Investigation into the Behavior of Upgraded Concrete-filled Tubular Columns

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Abstract

Many advantages of concrete-filled steel tube columns have led to their use in a variety of distinctive and important structures in recent years, including high-rise buildings, piers, stairs and other structures. The exterior steel wall of these columns is uncoated, which is the main disadvantage. When these columns are subjected to external pressure on the external steel wall, such as collision, explosion, fire or other accidents, the confinement of concrete will be lost owing to the weakness generated in the steel wall, resulting in an abrupt loss in column strength. In this study, a new upgraded section using internal steel mesh is designed and introduced, with this steel mesh and the external steel wall acting as a double-skin steel tube to protect the concrete core from being destroyed if the external steel wall is destroyed and the column's strength drops suddenly. Under axial and cyclic loads, a comparison of the behavior of this innovative section (with internal steel mesh) with the most commonly used sections of concrete-filled steel columns was conducted. Following the verification of the finite element modeling, various other studies were carried out. The results of the analyses clearly show that the suggested CFT section has increased strength, improved resistance to progressive collapse, and energy absorption capacity under axial and cyclic loading, particularly abrupt loads such as fire or explosion, and thus its use in construction is recommended.

1. Introduction

Concrete-filled steel tube (CFT) columns are gaining popularity in the building construction sector all over the globe. CFT columns provide many benefits over traditional steel or reinforced concrete columns, including the steel wall providing confinement and formwork for the concrete core, better stability and rigidity of the steel wall due to the concrete injected into the column, and cheaper construction costs. The axially compressed character of CFT columns makes them superior to traditional reinforced concrete and steel structural systems in terms of stiffness, strength, ductility, and energy absorption capacity. The proper behavior of these columns is owing to the steel wall's containment of the concrete core. To get the appropriate cross section of concrete-filled steel tubular columns, several studies on the presentation of new sections under different loads were conducted. Abedi et al. (2008) created a novel section that uses internal longitudinal symmetric stiffeners to boost the section's strength and ductility under axial and seismic loads. Because of the increased contact area between steel and concrete in this segment, load transferring may be done extremely effectively, resulting in greater strength and absorption capacity (Abedi et al. 2008). In the field, concrete-filled double-skin steel tubes (CFDSTs), a relatively novel configuration design with inserted inner steel tubes, has been produced, as proposed by Hsiao (2015). The authors report findings from an experimental research of CFT and CFDST columns employing ultrahigh-strength steel to further increase the seismic capacity of the CFT/CFDST columns. A total of eight scaled column specimens were subjected to axial and flexural cyclic loadings. The CFT and CFDST design configurations utilizing high-strength steel performed similarly, providing substantial moment and elastic deformation capabilities in both cases. The use of tougher inner steel tubes prevented local buckling on the outer steel tubes, according to the findings of this study. This reduced column moment capacity degradation and improved energy dissipation (Hsiao et al. 2015). Using ABAQUS and nonlinear material behavior and strain rates of steel and concrete, Wang et al. (2016) examined the behavior of CFDST members under low velocity lateral impact. Using existing experimental data, the finite element analysis (FEA) models of CFDST members are presented and confirmed. The influence of impact height, hollow ratio, and nominal steel ratio, diameter-to-thickness ratio of inner steel tube, and material strength on the impact force and global residual lateral deformation in CFDST members is investigated over the full loading range, followed by a parametric study investigating the influence of impact height, hollow ratio, and nominal steel ratio, diameter-to-

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thickness ratio of inner steel tube, and material strength on the impact force and global residual lateral deformation.

In addition, the dynamic increase factor (DIF) of CFDST members under lateral impact is presented, and the role of the confinement factor on the DIF is investigated. When the confinement factor is more than 1.03, the dynamic increase factor must be incorporated to appropriately evaluate the dynamic resistance capability of CFDST components exposed to lateral impact, according to the findings (Wang et al. 2016). The impacts of load-related factors on the response of concrete-filled double-skin steel tube columns exposed to lateral impact were investigated by Aghdamy et al. (2017). Zhou et al. (2016) researched the failure mechanisms and hysteresis curves of the concrete-filled double-skin stainless steel tube. The findings reveal that when the axial compressive force level is low, the hollow ratio and concrete strength have little effect on the behavior of the test specimens, whereas the axial compressive load level and thickness of outer tubes have a major impact (Zhou et al. 2016). By merging rectangular and circular tubes, Hassaneina et al. (2018) and Vernardos et al. (2019) investigated and researched the many types of double-skin steel tubular. Zheng et al. (2018) researched double-tube concrete-filled stainless steel tubular (CFSST) columns with circular outer stainless steel tube and circular inner carbon steel tube under cyclic loads. The results of this research show that if the slenderness factor is more than 60, these columns have a reduced lateral load capability. Chen et al. (2019) determined that a cross section with an octagonal steel tube filled with concrete may absorb a lot of energy. Wang et al. (2019) studied the seismic behavior of a square concrete-filled steel tube (CFT) column when subjected to cyclic stress. Nine large-scale square CFT columns were studied, each with a distinct shear stud configuration and axial compressive load ratio constructed according to engineering practice requirements. The damage mechanism, force-displacement relationship, deformation capacity, stiffness degradation, and energy dissipation capability of these specimens were all reviewed.

The axial compressive load ratio has a considerable influence on the hysteresis loops of the square CFT columns, according to test data. In specimens with a low axial compressive load ratio, the shear stud significantly improves local buckling of the steel tube (Wang et al. 2019). In a cyclic loading test of six full-sized square columns, including one traditional reinforced concrete (RC) column and five steel tube-reinforced concrete (STRC) composite columns, Zhang et al. (2019) investigated the cyclic behavior of steel tube-reinforced, high-strength concrete columns with high-strength steel bars. The test's main characteristics were the cross-sectional form of the inner steel tube, the strength matching of the outer concrete and the concrete core, and the presence of steel fiber in the outer concrete. The addition of steel fibers to concrete matrix can improve the tensile behavior and hinder cracking of concrete, thereby reducing the damage of concrete. Besides, it can slow down the slope of descending branch on the compressive stress-strain curves of concrete. In comparison to RC columns, steel fiber reinforced high-strength STRC composite columns showed improved seismic performance, according to the study. Steel fibers in exterior concrete successfully decreased the degree of damage and increased the specimens' ductility, robust ability, and energy dissipation capacity. Wang et al. (2020) tested six stiffened CFDST beam-columns under continuous axial load and cyclic lateral load. The axial load level and the hollow section ratio were the main test parameters. All specimens’ load, deformation, and strain were measured and studied. In terms of lateral resistance, ultimate displacement, ductility, and energy dissipation ability, the effect of parameters was addressed. They discovered that under cyclic stress, the stiffened specimens had a good energy dissipation capability. Additionally, the longitudinal stiffeners might significantly decrease local tube wall buckling. Stiffened CFDST members have stronger ductility and energy dissipation capability than unstiffened CFDST members.

In this research, a new section is proposed that can be utilized for high-rise building columns, offshore structure legs, and large-span bridge columns. Internal steel mesh is one of the most distinguishing features of the new portion. The effective parameters on the behavior of CFT columns should be considered while choosing this unique section. The following are the parameters:

a. The influence of concrete confinement on CFT column strength and energy absorption capacity
b. The kind of steel and concrete filling utilized in concrete-filled steel columns
c. Loading circumstances and connection type
d. The following are the study’s objectives:
e. Developing a novel section for concrete-filled steel columns
f. Using finite elements to model the real behavior of concrete-filled steel columns
g. A comparison of the proposed new section's behavior with that of typical CFT columns under axial and seismic loads

2. Suggested section for concrete-filled steel columns

Despite the proper behavior of concrete-filled steel columns under various loading conditions, including seismic loading, these columns are highly vulnerable to various accidental or intentional lateral impacts, such as those caused by vehicle collisions, vessel collisions, barge collisions, or flying debris from a nearby explosion. The column strength may quickly deteriorate if the confinement of the concrete core is reduced owing to a flaw in the exterior steel wall. Because structures are so important, it was required to develop a cross-section with more ductility and load-bearing capacity than the most frequent sections of this kind of column, as well as to lengthen the period of progressive failure against
unexpected loads like fire, collision, or explosion. Ferdousi presented the innovative recommended section of CFT shown in Fig. 1 and it is employed in vital projects such as tall building columns, bridge legs with huge spans. In this new proposed piece, a rolled internal steel mesh is employed. The performance of this new recommended section is comparable to that of concrete-filled double-skin steel tubes (CFDSTs), with the exception that the quantity of steel utilized in the internal steel mesh is much less than that used in the CFDSTs' internal skin. Furthermore, even if the exterior steel tube is lost due to collision, explosion, or fire, it will play a critical function in increasing concrete core confinement.

3. Finite element modeling of concrete-filled steel columns

These sections were simulated using the ANSYS finite element software to compare the behavior of the new recommended section with that of the most prevalent CFT columns. The SOLID 65 isoperimetric three-dimensional element is used to simulate the concrete core. This element is defined by eight nodes, each of which has three translational degrees of freedom. This solid element may fracture under tension and crush in compression, as well as deform plastically and creep. The steel wall was modelled using a three-dimensional solid element (SOLID 45). This element is compatible with the steel wall's shell element and the concrete core's isoperimetric solid element. Plasticity, stress stiffening, large deflection, and huge strain capacities are all features of this element. Steel mesh has been modelled using the BEAM 188 element. This element is made up of two nodes with three degrees of freedom in translation and three degrees of freedom in rotation in the three directions x, y, and z. Shear deformation effects are included since this part is based on Timoshenko beam theory. For linear, big rotation, and/or large strain nonlinear applications, the element is ideal. The shape and coordinates of the nodes in the element coordinate system are shown in Fig. 2 (ANSYS 2009). With the assumption of the cleanness of the steel and sufficient adhesion of the core concrete to the steel (perfect bond with no slip assumption), the coincident nodes of the steel and the core concrete can be connected to each other by merging (Abdalla et al. 2020). In this research, no contact element was used and assumed a perfect bond by merging the coincide nodes of steel and core concrete.

The behavior of materials used in the modeling of the samples of columns is indicated in Fig. 3 (Abedi et al. 2008).

![Fig. 1 Proposed part of CFT with internal steel mesh location: a) in the middle of concrete core, b) near the steel wall.](image1)

![Fig. 2 Geometry and node coordinate of elements used in modeling.](image2)
4. Verification of numerical modeling

The numerical findings obtained from material and geometric nonlinear static analyses were compared with the actual results of the CFT columns under axial and lateral cyclic loadings to validate the correctness and validity of the finite element modeling. Furthermore, after applying 30 percent of the column’s nominal load bearing capacity as an axial load, lateral cyclic displacement has been applied to the top of the column in this cyclic loading. In the first and second stages, when the column is under axial stress, the columns are subjected to axial loading; they are then exposed to cyclic loading until the column is damaged, as illustrated in Fig. 4.

The cross-section axial capacity ($N_c$) were adopted. The EN 1998-1 limits the design axial load to no more than 30% of the design column resistance for all composite columns in moment-resisting frames (Chen et al. 2019). In this paper, the axial bearing capacity of the circular CFST columns can be calculated according to the ACI design code as expressed by Eq. (1), where $A_s$ and $A_c$ are cross-sectional areas of the steel tube and the concrete core, respectively, $f_y$ and $f_c$ are the yield strength of the steel tube and the concrete compressive cylinder strength, respectively (Guler et al. 2013).

$$N_{c,ACI} = A_s f_y + 0.85 A_c f_c$$  \(1\)

Seamless pipes were employed in the steel walls of the columns. The steel walls have an inner diameter of 0.254 m and a thickness of 0.008 m, while the columns have a length of 2 meters. As a result, the diameter of outer steel wall is 0.27 m. Table 1 lists the other features of the tested columns, such as the steel mesh and concrete core properties. The interior steel mesh of the examined samples is also 0.227 m in diameter and near to the steel wall.

Compressive strain, and Young's modulus about concrete for both CFT and UGCFT samples is respectively 0.00126 and 25614 MPa. According to the EN ISO 6892-1 standard, the yield strength of the steel wall and steel mesh is 337 and 326 MPa, respectively, and its elastic modulus is about 200000 MPa.

The ATC-24 (ATC 1992) code is used to apply cyclic loading to experimental samples and finite element models, as illustrated in Fig. 5.

Table 1 Specifications of experimental samples.

| Sample name | Diameter of the internal steel mesh (m) | Wire diameter of the internal steel mesh (m) | Dimension of the steel mesh openings (m) | $f_c$ (MPa) |
|-------------|-----------------------------------------|---------------------------------------------|----------------------------------------|-------------|
| CFT         |                                         |                                             |                                        | 32.4        |
| UGCFT       | 0.227                                   | 0.003                                       | 0.0254                                  | 32.4        |

To model steel columns filled with concrete for both CFT and UGCFT samples is respectively 0.00126 and 25614 MPa. According to the EN ISO 6892-1 standard, the yield strength of the steel wall and steel mesh is 337 and 326 MPa, respectively, and its elastic modulus is about 200000 MPa.

Fig. 3 Stress-strain relationship curves for the a) core concrete and b) steel in steel wall and steel mesh.

Fig. 4 Experimental samples under axial and cyclic loading.
next step, the concrete between the outer steel wall and the concrete core of the previous step is modeled by SOLID 65 element. Finally, the steel wall is modeled by SOLID 45 element. In the meshing of these volumes, we have created coincide nodes. For the unique behavior of the column, we connected these volumes by merging these coincide nodes. The cross section of column modeled by ANSYS software is shown in Fig. 6. The interior steel mesh of the experimental sample has a diameter of 0.227 m and is located close to the steel wall. Furthermore, this steel mesh has a wire diameter of 0.003 m. Figure 7 shows the modeled samples of experimental columns. It is worth noting that the steel mesh at the two ends of the column has no special mechanical connection and is only kept in place by means of tools so that it does not move during concreting. However, in the implementation, it is recommended that the steel mesh is not cut at the connection joint of the beam to the column and continues as one piece. All the nodes of the external steel wall located at 0.20 m from the base plate column have been fixed.

The envelope curves of hysteretic loops for experimental samples listed in Table 1 and numerical samples modeled in ANSYS under the same situation are compared in Figs. 8 and 9. It worth nothing that lateral displacement in the test was measured by a gauge that it set up on a horizontal arm at the top level of the column. Unfortunately, it was not possible to record the stress or strain results for the internal steel mesh during the test due to the lack of embedding of the sensor on the steel mesh. Due to the very high bearing capacity of the sample column and the limited capacity of the loading tools regarding the continuation of loading until the complete destruction of the column, the sample column was not completely destroyed and at the end of the test, no signs of tearing of the steel mesh were observed. It seems that the steel mesh has maintained its performance until the end of loading in the laboratory. Considering that the outer steel wall of columns is thick and the cross section is compact, it can be said that before local buckling in the steel wall steel materials yielded.

Figure 10 shows the comparative envelope curves of laboratory samples and numerical models of these samples. This graph shows us that, the internal steel mesh has obviously increased the stiffness and capacity of the other sample column in lateral loading.

In this research, an attempt has been made to establish a good match between test and numerical results. However, due to the limitations and errors of laboratory tools, a slight difference between the boundary conditions of the experimental and numerical models and non-convergence of the analysis, due to increasing cracks in the concrete and yielding steel tube, the difference between the test and numerical results was inevitable. By studying the behavioral diagrams of laboratory and numerical samples of the CFT and UGCFT columns, it can be said that the line slope of the test results was less than the numerical results, which is due to a defect in fixing and looseness...
in the base of the laboratory columns. For the same reason, the maximum lateral load capacity of the numerical samples occurred at low displacements. In general, it can be said that the stiffness of the numerical samples is higher than the laboratory samples.

Experimental and numerical findings are near to each other in order to explore the behavior of experimental samples and numerical models, therefore finite element modeling of samples is trustworthy.

5. Numerical results of CFT and UGCFT columns under axial and lateral loading

Figure 5 shows the lateral cyclic loading according to ATC-24 (ATC 1992) historical loading applied to the CFT column. It is worth mentioning that in this cycle loading, the top of the column was subjected to a lateral cyclic displacement after 30 percent of the column’s nominal load bearing capacity was delivered as an axial force. In two groups of twelve, twenty-four samples were compared. The diameter of the internal steel mesh in the first group is 0.127 m, which is practically in the center of the concrete core. The diameter of this steel mesh in the second group is 0.227 m, which puts it close to the steel wall. For comparison examining, the parameters of the samples are altered in each group, such as the compressive strength of the core concrete ($f'_c$), the dimensions of the steel mesh openings, and the wire diameter of it, as shown in Table 2. In addition, three of the most typical CFT columns with core concrete compressive strengths of 30, 40, and 50 MPa were modeled.

Figures 11 to 13 indicate the deformed shape and failure pattern contours of the steel wall, internal steel mesh, and core concrete at the end of cyclic loading for the UGCFT1111 column, which is a sample from the first group. Similarly, Figs. 14 to 16 show the deformed shape and failure pattern contours of the steel wall, internal steel mesh, and core concrete, respectively, for the UGCFT1121 column as a representative of the second group. Figures 11 and 14 show that the critical three-dimensional stress in steel wall occurred near the foot on UGCFT1111 and UGCFT1121 columns Where if the thickness of the steel wall were less, local buckling could happen in these areas. The failure pattern as shown in Figs. 12 and 15; cracking is indicated with a circle outline in the cracking plane and crushing is indicated with an octahedron line. The first, second and third cracks are shown with a red, green and blue circle outlines at centroid of element, respectively. The software uses the Willam–Warnke criterion to introduce the failure surface in concrete material. With the increase of the amount of lateral load in the last cycles of loading and the yielding of steel mesh elements, the cracking in the concrete increases and spreads until the column is finally destroyed. Figure 13 shows that the steel mesh in UGCFT1111 column near to one third of column experiences maximum

| Row | Sample name | Group name | $f'_c$ for core concrete (MPa) | Diameter of the internal steel mesh (m) | Dimensions of the steel mesh openings (m) | Diameter of the internal steel mesh (m) |
|-----|-------------|------------|-------------------------------|---------------------------------------|----------------------------------------|---------------------------------------|
| 1   | UGCFT 1111  | 1          | 30                            | 0.003                                 | 0.0254                                 | 0.127                                 |
| 2   | UGCFT 1112  | 1          | 40                            | 0.003                                 | 0.0254                                 | 0.127                                 |
| 3   | UGCFT 1113  | 1          | 50                            | 0.003                                 | 0.0254                                 | 0.127                                 |
| 4   | UGCFT 1211  | 1          | 30                            | 0.005                                 | 0.0254                                 | 0.127                                 |
| 5   | UGCFT 1212  | 1          | 40                            | 0.005                                 | 0.0254                                 | 0.127                                 |
| 6   | UGCFT 1213  | 1          | 50                            | 0.005                                 | 0.0254                                 | 0.127                                 |
| 7   | UGCFT 2111  | 1          | 30                            | 0.003                                 | 0.0508                                 | 0.127                                 |
| 8   | UGCFT 2112  | 1          | 40                            | 0.003                                 | 0.0508                                 | 0.127                                 |
| 9   | UGCFT 2113  | 1          | 50                            | 0.003                                 | 0.0508                                 | 0.127                                 |
| 10  | UGCFT 2211  | 1          | 30                            | 0.005                                 | 0.0508                                 | 0.127                                 |
| 11  | UGCFT 2212  | 1          | 40                            | 0.005                                 | 0.0508                                 | 0.127                                 |
| 12  | UGCFT 2213  | 1          | 50                            | 0.005                                 | 0.0508                                 | 0.127                                 |
| 13  | UGCFT 1121  | 2          | 30                            | 0.003                                 | 0.0254                                 | 0.227                                 |
| 14  | UGCFT 1122  | 2          | 40                            | 0.003                                 | 0.0254                                 | 0.227                                 |
| 15  | UGCFT 1123  | 2          | 50                            | 0.003                                 | 0.0254                                 | 0.227                                 |
| 16  | UGCFT 1221  | 2          | 30                            | 0.005                                 | 0.0254                                 | 0.227                                 |
| 17  | UGCFT 1222  | 2          | 40                            | 0.005                                 | 0.0254                                 | 0.227                                 |
| 18  | UGCFT 1223  | 2          | 50                            | 0.005                                 | 0.0254                                 | 0.227                                 |
| 19  | UGCFT 2121  | 2          | 30                            | 0.003                                 | 0.0508                                 | 0.227                                 |
| 20  | UGCFT 2122  | 2          | 40                            | 0.003                                 | 0.0508                                 | 0.227                                 |
| 21  | UGCFT 2123  | 2          | 50                            | 0.003                                 | 0.0508                                 | 0.227                                 |
| 22  | UGCFT 2221  | 2          | 30                            | 0.005                                 | 0.0508                                 | 0.227                                 |
| 23  | UGCFT 2222  | 2          | 40                            | 0.005                                 | 0.0508                                 | 0.227                                 |
| 24  | UGCFT 2223  | 2          | 50                            | 0.005                                 | 0.0508                                 | 0.227                                 |
| 25  | CFT300      | -          | 30                            | -                                     | -                                      | -                                     |
| 26  | CFT400      | -          | 40                            | -                                     | -                                      | -                                     |
| 27  | CFT500      | -          | 50                            | -                                     | -                                      | -                                     |
stresses. The maximum stress of the longitudinal elements of steel mesh is equal to 278.4 MPa and the transverse elements of it is equal to 331.2 MPa at the end of cyclic loading. Figure 16 shows that the maximum stresses in the internal steel mesh occurred near the foot of the UGCFT1121 column. Unlike UGCFT111 column, the steel mesh in UGCFT1121 column is close to the steel wall, while in UGCFT111, the radius of the steel mesh is almost half the radius of the entire section and is located in the middle of the concrete core. The maximum stress of the longitudinal elements of steel mesh is equal to 356 MPa and the transverse elements of it is equal to 332 MPa at the end of cyclic loading. The steel mesh is working in both vertical and circumferential direction. It resists the bending action and confines the concrete. In the last steps of cyclic loading, the steel mesh elements start to yield, after which the crack in the concrete expands and the bearing capacity of the column decreases.

**CFT**

\[ P_{(EXP)} = 120 \text{ KN} , \quad P_{(FEM)} = 99.3 \text{ KN} \]

![Fig. 8 The lateral load- lateral displacement responses of CFT experimental sample and numerical model.](image)

**UGCFT**

\[ P_{(EXP)} = 128 \text{ KN} , \quad P_{(FEM)} = 114.4 \text{ KN} \]

![Fig. 9 The lateral load- lateral displacement responses of UGCFT experimental sample and numerical model.](image)

**Fig. 10 The envelope curves for CFT (without steel mesh) and UGCFT (with steel mesh) experimental samples and numerical models under axial and cyclic loading.**

**Fig. 11 Deformed shape and Von-Mises loading stress contours of steel wall for UGCFT1111 column at the end of cyclic loading.**

**Fig. 12 Crack pattern of concrete core for UGCFT1111 column at the end of cyclic loading.**

**Fig. 13 Deformed shape and stress contours of steel mesh for UGCFT1111 column at the end of cyclic loading.**
For a comparative study of two types of columns with steel mesh located in the middle part (UGCFT1111) and steel mesh near the steel wall (UGCFT1121), the steel mesh stresses in the same loading step (step 20) are shown in Fig. 17.

The maximum stress in the longitudinal and transverse elements of column UGCFT1111, which occurs near one-third of the length of the column, is equal to 359 and 459, respectively. While, in column UGCFT1121, the maximum stress in the longitudinal and transverse elements is observed in the amount of 330 MPa and 459 MPa, respectively, in the area near the foot of the column.

Due to the placement of the steel mesh at a distance from the neutral axis in column UGCFT1121 compared to column UGCFT1111, the stress value of the steel mesh and the number of steel mesh elements reached the yield point in column UGCFT1121 are higher than in column UGCFT1111. In other words, it can be said that the steel mesh close to the steel wall has a better performance in increasing the bending capacity (caused by lateral load) and increasing the confinement of concrete compared to the columns with steel mesh located in the middle part.

In Fig. 18, the hysteretic loops and their envelope curves resulting from numerical analyses for Group 1 of CFT columns with internal steel mesh located in the middle of the concrete core and a diameter steel mesh of 0.127 centimeters mentioned in Table 2 and the same $f_c'$ of 30 MPa under axial and cyclic loading are shown. Similarly, in Fig. 19, same curves for Group 2 of CFT columns with steel mesh next to steel wall with diameter equal to 0.227 centimeters specified in Table 2 and identical $f_c'$ equal to 30 MPa have been shown.

Under axial and cyclic stress, the ultimate lateral load capacity and lateral displacement of Group 1 and Group 2 samples with the same $f_c'$ equal to 30 MPa were compared in Table 3. The maximum load in the envelope diagram is accepted as the ultimate lateral load capacity of

| Row | Sample name | Group number | Ultimate lateral load capacity (KN) | Ultimate lateral displacement (m) | Percentage of the internal steel mesh to the steel wall cross section area | Percentage increase of load capacity to common CFT |
|-----|-------------|--------------|-------------------------------------|----------------------------------|--------------------------------------------------------------------------------|--------------------------------------------------|
| 1   | CFT300     | -            | 73                                  | 0.09                             | -                                                                              | -                                                |
| 2   | UGCFT1111  | 1            | 84                                  | 0.09                             | 1.72                                                                          | 15.07                                            |
| 3   | UGCFT1211  | 1            | 89                                  | 0.09                             | 4.76                                                                          | 21.92                                            |
| 4   | UGCFT2111  | 1            | 83                                  | 0.09                             | 0.86                                                                          | 13.70                                            |
| 5   | UGCFT2211  | 1            | 87                                  | 0.09                             | 2.38                                                                          | 19.18                                            |
| 6   | UGCFT1121  | 2            | 96                                  | 0.09                             | 2.45                                                                          | 31.51                                            |
| 7   | UGCFT1221  | 2            | 118                                 | 0.09                             | 6.81                                                                          | 61.64                                            |
| 8   | UGCFT2121  | 2            | 93                                  | 0.10                             | 1.22                                                                          | 27.40                                            |
| 9   | UGCFT2221  | 2            | 98                                  | 0.10                             | 3.40                                                                          | 34.25                                            |

Table 3 Comparison of ultimate lateral load capacity and lateral displacement of all samples with the same $f_c'$ equal to 30 MPa.

Fig. 14 Deformed shape and Von-Mises loading stress contours of steel wall for UGCFT1121 column at the end of cyclic loading.

Fig. 15 Crack pattern of concrete core for UGCFT1121 column at the end of cyclic loading.

Fig. 16 Deformed shape and stress contours of steel mesh for UGCFT1121 column at the end of cyclic loading.
the column and the maximum displacement that the column can withstand in cyclic loading is accepted as the ultimate lateral displacement of the column.

According to Table 1, the ratio of the quantity of steel used in the internal steel mesh to the amount of steel used in the steel wall in the new proposed sections of CFT columns is extremely low, ranging from 1 to 7 percent. When compared to most usual CFT columns, the findings of Table 3 and Figs. 18 and 19 reveal that the low proportion of steel employed in internal steel mesh (with diameter 0.127 m) has resulted in a 14 to 22 percent improvement in lateral load capacity and a 14 to 22 percent in lateral stiffness columns. Which means of lateral stiffness column is the ratio of lateral load to lateral displacement in envelope curves. However, in Group 2 of CFT

![Fig. 17 Deformed shape and stress contours of steel mesh at time step 20 for a) UGCFT1111 and b) UGCFT1121 columns.](image)

![Fig. 18 The hysteretic loops and their envelope curves for Group 1 of CFT columns with the same $f'_c$ equal 30 MPa under axial and cyclic loading.](image)

![Fig. 19 The hysteretic loops and their envelope curves for Group 2 of CFT columns with the same $f'_c$ equal to 30 MPa under axial and cyclic loading.](image)
columns, by using steel mesh nearby steel wall (with diameter 0.227 m), 27 to 62% improvement in lateral load capacity and 27 to 65% improvement in lateral stiffness is achieved. It seems that the little quantity of steel employed in the internal steel mesh increases the lateral load capacity and the amount of energy absorption in the new recommended CFT columns by enhancing the confinement of the concrete core. According to the provided data, the effect of internal steel mesh in increasing lateral load capacity and energy absorption in Group 2 samples with internal steel mesh near to the steel wall is more obvious than Group 1 samples (due to the greater confinement performance in Group 2 samples). The absorbed energy for specimens is equal to the area under the load-displacement envelope curves until the final loading step.

Furthermore, in samples UGCFT1211 and UGCFT1221, the beneficial impact of the internal steel mesh in enhancing ultimate load capacity is more visible than in their counterparts owing to the adequacy of confinement. Because the internal steel mesh in the UGCFT1221 sample is farther away from the neutral axis and has more confinement than the mesh in the UGCFT1211 sample, this impact is greater. This boost in capacity is 1.62 times that of a typical CFT column with no internal steel mesh and \( f'_{c} \) equal to 40 MPa (CFT300).

Under axial and cyclic loads, Fig. 20 shows the hysteretic loops and their envelope curves derived from numerical calculations for Group 1 of CFT columns, and Fig. 21 shows same curves for Group 2 of CFT columns with the same \( f'_{c} \) equal to 40 MPa. Table 2 lists the characteristics of all of these samples.

Under axial and cyclic stress, the ultimate lateral load capacity and lateral displacement of Group 1 and Group 2 samples with the same \( f'_{c} \) equal to 40 MPa were compared in Table 4.

In comparison to most conventional CFT columns, the findings of Table 4 and Figs. 20 and 21 show that employing internal steel mesh embedded with diameter 0.127 m (Group 1) has resulted in a 10 to 17% increase in lateral load capacity and a 4 to 17 increase in lateral stiffness columns. However, this improvement in lateral load capacity and lateral stiffness capacity is 15 to 40 percent and 14 to 33 percent respectively in Group 2 columns with internal steel mesh nearby steel wall (with 0.227 diameters). The lateral load capacity and quantity of energy absorption in the recommended CFT columns enhanced as a result of improving the confinement of the concrete core by the internal steel mesh. Due to increased

| Row | Sample name | Group number | Ultimate load capacity (kN) | Ultimate lateral displacement (m) | Percentage of the internal steel mesh to the steel wall cross section area | Percentage increase of load capacity to common CFT |
|-----|-------------|--------------|-----------------------------|----------------------------------|---------------------------------------------------------------|---------------------------------------------------|
| 1   | CFT400     | -            | 80                          | 0.09                             | -                                                             | -                                                 |
| 2   | UGCFT1112  | 1            | 89                          | 0.09                             | 1.72                                                           | 11.25                                             |
| 3   | UGCFT1212  | 1            | 94                          | 0.09                             | 4.76                                                           | 17.5                                              |
| 4   | UGCFT2112  | 1            | 88                          | 0.09                             | 0.86                                                           | 10                                                |
| 5   | UGCFT2212  | 1            | 90                          | 0.09                             | 2.38                                                           | 12.5                                              |
| 6   | UGCFT1122  | 2            | 103                         | 0.09                             | 2.45                                                           | 28.75                                             |
| 7   | UGCFT1222  | 2            | 112                         | 0.09                             | 6.81                                                           | 40                                                |
| 8   | UGCFT2122  | 2            | 92                          | 0.08                             | 1.22                                                           | 15                                                |
| 9   | UGCFT2222  | 2            | 104                         | 0.09                             | 3.40                                                           | 30                                                |

Under axial and cyclic loading, the ultimate lateral load capacity and lateral displacement of Group 1 and Group 2 samples with the same \( f'_{c} \) equal to 40 MPa were compared in Table 4.

In comparison to most conventional CFT columns, the findings of Table 4 and Figs. 20 and 21 show that employing internal steel mesh embedded with diameter 0.127 m (Group 1) has resulted in a 10 to 17% increase in lateral load capacity and a 4 to 17 increase in lateral stiffness columns. However, this improvement in lateral load capacity and lateral stiffness capacity is 15 to 40 percent and 14 to 33 percent respectively in Group 2 columns with internal steel mesh nearby steel wall (with 0.227 diameters). The lateral load capacity and quantity of energy absorption in the recommended CFT columns enhanced as a result of improving the confinement of the concrete core by the internal steel mesh. Due to increased

![Fig. 20 The hysteretic loops and their envelope curves for Group 1 of CFT columns with the same \( f'_{c} \) equal 40 MPa under axial and cyclic loading.](image-url)
confinement of core concrete in Group 2 samples with $f'_c=30$ MPa, the impact of internal steel mesh in enhancing lateral load capacity and boosting energy absorption is more effective than Group 1 samples with internal steel mesh adjacent to the steel wall. Furthermore, the favorable impact of the internal steel mesh in enhancing ultimate load capacity is more visible in samples UGCFT1212 and UGCFT1222, owing to the sufficiency of core concrete confinement. This impact is strongest in the UGCFT1222 sample. This boost in capacity is 1.40 times that of a typical CFT column with no internal steel mesh and $f'_c=40$ MPa (CFT400).

Under axial and cyclic loads, Fig. 22 depicts the hysteretic loops and their envelope curves derived from numerical calculations for Group 1 of CFT columns, whereas Fig. 23 depicts same curves for Group 2 of CFT columns with the same $f'_c$ equal to 50 MPa. Table 2 lists the characteristics of all of these samples.

Under axial and cyclic stress, the ultimate lateral load capacity and lateral displacement of Group 1 and Group 2 samples with the same $f'_c$ equal to 50 MPa were compared in Table 5.

Table 5 and Figs. 22 and 23 show that Group 1 of CFT columns with internal steel mesh (with 0.127 diameters) have a load capacity and stiffness column of 1 to 11% and 2 to 14% respectively greater than the most typical CFT without internal steel mesh. However, in new Group 2 columns with steel mesh nearby steel wall (with 0.227 diameters) have a load capacity and stiffness column of 1 to 29% and 2 to 33% respectively greater than the most common CFT columns without steel mesh. The same compressive strength of core concrete of 50 MPa, Group 1 improves load capacity and energy absorption more than Group 2. Furthermore, the UGCFT1223 sample's maximum load capacity is 1.29 times that of the CFT500 sample. Table 2 shows that CFT500 is a basic concrete-filled steel column with no internal steel mesh, having a core concrete compressive strength of 50 MPa. The increase in compressive strength of concrete under triaxial stresses depends on the amount of lateral compressive stresses due to the presence of steel wall and steel mesh. It is clear that factors such as steel wall thickness and compressive strength of concrete also affect it. Especially when the outer steel wall is damaged, the effect of the inner steel mesh in maintaining the confinement of the core concrete will be more obvious.
6. Suggestion for future research

It can be studied that by using the internal steel mesh, due to having a concrete cover and not being directly exposed to fire and preventing the rapid opening of cracks in concrete, which ultimately leads to increased destruction time in the progressive collapse of structures under abrupt loads. This score is very important for column members in sensitive structures such as tall buildings.

7. Conclusions

The confinement of the core concrete is enhanced in the new section proposed in this research for concrete-filled steel columns by employing the internal steel mesh. Under axial and cyclic loading, particularly sudden loads like fire or explosion, the results of the studies clearly show an improvement in strength, improved resistance to progressive collapse, and energy absorption capacity of the recommended innovative section. The quantity of steel utilized in the internal steel mesh compared to the amount of steel used in the steel wall is quite low, ranging from 1 to 7%. The findings of all experimental and numerical samples reveal that using a low proportion of steel in the internal steel mesh raises the columns' lateral load capacity to 1.57 times that of most conventional CFT columns. Internal steel mesh next to the steel wall samples have a higher load capacity than similar samples, with a wire diameter of 0.005 m and a steel mesh spring size of 0.0254 cm. The internal steel mesh utilized in these samples seems to have been able to keep the core concrete contained enough. It may also be argued that in samples with $f'_c = 30$ MPa, the favorable impact of steel mesh on column load capacity is more effective. In other words, to achieve proper confinement, we must utilize a steel mesh with a bigger cross-section area for columns with greater core concrete strength. In general, when there is an accident and a weakness in the steel wall, the internal steel mesh, particularly those close to the steel wall, provides adequate confinement based on the characteristics and dimensions of steel mesh openings, and thus plays a significant role in increasing ultimate load capacity and energy absorption.

Fig. 22 The hysteretic loops and their envelope curves for Group 1 of CFT columns with the same $f'_c$ equal 50 MPa under axial and cyclic loading.

Fig. 23 The hysteretic loops and their envelope curves for Group 2 of CFT columns with the same $f'_c$ equal 50 MPa under axial and cyclic loading.
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