Abstract- The most general use of high-strength concrete is for construction of high-rise buildings. High strength concrete has highly advantageous structural properties which can lead to significant cost and also time savings in heavily loaded concrete structures. In this paper a methodology is proposed for deriving a non-linear finite element model to analyse the behaviour of high strength concrete columns wrapped with carbon-fibre-reinforced-polymer (CFRP) jackets and reinforced with axial and helical steel bars. In the finite element analysis, the material and geometric nonlinearities are taken into consideration and the results of material testing of the constituent materials (concrete, steel and FRP) are explored to set up the model. Due to lack of axis of symmetry in this complex model, creating the whole column has had the most precise result among all the alternatives which were examined to achieve an appropriate model as a representative of the true specimens. There was a good correlation between the axial load-deflection results of the experimental tests and the finite element model of the concentrically loaded columns proving the accuracy of the developed model.

Keywords- High Strength Concrete (HSC), Fibre Reinforced Polymer (FRP), ductility, concentric loads, nonlinear analysis, lateral reinforcement, external confinement, finite element method (FEM), axial reinforcement, helical reinforcement.

I. INTRODUCTION

Some reinforced concrete (RC) structures may require increase in their load capacity due to development of new design standards or increase in the applied load. Therefore, some strengthening methods should be prepared for maintaining and upgrading structural elements.

High strength concrete (HSC) is more brittle than normal strength concrete (NSC). Using fibre reinforced polymer (FRP) and helical reinforcement has a considerable effect on improving the ductility and strength of HSC and can provide an efficient confinement for reinforced concrete columns. In this paper, the analysis of FRP wrapped helically reinforced high strength concrete columns under concentric loads are performed by using STRAND7 as the finite element software.

II. HIGH STRENGTH CONCRETE

In the past 20 years, there have been significant developments in the production and use of high strength concrete (HSC). The use of compressive strength as a gauge of concrete strength has been usually applied in concrete structures. Concrete can be generally classified into two categories: Normal Strength Concrete (NSC) and High Strength Concrete (HSC). There are different classifications for high strength concretes [1]; ACI440-02 [2] states that the compressive strength of high strength concrete is above 42 MPa and AS3600 [3] categorized that the concrete with compressive strength of more than 65 MPa is HSC. On the other hand Hadi [4] explains that concrete compressive strength of 50-120 MPa belongs to HSC and concrete with compressive strength of more than 120 MPa will be classified as Ultra High Strength Concrete (UHSC).

A major problem of high strength concrete is lack of ductility which is a very important factor to determine whether a large deformation and deflection on a structure can happen under overload conditions or it will otherwise experience catastrophic collapse [5].

Nawy [6] reveals that lack of ductility can be removed through adequate confinement. Applying FRP (Fibre Reinforced polymer), helical or spiral reinforcement are being found very helpful in an attempt to improve the ductility of this strong but brittle material, without any extensive change in stiffness [7].

III. FRP CONFINEMENT

Lateral confinement of concrete with FRP is an effective technique to enhance the load-carrying capacity and ductility of concrete columns. Confining a concrete member with FRP is achieved by orientating the fibres transverse to the longitudinal axis of the column. In this orientation, the hoop fibres are similar to conventional helical steel [2]. The concrete column shortens under a compressive load, and it will expand laterally due to Poisson’s effect.
This lateral expansion is resisted by the lateral confinement, so that the compressive strength of the concrete is increased.

Fibre Reinforced Polymer (FRP) is a relatively new composite material manufactured from fibres and resins and has proven to be efficient and economical for the development and repair of new and damaged structures in civil engineering and is also adequate for some architectural applications. The innovation of FRP allowed for materials other than steel to be used as confinement. The FRP allows the concrete to be externally wrapped with sheets, tapes or tubes. The most common types of FRP are carbon, glass and Aramid. Glass Fibre Reinforced Polymer (GFRP), Carbon Fibre Reinforced Polymer (CFRP) and many other types of reinforcing fibres in an epoxy, aluminium or polyamide matrix, have been used to retrofit the existing reinforced concrete structural members to enhance their load and displacement capacities, or in new structures to increase their strength, ductility and their resistance against environment [8, 9].

FRP jackets present passive confinement to the concrete structure, remaining unstressed until dilation and cracking of the wrapped compression member happens.

The bond between the FRP and concrete can be provided in a number of ways. Firstly, it may be an epoxy adhesive that provides even adhesion between the FRP and concrete over the entire surface of the column. Secondly, the bond may be provided by mechanical shear connector ribs. This type of bond is usually used to provide “leave in place” formwork for new concrete columns. Finally, in the case of an FRP tube which is filled with concrete, bond is provided by means of the weak natural bond between the two surfaces [10]. Generally in concentrically loaded columns the bond is not as important as beams.

IV. HELICAL REINFORCEMENT

External reinforcement enhances both the ductility and strength of the core concrete and therefore improves the whole behaviour of the column. Lateral reinforcement in the form of ties or helixes can supply lateral support for the main (axial) steel reinforcement and confines the interior concrete. The effect of lateral reinforcement on the behaviour of an axially loaded column is shown in Figure 1. When the load is not considerable, the lateral steel does not influence the load-strain capacity of the column; but, if insufficient lateral steel is provided, premature failure can arise due to external buckling of the axial steel (illustrated by Curve (a) in Figure 1.

If sufficient support for the main steel is provided by lateral reinforcement, the peak force is accomplished. Curve (b) characterizes a typical tied column, in which the concrete crushes and spalls at the initial load. At this level the main steel buckles and the load falls off harshly. For Curve (c) the concrete spalls at and the concrete cover is lost. In spite of having a fall off in load, the concrete core and the helical steel create lateral support for the axial bars. In addition, the strength and ductility of the middle concrete are improved by the confining action of the helical reinforcement. Curve (d) performs in a similar way, except added benefits match to a smaller pitch of helical reinforcement [11].

![Figure 1: Effect of lateral reinforcement](image)

V. EXPERIMENTAL STUDIES

In order to investigate the effect of the layers and the orientation of CFRP confinement and also helical reinforcement on the concrete, twelve columns with the same dimension were constructed and tested by Hadi [12].

The 12 specimens were subdivided into three groups as follows:

- **Group 1** consisted of reinforced concrete columns without any confinement.
- **Group 2** included RC columns with 3-layer horizontal CFRP confinement.
- **Group 3** embraced RC columns with 3-layer horizontal and vertical CFRP.

The concrete columns were 925 mm high and 205 mm in diameter including six N12 (12 mm deformed bars with 500 MPa nominal tensile strength) which were used as longitudinal reinforcement throughout the height of the column and were tied equally spaced around the inside of the helix (spirals).
R10 (10 mm plain bar with 250MPa nominal tensile strength) was used as the lateral reinforcement for the longitudinal reinforcement at a pitch of 60 mm. Lengths of 5 mm diameter bars were welded to the helix and longitudinal bars to ensure the sufficient cover of the concrete (12.5 mm each side). The compressive strength of the concrete was 75 MPa.

In the experimental work of Hadi [12] the uniformly distributed concentric loads were applied to the columns up to failure point and the test undertaken was displacement controlled.

VI. **FINITE ELEMENT MODEL (FEM)**

In this study a finite element model using Strand7 (as FEM software) has been developed to validate the load-deflection behaviour of the axial and helical reinforced high strength concrete columns with and without FRP confinement.

Strand7 is a general-purpose finite element analysis system consisting of a pre-processor, solvers and a post-processor. The graphical environment in Strand7 includes advanced tools for the creation of finite element models, the application of loading and boundary conditions, direct interfaces to popular CAD and solid modelling systems, and automatic mesh generators. Post-processing tools for the investigation of results include deformed displays, contour plots, point-and-click data inspection (peeking) and animation. The built-in report generator simplifies the task of compiling, printing and documenting results. The solver includes basic Linear Static and Linear Buckling Analysis, a range of Dynamic Analysis solvers including direct and mode superposition solvers, advanced Nonlinear Static and Dynamic solvers and both Steady State and Transient Heat solvers [13].

According to the complexity of the structure and variety of materials and geometry, the necessity of providing appropriate data and exploiting innovative meshing method in nonlinear analysis of the structure has been justified. Therefore some investigations have been dedicated to determine a deliberate modelling method which can assure the accuracy of results and provide an optimum prediction for the behaviour of the structure with specific material properties and loading systems.

Initially two fractional models (one sixth section and half height) were examined. As there were no axes of symmetry for helix, constructing that kind of reinforcement in a way that simulate the equivalent performance to the real structure has been found to be too complicated.

There were two choices for modelling the concrete. The first one was relying on stress-strain curve and max-stress yield criterion belonging to Elastic Nonlinear option of the FEM. The main defect of this alternative was neglecting the lateral expansion of the concrete. The second option was Drucker-Prager yield criterion (Elastic-Plastic nonlinear type) based on the cohesion and angle of friction of the concrete which was chosen as the preferred option to demonstrate the actual behaviour of the confined structure. As the experimental outcomes used as the basis of this study did not include triaxial compression tests of the concrete; those parameters (cohesion and angle of friction) were resolved by Mohr’s circles of compressive and tensile strength of the concrete while the confining pressure was assumed to be zero.

A. **Constructing the Geometry**

During the simulation procedure it is very important to determine the meshing shapes before modelling the structure and design the position of other materials. Generally the accuracy of the results is achieved through the complexity of the mesh, but there is a threshold where near full interaction between elements is attained and further increases in mesh complexity only lead to longer process time without any considerable effect in accuracy.

For modelling concrete, Tri6 and Quad8 were preferred to Tri3 and Quad4 for achieving more accuracy and providing more access to model the Beam elements for helical reinforcement.

Furthermore the 2D elements in R-T direction were extruded along the Z direction. The initial mesh of concrete elements was created simultaneously during extruding the plane elements including, sixteen Hexa20 and four Wedge 15 brick elements. The size of the brick elements depends on the positions of reinforcement which occupy the same nodes. Therefore the brick elements were divided by Group option in a way that creates access to nodes along axial and helical bars (Figure 2).
After the concrete volume was modelled; the reinforcing bars were constructed by creating the nodes inside the solid. The nodes of all bars were shared with the nodes of the brick elements. The meshing of both Beam2 and Brick elements share the same nodes which concrete’s region has same areas that were occupied by steel bars. It was assumed that both axial and helical bars were bonded perfectly in the concrete elements.

The most difficult part of the geometry to create correctly was the confining helical reinforcement The initial helical bars were constructed with two Beam2 elements along the nodes at right, left and centre of the external slice and assigned as ‘Helix’ in group option (Figure 3).

At the next level the FRP was modelled as plate elements on the outside of the concrete. The element type selected through Create option of Strand7 was Quad8. The nodes of these Quad8 elements coincided with the outer nodes of the Hexa20 brick element that represent the concrete (Figure 4).

At this stage the primary $60^\circ$ slice was copied by the repeat of 5 to create a $360^\circ$ disc while just brick and plate selects were toggled. The outcome shape again was copied by the increment of $Z = 10\text{mm}$ and $\theta = 60^\circ$ with the repeat time of 5 (while the select option of all types of element were toggled on) to create the 60mm height column which includes brick elements as concrete, beam elements as helix and plate elements as FRP (Figure 5).

The axial bars were also modelled as Beam2 elements along the nodes on the border of two slices which were created before. The constructed beams were subdivided to equal parts along the vertical direction to create some extra nodes at intersection with helix to provide lateral constraint for axial bars (Figure 6).
Figure 6: Modelling the axial bar.

The model was finalised by copying the 60 mm height segment along the axis of the model until the full height column was constructed (Figure 7).

Figure 7: Final model including reinforcement and FRP cover.

B. Material Properties

Creating a model which exactly shows the real behaviour of the concrete was not easy, especially lateral expansion of concrete column under compressive loading (which is often neglected) should be taken into consideration. As mentioned above material nonlinearity is strongly related to differences in stress-strain curves of constituent materials. Therefore the first step was based upon exploiting stress-strain relationship of the concrete specimens.

Since the nonlinear solver in Strand7 increases the applied load and calculates the corresponding deflections, calculating the downward sloping part of the load-deflection curve is impossible as this would require displacement control as used in the experimental testing equipment rather than load control as used in this study. Therefore stress-strain curve of concrete only includes the upward slope section of the tension and compression curves followed by a perfectly plastic section.

The concrete stress-strain table was arranged and entered into Strand7. A model of Column VCF9, a concentrically loaded FRP wrapped column, was run using the recent value of modulus of elasticity $E$ and found to exhibit maximum deflection of just 42% of the experimental results. Assuming a semi-linear relationship between axial strain and $E$ a value of 14.97GPa was chosen as the Modulus of Elasticity and entered into Strand7. At this level the final outcome of the FEA was similar to the experimental result.

Two additional options still remained and need to be specified. The first option was about the type of nonlinearity; Elastic or Elastic Plastic. Elastic nonlinear form which does not allow plastic flow (all strains can be recovered through removing the load) was firstly selected. There were three choices for the second option which determines the yield criterion; Von Mises (often used for metals), Tresca (relates to shear stresses) and Max Stress (for a material with the different stress-strain curve for tension and compression). The Max Stress criterion is more related to concrete and is recommended by Strand7 to be chosen but through selecting that item the software ignores the value of Poisson’s ratio and radial expansion of concrete column will be neglected. Ignoring the lateral expansion of the concrete as a result of choosing this option is almost inevitable, even in case of Column RC1 which had no confinement; the lateral strains of specimens were the result of strains in steel bars rather than the concrete.

According to foregoing problems a new concrete modelling method based on Elastic-Plastic option was chosen and yield criterion type was set to Drucker-Prager. This method is based on the Mohr-Coulomb theory and utilises the triaxial compression properties of concrete. The cohesion, $C$ and internal angle of friction, $\phi$ are specified instead of determining a stress-strain curve and verify the behaviour of concrete under triaxial loading.
Since there were no triaxial tests for the specimens, the values of those parameters can be calculated from the tensile and compressive strengths of the concrete with the confining pressure, $\sigma_3$ as zero [14].

According to the experimental results [12], concrete compressive strength was 75 MPa and concrete tensile strength was 5.9 MPa which were applied in Mohr’s circles to determine values of cohesion and angle of friction. By drawing the tangent line, the value of y-intercept was found to be 10.48 MPa and the angle of friction was 59.2°.

The modulus of elasticity is maintained in the Drucker-Prager models since the deflections until application of the unconfined compressive strength, $f_{co}$, are determined using the elastic modulus and Poisson’s ratio, the Drucker-Prager equations are used after this point to find out the yielding and failure point of the concrete.

The type of FFC340 U0075 Unidirectional Carbon Fibre was selected as the external confinement material and the epoxy system consisted of resin and hardener mixed in a ratio of 5:1 by weight were used in the experimental program to supply the bond action between the FRP and the concrete column. The values that were used in the finite element model are based on the uniaxial material testing which was carried out by Hadi [12] on single and triple layer of CFRP confined specimens.

The finite element model uses the average values of the three layered test results. According to the experimental tests the maximum stress is 884.6 MPa and the maximum strain is 0.020. On the other hand FRP is a non-yielding material with linear-elastic behaviour and therefore according to Hooke’s Law the modulus of elasticity can be calculated as a ratio of stress to strain ($E_t=884.6/0.020=44230$ MPa).

FRP has very small stiffness and strength in other two directions [15], that was the reason that material testing in other directions was not conducted and these values were calculated through the ratios of typical values provided by Kaw [15] for graphite/epoxy lamina.

The moduli of elasticity in other two direction (E2 and E3) were calculated from the ratio of typical values of E2 and E1 while the shear modulus (G) was extracted from the ratio of typical values of G and E1 [15]. The average thickness of three layers of CFRP according to the experimental work [12] was 1.58 mm. Furthermore, according to AS3600 [3] the Poisson’s ratio taken was 0.2.

For Specimen VCF9 that was wrapped in both directions, the model was the same as for the one direction wrapped columns except that the modulus of elasticity in the primary directions was adjusted to reflect the existence of the vertical layer. The new modulus was obtained from the values calculated by Strand7 when the two layers of FRP were modelled as laminate.

Finally the column was modelled with shared nodes between concrete and FRP to simulate full bond at the interface. This assumption is justifiable for concentrically loaded columns, in the case of the specimen chosen in this study. The type of plate elements was orthotropic as a material that has different properties and values of stiffness in each of three mutually perpendicular directions and 3D membrane as a plate element that has in-plane (membrane) stiffness only and can carry direct stress and in-plane shear stress without any bending stiffness [13].

As mentioned above there are two kinds of reinforcement in this project: axial and helical. Axial steel reinforcement is applied for providing enough strength in tension but can be taken to apply to compression as well. In this study the yield strength of axial bars is 550 MPa which is less ductile than a lower yield strength bar but as Hadi [1] noted assuming the steel does not yield has no considerable impact on the final model therefore the effect of brittle axial bars on collapsing the structure can be neglected.

Through using the tensile load-extension diagram, stress-strain curves of axial and helical bars were obtained and the simplified curve was entered into Strand7.

A summary of details for the finite element model which was provided by the application of Strand7 is presented in Table1. For concentric loading, the bottom plane of the column was defined as fixed support and all the nodes of that plane were restrained in all degrees of freedom (translation and rotation of R, T and Z).

Specimens RC1, CF5 and VCF9 were modeled with concentric loading. The load was applied as a uniform face pressure (MPa) over the top surface of the column and the Nonlinear Static option was chosen through solver menu. The load increment of 1(=100%) was applied as an automatic load stepping so that the software could employ smaller sub increments as the structure approached failure. The Save Sub-Increments option was also activated to ease the access to the results of defined steps.
VII. RESULTS AND ANALYSIS

The column was firstly analysed by Linear Static solver to assess the accuracy of the model. At the next level the Nonlinear Static solver has been chosen while the automatic load increment was activated.

Figure 8 provides comparison for the axial load–deflection of the three concentrically loaded columns for both the finite element analysis and experimental testing.

As can be seen in Figure 8, before the failure point of RC1 the trend of all three specimens is the same and finite element results have an optimum agreement with the experimental results. After this point (the failure point of RC1) the deflection of finite element models of FRP wrapped columns (CF5 and VCF9) increase gradually and both of them fail nearly at the same load but generally the FEM displacements of VCF9 is slightly lower than CF5. This result can be justified due to higher average thickness of VCF9 in two directions and consequently higher modulus of elasticity.

VIII. CONCLUSIONS

There was a very good correlation between the nonlinear results of the concentrically loaded columns and the experimental outcomes. The finite element model (FEM) has had the capability of ultimate load and axial displacement prediction with an acceptable margin of errors.

With further study in this field there is a potential to develop an appropriate symmetric fractional model that can represent the behaviour of the true column under both concentric and eccentric loading. Since the main concern of this study was modelling of the helix in a way that achieve the symmetric objectives, improving the parallel helical reinforcement through an optimised calibration (described in one-sixth model) or constructing the helix as shell elements can be the subject of future research.

Due to a wide increasing usage of the FRP (for its repair and strengthening characteristics) it is suggested to define some modelling template which includes the major material and geometric properties for this new element to ease the further design and FEM procedure.
Investigating a new way for failure point determination of the FRP is also needed to be considered, since the value of hoop strain at failure point in experimental testing does not correspond with ultimate strain of the material.

Finally, it can be concluded that the developed finite element model was viable in modelling the experimental results.

REFERENCES


