	0.001904	64.10979
Point 5	0.002204	73.12815
Point 6	0.002504	80.96215
Point 7	0.003104	90.39214
Point 8	0.003604	91

Concrete	
ShrCf-Op	0.4
ShrCf-Cl	0.9
UnTensSt	9.1
UnCompSt	91
BiCompSt	0
HydroPrs	0
BiCompSt	0
UnTensSt	0
TenCrFac	0

Table (A-4): Material properties for Ø8 rebar (longitudinal bar).

Linear Isotropic	
EX	200000 MPa
PRXY	0.3
Bilinear Isotropic	
Yield Stss	427 MPa
Tang Mod	448 MPa

Table (A-5): Material properties for Ø6 rebar (transverse bar).

Linear Isotropic	
EX	200000 MPa
PRXY	0.3
Bilinear Isotropic	
Yield Stss	451 MPa
Tang Mod	452 MPa

Table (A-6): Material properties for steel fibers.

Linear Isotropic	
EX	200000 MPa
PRXY	0.3
Bilinear Isotropic	
Yield Stss	1340 MPa
Tang Mod	1340 MPa

Table (A-7): Material properties for steel plate.

Linear Isotropic	
EX	210000 MPa
PRXY	0.3

A.1.4 Modeling and Meshing

A three-dimension model of the concrete structure is built using ANSYS. Modelling of the RC column in Cartesian coordinate system is created with number of elements about 9400.

Modelled specimen in this work have number of elements (5364). The process involved creating these types of columns into two regions. The process involved creating concrete column by pure concrete with steel reinforcement, then creating the concrete of the strengthening by concrete with steel fiber around the RC column.

Creating the model was done by generate the first node and take the advantage of the contract copying feature to complement the column modelling. Then the concrete element was created by using command prompt line input in each region and take advantage of the contract copying feature too.

The concrete column was modelled using a special concrete element SOLID65. It enables to define up to the rebar materials within the concrete. In the current work, modelling the reinforcement by Link 180 is assumed to be discrete modelling throughout the element.

The bond between the concrete and steel bars assumed as perfect bonding between them as shown in Fig. (3-19). Flexural and shear steel reinforcement have some shared nodes as revealed in Fig. (A-2).

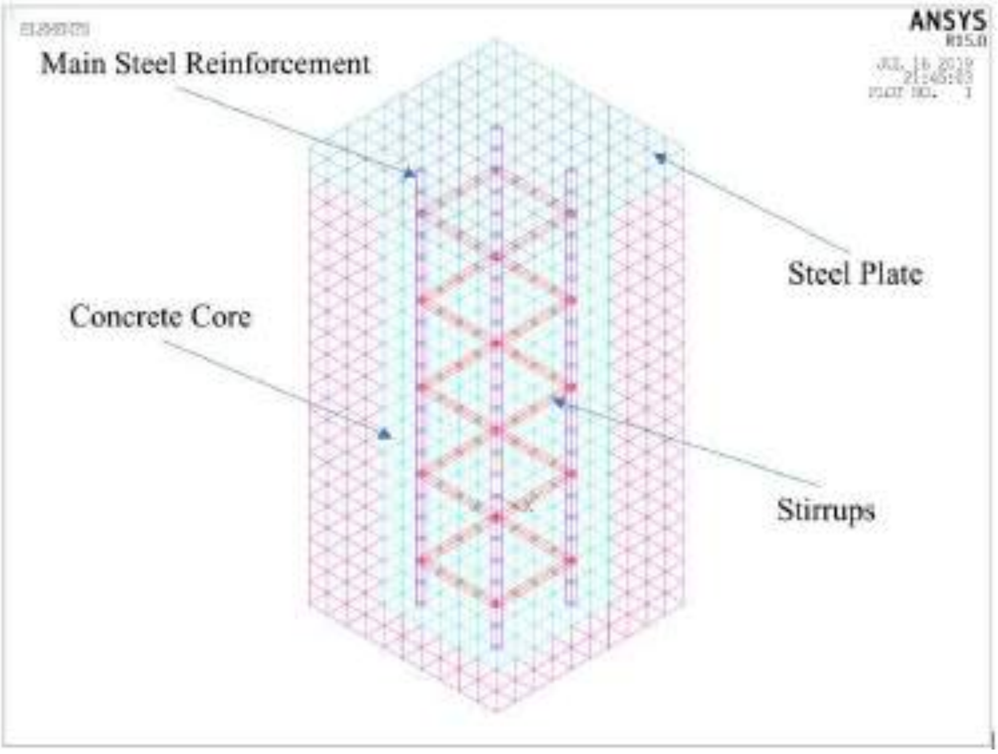


Figure (A-1): RC column model by ANSYS.

A.1.5 Boundary Conditions and Loading

It has been found that the simulation of the applied load and the supports has significant effect on the results of the finite element analysis. Bearing plate has been used to distribute the applied load on an area to get a unique solution. To ensure that the model acts the same way as the experimental column, the boundary conditions are:

1) The concrete column was supported by fixed at the bottom with no displacement of the nodes in all directions.

2) A concentrated load was applied on the all nodes of the steel plate. The distribution of the applied load on the nodes is that the load divided on the

number of nodes, the nodes at the edge of plate will carry the half value of internal nodes as illustrated in Fig. (3-20).

A concentrated load was applied on the single line of a steel plate which was placed in 100 mm wide from the vertex. The distribution of the applied load on the nodes is that the load divided on the number of nodes, the nodes at the edge of plate will carry the half value of internal nodes as illustrated in Fig. (A-3 b).

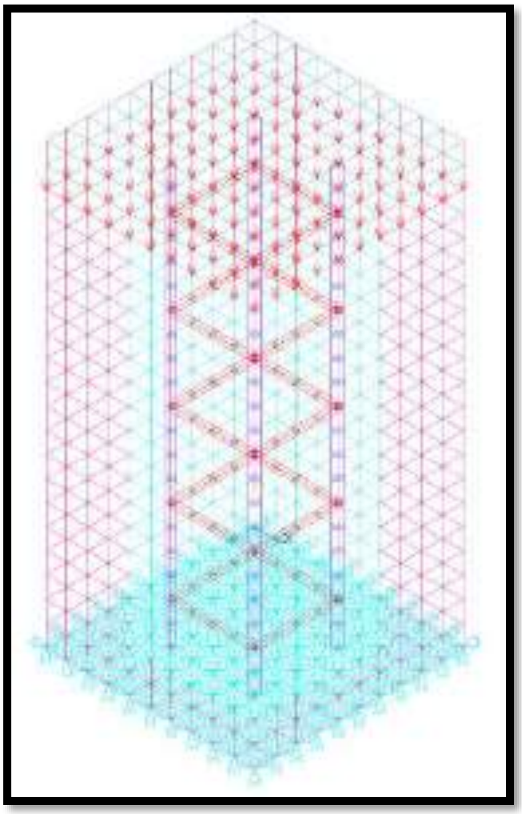


Figure (3-20): Loading and boundary condition of column.

A.1.6 Analysis Type

Actually, the static analysis was used for analyzed the models of all columns utilized in this study. The analysis was taken as small displacement and static which Performs a linear static analysis, i.e., a static analysis in which large deformation effects are ignored. The time at the end of the load step refers to the ending load per load step and the total time refers to total applied load. The time step refers to the time increment with maximum and minimum size. While the commands of the nonlinear algorithm and convergence criteria are used as tabulated in Table (3-10).

Table (A-8): The analysis commands in ANSYS.

Designation	Command
Analysis option	Small displacement
Calculate prestress effect	No
Time at end of Loadstep	Per applied load
Time step size	On
Maximum time step	1
Minimum time step	0.005
Write Items to Results File	All soluton items
Frequency	Write every substep

The aim in this study is to ensure that all selections are adequate to model the members. Tables (A-9) to (A-11) shows the commands of FE procedure used for the analysis of confined column.

Table (A-9): The analysis commands.

Equation Solvers	Sparse Direct
Number of restart file 1	1
Frequency	Write every substep

Table (A-10): The solution commands.

Line Search	on
DOF solution predictor	Prog. Chosen
Maximum number of iteration	200
Cutback control	Cutback according to predicted number of iter.
Equiv. Plastic Strain	0.15
Explicit Creep ratio	0.1
Implicit Creep ratio	0
Incremental displacement	10000000
Points per cycle	13
Set Convergence Criteria	
Label	F
Ref. Value	Calculated
Tolerance	0.005
Norm	Infinite norm
Min. Ref	Not applicable

Table (A-11): Commands for finishing analysis.

Program behavior upon non-convergence	Terminate but do not exit
Nodal DOF sol'n	0
Cumulative iter.	0
Elapsed time	0
CPU time	0

الخلاصـــــــة

أحد المتطلبات الرئيسية لتقوية أو تعزيز الهياكل الخرسانية المسلحة هو زيادة قابلية تحمل الأعمدة الخرسانية فيها لتحمل الأحمال المتوقعة الأكبر. هناك تقنيات مختلفة لزيادة قابلية العمود ؛ ومع ذلك ، تختلف هذه التقنيات في مزاياها وعيوبها. ان الهدف الرئيسي من هذه الدراسة هو التحقق من كفاءة حصر العمود الخرساني المسلح بالخرسانة المسلحة بالألياف الحديديةحيث تتكون الدراسة من جزئين عملي ونظري.

الدراسة الرئيسية التي تمت دراستها في الدراسة العملية هي أنواع ونسبة الألياف الحديدية ، واستخدام الإيبوكسي كمادة رابطة بين العمود الخرساني والسترة الواقية ، وسمك السترة ، وأنواع التقوية (التعزيز الجزئي والكامل) ، وأنواع الحبس (حيث تضمنت حالتين الاولى تكون السترة الواقية فيها على طول العمود والثانية تكون اقل من طول العمود بنسبة قليلة) بالاضافة الى متغير الطول.

بناءً على النتائج, فقد وجد أن السترة الخرسانية المسلحة بالالياف توفر دعماً جانبياً كافياً للعمود الخرساني وتزيد بشكل كبير من قوة وليونة الاعمدة تحت التحميل المحوري. زيادة نسبة الألياف الحديدية (0 ، 1 ، 1.5 ، و 2٪) بدون مادة الإيبوكسي أظهر زيادة في الحمل الأقصى بنسبة (4٪ و 20٪) لنسب (1٪ و 2٪) بالمقارنة مع العمود المقوى بواسطة السترة مع نسبة ألياف فولاذية صفرية ولكن استخدام (2٪) ألياف فد جعل العمود الخرساني يفشل في مرحلة مبكرة. أدى وجود الإيبوكسي إلى تعزيز السعة القصوى للإجهاد بنسبة (2٪ و 10٪ و 27٪) عند الانتقال في نسبة الألياف الفولاذية من (1 -2٪) على التوالي ولكن مع ليونة أقل. زيادة سماكة السترة (25 ، 35 ، 45) ملم عززت قدرة الحمل القصوى بنسبة (51٪ ، 128٪ ، و 164٪) على التوالي بالمقارنة مع العمود المرجعي. كان تحسين قدرة الإجهاد باستخدام الألياف المستقيمة أفضل من تلك الأعمدة التي استخدمت الألياف ذات النهايات المنبعجة. أظهر تغير ارتفاع السترة من حالة الطول الكامل إلى الطول الاقصر تحسينًا أفضل في سعة الضغط.

أدى استخدام السترة الواقية للعمود في جانبي العمود فقط إلى تحسين الحد الأقصى لقدرة الإجهاد القصوى بنسبة (35٪ و 44٪) للعمودين على التوالي مقارنةً بالعمود المرجعي بينما أظهرت تقوية الجوانب الثلاثة زيادة جيدة في سعة الحمولة القصوى للألياف المعلقة والمستقيمة بمقدار (88) ٪ و 96٪) على التوالي مقارنة بالعمود المرجعي غير المقوى. كان استخدام سترة الألياف الفولاذية في الأعمدة الدائرية أكثر أهمية من الأعمدة المربعة ، وزيادة الألياف الفولاذية بنسبة (1 ٪ ، 1.5 ٪ ، و 2 ٪) أظهرت زيادة في سعة الإجهاد للأعمدة الدائرية حيث أدت الزيادة

في نسبة الألياف الفولاذية إلى تعزيز سعة الحمولة القصوى بنسبة (194٪ و 71٪ و 126٪) على التوالي مقارنة بعمود التحكم غير المقوي الدائري. أظهر استخدام الإيبوكسي قدرة إجهاد أقل بنسبة (21٪ و 24٪) لنسبة (1٪ و 2٪) من ألياف الفولاذ المعلقة.

في الجزء الثاني من الدراسة ، يتم تحليل الأعمدة المختبرة باستخدام نماذج العناصر المحدودة ثلاثية الأبعاد غير الخطية. يستخدم برنامج ANSYS (15.0.7) لتحليل النموذج ثلاثي الأبعاد. تم تصميم العمود وسترة الخرسانة باستخدام عناصر ذات 8 عقدة (SOLID65) وعنصر ذو عقدتين (LINK180) ، في حين أن لوحة التحميل تم تمثيلها بواسطة العنصر (SOLID185) الذ يأتي مع 8 عقد. يفترض وجود رابطة مثالية بين العمود والسترة الخرساني للأعمدة بدون الإيبوكسي ولكن بالنسبة للإيبوكسي ، يتم استخدام عنصر (INTER205) لنمذجة الإيبوكسي. قدمت نماذج العناصر المحدودة المعتمدة تطابقاً جيدًا مع النتائج التجريبية.

بعد التحقق ، يتم إجراء دراسة نظرية مع متغيرات حيث تتكون هذه الدراسة من هذا الاستقصاء من 10 أعمدة خرسانية صغيرة الحجم وكاملة الحجم معززة بالعديد من المتغيرات مثل سماكة السترة ، والتسليح المائل والرأسي ، وأنواع الحبس. استخدام التسليح المائل قد عزز المقاومة بنسبة (35٪) المطيلية بنسبة (68٪).

فيما يتعلق بالأعمدة ذات الابعاد الحقيقية الكاملة ، أظهر العمود المعزز (سُمك 5 و 6 و 7) سم تحسنًا في سعة التحميل بنسبة (177٪ و 210٪ و 245٪) على التوالي مع حدوث زيادة طفيفة في الانفعال.

جمهورية العراق
وزارة التعليم العالي والبحث العلمي
جامعة ميسان
كلية الهندسة
قسم الهندسة المدنية

رسالة

دراسة حول الأعمدة الخرسانية المطورة بواسطة السترة الخرسانية من الألياف الفولاذية

مقدمة إلى كلية الهندسة في جامعة ميسان
كجزء من متطلبات نيل درجة الماجستير في علوم الهندسة المدنية/ إنشاءات

من قبل
واثق جاسم مهودر
(بكالوريوس هندسة مدنية 2001)

أشراف
الأستاذ المساعد الدكتور: سامر محمد جاسب

حزيران 2020 شوال 1441