Finite element analysis of square concrete columns strengthened with carbon fiber composites in different configurations

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Abstract. This paper offers the findings of numerical study carried out to evaluate the behavior of low strength concrete columns in square section strengthened by Carbon Fiber Reinforced Polymer (CFRP) composites and subjected to axial compression load. In this study, a three dimensional non-linear finite element formulation model has been presented using ANSYS-16.1 (software computer program) to analyse low strength concrete columns in square section strengthened by CFRP composites in different configurations, to evaluate the improvement in the ultimate load capacities of columns as a result of CFRP confinement, and to study the impact of the most important parameters such as: compressive strength of concrete, thickness of the CFRP layers jacket/stripst and the presence of steel reinforcement on the performance of the modelled columns subjected to compression load. A comparison has been made between the results obtained from the Finite Element Analysis (FEA) and experimental data. The results of the numerical FEA study showed that, external strengthening with CFRP composites are very effective in upgrading the ultimate load of square concrete columns. In addition, the findings revealed good agreement between the ANSYS and the experimental test results. The results obtained from the parametric study revealed that when the CFRP layers increased from 2 to 3 layers and from 2 to 4 layers the ultimate loads of the columns increased about (2% -10%) and (7%-15%) respectively. While the compressive strength showed significant effect on the upgrade of the ultimate load, when increase the compressive strength to 42MPa and to 60 MPa, the ultimate load increased to about 220% and to about 300% respectively. The presence of steel reinforcing did not show reasonable effect on the ultimate load.

1. Introduction
Columns behaviour are of critical importance for the performance and the safety of the concrete structures [1]. In the last two decades, it is commonly seen that there are require for strengthening and/or retrofitting of existing concrete structures and one of most common members are columns. There are many reasons could lead to damage the concrete structures like: the age of the concrete structures, design errors, amendments in the codes requirements or changing in the facility use, etc. External confinement of concrete columns with Fibre Reinforced Polymer (FRP) is an active technique can be used to upgrade the load carrying capacity and/or ductility of the concrete columns [2-9]. This technique has been studied theoretically through improving specific analysis procedures and experimentally on the performance of different substructures and structures.

The efficiency of the FRP jacket depends on several variables like: concrete type, shape and size of the cross-sectional area, corner radius for square and rectangular sections, section aspect ratio for non-circular sections, applied load condition and FRP stiffness (FRP type, number of layers and FRP configuration) [10].

Carlos and Silva [11], and Campione [12], observed that when increase the FRP layers resulting in increase of concrete strength. There are some studies conducted to investigate the most effective of FRP material type (aramid, glass, and carbon). Most of these studies indicate that the carbon fibre is...
the most effective material in providing strength and ductility [13-15].

Campione [12], observed from his study that ultimate strength decreased when the height of the column increased.

Sangeetha [16], carried out analytical study using FAE on a series of published experimental data of small scale of circular, square and rectangular columns strengthen by FRP. The study approved that the FEA can properly simulate the behaviour of confined concrete columns, moreover, the findings revealed that when strength of the columns increased due to the confinement by about (20-25%).

Tavio and Tat [17], approved that the actual stress-strain curves, upgrade in strength and ductility of confined reinforced normal strength rectangular concrete columns can be predicted through FEA modelling. This has been confirmed by comparison of the analytical results with experimental results.

Mazen and Zahraa [18], conducted a nonlinear FEA through using ANSYS computer program, three effective parameters including in the study: compressive strength of concrete, CFRP jacket thickness and radius of corners on the performance of RC columns having square section and strengthen by CFRP. This study made a comparison between the nonlinear results with published experimental results. The results from this study revealed that the CFRP jacket is very effective technique in upgrading the strength and the ductility of square columns, in addition, the study findings revealed good agreement between the FEA and the experimental results.

It’s widely approved that less benefit gained from applying the external reinforcement (FRP) in rectangular/square columns with respect to the circular columns. In the circular columns, the FRP jacket offers a uniform confinement pressure to the expansion of the concrete core as a result of the axial loading, while columns with rectangular/square sections, the FRP confining pressure is focused at the section edges instead of over the whole perimeter; therefore, evaluating analytically the behaviour of CFRP-confined square concrete columns under compression load became an objective of this paper.

2. Finite element modelling of CFRP confined concrete columns

The FEA by using computer program ANSYS-16.1 is carried out using solid brick elements (SOLID65) to represent concrete and quadratic-order membrane shell (SHELL41) to model CFRP.

Solid-65 element is commonly used to model concrete members. This element possesses eight nodes, each node has three degrees of freedom in the nodal, x, y, and z-directions as illustrated in Figure 1 (a). The element is able to crack in tension, crushing in compression, and large deflection nonlinearities. This element also used to analyse concrete members without reinforcing steel bars.

SHELL41 shown in Figure 1 (b) is a three-dimensional element having membrane (in-plane) stiffness but no bending (out of plane) stiffness. It is intended for shell structures where bending of the elements is of secondary importance. The element has three degrees of freedom at each node: translations in the nodal x, y, and z- directions. The element has changeable thickness, stress stiffening, and enormous deflection [19].

![Figure 1. Finite element representation [19].](image-url)
3. Properties of materials

FEA models for CFRP-confined concrete columns having square section are offered. In the first the material properties are specified, then specify the definitions of the material properties which are required to insert in computer software. Within this study, the computer program (ANSYS-16.1) is utilized to model the square concrete column and CFRP sheet. The nonlinear analysis is developed to simulate the nonlinear behaviour of the confined column. After whole model geometry definition, the material properties should be introduced. First, elastic behaviour of material is set. Hence, the elastic parameters such as: Young’s modulus of concrete, \( E_c \) and Poisson’s ratio, \( \nu \), are inputted. From experimental results \( E_c \) is calculated as \( E_c = 4700 \) where \( f_{c'} \) is given in MPa. The popular stress-strain relationship is used as illustrated in Figure 2 to make the uniaxial compressive simulation of the concrete column which is given by the following relationships [20].

\[
\begin{align*}
  f_{c'} &= \frac{\varepsilon}{1 + \left( \frac{\varepsilon}{\varepsilon_c} \right)^2} \\
  \varepsilon_c &= \frac{2f_{c'}^2}{E_c} \\
  E_c &= \frac{f_{c}}{\varepsilon_c}
\end{align*}
\]

\( f_c \) is the stress at any strain \( \varepsilon \), \( \varepsilon_c \) is the strain at stress \( f_c \), \( f_{c'} \) is the ultimate compressive stress and \( \varepsilon_c \) is the strain at the ultimate compressive strength \( f_{c'} \).

Poisson’s ratio of concrete is assumed to be \( \nu_c=0.2 \). The CFRP behaviour is orthotropic (Figure 3), the CFRP material is inputted as a linear elastic orthotropic material in the model. Indeed, it is necessary to introduce properties of the CFRP for each direction separately.

![Figure 2](image1.png)  
![Figure 3](image2.png)

**Figure 2.** Stress-strain relationship for concrete under uniaxial compression.  
**Figure 3.** CFRP stress-strain relationship.

4. Numerical integration

An alternative scheme of numerical integration is required to carry out the numerical analysis because it is not possible to perform analytically the integration necessary to set up the element stiffness matrix. It is thus very necessary to nominate a suitable integration scheme that is both accurate and computationally efficient.

In the present work, the Gauss quadrature method is selected, since it has proved useful in finite element work [19]. The element stiffness matrix for brick element may be written in local element axes as:

\[
I = \int_{-1}^{1} \int_{-1}^{1} \int_{-1}^{1} F(\xi, \eta, \zeta) d\xi d\eta d\zeta
\]

Which can be calculated numerically as:
\[ I = \sum_{i=1}^{p_1} \sum_{j=1}^{p_2} \sum_{k=1}^{p_3} W_i W_j W_k F(\xi, \eta, \zeta) \] (5)

Where, \( p_1, p_2, \) and \( p_3 \) represent Gaussian points in the \( \xi, \eta, \) and \( \zeta \) directions respectively. The function \( F(\xi, \eta, \zeta) \) represents the matrix multiplication generally the number of integration points is taken to be equal in the three directions: \( p_1 = p_2 = p_3 = p. \)

The integration rule used in this work is the 8 (2×2×2) points rule, Figure 4. Sampling point locations and weighting factors for 2×2×2 integration rule shown in table 1.

Table 1. Sampling point locations and weighting factors for 2×2×2 integration rule [19].

| Sampling Point | Load Coordinates | Weight |
|----------------|------------------|--------|
| 1-8            | ±0.57734, ±0.57734, ±0.57734 | 1      |

5. Nonlinear solution techniques

The solving of the nonlinear structural problems by using finite element discretization results in a set of nonlinear algebraic equations of the form:

\[ [K] \{U\} = \{F_a\} \] (6)

where:
\( [K] \): stiffness matrix.
\( \{U\} \): vector of nodal displacements.
\( \{F_a\} \): vector of applied loads.

For linear elastic problems, equation (6) is used to seek out the answer of the unknown displacements \( \{U\}. \) In the case of nonlinear system, the stiffness matrix \([K] \) is a function of the unknown displacements (or their derivatives). Then equation (6) cannot be exactly computed before determination of the unknown displacements \( \{U\}. \)

There are several techniques can be used to solve the non-linear equations (6); the basic techniques can be classified into:

1. **The incremental procedure**: load is applied in several small increments, and the structures is assumed to respond linearly within each increment with its stiffness recomputed based on the structural geometry and member end actions in the final of the previous load increment. This is a simple procedure, which requires no iterations, but errors are likely to accumulate after several increments unless very fine increments are used, Figure 5 (a).

2. **The iterative procedure**: the total load is applied in one increment at the first iteration, the out of balance forces are then computed and used in the next iteration to obtain progressively
improved solution. The iterative process is continued until the end of converging solution to be in balance, this internal load vector would equal the subjected load vector or within some tolerance. This process can be written as:

$$[K_i^T]\{\Delta U_i\} = \{F^e\} - \{F_{nr}\} \quad (7)$$

$$\{U_{i+1}\} = \{U_i\} + \{\Delta U_i\} \quad (8)$$

where:
$$[K_i^T]$$: stiffness matrix.
i: subscript representing the current equilibrium iteration.
$$\{F_{nr}\}$$: internal load vector.

This procedure fails to produce information regarding the intermediate stage of loading. For structural analysis including nonlinear (like plasticity), the solving method needs some average increments to be in balance to follow the correct load route. This could be gained by employing combined incremental-iterative method, Figure 5 (b).

3. **The combined Incremental-iterative procedure**: it can be used the combination of the incremental and iterative scheme. The load is subjected incrementally, and iterations are performed in order to obtain converged solution corresponding to the stage of loading under consideration, Figure 5 (c).

![Figure 5](image.png)

**Figure 5.** Basic technique to solve non-linear equations [21]: (a) Incremental, (b) Iterative, (c) Incremental-Iterative.

**ANSYS** program implemented incremental-iterative solution procedures incorporated with Full and modified Newton-Raphson algorithm.

6. **Finite element idealization**
The structural behaviour of concrete columns having square section and strengthen by CFRP in different configurations is simulated depending on experimental test results [22] is presented in this study. Four column specimens have been chosen and analysed by using FEA. The columns verification study was carried out to check the validity of the analytical results with experimental test results, then carried out a parametric study to evaluate the influence of some effective parameter on behaviour of square concrete columns strengthen by CFRP composites.

6.1. **Verification of the tested square concrete columns strengthen by CFRP composites**
Four columns specimens were selected from experimental test data [22] to be analysed by FEA. All columns had the same cross section dimensions 300×300 and 1000 mm in height. The details of the tested columns are shown in table 2.
Table 2. Experimental results [22].

| Specimen ID | $f_c$ (MPa) | CFRP Strengthening Details | $P_{u, \text{exp}}$ (kN) |
|-------------|-------------|----------------------------|------------------------|
| CP-Cnt 1    | 18          | -                          | 1830                   |
| CP-3        | 18          | 2 CFRP ply                 | 2250                   |
| CP-5        | 19.3        | 2 CFRP ply+3 CFRP strip+10 CFRP anchors | 2150                   |
| CP-7        | 18.8        | 1 CFRP ply+3 CFRP strip+10 CFRP anchors | 2200                   |

*Note:* The ultimate axial load values were the actual values obtained through an experimental test conducted by the author during the Ph.D. thesis’ research.

From the table, it can be noticed that column specimen CP-5 has additional compressive strength and additional confinement (CFRP strips) than CP-3, but the ultimate load capacity was less than CP-3 (2150 kN vs 2250 kN). These unexpected values regarding to the ultimate capacities could happen because of the alternative arrangement of the anchors (not in the same position in each side). But on the other hand, the anchorage system improved based on the final failure mode of these confined columns where the failure didn’t happen in the anchors zones, which was the aim of the research. In order to achieve a more effective confinement, the FRP strips be constrained on all sides through the use of anchors, thereby creating shorter distances which are confined between anchors.

These columns have been modelled using different element types with real constants and material properties. For these columns, the 1000 mm clear height of the columns was considered in the analysis and was divided into 3 divisions; first division is 50 mm in length, the second division is 900 mm in length (mid span) and the last division is 50 mm in length.

For control column CP-Cnt 1 (without CFRP), the FEA model consists of a total 12375 elements. For strengthen columns (with CFRP), the FEA model consists of a total 15675 elements for CP-3, 16590 for CP-5 and CP-7. The mesh and the boundary conditions used in the FEA for columns (control and strengthen) under concentric load are shown in Figure 6. The corner four nodes are restrained in the x direction for prevention solid shape to move of the idealized model. The distribution of the axial forces at the end elements of the model is shown in Figure 7. The middle nodes take full load and the nodes at the edges take half load, finally the nodes at the corners take quarter load as seen in Figure 8.

![a) Unconfined column](image-a)
![b) Strengthen column](image-b)

**Figure 6.** Mesh and boundary conditions of unconfined and confined columns

![Figure 7.](image-c)

**Figure 7.** The distribution of the axial forces at the end.

![Figure 8.](image-d)

**Figure 8.** The nodes at the corners take quarter load.
The external nodes of concrete surfaces were used to represent the elements of CFRP element (in case of wrapping the column with full and partial jacketing). The both full and partial jacketing of CFRP elements were represented by using these nodes by connected to concrete elements or CFRP elements assuming full contact. In addition, the meshes size were bigger for the top and bottom part of the columns because of the load distribution. Figure 9 (a) and (b) shows the CFRP meshing.

Figure 9. CFRP meshing of jacket and strips: (a) CFRP meshing for the full confinement, (b) CFRP meshing for the partial confinement.

7. Results of the numerical analysis (ultimate load capacity)

The ultimate loads of the modelled columns were indicated by the state that the columns no longer can support additional load as indicated by the convergence failure of ANSYS program in failing to find a solution.

Table 3 shows the maximum failure load for each modelled columns obtained from the software computer program (ANSYS 16.1) and experimental tests. It can be observed from table 3 that the obtained ultimate load from the numerical analysis is a little less than the experimental ultimate load of square concrete columns strengthen by CFRP jackets/strips with least ratio $P_{u,Exp} / P_{u,Ansys}$ was 2% for CP-3 and maximum ratio value was 9% for CP-Cnt 1 while the ratio CP-7 was 4%, except for CP-5 the ANSYS value was a little higher than the experimental value by 5%.

Table 3. Experimental and numerical (ANSYS) of ultimate load capacities.

| Column ID | Ultimate Load kN | $P_{u,Exp}$ | $P_{u,Ansys}$ | $P_{u,Exp} / P_{u,Ansys}$ |
|-----------|------------------|-------------|--------------|--------------------------|
| CP-Cnt 1  | 1830             | 1680        | 1.09         |                          |
| CP3       | 2250             | 2193        | 1.02         |                          |
| CP-5      | 2150             | 2260        | 0.95         |                          |
| CP-7      | 2200             | 2111        | 1.04         |                          |

The axial load - deformation curves of columns CP-Cnt 1, CP-3, CP-5 and CP-7, obtained from the numerical FEA along with the experimental curves reported by experimental data [22] presented and compared in Figure 10. These figures show good agreement between the experimental and finite element axial load-axial deformation results. In addition, complete curves have been obtained from ANSYS while the curves from the experimental were up to 75% of the $P_a$ because mechanical dial gage used in reading the axial deformation during the experimental test. Figure 11 (a) – (i) shows the the nodal stress in the y direction at failure load of the modelled columns.
Figure 10. The axial load-axial deformation curves of columns CP-Cnt 1, CP-3, CP-5 and CP-7 for both analytical and experimental.
Figure 11. The nodal stress in the y direction at failure load of modelled columns
8. Parametric study
The evaluation to the most important parameters impact on the performance of concrete columns in square section that could not be performed during the experimental test were performed by using the numerical finite element ANSYS. In the parametric study, three parameters were included to evaluate their effect on the failure load (ultimate load) of the modelled columns. The parameters were included in this parametric study are:

8.1. Effect of number of CFRP layers
In this numerical study, the effect of CFRP jacket thickness applied on column specimens CP-3 and CP-5, the columns have been numerically analysed using different numbers of CFRP layers (full layers). The selected numbers of layers were 3 and 4, it is worth noting that in the experimental test, the amount of CFRP layers used for columns CP-3 was two and the amount of CFRP layers used for column CP-5 was also two but combined with CFRP strips and CFRP anchors. The comparison between the numerical maximum failure loads got from different amounts of CFRP layers (3 and 4 layers) of the tested columns are listed in table 4 and 5 respectively. It is obvious that the maximum load increases as the amount of CFRP layers increase. When the amount of CFRP layers increased from 2 to 3 layers as listed in table 4, the ultimate load increased by about 5% for both CP-3 and CP-5 respectively, while when the amount of CFRP layers increased from 2 to 4 layers as listed in table 5, the ultimate load increased by about 10% for both CP-3 and CP-5 respectively.

Figure 12 and 13 shows the concrete stresses and CFRP jacket distribution in the y direction at the maximum load of column specimen CP-3 with 3 and 4 CFRP layers respectively.

Table 4. Ansys ultimate load results with 2 and 3 CFRP layers.

| Specimen ID | $f_c$ (MPa) | Exp. Strengthening (No of Layers) | $P_{u, Ansys}$ (kN) | FEA Strengthening (No of CFRP layers) | $P_{u, Ansys}$ (kN) | $P_{u, Ansys} / P_{u, Ansys}$ |
|-------------|-------------|----------------------------------|----------------------|----------------------------------------|----------------------|-------------------------------|
| CP-3        | 18          | 2 CFRP ply                       | 2193                 | 3                                      | 2305                 | 1.05                          |
| CP-5        | 19.3        | 2 CFRP ply+3 CFRP strip+10 CFRP anchors | 2260                 | 3                                      | 2375                 | 1.05                          |

Figure 12. Concrete stresses and CFRP jacket of CP-3 with 3-CFRP layers: (a) Stresses of concrete of CP-3 with 3-CFRP layers, (b) Stresses of CFRP jacket of CP-3 with 3-CFRP layers.
Table 5. Ansys ultimate load results with 2 and 4 CFRP layers.

| Specimen ID | $f_c$ (MPa) | Exp. Strengthening | $P_{u,\text{Exp}}$ (kN) | $P_{u,\text{Ansys}}$ (kN) | $P_{u,\text{Ansys}}/P_{u,\text{Exp}}$ |
|-------------|--------------|---------------------|--------------------------|--------------------------|------------------------------------------|
| CP-3        | 18           | 2 CFRP ply          | 2193                     | 2413                     | 1.10                                     |
| CP-5        | 19.3         | 2 CFRP ply +3 CFRP strip + 10 CFRP anchors | 2260                     | 2475                     | 1.10                                     |

Figure 13. Concrete stresses and CFRP jacket of CP-3 with 4-CFRP layers: (a) Stresses of concrete of CP-3 with 4-CFRP layers, (b) Stresses of CFRP jacket of CP-3 with 4-CFRP layers.

8.2. Effect of Concrete Compressive Strength

In this numerical study the selected values of the concrete strength ($f_c$) were 42 and 60 MPa for columns CP-3, CP-5 and CP-7. The numerical ultimate axial loads got from different values of concrete strength of the tested columns together with the experimental values are listed in table 6. It can be noted from the table that the concrete strength has a significant effect on ultimate axial loading. When the compressive strength of columns CP-3, CP-5 and CP-7 increased to 42 MPa, the ultimate load capacities increased by 222%, 221% and 223% respectively. When the concrete strength of columns CP-3, CP-5 and CP-7 increased to 60 MPa, the ultimate load capacities increased by 304%, 301% and 313% respectively. This effect was expected because the column (compression member) is essentially depends on the concrete compression strength as more as in this study the applied loads were pure compression (concentric load). Figure 14 shows the stresses of concrete, CFRP jacket and the CFRP strips of columns CP-3 for compressive strength 42 and 60 MPa.

Table 6. Experimental and numerical results of ultimate load with different compressive strength

| Column ID | $f_c$ (MPa) (Exp.) | $P_{\text{Exp}}$ (kN) | $f_c$ (MPa) (ANSYS) | $P_{u,\text{Ansys}}$ (kN) | $P_{u,\text{Ansys}}/P_{u,\text{Exp}}$ |
|-----------|-------------------|------------------------|---------------------|--------------------------|------------------------------------------|
| CP3       | 18                | 2250                   | 42                  | 5004                     | 2.22                                     | 60                         | 6840                     | 3.04                     |
| CP-5      | 19.3              | 2150                   | 42                  | 4770                     | 2.21                                     | 60                         | 6470                     | 3.01                     |
| CP-7      | 18.8              | 2200                   | 42                  | 4914                     | 2.23                                     | 60                         | 6903                     | 3.13                     |
Stresses of concrete of CP-3 with $f_c = 42$ MPa

Stresses of CFRP jacket of CP-3 with $f_c = 42$ MPa

Stresses of concrete of CP-3 with $f_c = 60$ MPa

Stresses of CFRP jacket of CP-3 with $f_c = 60$ MPa

Figure 14. The stresses of concrete and CFRP jacket CP-3 with compressive strength 42 MPa

8.3. Presence of steel reinforcement

In order to evaluate the effectiveness of the presence of steel reinforcement on the maximum axial loading capacity of the tested concrete columns in square section strengthen by CFRP, assuming that the columns (CP- Cnt 1, CP-3, CP-5 and CP-7) having a steel reinforcing details (4 - $\varnothing$ 16 for main reinforcement and 7 $\varnothing$ 10 for stirrups) as seen in Figure 15. All columns were modelled in finite element software ANSYS. In addition, three-dimensional spar element (LINK180) was used to represent the steel.

The numerical findings got from the finite element software ANSYS revealed that the reinforcement steel did not have a reasonable effect on the ultimate load capacity of the confined columns, the reason of that could be because the confined columns have been applied to compression loading and it have been approved previously that the concrete take the most action in compression than the steel, in addition to that the columns specimens have CFRP confinement which enhanced in the confinement instead of stirrups or spiral reinforcement. table 7 shows the ultimate load capacities of the modelled columns with and without steel reinforcement.

Figure 15. Reinforcement details in each modelled column.
Table 7. The ultimate load capacities with and without steel reinforcement

| Column ID  | Ultimate Load without reinforcing steel | Ultimate Load with reinforcing steel | \( \frac{P_{u, Ansys}}{P_{u, Ansys}} \) |
|------------|-----------------------------------------|-------------------------------------|-------------------------------------|
|            | \( P_{u, Ansys} (kN) \)                  | \( P_{u, Ansys} (kN) \)               |                                     |
| CP-Cnt 1   | 1680                                     | 1900                                | 1.13                                |
| CP3        | 2193                                     | 2315                                | 1.05                                |
| CP-5       | 2260                                     | 2230                                | 0.98                                |
| CP-7       | 2111                                     | 2190                                | 1.03                                |

9. Conclusion

Based on the analysis of comparison obtained from the analytical (ANSYS) results and experimental results it can be concluded that:

1. The numerical results showed that external strengthening with CFRP composites are very effective in upgrading the maximum compression loading capacity of the square concrete columns.
2. The numerical findings revealed good agreement between the ANSYS and the experimental test results.
3. The numerical results obtained from the parametric study revealed, when the CFRP layers increased from 2 to 3 and from 2 to 4 layers, the maximum compression loading of column specimen CP-3 increased by 5% and 10% respectively. While, when the CFRP layers increased from 2 to 3 and from 2 to 4 layers, the maximum compression loading of column specimen CP-5 increased by 5% and 10% respectively as listed in table 4 and 5.
4. The numerical results confirmed that the compressive strength showed significant effect on the upgrade of the ultimate load, when the concrete compression strength of modelled columns increased to 42MPa and to 60 MPa, the maximum compression loading of column specimens CP-3, CP-5 and CP-7 increased to about 220% and to about 300% respectively as shown in table 6. This effect was expected because the column is a compressive member and is essentially depends on the concrete compression strength as more as in this study the applied loads were pure compression (concentric load).
5. The numerical results revealed that the presence of steel reinforcing did not have a reasonable effect on the maximum axial loading of modelled confined columns. The reason of that could be because the columns were applied to pure compression load and it have been approved previously that the concrete take the most action in compression than the steel, in addition to that the columns specimens have CFRP confinement which enhanced in the confinement instead of stirrups or spiral reinforcement, while the unconfined column CP-Cnt 1 gained the higher increase in the ultimate load capacity by 13% due to the presence of steel reinforcement, which did not have external CFRP.

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