Performance of Steel-Jacketed RC columns using various cementitious materials

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Performance of Steel Jacketed RC Columns Using Various Cementitious Filling Materials

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DEDICATION

This thesis is dedicated to my family who were always encouraging and loving throughout the whole journey of my graduate studies. I owe them any success I achieve in life.

This thesis is also dedicated to Basem Ahmed, my best friend who passed away several years ago. I wish you were here to share this moment with me. May you soul rest in peace.

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Thanks and praises to Allah who blessed me with the strength, capability, and knowledge to undertake and complete the research.

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ABSTRACT

Repair and strengthening techniques of RC elements are considered to be challenging due to time, cost, and space constraints. Conventionally, several techniques have been used in the retrofitting of RC element. These techniques includes epoxy repair, concrete and steel jacketing and FRP. Recently a new technique has been introduced which is concrete filled steel jackets. Although the previously mentioned techniques have been thoroughly studied, little research has been found in this area. Consequently, more data is required towards a safe and efficient design of this technique.

In this work, a strengthening technique for RC columns is proposed, which is concrete filled steel jackets. This technique comprises a steel cage consisting of four steel angles with steel strips at fixed spacing to prevent the buckling of the angles. The space between the RC column and the steel cage is filled with different classes of concrete. The experimental program consisted of ten RC columns, two of which are unstrengthened columns. Eight steel cages were used with the same length of the column to confine the RC columns. Four different concrete mixes of filling concrete were prepared with different grades to be used as the filling concrete. No interface or shear connectors were used between the old and new filling concrete. LVDT's and strain gauges were mounted on the specimens to record the load displacement and stress strain curves of the specimens. The properties of hardened concrete mixes were assessed using the cube strength at 28 days. The specimens were then uniaxially loaded until failure. Afterwards, the results of jacketed specimens were compared to each other as well as control specimens i.e. specimens without jacketing. In order to address the effect of the composite jacketing, the strength of the columns are to be compared with the Eurocode 4 and Regalado design equations for composite sections.

The results of this study reveal that the proposed technique have significant effects on the capacity, ductility and stiffness of the strengthened columns for different types of filling concrete. Also, this technique is more effective and economic for lower strength filling concrete as it behaves as a composite section. Moreover, the Eurocode 4 design equations tends to overestimate the capacities of the columns and Regalado's equation provide reasonable design values.

For future work, it is recommended to examine wider set of concrete mixtures to confirm the findings of this study, the bond between concrete and steel should be thoroughly studied and observe the change in the confinement action on the RC columns and compare the performance of the jacket under eccentric and lateral loads with the results of this study

Keywords (Repair, Strengthening, RC Columns, Steel Jacket, Concrete Class)

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LIST OF ABBREVIATIONS

ACI	American Concrete Institute
ASTM	American Society for Testing and Materials
BS	British standards
CFRP	Carbon Fiber Reinforced Polymers
FRP	Fiber Reinforced Polymers
RC	Reinforced Concrete
SCC	Self Compacting Concrete
SI	Strength Index
w/c	water to cement rati

Chapter 1 INTRODUCTION

Repair and retrofitting works are considered to be very challenging as most of the times the reasons which lead to the damage of concrete are vague. Since there is no clear guidelines or codes for the design of the repair works, so it is mainly dependent on the experience, judgement and inspection of the responsible engineer. Time and cost represent additional constraints to the repair works. In many cases the damaged structures have to be repaired while they are in service. Also, the performance and lifetime of the repaired or strengthened structure is mainly dependent on the repair process. That is why choice of the appropriate repair or strengthening technique is thought to be very crucial.

The following can be considered as a summary for the deterioration and damage that the concrete is subject to: poor quality concrete, corrosion of reinforcement steel, carbonation, freeze-thaw damage, earthquake damage, underestimated design and environmentally related problems.

Figure 1-1: Corrosion of bridges (www.cbc.ca)

Figure 1-2: Building Collapse due to an earthquake in Nepal (www.concrete.org)

Figure 1-3: Shear Failure in a column (www.pinsdaddy.com)

In the repair and strengthening works, it is so important to understand clearly and differentiate between the following expressions: repair, strengthening, restoration and maintenance. The following figure shows the difference between the different terminologies. Repair can be defined as increasing the structure performance after a damage to the performance that the element would exhibit with ageing. Restoration is to recover the original performance of the element to the initial performance. While strengthening is to increase the performance and load capacity of the element more than its initial capacity. On the other hand, maintenance is a systematic simple repair process that is carried out at periodic times to raise the performance of the element but not to the original point. The following diagram illustrates the relationship between different techniques.

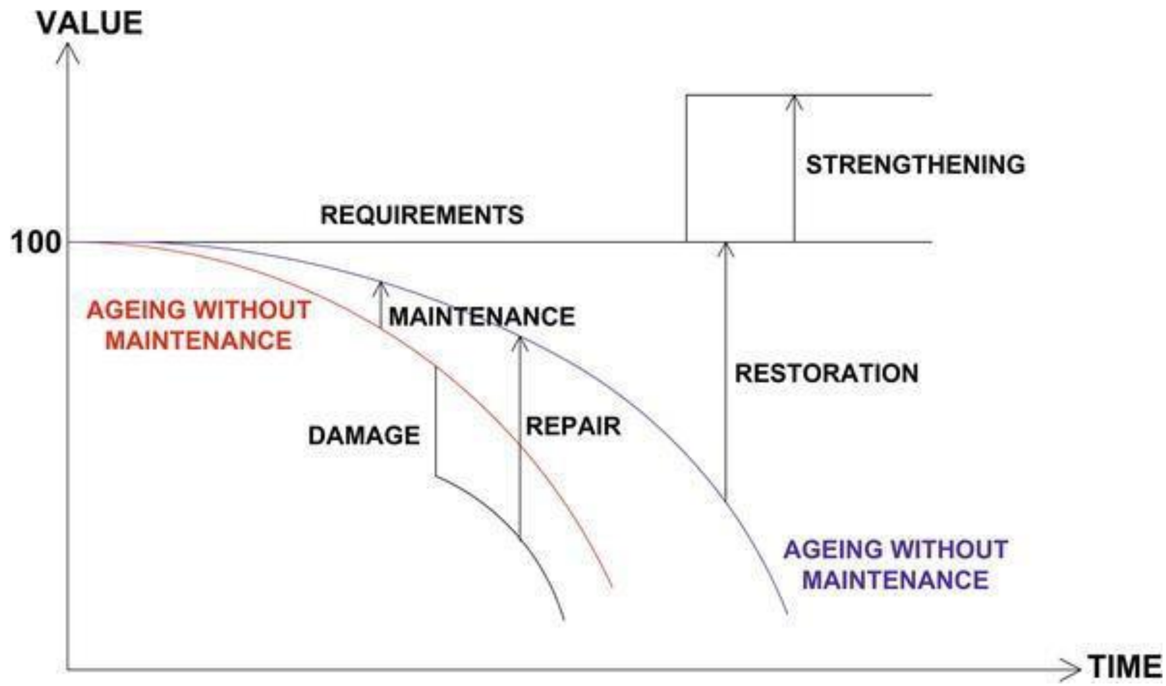

Figure 1-4: Relationship between different techniques (Mazzolani, 2006)

1.1. Buildings and infrastructure status in Egypt

A problematic phenomenon appeared in Egypt in recent years is the collapse of buildings. These buildings exist across Egypt in urban areas such as Cairo, Suez, Alexandria and Giza as well as other rural areas. This problem endangers lives, homes and the economy. According to experts, two types of buildings are susceptible to collapse in Egypt. The first one is old buildings that were constructed hundreds of years ago. The second type is new buildings. What should be highlighted is that the last string of collapsed buildings were new. This is due to the fact that many buildings after the 2011 revolution were constructed with no building licenses or with violations to these licenses. Some buildings increased the number of floors more than the obtained license which led to an increase in loads and hence imminent danger. According to recent reported numbers by experts and studies by governmental organizations, 12% of Egypt's real estate is in danger, 600,000 buildings are violating the building licenses, and 100,000 of them are susceptible to collapse. (www.albawabhnews.com) (www.atlanticcouncil.org)

Figure 1-5: Building Collapse, Egypt 2006 (www.atlanticcouncil.org)

1.1. Research Motivation

This study is of crucial importance particularly in these days in Egypt. As discussed in section 1.2, the buildings in Egypt are in dire need of repair works and strengthening due to the large number of buildings constructed after Jan, 2011. These buildings subject the life of civilians to danger. Also many governmental buildings were exposed to major damages due to explosions during the recent terroristic attacks. Moreover, the current status of Egypt's infrastructure shows that a lot of them in a questionable state. As the demolition of these buildings, in such circumstances, is neither practical nor accepted option, so rapid and economic repair and strengthening techniques have to be implemented. Three main aspects have the major contribution behind this study: 1) Egypt's need for widely accepted rapid and economic repair and strengthening techniques. 2) The influence of endangered of buildings and infrastructure on the Egyptian economy. 3) The effectiveness of the proposed repair and strengthening technique that is already used in some repair projects.

1.2. Research Objectives and Scope

This study aim is to investigate the influence of different classes of concrete used to fill steel jacket around concrete columns. This work is dedicated to study the applicability and economy of this repair and strengthening technique in different types of projects in Egypt. Detailed objectives of this work are:

1. Investigate the development, various techniques, and performance of repair and strengthening works.
2. Explore the most appropriate concrete class to be used in concrete filled steel jacket around a concrete column and its effect compared with normal concrete.
3. Evaluate the strengthening technique currently used in the structural repair in Egypt.

1.3. Research Methodology

The methodology used in this study to attain the above mentioned objectives is:

1. An extensive literature review on confinement of concrete, development, theory, design methods and applications.
2. Perform an experimental program to evaluate the compressive strength of confined concrete using the concrete filled steel jacket around concrete columns for different filling concrete classes under room temperature.
3. Compare the results to the design equations provided in the literature for steel jacketed RC columns.

1.4. Organization of Chapters

This study will consist of four other chapters outlined as follows:

Chapter 2 provides a literature review regarding the concrete repair/strengthening methods, factors affecting the confinement, design of confining methods, factors affecting the concrete properties. Also, it contains an evaluation in details about the steel jacketed RC columns. This is performed through reviewing recent papers to achieve comprehensive background about this topic.

Chapter 3 comprises the materials used and mixtures proportions of different classes of concrete. The experimental work will be illustrated in details including the equipment, data acquisition system, the structural details of the confined specimens. Also, testing methods and purposes shall be introduced

Chapter 4 i ncludes the results of different concrete mixtures and ultimate loads and properties of tested RC columns. Results of concrete filled steel jacketed columns shall be compared with the results of the reference columns. Results are to be interpreted and justified.

Chapter 5 presents conclusions to the whole study. Conclusions are warranted from experimental work provided in Chapter 4. Recommendations for future work are also addressed.

Chapter 2 LITERATURE REVIEW

2.1. Repair and Strengthening of Concrete: A Review

Various techniques are used in the repair of reinforced concrete elements. These techniques are epoxy repair, concrete and steel jackets, and FRP. The choice of the technique depends on the nature of the problem, cost, and the skilled labor. Moreover, the external strengthening is used whenever there is a need to increase the capacity of the building (Papanikolaou et al., 2013) (Karayannis et al., 2008). The research in this area covered various variables that affect the process of repair and strengthening. These factors include the preloading of old concrete, spacing of transverse reinforcement, interface between concrete and repair material, type of confinement, the type of member, shape and size effects, and concrete class. Hence, a lot of research is needed to determine and confirm the suitability of the methods used for repair and strengthening of concrete elements.

2.2. Factors Affecting Repair Experimental Work

2.2.1. Type of Confinement

The type of confinement used in the repair or strengthening process would significantly impact the behavior of the structural element. For example, using thin reinforced concrete jackets in repair and rehabilitation was shown to be effective in terms of enhancing the flexural and shear capacity of beams, when applied to a beam-column joint, without changing the mass or dynamic characteristics of the buildings. On the other hand, FRP applications have been used since the 90s in the RC beam-column joints. The FRP composites is commonly bonded to the RC members using epoxy fabrics. FRP jacketing has an advantage over the reinforced concrete jacketing as the latter changes the initial dimensions of the repaired or strengthened elements. This alters the dynamic characteristics of the building as well as the structural system geometry and mass. In addition, beam failure is characterized with a ductile mode failure instead of brittle one. The main disadvantage of using FRP jacketing is that it is dominated by debonding of the fabrics from the concrete elements. The failure mode of FRP jacketing hinders the effectiveness of this technique. Another well-known technique used in the repair and strengthening of concrete is the steel jacketing. Xiao and Wu (2003) concluded that steel jacketing gives better performance in the ductility of concrete over other repair and rehabilitation techniques.

In a study by Chen (2017), a comparison was conducted to show the difference in the failure mode of a repaired circular hollow section steel columns using grout or CFRP. The grout-repaired specimens exhibited brittle failure with spalling of grout. Also, the deformability of the grout specimens were more compared to the CFRP-repaired ones. On the other side, the grout repaired specimens showed higher stiffness and post yield ductility. To conclude, the grout repaired specimens can be considered to be more effective than those repaired with CFRP

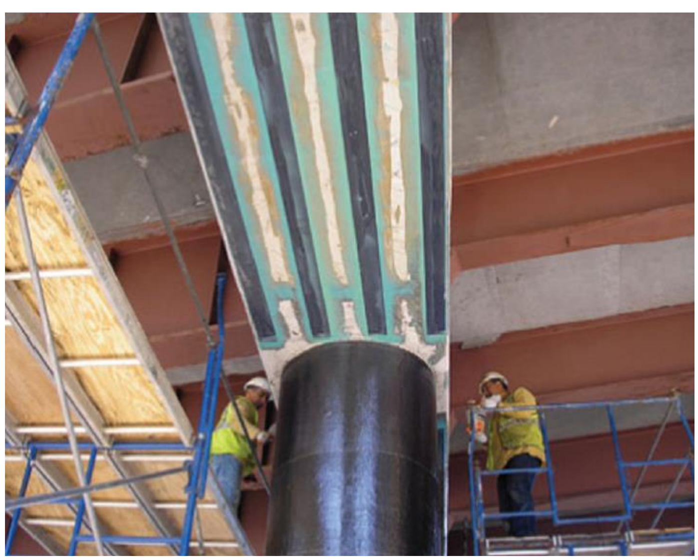

Figure 2-1. FRP in repair of concrete bridge (Ma et al., 2017)

2.2.2. Preload

The preloading plays an important role in the repair and rehabilitation process. Since, the seismic loading is naturally applied on preloaded columns under gravity loads, it is practically impossible to unload the columns before constructing the concrete jacket. The effect of preloading on strengthened concrete specimens using concrete jackets can be explained as follows. The non-preloaded specimens experience lower strength and lower displacement than the preloaded specimen. This is attributed to the dissipation of energy under the effect of preloading. Also, the preloading causes lower initial stiffness for the specimens but the stiffness degradation of the preloaded specimen is less than the preloaded specimen. (Vandoros et al., 2006). Preloading has a significant effect on the repair of damaged elements. The preloaded specimen has almost half of the axial capacity compared to the non-preloaded specimens. However, it does improve the capacity of strengthened members but only has a minor effect that can be neglected. (Ersoy et al., 1993, Takeuti et al., 2007). Thus, it can be understood that neglecting the preloading effect is on the conservative side of the design.

In a study by Chen (2016) to analyze the effect of preloading on steel jacketed concrete columns. Concrete columns retrofitted with steel tubes were investigated under different preloading and eccentricity values. The preloading effect on steel jacket retrofitted reinforced concrete columns was studied experimentally and numerically. The outcome of this study was that preloading effect became less effective as the ratio of D/t ratios decreased or with the increase of the yield strength of steel tube. On the contrary, the preloading did not have a significant effect with the variation of the strength of the concrete core. In addition, Preloading using loads that caused flexural failure of the concrete column were found to have more adverse effect on the ultimate strength of the retrofitted reinforced concrete column.

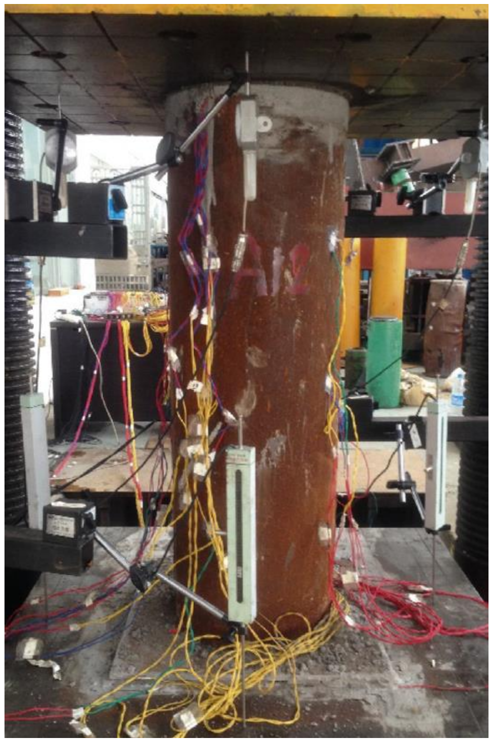

Figure 2-2. Concrete columns retrofitted with steel tubes (Chen, 2016)

A study was carried out by Papanikolauo (2012) to analyze the effect of the preloading on the repair and rehabilitation of concrete columns under axial and bending moment loads. The results of this study were that the favorable effect of preloading is only in the case of axial loading. The combination of axial loading and bending moment resulted in adverse results. Also, that the preloading has a significant effect in case of medium to high axial compression loads.

It is worth to be noted that in all of this studies the same thickness of concrete jackets and reinforcement was used for all the specimens. However, different concrete classes for the concrete core and jacket were used

2.2.3. Interface

The bond between the concrete jacket and core concrete is of critical importance. Epoxy can be used in the interface to increase the bond between the concrete core and the new concrete. Also, dowel bars or steel connectors might be used to strengthen the bond. In some cases no mechanical anchorage is used, but only a layer of high bonding material was applied with a brush on the surface of the repaired specimen, to focus on the interaction between the jacket and original concrete core. (Kumar, 2016)

2.2.4. The Scale of the Test

Many experimental tests were carried out on evaluate the efficiency of repair methods. These tests can be divided into three categories according to their sizes namely small, medium and full scale tests. According to the research budget, most of the tests were either small or medium scale tests. Most of the small scale tests were carried out to investigate FRP confinement technique. Ma et al. (2017) summarized the experimental work done in this area according to the size of the scale of the test, type of confinement and parameters tested. It was clear that the medium scale tests gave reliable results compared with full scale tests. Also, there is a lack in the research in testing the variables regarding the steel jacketing.

Table 2-1: Different scales of structure test (Ma et al., 2017)

Scale of test	Specimen	Testing Variables	Loading Scheme	Repair Technique
Small	Cylinder	Concrete Strength, Confinement Modulus, Type of confining Material, Size effects, Pre-damaged levels, Characteristic of confining materials, Load History	ML, CA, MA	FRP, Others
Medium	Column, Beam Column joint, Bridge Piers	Load History, Concrete Strength, Confinement Modulus, Partial Interaction, Wire Mesh Orientation, Numbers of wire layers, Axial Load Levels, Damaged Degree, Types of Mortar, Types of Concrete Core, Shape Effects, Damaged Condition, Types of Confining	ML + CA, MA, FL, CSF	Concrete Jacket, Steel Jacket, Ferrocement Jacket, FRP, Others

		Material		
Full	Column-footing Column, Beam, T-beam, Bridge Piers	Interface Treatment, Types of Confining Material, Size Effects, configuration, Transverse reinforcement, Height of repaired part, Spacing of Shear Connectors, Axial Load Levels, Confining Volumetric Ration	MA, ML+CA, FL, CL, SL	Concrete Jacket, Steel Jacket, Ferrocement jacket, FRP

2.2.5. Concrete Class

The effect of concrete compressive strength on concrete elements confined with CFRP was studied in another research by Sallam et al. (2016). The strengthening of the CFRP confinement was examined under the condition of changing the confined concrete compressive strength. Normal strength concrete of 15-MPa and 35 MPa was used in the investigation. Both of them showed linear increase in strength with the increase in number of CFRP layers. Hence, the effectiveness of the CFRP confinement is greater for the 15-MPa concrete. This is due to the lower value of volumetric strain exhibited by the lower strength concrete. The results are shown in the following figure.

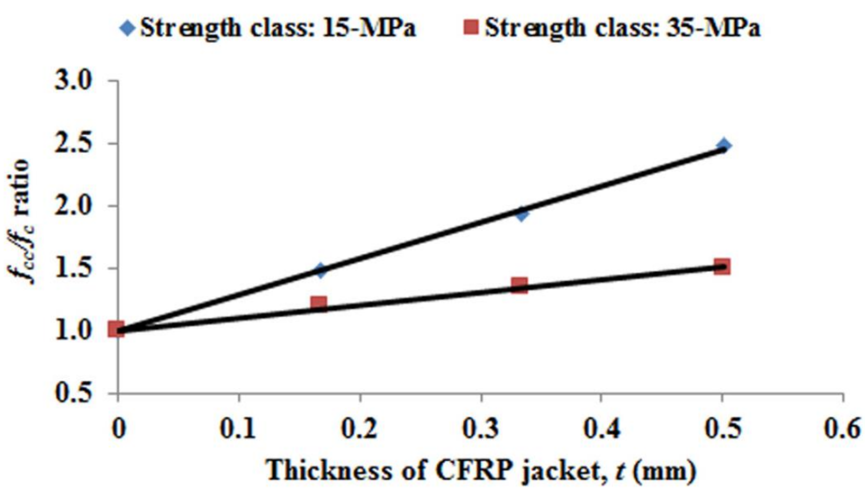

Figure 2-3: Effect of concrete class on CFRP confined concrete for different jacket thickness (Sallam,2016)

2.2.6. Active and Passive Confinement

There are two types of lateral confinement of concrete which are the active and passive confinement. The difference between them lies in the way the confining pressure is applied on the section before the axial loading. The concrete core is passively confined using circular or spiral hoops, and different kinds of jacketing. So, the confining pressure is not generated until the section is axially loaded. While, it can be actively confined through pre-stressing the concrete element laterally before applying the load. Additional axial load is needed to overcome the pre-stressing force and hence the load capacity in increased. According to Shin and Andrews (2009), the load capacity of actively confined concrete is greater than that of the passively confined. The active confinement results in an increase in the compressive strength, the value of prestressing has a minor influence on the axial capacity. Moreover, using passive confinement where active confinement is used results in higher strength and ductility. (O’shea et al., 2014)

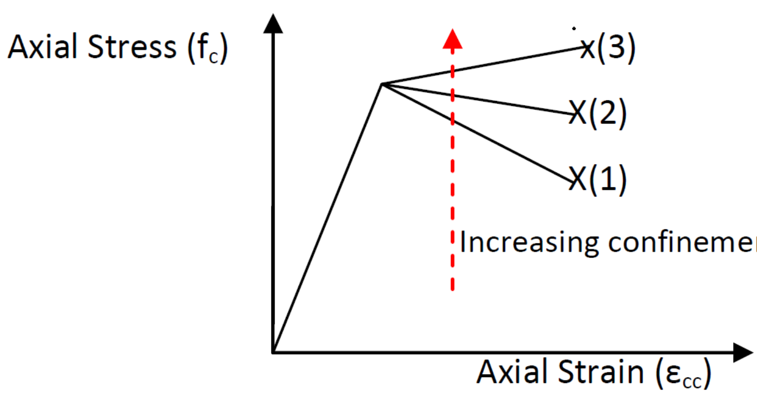

Figure 2-4: Effect of Confinement on Axial Stress (O'shea et al., 2014)

2.2.7. Temperature

Shehab El-Din (2013) investigated the behavior of CFRP confined concrete under different elevated temperature of 100,150 and 200C). The main objective was to investigate the effect of elevated temperature on compressive and tensile of concrete strengthened with FRP.

Figure 2-5: Heating the Specimens (Shehab El-Din, 2013)

Both the compressive and tensile strength of the concrete decreased when no CFRP confinement was used. The compressive and tensile strength increased as the number of confining strips increased.

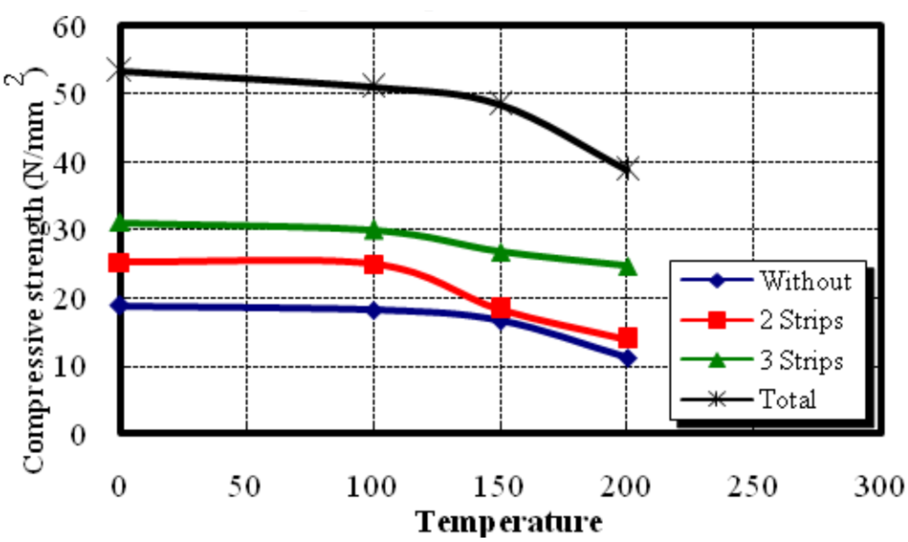

Figure 2-6: Effect of temperature on the compressive strength of CFRP confined concrete (Shehab El-Din, 2013)

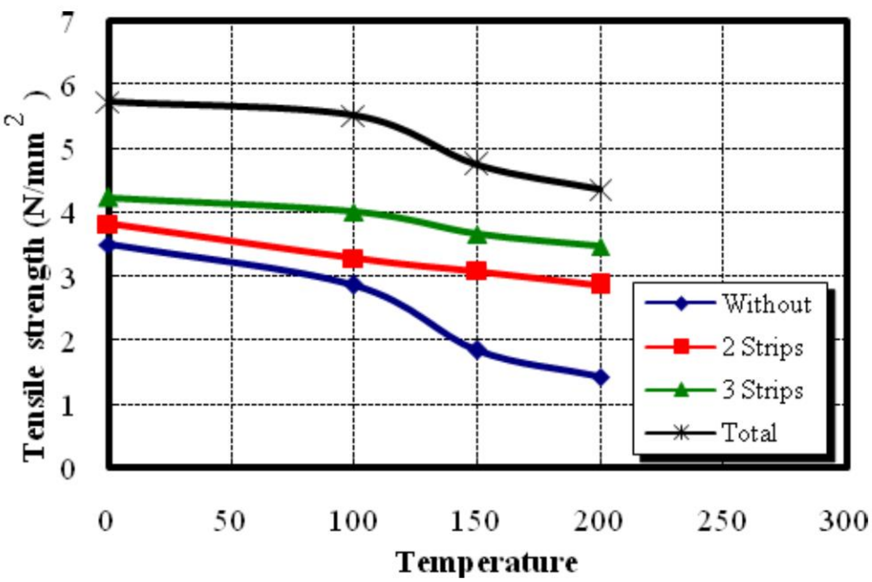

Figure 2-7: Effect of temperature on tensile strength of CFRP confined concrete (Shehab El-Din, 2013)

2.3. Design of Confining Methods

Repair techniques can restore the original capacities of concrete members. The money consumed in repairing of concrete structures exceeds the money used in building new structures. A lot of research is needed in order to make the repair techniques more economic. In addition, there is no well-established standards for the design of repair and rehabilitation works. Therefore, more experimental data is needed for adequate design of repair and rehabilitation works. Also, no clear guidelines to determine the level of damage are available for design purposes, the main design assumption that the concrete core is unloaded. This can be referred to as conventional design methods. As shown in Figure 2-9, the design using the conventional method assumes that the behavior of the concrete follows the path ABC in loading without confinement. It is well known that the repair can significantly increase the post peak strength of concrete elements and hence follows the path ADE after retrofitting. For repaired elements, the behavior exhibits a totally different path which is RST due to permanent deformations from the loading stage before damage.

The totally different mechanical behavior of concrete is the reason that hinders the effectiveness of the conventional design method. The design of confining methods for repair of concrete members should involve the load capacity, restorability and deformability of the concrete member.

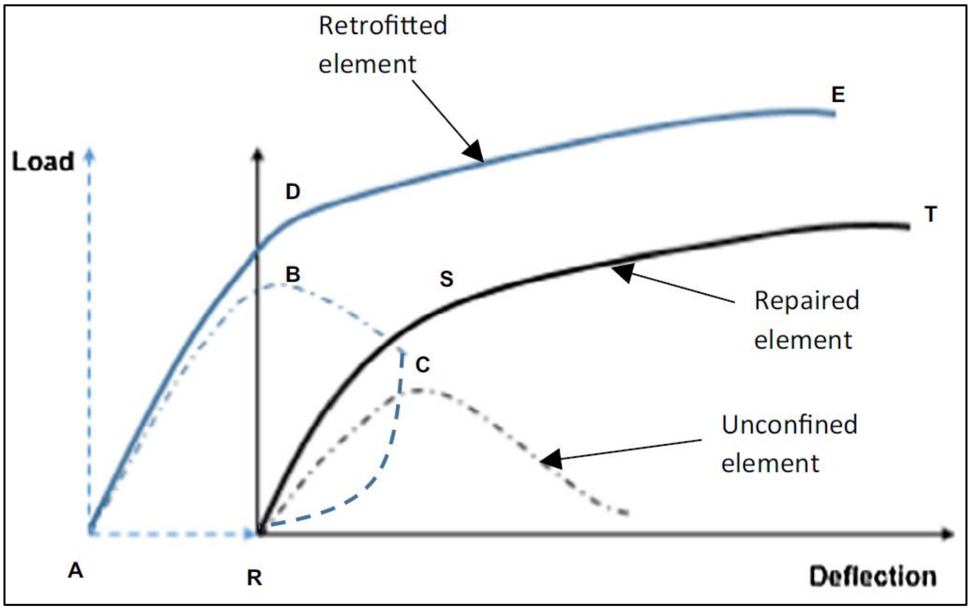

Figure 2-8: Load Deflection Curve of Repaired Elements (Ma et al., 2017)

2.3.1. Load Capacity

The factors affecting the load capacity of the repaired and retrofitted elements are the damage level of old concrete, confining pressure, confining efficiency, composite effect between concrete core and repair material and finally the type of confinement. Currently the load capacity of repaired concrete is determined through estimating the capacity of retrofitted columns using correlations or theoretical equations. According to Figure 2-8, the load capacity of retrofitted and repaired columns can vary clearly. For example for a given load, the deformation of the repaired column is more than that of the retrofitted columns at the same load. This leads to a difference in the behavior of the columns due to slenderness effects and consequently the load capacity decreases for the same cross section dimensions.

Richart et al. (1928) strength model can be used to estimate the load capacity of strengthened concrete. The enhanced strength of concrete is as follows:

$$F_{cc} = F_{co} + Kf_1$$

[Equation 2.1]

Where F_{cc} is the compressive strength of the repaired concrete, F_{co} is the unconfined concrete strength, K is the confinement effectiveness coefficient and f_1 is the confining pressure. The equation was proposed for confined concrete thus, for the case of repaired concrete the F_{co} should be replaced with F_{cd} which is the compressive strength of unconfined concrete after damage. As conveyed above, there are no clear guidelines or practical methods to determine the damage degree of a concrete element. As well as, the value of confining pressure is different for different types of confinement. So, the use of unified equation to design the repaired concrete section.

2.3.2. Deformability

The ultimate deformation can be determined through empirical correlations only because the plastic strain is mainly dependent on the confinement of concrete. Also, the ultimate deformations depend on the bond slip, plastic hinge of the member and confinement effects on concrete tension between cracks.

2.3.3. Serviceability

Serviceability of a building is measured in terms of excessive deformations or cracks. A structure can be considered unsafe if the serviceability requirements are exceeded even if it is safe structurally. The excessive deformations lead to, by nature, to excessive cracks which affect the durability and appearance of concrete members. In Numerous studies, the repaired concrete was reported to have larger deformations compared to retrofitted concrete elements. In other studies, short columns were reported to exhibit a slender behavior due to these large deformations. Till now, this matter is not clearly understood for an adequate design of the repair of concrete.

2.3.4. Restorability

Most of the studies focused on the effect efficiency of repair methods which can be measured according to the restorability of confinement. The restorability of confinement can be defined as the new capacity of concrete compared with old capacities before repair. Most of the studies are focused on repair using FRP fabrics. Hence, the outcomes of these studies are not valid to be used for other techniques. Also worth to mention that the concrete damage effect was not considered on the restorability of confinement.

2.4. Self-Compacting Concrete

SCC is used due to its remarkable fluidity. Self-compacting concrete can increase the lateral stiffness of heavily damaged columns. In a study by (Chalioris et al., 2012), self-compacted concrete was proven to be an efficient technique in repair and rehabilitation of reinforced concrete beams. Also, the results of the study agreed well with the predicted results. (Carballosa et al., 2012) used expansive self -compacting concrete to fill the gap between circular column and the formwork of CFRP. The self-compacting concrete was used because the compaction of normal concrete is neither appropriate nor satisfactory in this area. While the expansive characteristic of the concrete increases the axial capacity of concrete as it is an actively confined system. The specimens were prepared according to standard ASTM C878. The results provided showed that filling the gap with self-compacting micro concrete is an appropriate technique in the repair and strengthening of reinforced concrete element. (Dubey et al., 2016) conducted a study to investigate the effectiveness of using self-compacting concrete for rehabilitation of reinforced concrete columns. The strength gain of the repaired columns was analytically quantified in this study. The strength gain factor was defined as the ratio between the strength of repaired/retrofitted concrete to the original one. Using concrete of the same grade or weaker grade was proved decrease the strength gain factor comparatively.

2.5. RC Columns Strengthened with Steel Angles and Battens

2.5.1. Application

Four angles are used at the corners of the columns and steel battens are welded at a fixed spacing to prevent the buckling of the angles. The gap between the steel cage and the concrete column is filled with epoxy or cement to guarantee the bonding between them (Tarabia, 2014). The steel battens are used to prevent the bulging of the concrete i.e. increase the confinement.

Figure 2-9: Steel jacketed RC column (Amulya, 2010)

2.5.2. Advantages

The advantages of this system is that it does not enlarge the area of RC column. Also, it has adequate durability and ease of application. This system is also known to be protective against fire and corrosion (Adam, 2008, Campione, 2013). Tarabia (2014) proved that the ductility of the columns increased by about 50% in the strengthened columns.

2.5.3. Failure Mechanisms

Two mechanisms can lead to the failure of the strengthened columns. The first is the yielding of the angles and yielding of the steel strips. The former is not common if the strips are fixed at adequate spacing that prevent the buckling of the steel angles (Adam, 2008, Calderon, 2009, Tarabia, 2014).

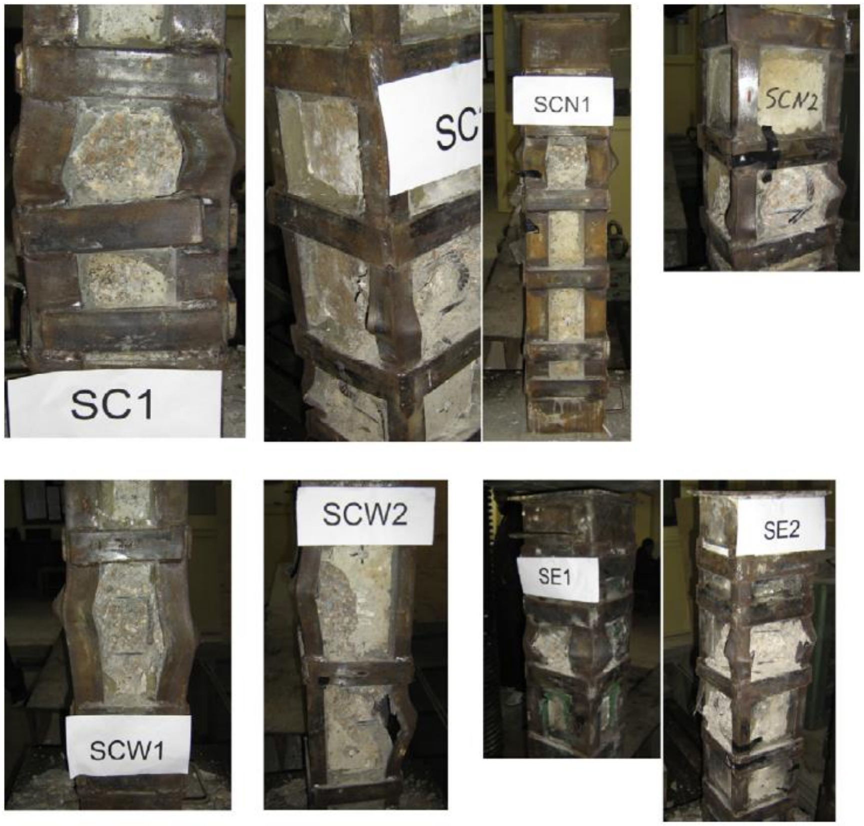

Figure 2-10: Failure Shape of strengthened columns (Tarabia, 2014)

2.5.4. Behavior of the System

The section does not behave as a composite section as there is no compatibility in the deformation between the RC column and the steel cage. This is due to the slippage between the layer of mortar and the strengthening.

Most of the work done in this field was done on low compressive concrete (F_{cu} = 15-20 MPa). The effectiveness of the strengthening technique increases for the low strength concrete. This means that the steel will absorb more load due to the high deformability of the low strength concrete which means higher lateral deformation due to Poisson effect. (Adam, 2008). Ramirez (1997) showed that injection with epoxy resin with fine sand is more effective in the bonding than the epoxy adhesive. Although, epoxy grout yielded better results than cement grout as a bonding material. It is recommended to use cement grout because the epoxy is more expensive

2.5.5. History of the Strengthening Technique

Montouri et al. (2009) evaluated the behavior of RC columns strengthened by steel angles and battens through conducting a set of experimental tests. Several outcomes were concluded out of this work. The strengthening using steel angles and battens increased the effectively confined area as well as the degree of confinement of concrete that was already confined before the strengthening intervention. Moreover, the steel cage provided lateral restrain to the longitudinal bars and prevented the spalling of the concrete at the corner sections. Finally, the steel angles can act in both compression and tension depending on the structural detail of the joint. The results of the experiments were compared with the EC8 provisions and was found to be fairly accurate.

(Adam, 2008) conducted an experimental work as well as a finite element parametric study on a group of axially loaded concrete columns strengthened with steel angles and battens. In this study the following parameters were addressed: size of the angles, yield stress of the cage, the compressive strength of the concrete in the column, the size of the strips and the friction coefficient between the bonding layer and the steel. The results of this work was that the strengthened columns do not behave as a composite section. In addition to, the effectiveness of confinement is increased by increasing the size of the angles, decreasing the compressive strength of the core concrete, using bigger battens.

Campione (2013) provided an analytical model to calculate the capacity of RC columns strengthened by steel cages. This model relates the increase in the load capacity to the mechanical and geometrical properties of the strengthening steel and concrete core. Experimental work was performed to verify the analytical model. Cases of directly and indirectly loaded columns were investigated. It was found that the load capacity of the columns and ductility increase with the decrease in the pitch of the steel battens.

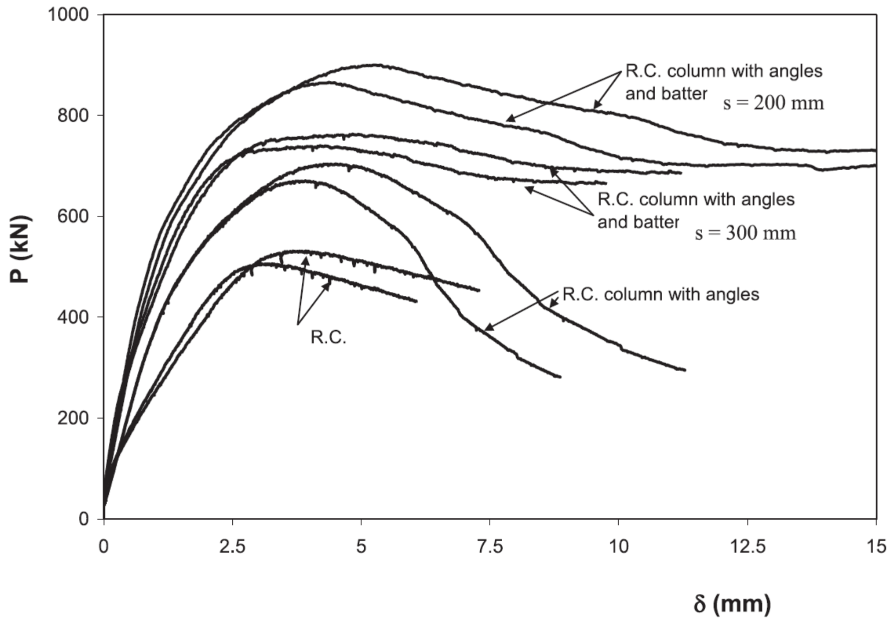

Figure 2-11: Load-Axial shortening curves for compressed columns (Campione, 2013)

Tarabia (2014) conducted a research on a group of RC columns strengthened with angles and steel battens. Some of the angles were directly connected to the head of the columns and other not. Also, an analytical model was developed using simple mechanics equations to obtain the failure load of strengthened columns. The results of the experiment work of Tarabia were promising as the columns gained increase in axial capacity between 210% and 135%. This was explained due the ability of the angles to share a part of the compression load with the steel angles as well as increasing the confinement of the concrete column. The directly connected angles to the column could transfer the axial load. On the other side, the other angles transferred the load by friction. The improvement in the latter case was between 190% and 135%.

2.5.6. Design Proposals

Several Design Equations were proposed for the design using the above mentioned technique.

According to the Eurocode 4; the section is assumed to be a composite section and could be designed according to the following equation

$$P_{EC4} = 0.85 \text{ x } A_c \text{ x } F_{c\,+}\ A_L \text{ x } F_{yL} + A_s \text{ x } F_{ys}$$

[Equation 2.2]

where; where A_c is the cross-section area of the RC column to be strengthened, f_c the compressive strength of the concrete, As the cross-section area of the longitudinal reinforcement of the column, f_{ys} the yield stress of the longitudinal reinforcement, A_L the cross-section area of the angles forming the cage, and f_{yl} the yield stress of the steel used in the angles.

Regalado (1999) reduces the ultimate load obtained by EC4 to account for the slippage between the steel cage and the mortar and that the column does not behave as a composite section. The following equation was proposed

$$P_{Reg} = 0.6 \text{ x } (0.85 \text{ x } A_c \text{ x } F_{c\,+}\ A_L \text{ x } F_{yl} + A_s \text{ x } F_{ys})$$

[Equation 2.3]

A design method was proposed by Calderon (2009). Two possible mechanisms were considered in the formulation of this calculation methods. The results of this method along with calculations of the EC4 and Regalado were compared with the finite element results of Adam (2008). Calderon proposal yielded more effective and reliable results than other proposals

Calderon proposed a new design method for RC columns strengthened with steel angles and battens. The results were compared with the output of the FE element models of Adam (2008). The ultimate loads of this method were found to be more reliable than those of Regalado and Eurocode No.4. Regalado’s assumption was found to be very conservative which induces more costs for design using this method. On the other side, it is non conservative to assume that the section acts as a composite section (EC no.4 assumption) due to the incompatibility in the deformation between the steel and reinforced concrete.

Table 2-2: Comparison of ultimate load obtained by Adam et al. (Calderon, 2009)

Specimen	P_{Adam}(kN)	Design proposal			Comparison		
	Adam et al. [7]	P_{EC4} (kN)	P_{Reg} (kN)	P_u (kN)	P_{Adam}/P_{EC4}	P_{Adam}/P_{Reg}	P_{Adam}/P_u
FEM-0	2185.7	2614.0	1568.4	2152.5	0.84	1.39	1.02
FEM-L40	1611.2	1599.8	959.9	1546.7	1.01	1.68	1.04
FEM-L60	1842.7	2021.1	1212.6	1781.2	0.91	1.52	1.03
FEM-L100	2582.5	3373.0	2023.8	2549.4	0.77	1.28	1.01
FEM-L120	3065.8	4286.0	2571.6	3108.5	0.72	1.19	0.99
FEM-fy235	2109.8	2417.2	1450.3	2048.4	0.87	1.45	1.03
FEM-fy355	2349.4	3007.6	1804.5	2339.8	0.78	1.30	1.00
FEM-fc4	1572.8	1894.0	1136.4	1442.6	0.83	1.38	1.09
FEM-fc20	2944.6	3334.0	2000.4	2873.9	0.88	1.47	1.02
FEM-fc30	3992.3	4234.0	2540.4	3788.2	0.94	1.57	1.05
FEM-St80	1898.1	2614.0	1568.4	1845.1	0.73	1.21	1.03
FEM-St120	1961.9	2614.0	1568.4	1947.6	0.75	1.25	1.01
FEM-St200	2396.1	2614.0	1568.4	2364.4	0.92	1.53	1.01
FEM-aSt	2678.3	2614.0	1568.4	2473.9	1.02	1.71	1.08
Exp-A	1954.8	2281.0	1368.6	1996.4	0.86	1.43	0.98
Exp-B	2324.1	2650.0	1590.0	2370.7	0.88	1.46	0.98
Exp-C	2599.4	2929.0	1757.4	2569.8	0.89	1.48	1.01
Exp-D	2451.9	2281.0	1368.6	2280.9	1.07	1.79	1.07
Mean	–	–	–	–	0.87	1.45	1.03
COV	–	–	–	–	0.101	0.169	0.033

2.6. Concrete Mixtures

The following properties of concrete are addressed in the following section

- workability
- strength
- durability

The concrete properties are affected by several factors. Only the three following factors are discussed in this thesis

- cement content
- water-to-cement ratio (w/c)
- aggregates

2.6.1. Workability

American Concrete Institute (ACI) 116R defines workability as “that property of freshly mixed concrete or mortar that determines the ease and homogeneity with which it can be mixed, placed, compacted and finished to a homogenous condition”.

i. Water Content

Increasing the water content increase the workability of the concrete. However, excessive water can cause bleeding and segregation (Mindess et al. 2003).

ii. Cement Content

As the workability is affected by paste volume, when the cement content increases the friction between aggregates will decrease. For a given water cement ratio, the water content per unit volume will increase and thus the workability will increase.

iii. Aggregates

The aggregates represent 60% to 75% from the volume of the concrete. That is why its selection is very important in the concrete mix design. The workability of the concrete is affected by the properties if the aggregates such as porosity, gradation, and shape. (Kosmatka et al., 2002).

2.6.2. Strength

Kosmatka et al. (2002) define strength as “the measured maximum resistance of a concrete specimen to axial loading”. Strength is frequently used to assess the quality of concrete

For a given water to cement ratio, the strength is independent of the cement content.

i. Water-to-Cement Ratio

The strength of the concrete is inversely proportional to the water to cement ratio. This is due to the influence of the w/c ratio on the porosity of concrete (Mindess, 2003).

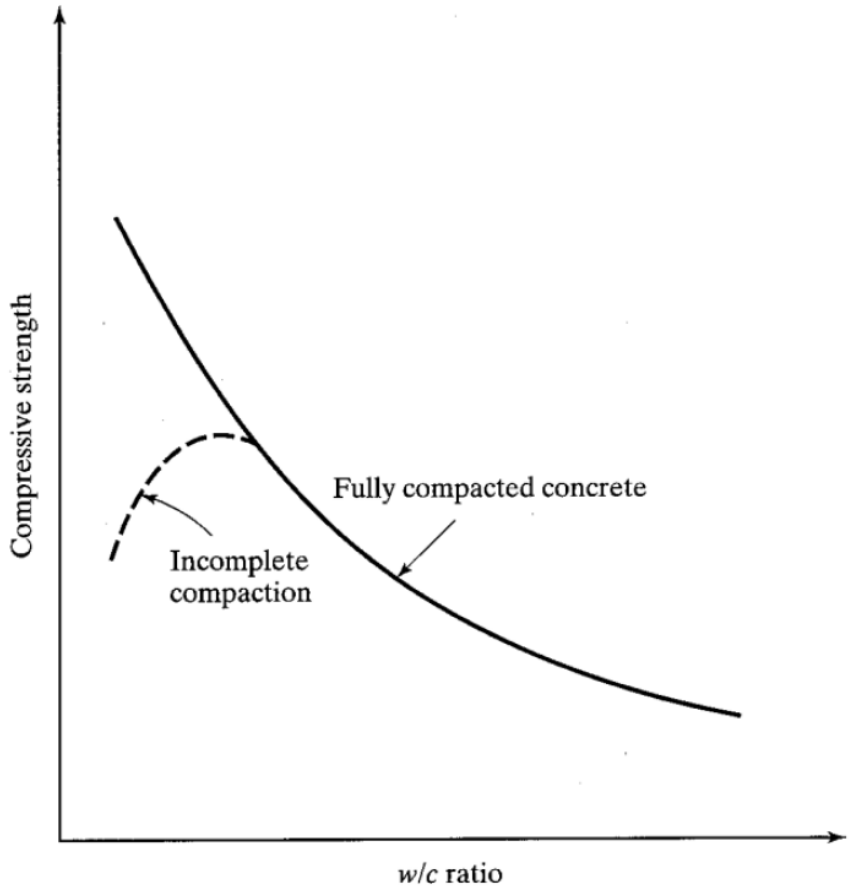

Figure 2-12: Relationship between compressive strength and water-to-cement ratio (Mindess et al., 2003)

ii. Aggregates

As per Mindess (2003), the rough and angular aggregates yields higher strength concrete due to the better bond to the cement paste. As for the aggregate maximum size, it is also worth to mention that larger aggregate particles reduce the concrete strength.

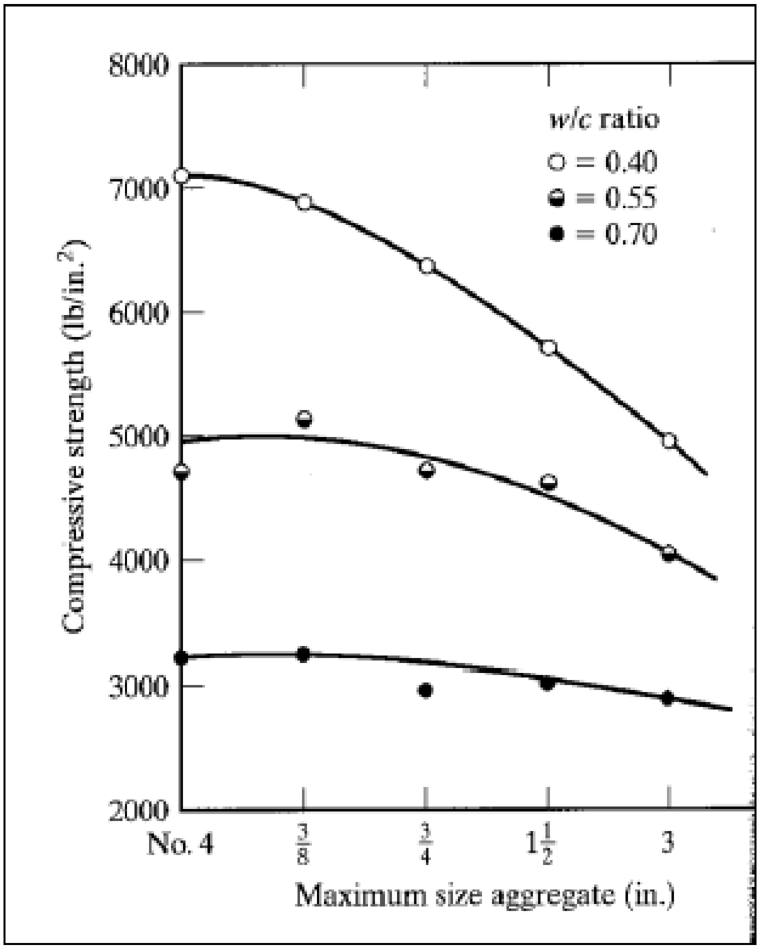

Figure 2-13: Effect of maximum size of aggregate on compressive strength (Cordon and Gillespie, 1963)

2.6.3. Durability

ACI Committee 201 (2008) defines durability of concrete as "the ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration and retain its original form, quality, and serviceability when exposed to its environment".

i. Water-to-Cement Ratio

The w/c is a very important parameter for durability. As w/c decreases, the porosity decreases which means better durability against chlorides and aggressive material (Mindess, 2003, Kosmatka et al., 2002).

ii. Cement Content

The increase of cement content for a given w/c increases the shrinkage. Increasing shrinkage causes more cracks to the concrete and hence decreases its durability and it becomes subject to aggressive compounds (Mehta and Monteiro, 1993).

iii. Aggregates

The maximum aggregate size affects the durability of the concrete. When the maximum aggregate size decreases, this will increase the cement paste that is subject to chemical attack (Mindess, 2003).

2.6.3.1. Factors that indicate durability

The following factors play an important role in the service life of reinforced concrete.

i. Permeability

The high permeability of concrete increases the sulfate penetration and chlorides absorption and chemical compounds attacks. This leads to the deterioration and reduction of service life of concrete

ii. Chloride Penetration

The concrete durability decreases when the chloride penetration increases. The chlorides attack the steel and causes its corrosion.

iii. Carbonation

Carbonation occurs when the carbon dioxide reacts with the hydroxides in the concrete to form carbonates. The reinforcement steel in this case is subject to corrosion.

Chapter 3 WORK METHODOLOGY

3.1 General

This chapter presents the experimental work performed in this study. The experimental work consists primarily of 10 cast in-place concrete columns. Eight of the ten columns were jacketed using concrete filled steel jackets. The steel jackets were formed from four vertical angles with steel battens at a suitable spacing to prevent buckling of the steel angles. The mechanical properties of the filling concrete is expected to have a direct effect on the efficiency of the concrete jacketing. Thus, the concrete grade of the filling concrete was changed for different specimens to evaluate the overall performance of the jacket. This chapter describes the procedures, material and equipment used in the laboratory as well as the different mixes used.

In this work, different water to cement (w/c) ratios were used as well as cement content. The values of the cement content adopted were chosen to simulate commonly used practices in the concrete columns in Egypt. The w/c ratio ranged between 0.35 to 0.55, while the cement content was between 350 to 430 kg/m^3. This variation was selected in order to be able to evaluate the performance of the jacketing under different strengths of filling concrete.

3.2 Materials and Proportioning

This section addresses the material selection used in this study. All the material used in the experimental work were acquired from local Egyptian sources. The material were selected from the frequently used types and brands in the Egyptian market. The following sections describe the materials used in the study. Standard tests were performed on different constituents of the concrete and steel jacketing.

3.2.1 Portland Cement

The cement used was ordinary Portland cement (ASTM C 150 Type I). The cement was produced by Lafarge cement Egypt in Ain Sokhna plant. The cement had a specific gravity of 3.15 and a Blaine fineness of 313 m^2/kg. The cement consisted of the following Bogue compounds: C_2S = 28.64%, C_3A = 11.75%, C_3S= 44.34%, C_4AF= 9.26%. Table 3-1 shows the chemical composition of the used cement.

Table 3-1: Type I Portland cement characteristics

Element	CaO	SiO_2	Al_2O_3	Fe_2O_3	MgO	Na_2O	K_2O	SO_3	CI
Weight %	63.54	22.13	5.25	3.44	1.87	0.25	0.3	2.13	0.3

Table 3-2: Typical results of standard testing of the cement used

Test	Standard(s)	Property	Results
Fineness of Portland Cement	ASTM C204	Fineness	313 m^2/kg
Density of Portland Cement	ASTM C188	Density	3.15
Setting Time of Portland Cement	ASTM C191	Initial setting	145 minutes
		Final setting	235 minutes
Compressive Strength of Cement Mortar	ASTM C109	3-day Comp. Strength	17.9 MPa
		28-day Comp. Strength	47.3 MPa

3.2.2 Fine Aggregates

Different concrete mixtures had the same type of siliceous sand. Fine aggregates were obtained from natural Wadi Sand, Bani Youssef. The fineness modulus of sand is 2.557, a saturated surface dry specific gravity of 2.66 and a percent absorption of 0.62%. Table 3-3 presents the sieve analysis results of the sand. (Along with the ASTM C33 limits for fine aggregate grading). Sieve analysis test was conducted according to ASTM C136. In order to determine the other properties of the sand, several other tests were also conducted. The results of these tests are presented in Table 3-4.

Table 3-3: Fine aggregates Sieve analysis, % retained cumulative

Sieve Size (mm)	% Retained	ASTM C33 Limits
10.0	0	0
5.00	0	0-5
2.36	6.0	0 - 20
1.18	15.0	15 - 50
0.60	52.0	40 - 75
0.30	84.8	70 - 90
0.15	97.9	90 - 98
0.0075	99.5	98 - 100

Table 3-4: Typical results of standard testing of the fine aggregates used

Test	Standard(s)	Property	Results	ASTM C33 Limits
Materials Finer Than 75µm (No. 200)	ASTM C117	Percent of Materials Finer Than 75µm	**0.50 %**	**3.0%**
Chemical Analysis	BS 812 – Part 117/118	Chloride (CL)	**0.0453%**	-
		Sulphate (SO3)	**0.40%**	-
Clay Lumps & Friable Materials	ASTM C - 142	Percent of Clay Lumps & Friable	**0.65%**	**3.0%**

Specific Gravity & Absorption	ASTM C128	Bulk S.G (SSD)	**2.638**	-
		% Absorption	0.62 %	-

3.2.3 Coarse Aggregates

Crushed dolomite aggregate was used in different concrete mixtures. Coarse aggregates were obtained from OCI Crusher, Attakah. The maximum nominal size of dolomite was 20mm, a saturated surface dry specific gravity of 2.55 and a percent absorption of 1.96%. Table 3-5 presents the typical sieve analysis results of dolomite. (Along with the ASTM C33 limits for coarse aggregate grading). Sieve analysis test was conducted according to ASTM C136. In order to determine the other properties of the sand, several other tests were also conducted. The results of these tests are presented in Table 3-6.

Table 3-5: Coarse aggregates sieve analysis, % retained cumulative

% Retained			
Sieve size (mm)	Dolomite Size 1	Dolomite Size 2	ASTM Limits
	0	0	
20.00	0	19.4	0 - 10
14.00	3	72.2	-
10.00	42.7	87.3	40 - 70
5.00	93.9	96.4	90 - 100
2.36	97.2	97.2	-
0.075	0.2	0.2	99 - 100

Table 3-6: Typical results of standard testing of the coarse aggregates used

Test	Standards	Property	Dolomite Size 1	Dolomite Size 2	ASTM C33 Limits
Materials Finer Than 75μm (Sieve No. 200)	ASTM C117	% of Materials Finer Than 75μm	0.8 %	**0.8%**	1%
Specific Gravity and Absorption of Coarse Aggregate	ASTM C127	Bulk S.G	2.570	**2.572**	-
		Absorption	1.96%	**1.88%**	-
Clay lumps & Friable Materials	ASTM C - 142	Clay Lumps & Friable	0.07%	**0.05%**	5%
Chemical Analysis	BS 812 – Part	Chlorides (CL)	0.021%	**0.020%**	-
		Sulphates	0.28%	**0.25%**	-
Resistance to Abrasion (LAA)	ASTM C131	Percent loss	19.5%	19.5%	50%

Figure 3-1: Coarse Aggregates size 1

3.2.4 Admixtures

Two admixtures were used for different grades of concrete. The first is a superplasticizer which has a commercial name Sikament R2004 and complies with ASTM C494 Type G. It provides the following properties; a superplasticizer and a high range water reducer. It has a density of 1.195 kg/l at 20^{o}. The Second admixture used is Plastiment RX SRL and complies also with ASTM C494 Type A. It has the following advantages; a water reducer and increases the workability and strength of the concrete. It has a density of 1.155 kg/l at 20^{o}.

Figure 3-2: Admixtures used in this work

3.2.5 Grout

The grout was acquired from SIKA. It complies with ASTM C 1107. Grout was used due to its adhering nature ensuring monolithic bond with concrete surface. Its compressive strength can reach up to 60 MPa. The mortar is obtained by mixing tap water with grout, approximately 2.5 L per each 25 kg.

Figure 3-3: Grout

3.2.6 Silica Fume

The silica fume was acquired from SIKA. It had a bulk density of 0.5 kg/l, particle size of 0.15 μm and a specific surface 20 m^2/gm.

3.2.7 Mixing and Curing Water

Clean Potable water was used for washing aggregates and process of mixing of concrete.

3.2.8 Reinforcement Steel

Steel rebars with diameter 8 mm was used for the longitudinal steel bars and 6 mm for the stirrups. The steel was produced by Egyptian steel. The steel had a specific gravity of 7.85 and modulus of elasticity of 220 GPa. The steel is mild steel which is known to have a yield stress of 240 MPa and elongation at fracture was 20%. The steel complies with ASTM standards A615-79.

3.2.9 Structural Steel

Structural steel was used to manufacture the strengthening steel cage. The steel was of grade 37 which have the following properties. The steel had a specific gravity of 7.85. The yield stress was 240 MPa and ultimate stress equals 360 MPa, and the modulus of elasticity was 220 GPa.

3.2.10 Strain Gauges

Two strain gauges were connected at the mid height of the column in order to measure the strain with load progression

3.2.11 LVDT

In order to measure the displacement in the concrete and steel cage, a linear variable differential transformer was connected at the top of the tested specimen.

3.2.12 Mixture Proportioning

Concrete mixtures had w/c of ranging between 0.35 to 0.55 and cement content ranging between 350 to 430 kg/m^3. Only 1 mix grout. Figure 3-2 illustrates the mixtures used in this study.

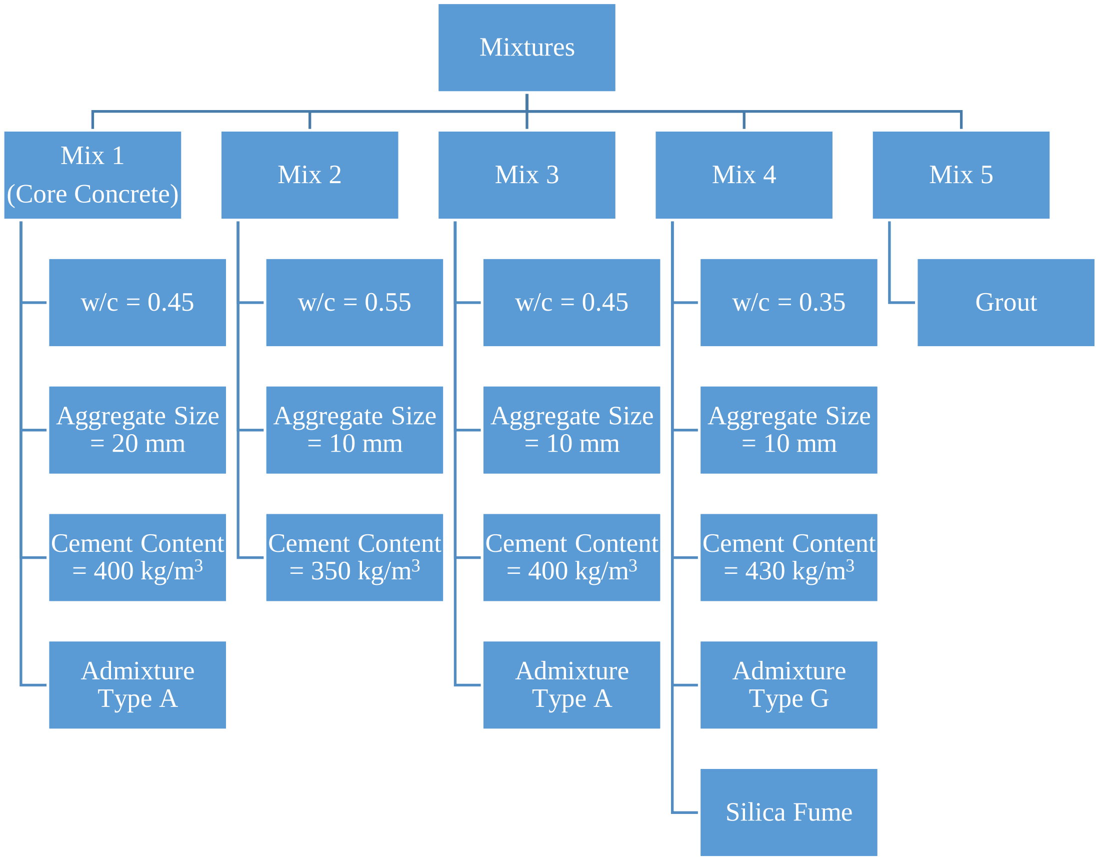

Figure 3-4: Diagram illustrates the mixes used in this study

The First mix of concrete is used for the core columns while the other four are used as a filling concrete between the core columns and the steel jacket.

The constituents of the sets of concrete were as follows:

Table 3-7: Mix constituents used in this work

Material kg/m^3	Mix 1	Mix 2	Mix 3	Mix 4	Mix 5
Cement	400	350	400	430	-
Aggregate Size 1 (10mm)	-	1135	1125	1080	-
Aggregate Size 2 (20mm)	1125	-	-	-	-

Fine Aggregate	625	630	625	600	-
Water	180	192	180	170	-
Admixture Type A (Plastiment)	2.4 L	-	2.4L	-	-
Admixture Type G (Sikament)	-	-	-	10L	-
Silica Fume	-	-	-	50	-

3.3 Equipment

The testing machine consists of a hydraulic jack, loading frame and a strong floor. A steel base was manufactured with different slots so that both the control and jacketed specimens can fit in. The loading frame was made up from 2 steel columns and a stiffened steel beam.

Figure 3-5: Equipment used to test the jacketed columns

3.4 Data Acquisition System

The data acquisition is the process of measuring the voltage and digitizing the analog signals so that the computer can interpret them. It consists of the following parts: a load cell, strain gauges and LVDT, control unit and a computer software.

3.4.1 Load Cell

2 load cells were used in testing the specimens. The capacity of the first is 2000 kN and was used to test the 2 reference specimens. A larger load cell of capacity 3000 kN was used to test the jacketed specimens. Pumps and regulators were used to adjust the load increments to avoid premature failure of concrete.

3.4.2 Control Unit

The Japanese "TMR 211 "control unit was used in the experimental work. It acts as an interface between the computer and the signals produced by the load cells, strain gauges and LVDT. The voltage is measured at a suitable predefined rate. The control unit changes the voltage into a digital form that can be read by a computer. The control unit is connected to a laptop using USB port.

3.4.3 Computer Software

The software TMR 211 was used to process, visualize and store the data.

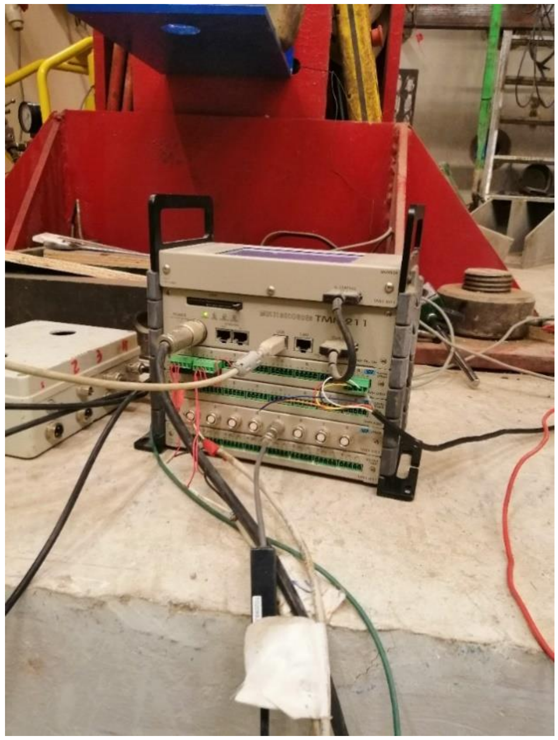

Figure 3-6: Data Acquisition Control Unit

3.5 Experimental Work

3.5.1 Specimen Preparation

Ten columns were prepared with the following dimensions (150 x 150 x 1250 mm) to simulate the core concrete that need repair/strengthening. The core concrete columns were casted using concrete mix 1. The concrete was reinforced with four steel bars 8mm at the corners and stirrups 6 mm each 200 mm. Eight out of the ten specimens were jacketed using concrete filled steel jackets with dimensions (240 x 240 x 1250mm), while two columns were left as control specimens. The jackets were filled with concrete mixes 2-5; two columns from each mix as shown in Table 3-8.

3.5.2 Steel Jacket

Eight Steel jackets were utilized in the experimental work. The steel cages consisted of 4 steel angles 30 x 3mm and welded steel strips with dimensions 40 x 3mm. The clear spacing between the strips was 26.5 cm to prevent the buckling of the steel angles and increase concrete confinement.

Table 3-8: Filling concrete options for different columns

Specimen	Dimensions (mm)	Core Concrete	Filling Concrete
C11-C12	150 x 150 x1250	Mix 1	No jacket
C21-C22	240 x 240 x1250	Mix 1	Mix 2
C31-C32	240 x 240 x1250	Mix 1	Mix 3
C41-C42	240 x 240 x1250	Mix 1	Mix 4
C51-C52	240 x 240 x1250	Mix 1	Mix 5

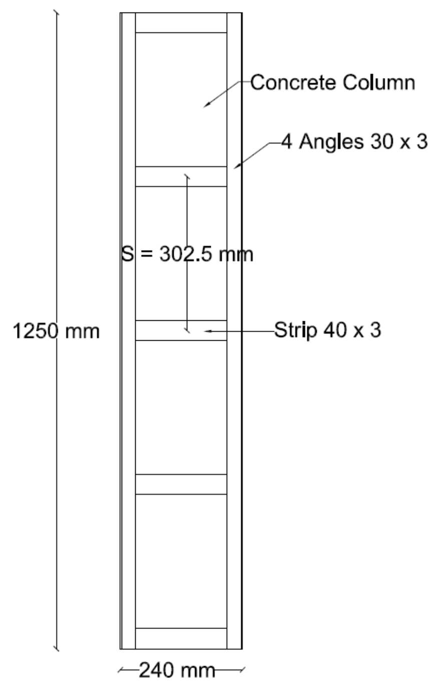

Figure 3-7: Diagram shows the dimensions of the steel cage

Figure 3-8: Steel Cage

3.5.3 Casting

- Ten steel meshes were prepared for the core concrete columns. The steel mesh consisted of 4 ϕ 8 longitudinal bars at the corners and stirrups ϕ 6 @ 0.2m.
- Four wooden forms were prepared as moulds for the concrete columns.
- The steel mesh was inserted in a wooden mould of inner dimensions (0.15 x 0.15 x 1.25m). Afterwards, the concrete was mixed using a 0.11m^3 mixer and poured in the moulds. The volume of the concrete columns was about 0.05m3 so two columns were casted in each batch.
- The concrete was consolidated using a vibrator to ensure the filling of all gaps and having a smooth concrete surface.

- After three days the moulds were removed and the other four columns were casted using the same procedure.
- After 28 days, 8 wooden forms were prepared but with different dimensions (0.24 x 0.24 x 1.25m).
- The steel cages were inserted inside the wooden forms as well as the steel mesh of the filling concrete. The steel mesh of the filling concrete consisted of 8 ϕ 8 longitudinal bars and stirrups ϕ 6 each 0.2m.
- The filling concrete is then poured following the same procedure as the normal core concrete. (2 columns for each mix).
- Two strain gauges are placed at the mid height of the columns on 2 perpendicular faces and 1 LVDT on the top of the surface of the concrete.

Figure 3-9: Wooden forms

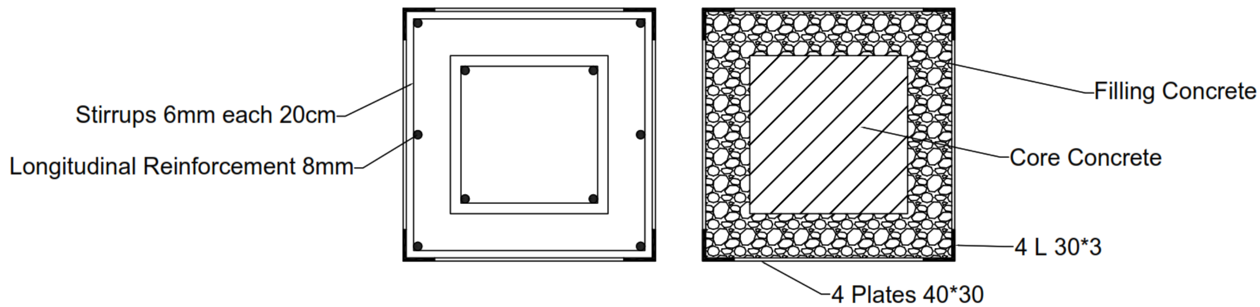

Figure 3-10: Cross section and reinforcement of the specimen after casting

3.5.4 Hardened Concrete Testing

Compressive strength of Concrete Cubes was carried out according to BS standards after 28 days using an “ELE” brand machine of 2000 kN capacity.

3.5.5 Testing of Concrete Columns

Compressive strength of jacketed concrete columns and the control specimens was tested after 28 days using the above mentioned equipment. A steel plate with thickness 20mm was placed over the specimen to distribute the stresses.

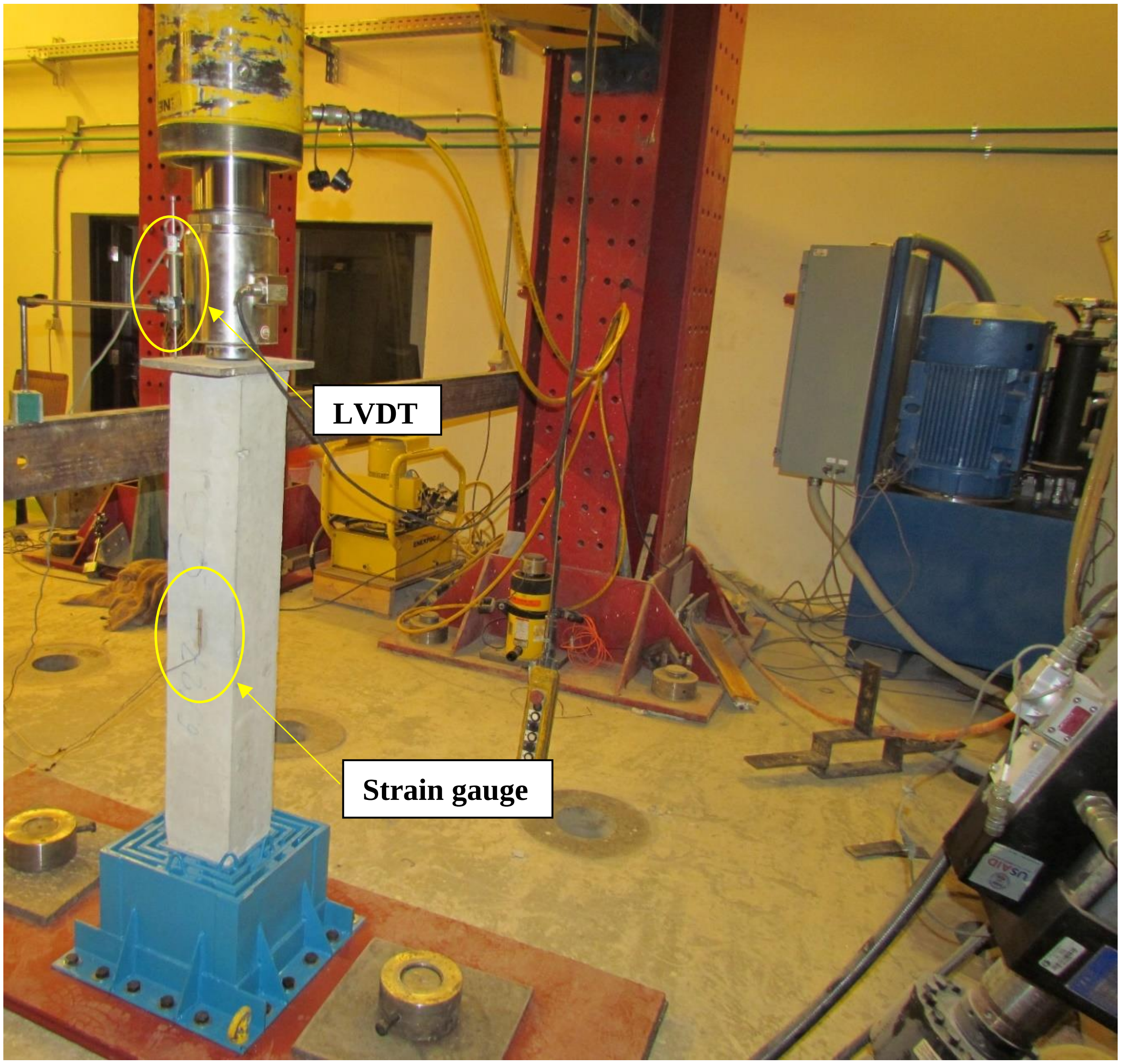

Figure 3-11: Compressive Strength Test Setup

Chapter 4 RESULTS AND DISCUSSION

In this section, test results of compressive strength of hardened concrete and failure loads of the ten columns are presented. Results are analyzed to understand the efficiency of the composite jacketing in terms of strength, stiffness and ductility. As well as exploring the most appropriate concrete class to be used with the composite jacketing. Also, in this chapter, the results are compared with several design proposals of steel jacketed concrete columns.

4.1. Properties of Concrete Mixtures

The experimental tests started at 28 days and were completed after 120 days. A concrete time dependent strength has been adopted (Montouri et al., 2009) to evaluate the strength of concrete at the time of testing the specimens.

$$Fcu(t) = Fcu\,(28) \times \exp\,(0.38 \times (1 - (28/t)^{0.5}))$$

[Equation 3.1]

Where Fc (28) is the cubic strength after 28 days, t is the time in days. By means of this equation the concrete strength was obtained at 28 days. Also, from the same equation the strength of the concrete at the days of testing of column specimens was evaluated. These values are shown in the Table 4.1 and 4.2.

Table 4-1: Compressive strength of concrete mixtures at 28 days

Concrete Mix	Day of test	Fcu(t) (MPa)	Fcu(28) (MPa)
Mix 1 (No jacket)	28	37	37
Mix 2	56	23.7	18.2
Mix 3	56	44	33.6
Mix 4	56	46.9	35.9
Mix 5 (Grout)	28	46.9	46.9

Table 4-2: Compressive strength of concrete specimens at time of testing

Concrete Mix	Day of test	Fcu (t)
Mix 1 (No jacket)	28	37
Mix 2	56	23.7
Mix 3	56	44
Mix 4	90	49.2
Mix 5 (Grout)	120	65.4

It is worth to mention that Equation 3.1 might me a little bit exaggerating. The compressive strength of concrete at 56 days is about 30% more than its compressive strength at 28 days. The concrete is known to gain almost 90% of its final strength at 28 days. However, adopting this equation does not have a significant influence on the results. Mixes one, two and three are tested at the same day of testing the concrete cubes. For mix number 4, due to the exponential equation the compressive strength at 90 days compared to 56 days also seems reasonable. The strength of the mix five (grout) might be questionable as the cubes were tested at 28 days and the columns were tested after 120 days. However, the strength of the grout in all cases will be higher than mix four but still can be less than the calculated value in Table 4-2.

4.2. Load Bearing Capacity

The failure loads of the column specimens are shown in Table 4.3. Generally, the loads of the strengthened columns are much higher than those of the reference columns. This cannot be attributed only to section enlargement but also to the confinement provided by steel angles and jackets. This shall be discussed in details the next section. Figure 4-1 shows the ratio of the failure load of the specimens compared to the reference columns. It is clear that Group number 5 (grout) yielded the highest capacity (about 20% higher than other groups) due to the high strength of grout and the non-shrinking property of grout. This leads to a better bond between the steel jacket, filling material and the core concrete column. On the other side, the other groups produced comparably equal capacities despite using different classes of concrete jacketing. These analogous results shows that the confinement effects is more effective and functional using lower strength concrete as a filling material. The strengthening effects might be a exaggerating as the cross section after enlargement is 2.5 times the old cross section. The reason behind these dimensions is to leave a suitable space for the concrete jacketing for compaction.

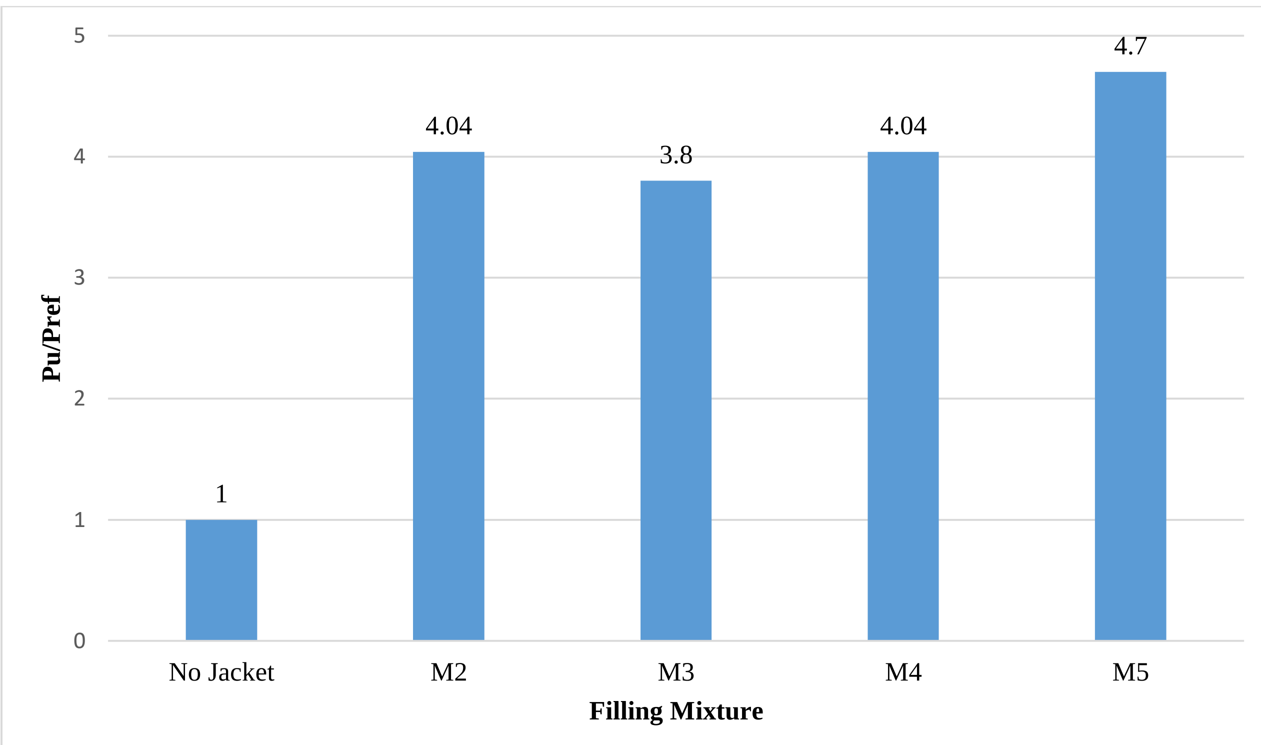

Figure 4-1: Ratio of failure load bearing capacity of strengthened columns to reference columns

4.3. Strength Index

To reduce the exaggeration and focus on the beneficial effects of the concrete filled steel jackets, non-dimensional curves are plotted in which the capacity of each column is compared with the nominal strength of each column. The nominal strength represents the sum of the strength of the material of the column. It can be calculated using the following equation,

$$\text{Nominal Strength} = (A_s \times F_{ys} + A_r \times F_{yr} + 0.85 \times A_c \times F_{cu} + C)$$

[Equation 4]

where, A_s, A_r and A_c are the areas of the steel angles, longitudinal reinforcement bars and concrete jacket respectively, F_{ys} and F_{yr} are the yield stress of the steel angles and longitudinal reinforcement bars respectively, F_{cu} is the cubic strength of the concrete jacket and C is the capacity of the inner concrete column. As shown in Figure 4-2, the beneficial effects of steel jacket are clear. The strength index values range increased by 140% to 210% compared to the reference columns. The confinement is efficient in case of lower strength concrete as it makes full use of the composite action between steel and concrete jackets. It is clear also that as the strength of the concrete jacket increases, the confinement effects becomes almost the same. Although, using grout as a filling material between the inner column and the steel jacket gives the highest capacity, yet it does not make use of the full capacity of the section compared to lower strength concrete. Hence it is thought that it is uneconomic to use grout as a filling material.

Table 4-3: Strength Index for different specimens

Column	Pexp (ton)	Pnom (ton)	SI
C11	37.3	73.6	0.51
C12	36.2	73.6	0.49
C21	143.7	142.9	1.01
C22	153	142.9	1.07
C31	130.9	199.7	0.65
C32	147.7	199.7	0.74
C41	141.2	213.7	0.66
C42	155.8	213.7	0.73

C51	174.5	258.6	0.67
C52	173.1	258.6	0.67

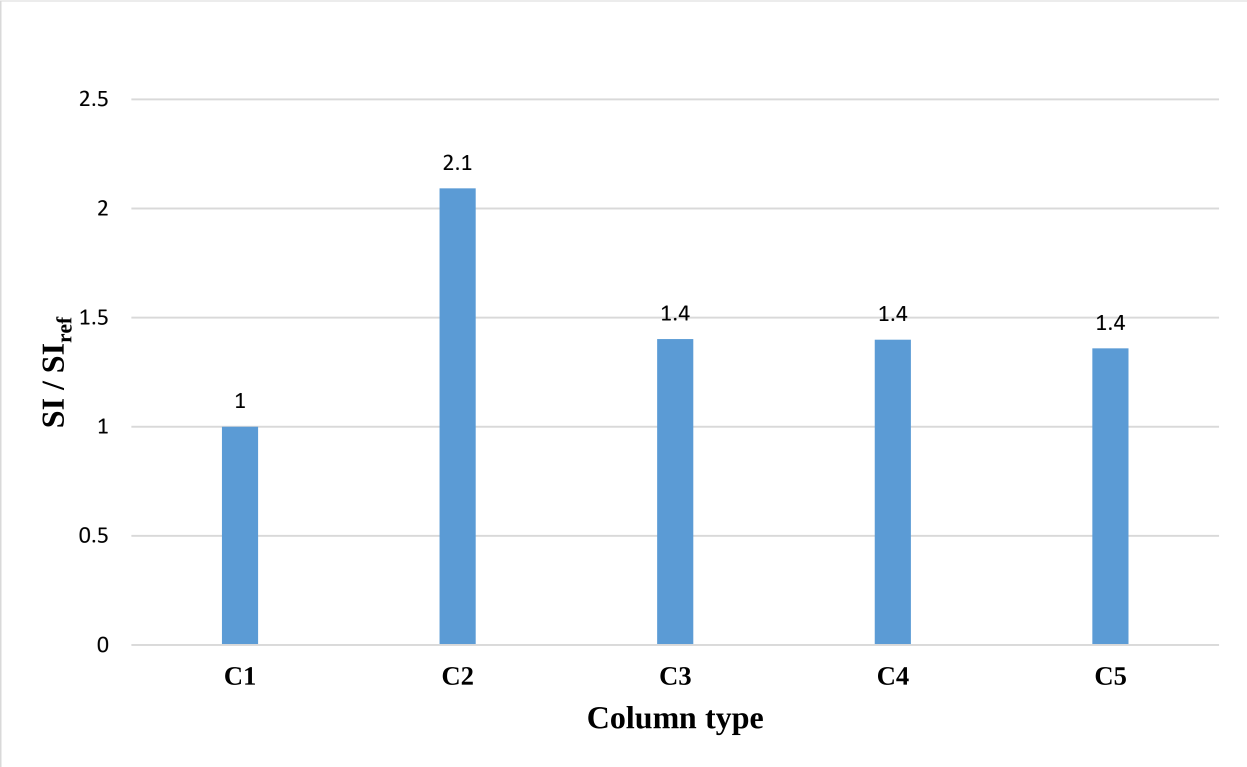

Figure 4-2: Ratio of strength Index of strengthened specimens to reference columns

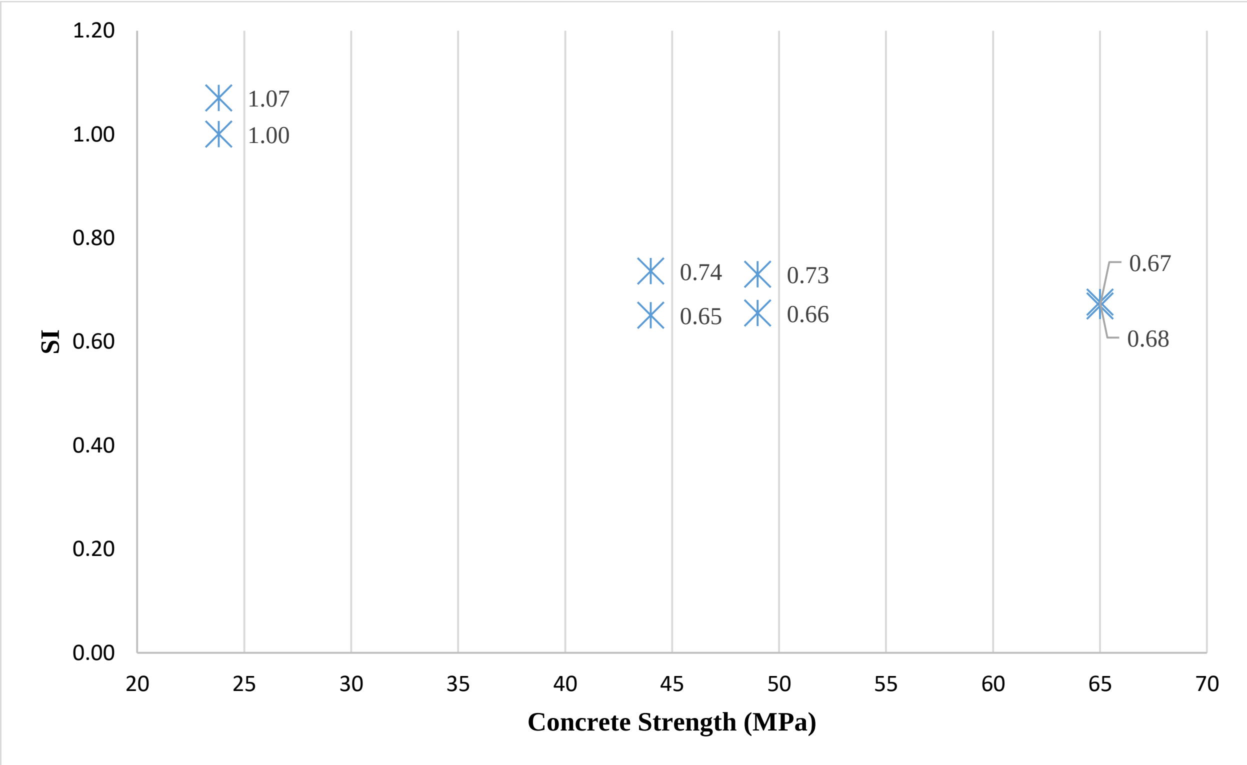

Figure 4-3: Strength Index for different concrete mixtures

4.4. Failure Pattern

For the reference columns, both of them exhibited the same behavior. The axial shortening as well as the stress strain curve increased linearly till a brittle failure happened. The concrete spalled off and buckling of the longitudinal reinforcement bars occurred.

As for the strengthened columns, all the columns showed a significant increase in both strength and ductility as shown in Table 4-4 and Figures 4-4 to 4-13 . The failure load was higher than that of the control specimens and reached up to 460% of its ultimate load.

The failure began with having some cracks on the surface of the concrete. Next the concrete started to expand which led to the bending of the confining steel. All the specimens were characterized by the buckling of the vertical steel angles at failure followed by buckling of the longitudinal steel reinforcement bars. For some specimens, the weld between steel battens and the vertical steel angle was broken near the column head. This happened after the buckling of the angles. The inner concrete columns of specimens 21, 22, 31 and 32 were crushed and the inner longitudinal reinforcement bars buckled. While for the other columns only the concrete jacket spalled off. This indicates that the concrete filled steel jackets can totally change the mode of failure of concrete columns from brittle to ductile failure. This type of failure is beneficial in warning occupants if the columns of the buildings are exceeding the ultimate capacity. The failure patterns are shown in the following figures.

Table 4-4: Load bearing capacity and failure pattern of different specimens

Specimen	Failure Load (ton)	Failure Pattern
C11	37.3	Shear
C12	36.2	Shear
C21	143.7	Buckling of steel angles
C22	153	Buckling of steel angles
C31	130.9	Weld broken
C32	147.7	Buckling of steel angles
C41	141.2	Buckling of steel angles
C42	155.8	Buckling of steel angles
C51	174.5	Weld broken
C52	173.1	Weld broken

Figure 4-4: Failure of column C11

Figure 4-5: Failure of column C12

Figure 4-6: Failure of column C21

Figure 4-7: Failure of column C22

Figure 4-8: Failure of column C31

Figure 4-9: Failure of column C32

Figure 4-10: Failure of column C41

Figure 4-11: Failure of column C42

Figure 4-12: Failure of column C51

Figure 4-13: Failure of column C52

4.5. Stress – Strain Behavior

The relationship between the load and the column axial strain of the five specimens are presented in Figures 4-4 to 4-8. The curves show that the ductility of the strengthened columns is significantly greater than that of the reference column. The ductility here is defined as the maximum strain that the specimen can attain at failure. The results also reveal that ductility increase with increasing the compressive strength of then concrete jacket except for group 3. This can be explained due to breaking of the weld between the angle and the steel strips of specimen C31. Correspondingly, the values of the secant modulus at failure are decreasing as the concrete strength increase. The secant modulus better represents the behavior of the specimens at failure than the tangent modulus. The results are listed in Table 4-5. The table also shows that all the strengthened columns can bear higher stresses than the reference columns, up to 80% increase in the maximum stress. As shown in Figure 4-14, the strain of the reference column at failure is about 0.0012. Commonly, the concrete maximum compressive strain is 0.003. This discrepancy can explain why the reference column maximum capacity was almost 50% of the expected capacity.

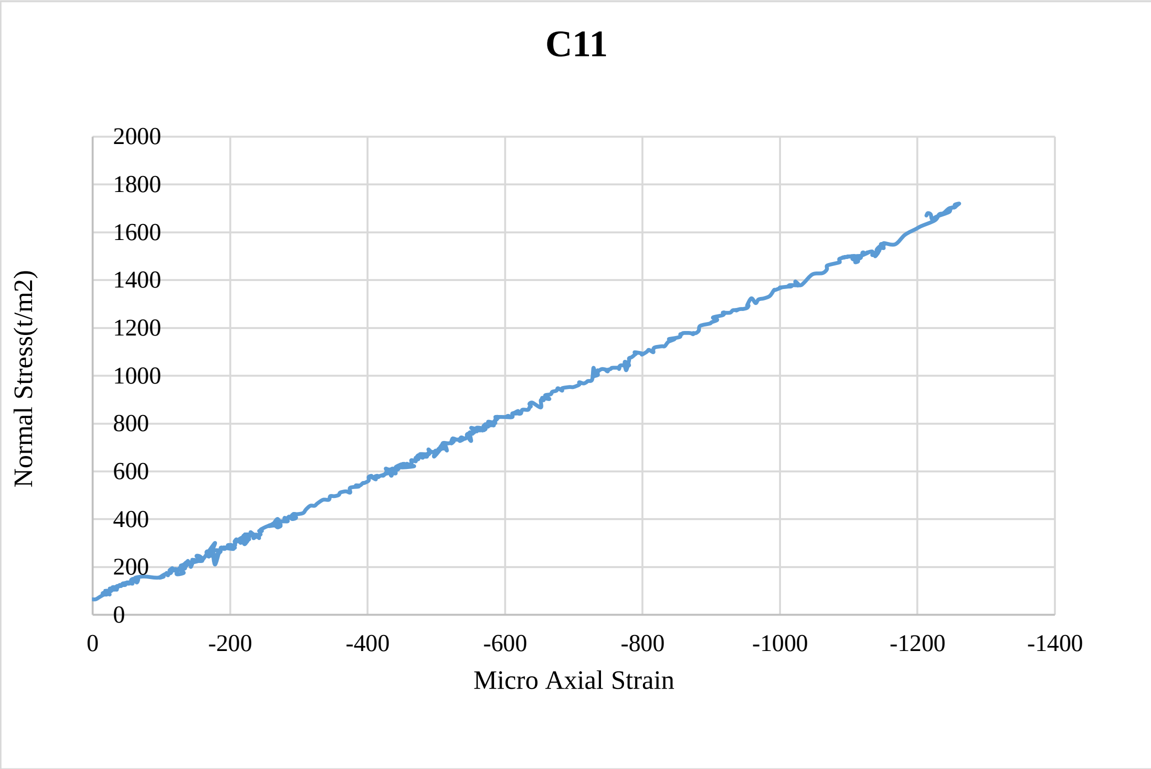

Figure 4-14: Stress – strain curve of specimen C11

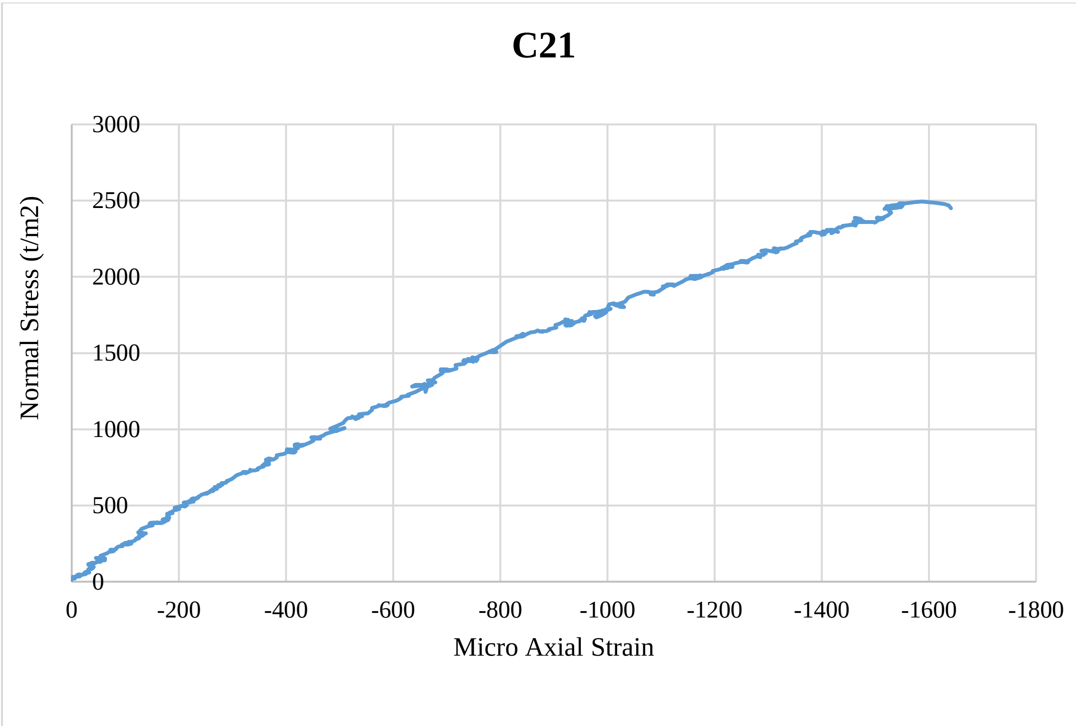

Figure 4-15: Stress – strain curve of specimen C21

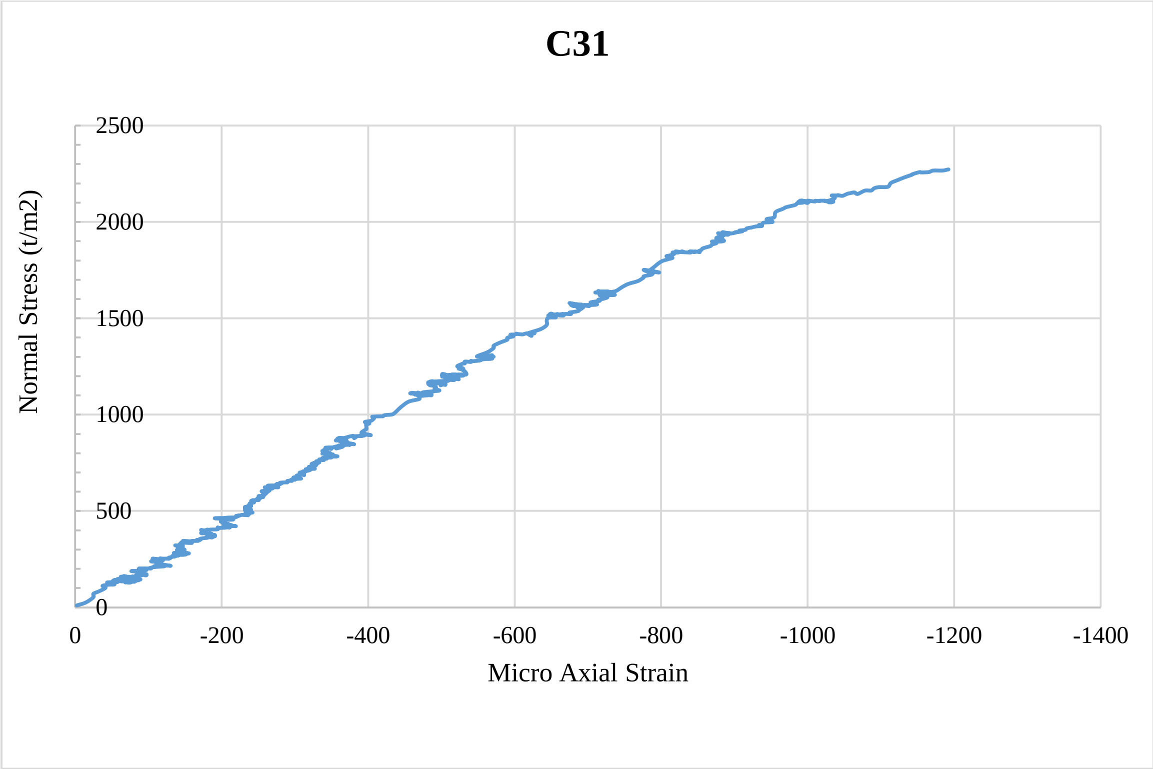

Figure 4-16: Stress – strain curve of specimen C31

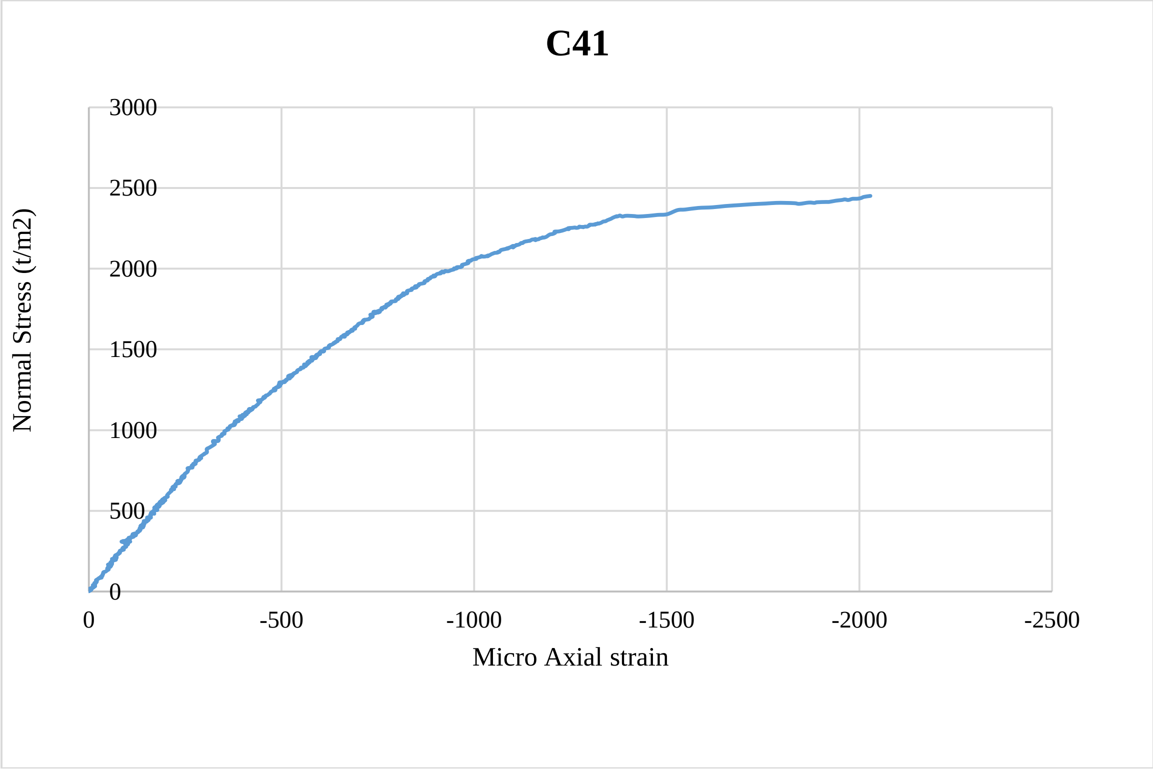

Figure 4-17: Stress – strain curve of specimen C41

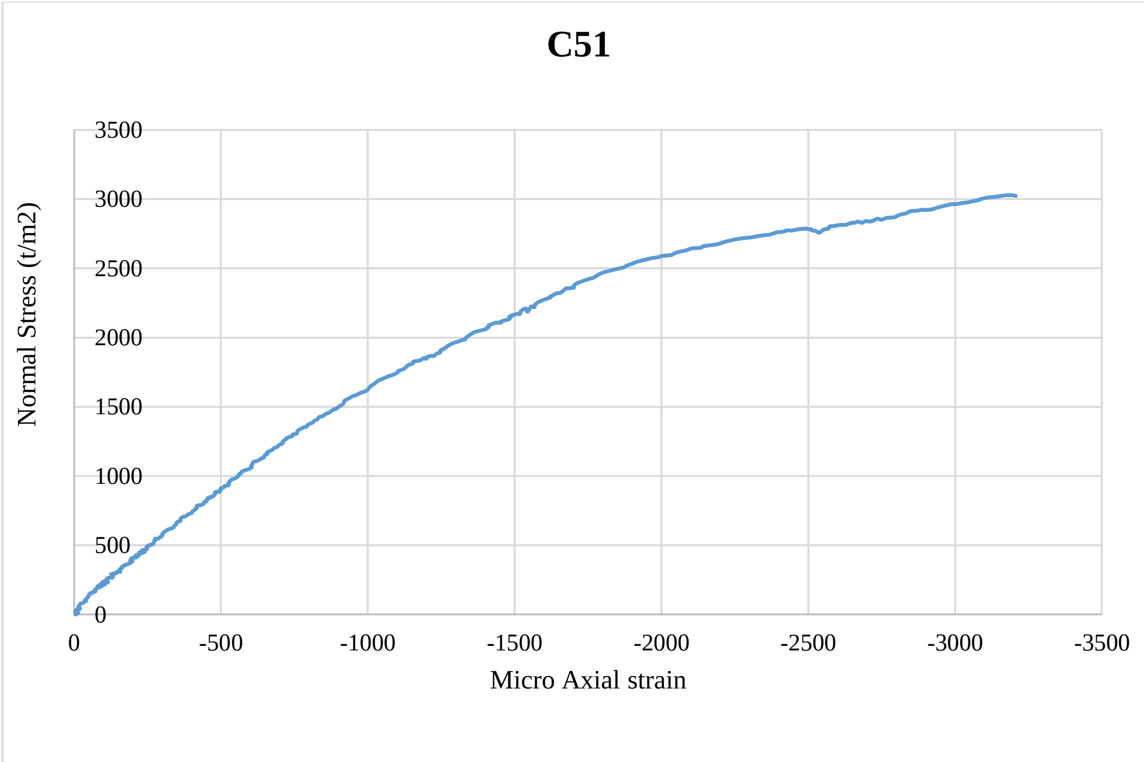

Figure 4-18: Stress – strain curve of specimen C51

Table 4-5: Maximum stress and strain values for different specimens

Specimen	Max. Stress (kN/m^2)	Corresponding strain (µstrain)	Secant Modulus (GPa)
C11	1670	1213	1.37
C21	2450	1641	1.49
C31	2272	1192	1.9
C41	2450	2028	1.2
C51	3023	3206	0.94

4.6. Load – Axial Deformation Behavior

The relationship between the load and the column axial shortening of the 10 specimens are presented in Figures 4-19 to 4-28.

Generally, the vertical axial deformations are greater in the strengthened columns than the reference columns. Neglecting the seating deformations, the load-deformation curves exhibited a linear behavior for almost all the strengthened specimens. The stiffness of the columns was determined using the linear portion of the load deformation curve to avoid the influence of the flat part at the beginning of the curve associated with the seating of the specimen. The stiffness values are listed in Table 4-6. As clear in Figures 4-9 to 4-18, the stiffness of the strengthened columns is much higher than the reference column. It reaches up to twice the value of the reference columns. This is not surprising due to the effects of the section enlargement. The axial shortening of the strengthened columns increased by 240 to 345% despite the increase in the columns stiffness. The results reveals the role of concrete filled steel jackets in increasing the deformability of concrete columns. Group 5 recorded the highest axial shortening values compared to other specimens. It confirms that the ductility of the concrete filled steel jacketed columns increases with the increase of the concrete jacket class. On the other side, the stiffness of the columns increases with the increase of the concrete strength as expected however, the stiffness of Group 5 showed a reduction by almost 10%. The increase in the elastic stiffness leads to less deformation at working loads. So, for the working load stage, using concrete with silica fume and can yield better results than using grout. Although, as previously concluded at the ultimate load stage, grout can bear higher stresses with higher ductility. The axial strain for the reference column C11 can be calculated from Figure 4-19 by dividing the axial deformation by the total length of the column. The axial strain is about 0.0024. The discrepancy between the maximum reached strains from Figures 4-14 and 4-19 can be explained due to the seating deformation associated with compression loading of the columns which is noticed all of the specimens. To avoid the effect of the premature failure of the reference columns and stay clear from any misconceptions, the capacity of the columns was considered to be as constant in calculating the nominal capacity of the jacketed columns. Accordingly, this was also taken into consideration when the strength index was calculated. Hence, the failure load of the inner column has no effect when the nominal capacities of the jackets are compared to each other. This implies on the main objective of this study which is investigating the influence of different filling materials on the performance of the composite jacketing.

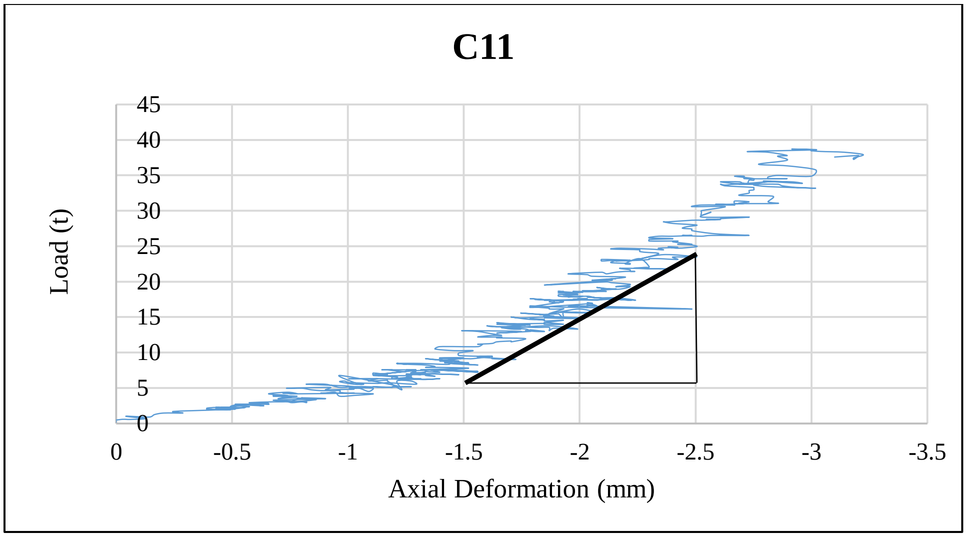

Figure 4-19: Axial Load Deformation Curve of Specimen C11

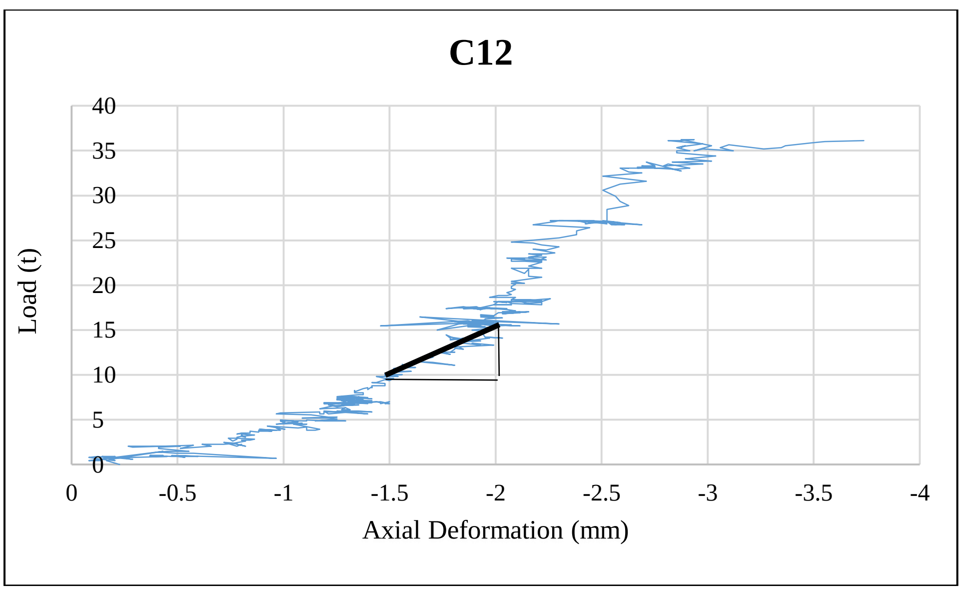

Figure 4-20: Axial Load Deformation Curve of Specimen C12

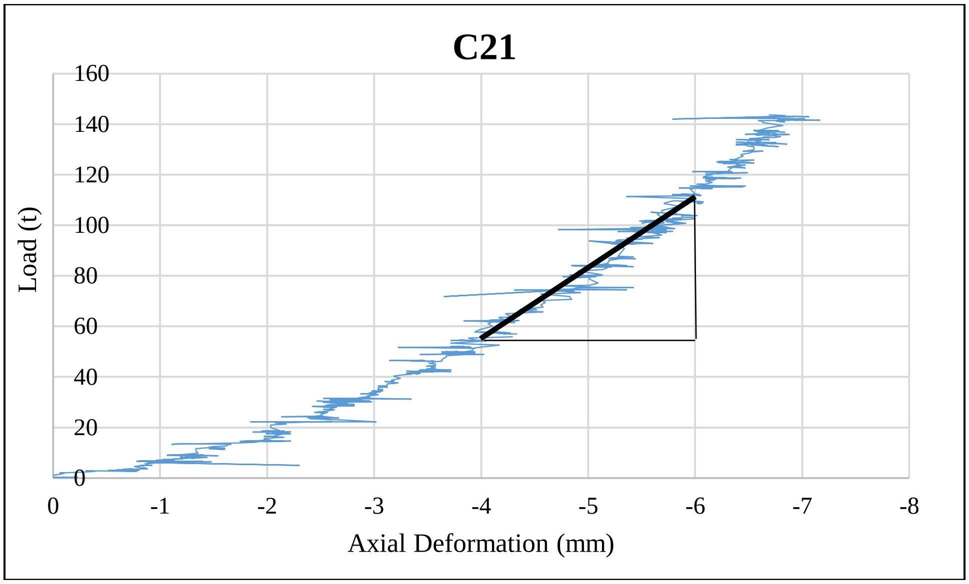

Figure 4-21: Axial Load Deformation Curve of Specimen C21

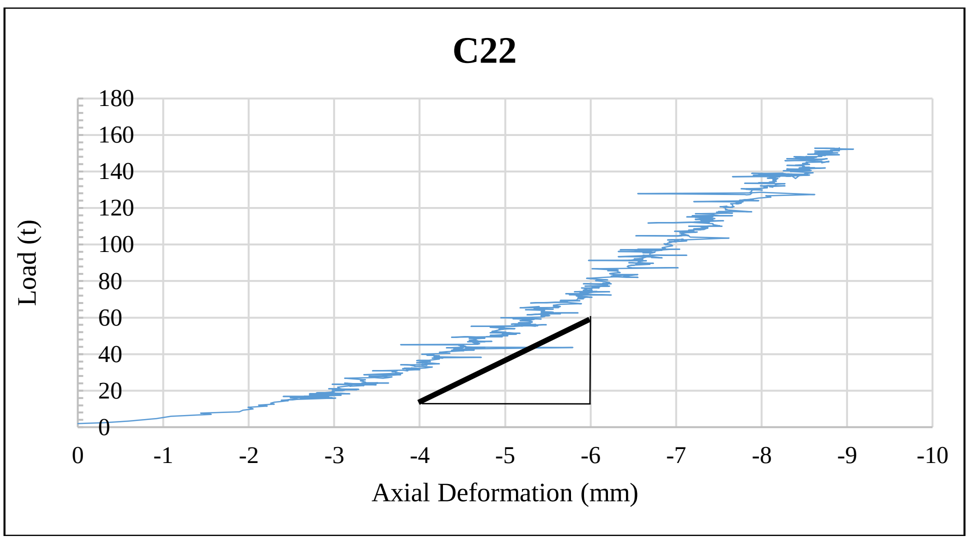

Figure 4-22: Axial Load Deformation Curve of Specimen C22

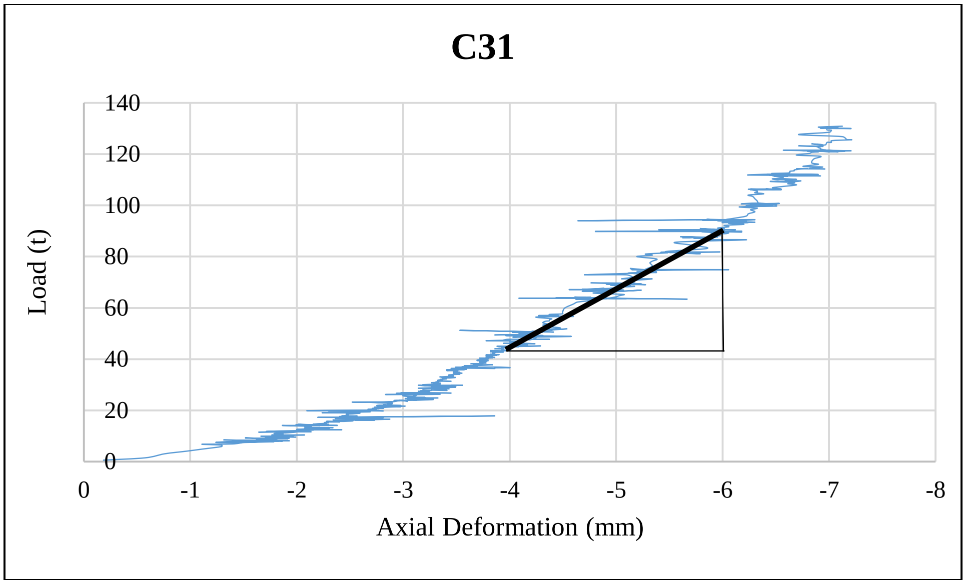

Figure 4-23: Axial Load Deformation Curve of Specimen C31

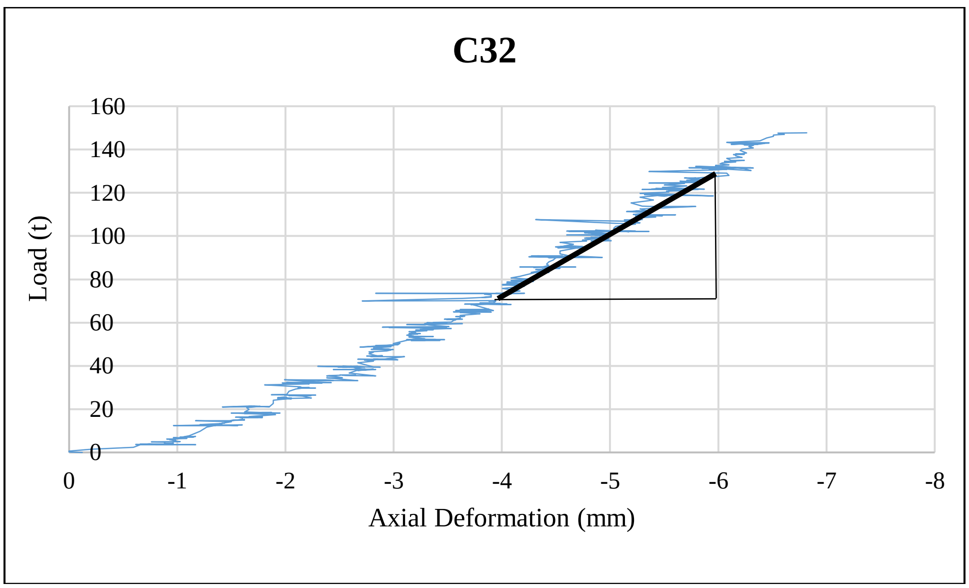

Figure 4-24: Axial Load Deformation Curve of Specimen C32

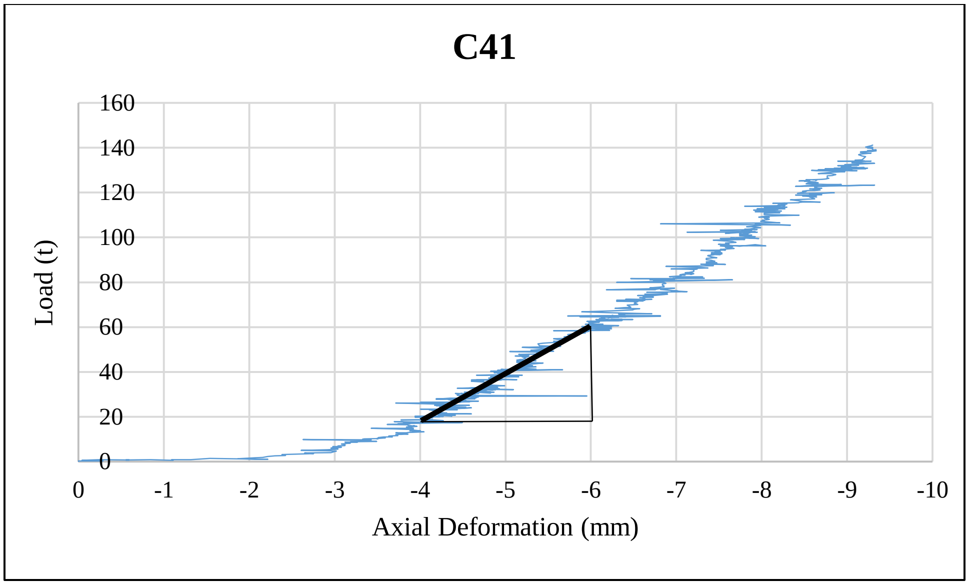

Figure 4-25: Axial Load Deformation Curve of Specimen C41

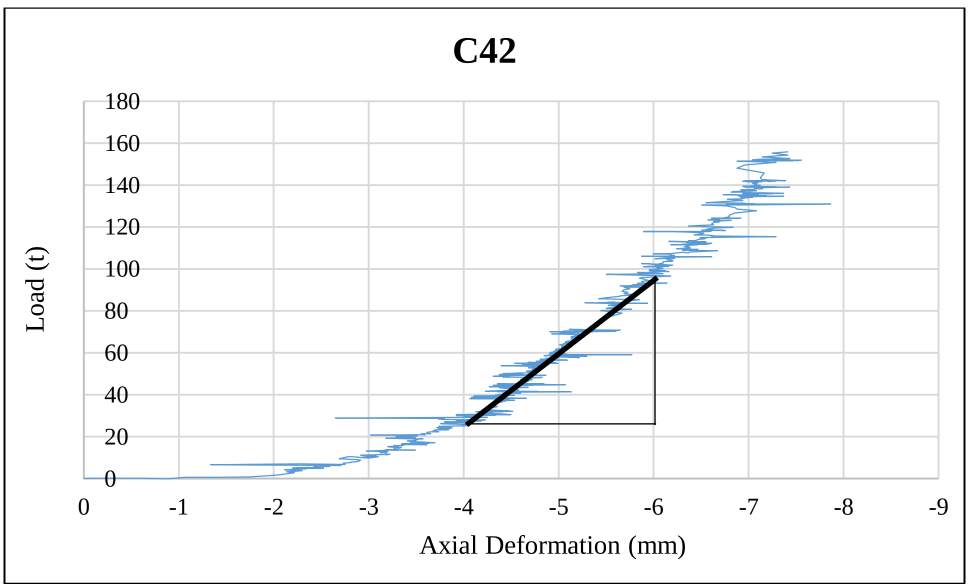

Figure 4-26: Axial Load Deformation Curve of Specimen C42

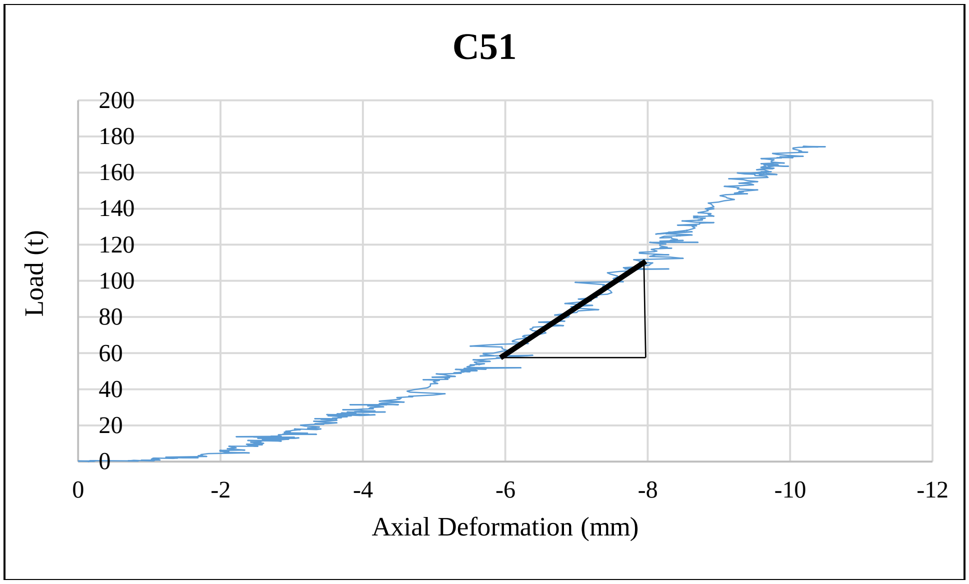

Figure 4-27: Axial Load Deformation Curve of Specimen C51

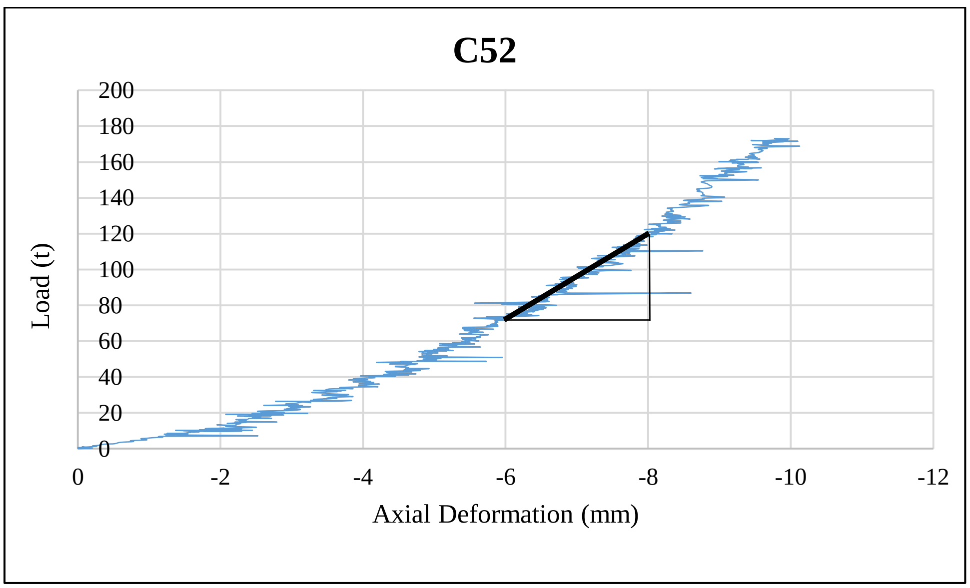

Figure 4-28: Axial Load Deformation Curve of Specimen C52

Table 4-6: Linear stiffness values of different specimens

Specimen	Stiffness (t/mm)
C11	16.5
C12	10
C21	23
C22	20
C31	23
C32	28
C41	21
C42	35
C51	26
C52	24

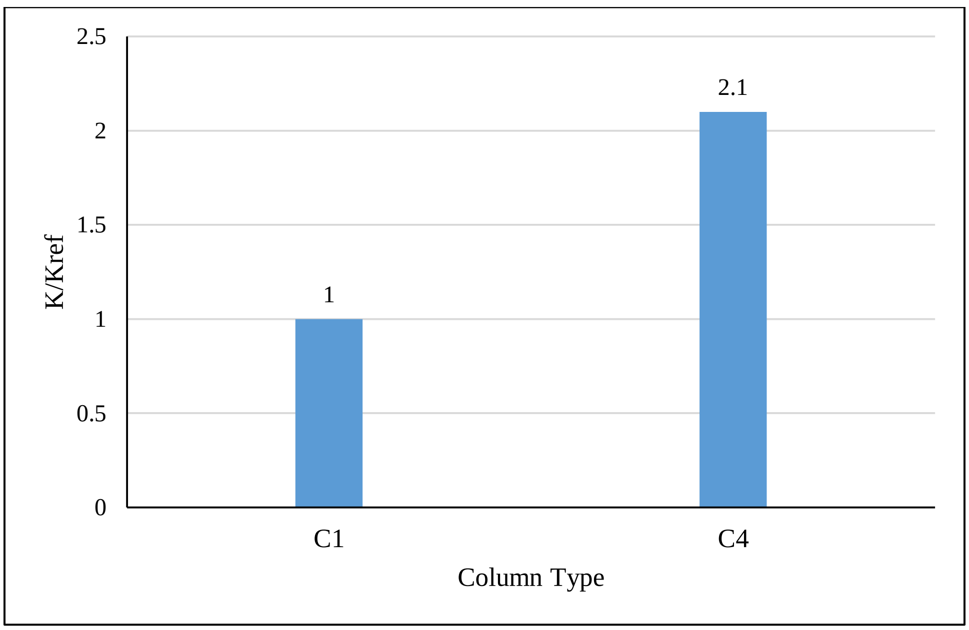

Figure 4-29: Ratio of stiffness of strengthened columns to reference column

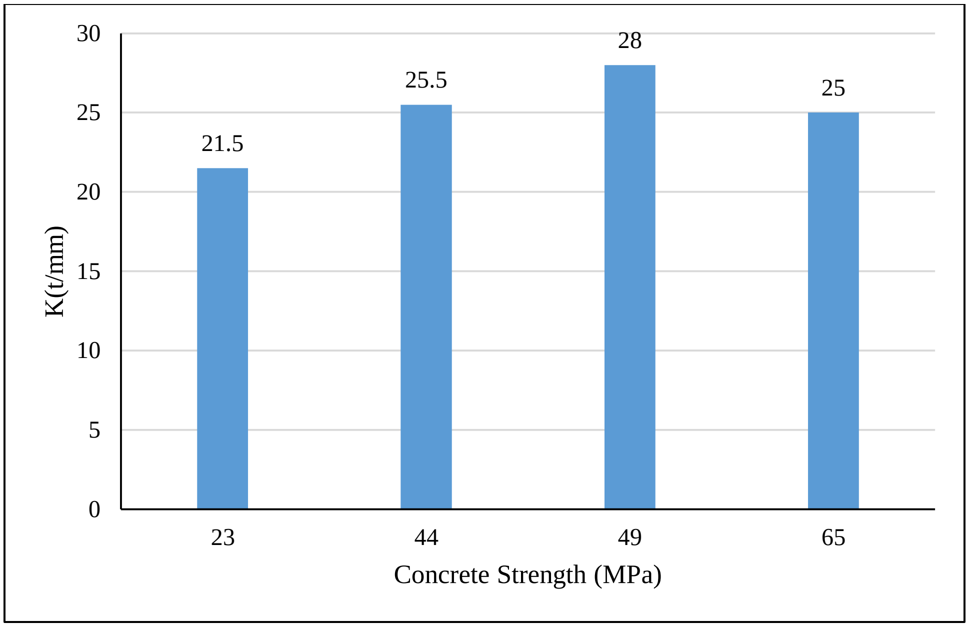

Figure 4-30: Stiffness of specimens for different concrete strengths

4.7. Design Proposals

As highlighted in the literature, there are several methods for the design of the steel jacketed columns. Referring to equation 2.2 and 2.3, the ultimate loads of the jacketed columns were compared with the design loads of the EC4 and Regaldo equations as they represent the upper and lower bounds of the design proposals. The EC4 equation considers the jacketed column to act as a composite section. While, Regaldo reduces the capacity by a constant factor taking into consideration the incompatibility in the deformations between the steel jacket and concrete. The results are summarized in Table 4-7.

Table 4-7: Comparison of ultimate load with EC4 and Regaldo

Column	Pexp (ton)	EC4 (ton)	Regalado (ton)	Pu/P_{EC4}	Pu/P_{Reg}
C21	143.7	142.9	85.7	1.01	1.68
C22	153	142.9	85.7	1.07	1.79
C31	130.9	199.7	119.9	0.65	1.09
C32	147.7	199.7	119.9	0.74	1.23

C41	141.2	213.7	128.2	0.66	1.10
C42	155.8	213.7	128.2	0.73	1.22
C51	174.5	258.6	155.1	0.67	1.13
C52	173.1	258.6	155.1	0.67	1.12

From the above table it is clear, that generally the EC4 method is overestimating the capacity of the jacketed columns. On the other hand, Regaldo equation underestimates it. However, for the lower strength filling concrete the EC4 equation yields better results and the columns behave almost as a composite section and . While, for higher strength concrete there is a noticeable reduction in the capacities of the composite section. Hence, design using the EC4 equation is non conservative and Regalado's equation represents more efficient and safer solution.

Chapter 5 CONCLUSIONS AND RECOMMENDATIONS

In this chapter, the overall summary of the study and conclusions are presented as well as the recommendations for future work are provided.

5.1. Conclusions

In light of scope, material, equipment and other parameters and variables associated with this study, the following can be considered as the most important findings of this study:

1. The ultimate loads of the strengthened columns are much higher than those of the reference columns i.e. unstrengthened columns.
2. Using grout as a filling material produces the highest ultimate load for the tested groups of columns. On the other side, using various filling materials with different compressive strengths produced almost the same failure load. This is not only attributed to high strength but also to the non-shrinking property of grout leading to better adhesion between the different parts of the composite jacket,
3. The strength index values increased by 140% to 210% for different groups of strengthened columns. The strength index tends to be higher for lower strength filling concrete and almost unchanged for higher strength concrete and grout.
4. The concrete filled steel jackets change the mode of failure of concrete columns from brittle to ductile failure. Such failures are beneficial in warning occupants if the columns of the buildings are exceeding the ultimate capacity.
5. The ductility of the strengthened columns showed to be significantly greater than that of the reference column.
6. The results reveal that ductility is clearly affected by the compressive strength of the filling concrete jacket. This was shown as it increases with the increase of the compressive strength.

7. The values of the secant elastic modulus at failure are decreasing as the concrete strength increase indicating higher ductility.
8. All the strengthened columns can bear higher stresses than the reference columns, up to 80% increase in the maximum stress.
9. The load-deformation curves exhibited almost a linear behavior for almost all the strengthened specimens.
10. The results reveal that stiffness of the jacketed columns is clearly affected by the compressive strength of the filling concrete jacket. It increases with increasing the compressive strength of the filling concrete. This finding is not valid for the grout.
11. For the working load stage, using concrete with silica fume and can yield better results than using grout as it produces the highest stiffness i.e. the least deformations.
12. For the ultimate load stage, grout can bear higher stresses accompanied with higher ductility.
13. Design using the EC4 equation is non conservative and Regalado's equation represents a better and safer solution.
14. Using lower strength filling concrete drives the jacketed column to behave as a composite section with its maximum capacity.
15. The steel jacketed RC columns using filling material represents a midway solution between area and economy in strengthening techniques. Achieving the same increase in strength and deformability using only concrete jacketing requires enormous space and using only steel jacketing entails high costs.

5.2. Recommendations for Future Work

Similar to other research work, further investigations need to be conducted to cover the following:

1. Wider set of concrete mixtures need to be examined to confirm the findings of this study.

2. The influence of preloading on the behavior of the concrete filled steel jackets needs to be further examined.

3. It is recommended to experiment on other thicknesses of the filling material as well as the steel jacket and monitor the performance of the composite jacketing.

4. The bond between concrete and steel should be thoroughly studied with various techniques such as using epoxy or dowels and observe the change in the confinement action of the RC columns.

5. The durability of the filling concrete should be investigated to be able to judge the economy of the composite jacket compared to other alternatives.

6. The performance of the composite jacket can be examined under eccentric and lateral loads.

5.3. Recommendations for Applicators

The execution of steel jacketed RC columns using filling concrete is fundamentally different than conventional steel jacketed columns. The following recommendations are provided for the application of this technique.

1. Applicators must be aware that the behavior of the jacket depends primarily on the finishing of the filling concrete as it acts as the interface between the steel jacket and the core concrete. Improper finishing of concrete may result in adverse results.

2. It is recommended to use grout or concrete with expansive agent as a filling material to yield the highest possible strength and ductility. These filling materials are expected to produce better adhesion and bond between the parts of the composite jacket.

3. The spacing between the steel and concrete jacket shall be carefully chosen based on the type

of the filling material and nominal size of aggregates to avoid segregation

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