Numerical study of concrete-filled aluminium alloy tubular columns under eccentric compression

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Abstract. Concrete-filled aluminium tubular (CFAT) columns have numerous benefits, such as aesthetic appearance, less self-weight and good corrosion resistance, over conventional concrete-filled tubular columns. A limited body of research exists for CFAT columns, and the research on the behaviour of CFAT columns under eccentric loading is rather rare. Therefore, the present study aims at investigating the eccentric compressive response of CFAT columns by means of numerical analysis. The numerical models were developed considering nonlinear behaviour of the materials. The models were validated against experimental data obtained from concentric compression tests of CFAT columns. The columns used in the experiments were 6082-T6 heat-treated aluminium alloy square tubes filled with 30 MPa cube strength concrete. The validated models were used to study the influence of different loading eccentricities on the behaviour of CFAT columns. The results are discussed in terms of ultimate strengths and load-lateral displacement responses. A comparison is drawn between the column capacities obtained from the numerical analysis and calculated by European standards. It is found that the design formulae provided in Eurocode are conservative for the eccentrically loaded CFAT columns.

1. Introduction
The use of aluminium alloys in structural engineering has increased due to their beneficial characteristics such as less self-weight, good corrosion resistance, high recyclability and aesthetic appearance. Despite these advantages, previous research showed that aluminium alloy structural members are more prone to buckling failure due to the low Young’s modulus of aluminium [1-3]. To overcome this problem the concrete-filled aluminium alloy tubular (CFAT) members are used which combine the benefits of both materials and enable achieving higher strengths. However, research on CFAT members is limited compared to the concrete-filled steel tubular members. Zhou and Young [4,5] experimentally investigated the behaviour of CFAT stub columns subjected to axial compressive loading. It was shown that due to concrete infill the ultimate strength of CFAT columns improved significantly compared to bare columns. Zhou and Young [6] conducted a numerical study on circular CFAT columns and proposed design formulae considering the advantageous composite behaviour of aluminium tube and infilled concrete. Wang et al. [7] carried out an analytical investigation on the response of CFAT stub columns subjected to axial compression and proposed design expressions for
composite columns. Patel et al. [8] numerically studied the compressive response of high strength CFAT columns by adopting a new analytical model of confined concrete.

Previous studies are mainly dealt with the structural response of CFAT short columns loaded concentrically. However, research on the compression behaviour of CFAT slender columns under eccentric loading has been reported rarely. Therefore, the present study aims at investigating the eccentric compressive response of CFAT slender columns by means of numerical analysis. The columns used herein consist of 6082-T6 heat-treated aluminium alloy square tubes with 30 MPa cube strength concrete. Numerical models of all tested columns were developed considering the nonlinear behaviour of the materials. The models were validated against experimental results. Finally, a comparison is drawn between the column capacities obtained from the numerical analysis and calculated by European standards.

2. Selected test data
The experiments on square CFAT columns tested under concentric compression presented by Georgantzia et al. [9] were used to validate the numerical models. The specimens were fabricated by 6082-T6 aluminium alloy with 1000 mm nominal length. The aluminium properties were determined by testing the dog-bone coupon samples under tension. The concrete properties were obtained by compression tests of concrete cubes and the average strength recorded was 31.57 MPa after 28 days. To avoid the localised failure during the tests, the top and bottom ends of each column were strengthened by carbon fibre reinforced polymer wraps. Moreover, both ends of each specimen were milled flat and plastered for uniform distribution of the applied load. Before the test, the geometric imperfections of all columns were measured. A bearing system was used to test the specimens under pin-ended conditions. During the tests, strain gauges and linearly varying displacement transducers (LVDTs) were instrumented to capture the axial strain and mid-height displacement of the specimens. For reference, Table 1 presents the measured geometric dimensions and material properties including Young’s modulus of aluminium tube ($E_a$), 0.2% proof stress ($f_{0.2}$) and ultimate tensile stress ($f_u$) of all columns. In this table the columns are designated based on their nominal section dimensions (i.e., depth, width and thickness).

| Specimen | Depth, $h$ (mm) | Width, $b$ (mm) | Thickness, $t$ (mm) | Length, $L$ (mm) | $E_a$ (GPa) | $f_{0.2}$ (MPa) | $f_u$ (MPa) |
|-----------|----------------|----------------|-------------------|-----------------|-------------|----------------|-------------|
| 50.8×50.8×1.6 | 50.7            | 51.0           | 1.61              | 1000            | 65          | 289.1          | 315         |
| 50.8×50.8×3.3 | 50.6            | 50.6           | 3.13              | 999             | 71.7        | 302.2          | 330         |
| 50.8×50.8×4.8 | 50.6            | 50.6           | 4.67              | 1000            | 67.5        | 305.9          | 325         |
| 76.2×76.2×3.3 | 76.4            | 76.4           | 3.23              | 1000            | 66.2        | 299.1          | 321         |
| 76.2×76.2×4.8 | 76.2            | 76.1           | 4.76              | 1000            | 64.7        | 306.1          | 316         |
| 76.2×76.2×6.4 | 76.3            | 76.3           | 6.28              | 1000            | 69.3        | 295.3          | 326         |

3. Numerical model

3.1. Development of finite element (FE) model
The ABAQUS software [10] was used in this study to develop the FE models of CFAT columns. The material properties and geometries of all columns reported in Table 1 were used to build the models. The eight-node hexahedral solid elements (C3D8R) were used for modelling both the aluminium tube and infilled concrete [6]. Average mesh size of 5 mm was assigned to obtain accurate results in
reasonable computation time. The wall thickness of hollow tubes was discretised in three elements to predict the nonlinear behaviour of the specimen [11].

The elastic-plastic model available in ABAQUS was applied to consider the material properties of aluminium alloy. The measured stress-strain responses were transferred into true stress-strain responses to incorporate into FE models. For elastic material properties, the Young’s modulus of all specimens reported in Table 1 were used and the Poisson’s ratio was taken as 0.33. To consider concrete plasticity in the models, the Concrete Damaged Plasticity (CDP) model provided by ABAQUS was employed. The Young’s modulus was determined according to EN 1994-1-1 [12] and the value of Poisson’s ratio was used equal to 0.2. The plasticity parameters in CDP model were calculated according to the recommendations provided in Tao et al. [13]. A compressive stress-strain response for confined concrete recommended by Tao et al. [13] was employed to consider the beneficial composite response between the aluminium tube and infilled concrete. For accounting the tensile behaviour of concrete, the stress-strain relationship was considered linear up to 10% of compressive cylinder strength [13], while the inelastic region was defined by the stress-crack opening displacement relationship using fracture energy [14-16].

The interaction of aluminium column and infilled concrete was considered by the surface-to-surface contact algorithm. In this algorithm, the hard contact relationship are used in normal direction and Coulomb friction model are considered in tangential direction. In tangential direction, the coefficient of friction was taken equal to 0.3 [17]. Moreover, the existing geometric imperfections of all specimens were accounted in the modelling by incorporating the lowest buckling mode obtained from eigenvalue buckling analysis. The imperfection amplitude was taken L/1000 based on a sensitivity study [18]. Regarding the modelling of boundary conditions, the top and bottom ends of each specimen were connected to reference points that were created at the centroid of both ends. The concentric load was applied through a displacement control approach at the top reference point. Both reference points were restrained to move or rotate at all directions, except the displacement at the loading direction and the rotation about the buckling axis. A typical FE model of CFAT column is shown in Figure 1.

![Figure 1. FE model of a typical CFAT column.](image)

3.2. Validation of FE model
The FE models were validated by comparing the experimental and numerical ultimate strengths, load versus mid-height lateral displacements and modes of failure. Table 2 reports the columns ultimate capacity obtained from the experiment \((N_{u,EXP})\), FE analysis \((N_{u,FE})\) and their ratios \((N_{u,FE}/N_{u,EXP})\). It is observed from the table that the ultimate capacities obtained from FE models are very close to experimental results with the mean value of \(N_{u,FE}/N_{u,EXP}\) equal to 1.037 and the coefficient of
variation (COV) equal to 0.068. Comparisons of test and FE load-mid-height lateral displacement and failure mode of a typical specimen are shown in figure 2(a) and (b), respectively, where good agreements were also observed. Therefore, it can be stated from the above comparison that the developed FE models are capable to correctly capture the structural response of CFAT columns.

Table 2. Comparison of experimental [9] and FE ultimate capacities.

| Specimen       | $N_{u,EXP}$ (kN) | $N_{u,FE}$ (kN) | $N_{u,FE}/N_{u,EXP}$ |
|----------------|------------------|------------------|-----------------------|
| 50.8×50.8×1.6 | 103.71           | 97.48            | 0.94                  |
| 50.8×50.8×3.3 | 141.18           | 151.31           | 1.07                  |
| 50.8×50.8×4.8 | 195.77           | 185.45           | 0.95                  |
| 76.2×76.2×3.3 | 344.07           | 388.23           | 1.13                  |
| 76.2×76.2×4.8 | 449.68           | 470.36           | 1.05                  |
| 76.2×76.2×6.4 | 532.08           | 578.60           | 1.09                  |
| Mean           |                  |                  | 1.037                 |
| COV            |                  |                  | 0.068                 |

Figure 2. Comparison of experimental [9] and FE ultimate results of specimen 76.2×76.2×4.8: (a) load versus mid-height lateral displacement, (b) failure mode.

4. Numerical simulation of eccentric compressive behaviour

To investigate the eccentric compressive behaviour of CFAT columns, six different loading eccentricities, which ranged from 0 mm to 65 mm, were considered in the FE models, leading to a total of 36 nonlinear analyses. The eccentricity was applied by moving the top reference point at the desired position.

Table 3 presents the numerical results including ultimate capacities for different eccentricities ($e$) and the mid-height lateral displacements corresponding to ultimate capacities ($\delta_u$) for $e = 0$ and 65 mm. It is observed that the ultimate capacity of specimens decreased gradually with the increase of loading eccentricity. In addition, the lateral displacement increased together with the increase in the eccentricity. To determine the effect of the highest eccentricity considered in this study on the columns’ capacity, the percentage decrease of ultimate load compared to zero eccentricity also included in the table. It is found that for all specimens the ultimate capacity decreased more than 70%
compared to the ultimate capacity obtained under concentric loading. This is accompanied by a corresponding increase in the bending moment capacity.

Table 3. Summary of key results from FE analysis.

| Specimen   | $N_{u,FE}$ (kN) | $\delta_u$ (mm) | % decrease of $N_{u,FE}$ for $e = 65$ mm | $\delta_u$ (mm) | $e$ (mm) |
|------------|-----------------|-----------------|-----------------------------------------|-----------------|---------|
| 50.8×50.8×1.6 | 97.48           | 0               | 77.78                                   | 7.75            | 25.42   |
| 50.8×50.8×3.3 | 151.31          | 50.16           | 9.30                                    | 29.10           |         |
| 50.8×50.8×4.8 | 185.45          | 76.14           | 74.32                                   | 31.09           |         |
| 76.2×76.2×3.3 | 388.23          | 76.14           | 73.95                                   | 21.89           |         |
| 76.2×76.2×4.8 | 470.36          | 72.73           | 72.73                                   | 23.19           |         |

Figure 3(a) and (b) illustrate the effect of the eccentricity on load-mid-height lateral displacement relationship of columns 50.8×50.8×3.3 and 76.2×76.2×4.8, respectively. It is observed that all curves displayed a clear elastic region in which the lateral displacements increase linearly with the increase of loading. In the subsequent elastic-plastic region, the curves started to show a nonlinear relationship between load and displacement. For columns with higher eccentricities, a relatively flat load-displacement curve has been observed. For columns with lower eccentricities, a descending branch after the ultimate load was reached is evident.

Figure 3. Effect of loading eccentricity on the load-mid-height lateral displacement curves for specimens (a) 50.8×50.8×3.3, (b) 76.2×76.2×4.8.

5. Assessment of Eurocode

In this study, the design approach provided by EN 1994-1-1 [12] for composite steel-concrete columns under eccentric loading, with the material properties of steel replaced by that for the aluminium alloy appropriately, is assessed based on the numerical results. The interaction diagrams of all CFA T columns are prepared according to the simplified method provided in EN 1994-1-1 [12]. These polygonal diagrams are constructed with four points as shown in figure 4.
Figure 4. Simplified interaction curve proposed in EN 1994-1-1 [12].

According to EN 1994-1-1 [12], Point A (pure compression) presents only the plastic resistance to compression ($N_A = N_{pl,Rd}$) of the column where the moment ($M_A$) is zero. $N_A$ is calculated by equation (1).

$$N_A = N_{pl,Rd} = A_n f_{0,2} + 0.85 A_c f_c$$  \hspace{1cm} (1)

where $A_n$, $A_c$ are the cross-sectional area of the aluminium tube and infilled concrete, respectively. $f_c$ is the compressive strength of concrete cylinder and $f_c = 0.8 f_{cu}$, where $f_{cu}$ is the compressive strength of concrete cube [19]. It is worth mentioning that $A_n$ is replaced by the effective cross-sectional area ($A_{eff}$) for Class 4 aluminium sections, evaluated according to EN 1999-1-1 [20].

At point B (pure bending), the load ($N_B$) is zero and the moment is considered the plastic resistance moment of the column $M_B = M_{pl,Rd}$ which is determined by equation (2). In this equation $\alpha_c$ is taken as 1 to consider the confinement effect.

$$M_B = M_{pl,Rd} = (W_{pc} - W_{pa,n}) f_{0,2} + 0.5(W_{pc} - W_{pc,n}) \alpha_c f_c$$  \hspace{1cm} (2)

where $W_{pc}$ and $W_{pa}$ are the plastic section modulus of the concrete and aluminium tube which are calculated by equation (3) and (4), respectively. $W_{pc,n}$ and $W_{pa,n}$ are the plastic section modulus of the infilled concrete and aluminium tube from $2h_n$ which are determined by equation (5) and (6), respectively; $h_n$ is the height of the neutral axis which is obtained using equation (7).

$$W_{pc} = \frac{(b - 2t)(h - 2t)^2}{4} - \frac{2}{3} r_{int}^3 - r_{int}^2 (4 - \pi) \left( \frac{h}{2} - t - r_{int} \right)$$  \hspace{1cm} (3)

$$W_{pa} = \frac{bh^2}{4} - \frac{2}{3} (r_{int} + t)^3 - (r_{int} + t)^2 (4 - \pi) \left( \frac{h}{2} - t - r_{int} \right) - W_{pc}$$  \hspace{1cm} (4)

$$W_{pc,n} = (b - 2t) h_n^2$$  \hspace{1cm} (5)

$$W_{pa,n} = bh_n^2 - W_{pc,n}$$  \hspace{1cm} (6)
\[ h_n = \frac{A_f f_c}{2bf_c + 4t(2f_{0.2} - f_c)} \]  

where \( r_{out} \) is the internal corner radius of the hollow section which is zero herein.

At point C, the load \( (N_{pm,Rd}) \) is considered as the resistance of concrete only under compression (equation 8) and the moment is the same as \( M_B \).

\[ N_c = N_{pm,Rd} = A_c f_c \]  

At point D, the moment is considered the maximum resistance moment \( (M_{max,Rd}) \) which is calculated by equation (9) and the load is taken as the half of the load of point C.

\[ M_D = M_{max,Rd} = W_{pu} f_{0.2} + 0.5W_{pc} \alpha_f f_c \]  

The calculated theoretical capacities are then converted to member capacities by multiplying the reduction coefficient \( \chi \) according to EN 1999-1-1 [20], which is expressed by equation (10).

\[ \chi = \frac{1}{\phi + \left(\phi^2 - \lambda^2\right)^{0.5}} \leq 1.0 \]  

The parameter \( \phi \) and the member slenderness \( \lambda \) are estimated by equation (11) and (12), respectively.

\[ \phi = 0.5 \left[ 1 + \alpha \left( \lambda - \lambda_0 \right) + \lambda^2 \right] \]  

\[ \lambda = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}} \]  

where for 6082-T6 \( \alpha \) is 0.2 and \( \lambda_0 \) is 0.1 [20] and \( N_{cr} \) is the critical elastic buckling load which is calculated by equation (13).

\[ N_{cr} = \pi^2 \left( \frac{E_c I_c + k_e E_a I_a}{L_{cr}^2} \right) \]  

where \( E_c \) and \( E_a \) are the Young’s modulus of infilled concrete and aluminium tube, respectively, \( I_c \) and \( I_a \) are the moment of inertia of infilled concrete and aluminium tube, respectively, \( k_e \) denotes the correction factor for the concrete which is considered as 0.6 in line with EN 1994-1-1 [12] and \( L_{cr} \) represents the effective buckling length of the specimen.

Figure 5 presents a comparison between the results obtained from FE analysis and the interaction curves constructed according to EN 1994-1-1 [12] for all specimens. In the curves, the ultimate strengths and bending moments obtained from FE analysis are inputted as non-dimensional forms, i.e., \( N / N^* = N / (\chi N_{pl,Rd}) \) and \( M / M^* = M / M_{max,Rd} \). The ultimate bending moment of each specimen was calculated using equation (14).

\[ M = \left( e + \delta \right) \times N \]  

where \( \delta \) is the mid-height lateral displacement at peak load \( N \).

From figure 5, it is observed that all the results obtained from numerical analysis are higher than results predicted by EN 1994-1-1. It can be concluded that the design formulae provided in EN 1994-1-1 are conservative in general and conservative for the design of 6082-T6 CFAT columns subjected to eccentric loads.
Figure 5. The comparison between the FE results and the interaction curves constructed according to EN 1994-1-1 [12].

6. Conclusions
In this study, a numerical investigation of the structural behaviour of CFAT columns subjected to eccentric compression is presented. The numerical models were developed considering the nonlinear behaviour of materials. The models were validated by comparing the results obtained from concentric compression tests of CFAT columns. The validated models were used to study the influence of
different loading eccentricities on the response of CFAT columns. The observed ultimate strengths and load versus mid-height lateral displacement relationship are presented. It is observed that for all specimens the ultimate capacity decreased and lateral displacement increased gradually with the increase of loading eccentricity. A comparison is drawn between the column capacities obtained from the numerical analysis and the design capacities calculated by European standards. It is found that the design formulae provided in EN 1994-1-1 for steel-concrete members, with the material properties of steel replaced by that for the aluminium alloy appropriately, are in general conservative for the design of 6082-T6 CFAT columns subjected to eccentric loads.

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