Research Article

Ultimate Limit Design of Strengthened Steel Columns by Mortar-Filled FRP Tubes

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The present study investigated the various failure modes of strengthened steel columns by mortar-filled fiber-reinforced polymer (FRP) tubes to analytically formulate the ultimate capacities of these steel columns. A simple and effective method, wherein a mortar-filled FRP tube was sleeved outside the steel member, was also formulated to enhance the buckling resistance capacity of compressed steel members. In addition, to facilitate the connection of the column to other structural members, the length of the sleeved mortar-filled FRP tubes is less than that of the original steel columns. Theoretical analyses were also performed on the critical sections of such composite columns at their ultimate states to identify their potential failure modes, such as FRP-tube splitting at the ends or on the insides of wrapped areas, local buckling at the steel ends of transition zones, and global buckling of the composite columns. The corresponding ultimate capacity of each failure mode was then analytically formulated to characterize the critical failure mode and ultimate load capacity of the columns. The current theoretical results were compared with those from literature to validate the applicability of the developed ultimate limit design approaches for FRP-mortar-steel composite columns.

1. Introduction

Compressive buckling is considered the most significant failure mode in steel columns due to its prevalence in many engineering accidents, such as the collapse of the Québec Bridge in 1907 [1] and transmission towers in South China in 2008 [2]. These events have prompted scholars and engineers to further investigate the buckling resistance capacity of compressed steel components to improve the design or repair of steel structures to effectively prevent compressive buckling and to improve buckling resistance capacity of its compressed members.

Fiber-reinforced polymer (FRP) composites are lightweight, high strength, and corrosion-resistant materials that have been increasingly employed to strengthen and repair existing steel and concrete structures [3–5]. The application of pultrusion technology (pulling of continuous fibers through a bath of resin, blending with a catalyst and then into preforming fixtures, and then passing through a heated die) as an automated and continuous manufacturing process has significantly reduced FRP section production costs. Additionally, FRP sections, specifically glass fiber-reinforced polymers (GFRP), are affordable materials with low thermal conductivity and low embodied energy for building construction [6, 7]. In general, FRP fabrics are pasted or wrapped onto the surfaces of steel columns with FRP composites to enhance their strength by improving their load-bearing capacity and durability.

Teng and Hu [8] experimentally and numerically studied the mechanical properties of FRP-jacketed circular steel tubes under axial compression, which showed that FRP-jacketed reinforcement can effectively restrict local buckling and improve ductility. Shaat and Fam [9, 10] also experimentally and theoretically examined compressed steel columns pasted with GFRP composites and carbon fiber-reinforced polymer (CFRP) composites, whose results indicated that FRP composites significantly strengthened the compressive steel members. However, strengthening by
pasting or wrapping FRP fabrics onto the surface of steel components requires too much labor work, such as sanding all the exposed surfaces. In addition, it is more difficult to develop high-quality strengthening construction methods on steel members with complex plane shapes.

In recent years, Liu et al. [11] and Feng et al. [12, 13] pioneered a strengthening technique to improve the buckling resistance capacity of compressed steel columns by wrapping or sleeving FRP jackets onto the outside of steel members, specifically within corrosion-promoting zones or along the entire length of the member except for the nodes. The steel members are subsequently filled with concrete or mortar between the FRP tube and the original steel member. Experimental and theoretical analyses have validated the applicability of the aforementioned strengthening technique due to its easy construction applications and minimal labor requirements. In addition, based on the research results of the above references, it can be found that the method is very effective for improving the buckling resistance and compression stiffness of steel members. An ultimate design approach based on the segmentation model has been proposed for the calculation of the load-bearing capacity in [13]. However, the ultimate design approach was only developed and focused on the global buckling of composite columns, and the approach did not consider the failure modes of FRP-tube end-splitting and local buckling at the transition zone ends of steel materials. And for different failure modes, there was no in-depth theoretical analysis and no reasonable mechanical model of limit state. Moreover, the prediction results of the design approach were not in good agreement with the experimental results.

The present study investigated possible experimental failure modes of strengthened steel columns by sleeved mortar-filled FRP tubes and developed a theoretical formulation to estimate the ultimate capacity of each presented failure mode, including FRP-tube end-splitting, local buckling at the steel end of the transition zone, and global buckling of the composite columns. Sectional stress and strain analyses and force equilibrium principles were applied to generate a theoretical formulation to determine the most critical failure mode, thereby characterizing the ultimate capacity of the composite columns. A comparison between the theoretical formulation and experimental results was generated to verify the applicability of the presented formulations for composite columns and for the ultimate design of steel columns strengthened by sleeved mortar-filled FRP tubes.

2. Failure Characteristics of the Test Specimens

Two groups of steel columns with cross- or I-sections were strengthened by sleeved mortar-filled FRP tubes [13]. The configurations of the strengthened specimens are shown in Figure 1. The length $l_0$ of the steel member outside the FRP tubes at both ends of the column was reserved for the connection between the strengthened columns and other structural members. The inner diameters of the sleeved FRP tubes were slightly larger than the boundary dimensions of the original steel columns. Considering the easy splitting of both ends of the sleeved FRP tubes, the ends of some specimens were circumferentially wrapped with FRP fabrics to improve the end-bearing capacities of the sleeved FRP tubes, and the length of reinforced region is $l_1$. In example specimen C/I X-Y, C or I refers to the cross section or I-section steel member in the center of the specimen, and X and Y refer to the nominal slenderness ratio and the FRP fabrics layers, respectively. The geometric dimensions and material properties of the strengthened columns are shown in Tables 1 and 2 [13], respectively.

The strengthened composite columns were examined under axial compression to investigate their failure modes and load-carrying capacities. Previous experimental results identified potential failure modes, such as FRP-tube splitting at the ends (Figure 2(a)) or on the inside of the wrapped areas (Figure 2(b)), FRP fabric ruptures (Figure 2(c)), local buckling at the ends of steel members (Figure 2(d)), and global buckling (i.e., global buckling in common steel columns) [13]. A summary of the possible failure modes is presented in Table 3. FRP fabric rupture can often be avoided by increasing the wrapped FRP fabric layers. Therefore, the present study did not discuss the failure mode for such strengthened composite columns.

3. Ultimate Limit Design

3.1. FRP-Tube Splitting at the Ends or on the inside of the Wrapped Area. Given that the axial load was only directly applied to both ends of original steel columns, a development or transition length $L_T$ was proposed in the transition region [11], by which the applied axial load can be transferred to the jackets. The transition region of strengthened specimen is shown in Figure 3. The sectional stiffness of the outer half of the $L_T$ was assumed to be equal to that of the original steel section ($E_s I_s$), and the sectional stiffness of the inner half of the $L_T$ was assumed to be identical to the strengthened steel section such that

$$E_W I_W = E_s I_s + \beta (E_m I_m + E_f I_f),$$

where $E_m I_m$ and $E_f I_f$ are the sectional stiffness of the filled mortar and FRP, respectively; and factor $\beta$ is equal to 0.6 for the expansive mortar and 0.3 for nonexpansive mortar [11].

When the nominal slenderness ratio of strengthened specimens is smaller and the splitting shear strength of FRP-tube walls and the wrapped FRP fabrics layers at the ends of FRP tubes are also smaller, the FRP-tube end-splitting failure mode may occur. FRP-tube end-splitting may be induced by the larger shear force in the transition region, which is caused by second-order bending. The shear force-induced local FRP-tube end-splitting is mainly controlled by the deformation and section stiffness of the strengthened composite column ends. On the other hand, because of the smaller nominal slenderness ratio and splitting shear strength of FRP-tube walls in the failure mode, there is no damage in the middle section of strengthened specimens, and its deformation is very small. Therefore, the influence of the middle section of the strengthened composite columns (section stiffness equals equation (1)) on the local FRP-tube
splitting may be ignored. Figure 4 presents the force balance of the analytical specimen due to axial compression, in which the length and section stiffness of specimen are approximately regarded as $2l_0 + L_T$ and $E_s I_s$, respectively. The experimental results suggested the presence of FRP-tube splitting at the weaker restrained ends of the strengthened specimens.

Table 1: Geometric dimensions of specimens.

| Specimens  | Cross section | Nominal slenderness ratio | FRP fabrics layers | Length (mm) | $l_0$ (mm) | $t_c$ (mm) | $b_c$ (mm) |
|------------|---------------|---------------------------|--------------------|-------------|------------|------------|------------|
| C70-5      | Cross         | 70                        | 5                  | 1140        |            |            |            |
| C105-0     | Cross         | 105                       | 2                  | 1720        | 70         | 6          | 80         |
| C105-2     | Cross         | 140                       | 5                  | 2310        |            |            |            |
| C105-5     | Cross         | 175                       | 5                  | 2890        |            |            |            |
| C140-5     | I             | 140                       | 5                  | 1580        |            |            |            |
| C175-5     | I             | 175                       | 5                  | 1990        |            |            |            |

Table 2: Material properties of specimens.

| Material   | Modulus (GPa) | Strength (MPa) | Notes |
|------------|---------------|----------------|-------|
| Steel      | 204.2         | 362.0/575.6    | In tension |
| FRP tube   | 36.4          | 293.4          | In compression |
| Mortar     | 23.1          | 36.0           | In compression |
| FRP fabrics| 96.8          | 2905.0         | In tension    |

1Yields strength/ultimate strength.
composite columns [13] such that the end-restrained condition for both ends of the analytical specimen in Figure 4 is the same as the weaker restrained ends of the composite columns.

The experimental failure phenomena (Figure 2(a)) and the force balance analysis at the critical cross section indicate that the splitting starting positions, SP1 and SP2 (see Figure 5(a)), were located on both sides of the FRP-tube wall ends, which correspond to two possible contact points or areas between a pair of deformed steel cross section or I-section flanges and the FRP-tube wall. The angles $\theta_1$ and $\theta_2$ are approximately equal given the smaller longitudinal distance between SP1 and SP2 (Figure 5). Therefore, the shear forces at positions SP1 and SP2 can be determined as follows:

$$Q_1 + Q_2 = P_1 \sin \theta_1 + P_2 \sin \theta_2 = P \sin \theta_1.$$  \hfill (2)

The effective area that carries and transfers the splitting shear stress from positions SP1 and SP2 must first be evaluated given the approximate point of contact between the deformed flanges of the steel member and the FRP-tube wall. In order to investigate the distribution of splitting shear stress near the contact point, we carried out a series of finite element numerical analyses. In the finite element models, the steel members were simplified into the steel blocks of 3 mm square, whose sizes were much smaller than the FRP-tube walls. The FRP tubes were simplified into the rectangular plates 10 mm wide and 30 mm long, varying FRP-tube wall thicknesses. Steel and FRP were regarded as elastic-plastic.
and elastic, respectively, and the material properties are shown in Table 2. According to a finite element analysis of the composite columns of varying FRP-tube wall thicknesses, an area within the distance of the FRP-tube wall thickness from splitting starting positions SP1 or SP2 would be the most effective region to transfer the splitting shear stresses; in other words, about 90% of the splitting shear stress would be carried within this region (Figure 5(b)). Experimental results on pultruded GFRP box sections under concentrated loading suggested similar conclusions [14]. Therefore, only the contribution of the splitting shear capacity of FRP-tube walls near splitting starting positions SP1 and SP2 (within one FRP-tube wall thickness range) was considered, and the splitting shear capacity outside this region was neglected.

Two potential splitting shear contact points between FRP-tube wall and steel member may be present on both sides of the weak bending axis in the composite column with the I-section steel member, thereby allowing two theoretical splitting starting positions on both sides of the FRP-tube wall; that is, two potential splitting cracks may be present on both sides of the FRP-tube wall. However, because of the randomness and asymmetry of the actual section distribution and material properties, only one splitting failure section is actually located on each side of the FRP-tube wall. In consideration of the above conditions, an amplification factor $\alpha$ for the effective splitting area can be introduced for composite columns with I-section steel members. Therefore, a unified effective splitting area for the composite columns with cross section or I-section steel members can be defined as

$$A_{fs} = \alpha \cdot t_{ft} \cdot \left(2t_{ft} + t_{ft} \right) = 3\alpha \cdot t_{ft}^2,$$

where the amplification factor $\alpha$ is 1 and more than 1 (taken as 1.2 in this study) for cross section and I-section steel members, respectively.

The splitting shear stress $\tau$, which resulted from the second-order effects, can be calculated in comparison to the splitting shear strength $f_s$ of the FRP-tube wall [15] as follows:

$$\tau = \frac{Q_1 + Q_2}{A_{fs}} = \frac{P}{3\alpha \cdot t_{ft} \cdot \sqrt{\pi^2 \cos^2 \left(\pi/2l_0 + L_T \cdot l_0 \right) + k^2 \left(1 - P/P_{El} \right)^2}} \leq f_s.$$

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Figure 5: Schematic diagram and shear stress distribution of splitting starting position: (a) splitting starting position; (b) shear stress distribution near splitting starting position.
where \( k \) is the imperfection factor (out-of-straightness) and \( P_{E1} \) is the Euler load corresponding to the analytical specimen such that
\[
P_{E1} = \frac{\pi^2 \cdot E_s I_s}{\mu^2 \cdot (2l_0 + L_T)^2},
\]
where \( \mu \) is the length coefficient determined by the end-restrained condition.

Equation (4) is a quadratic equation about \( P \), and its solution for \( P \) can be expressed as
\[
P = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a},
\]
where \( a, b, \) and \( c \) are defined as the coefficients of a quadratic equation such that
\[
a = \left( \frac{k \cdot f_y \cdot 3a \cdot t_{j1}^2}{P_{E1}} \right)^2 - \left( \pi \cdot \cos \left( \frac{\pi}{2l_0 + L_T} \cdot l_0 \right) \right)^2,
b = -2 \left( \frac{k \cdot f_y \cdot 3a \cdot t_{j1}^2}{P_{E1}} \right)^2,
c = \left( \frac{k \cdot f_y \cdot 3a \cdot t_{j1}^2}{P_{E1}} \right)^2 + \left( \pi \cdot f_y \cdot 3a \cdot t_{j1} \cdot \cos \left( \frac{\pi}{2l_0 + L_T} \cdot l_0 \right) \right)^2.
\]

For FRP-tube splitting on the inside of the wrapped area, the only deviation observed from the FRP-tube splitting at the end is the deviation of the splitting starting positions SP1 and SP2 with those on the inside. However, due to the relatively small width of the wrapped fabrics, the ultimate load-carrying capacity \( P \) for the failure mode can still be conservatively calculated by equation (6).

3.2. Local Buckling at the Ends of the Steel Member. When the nominal slenderness ratio of strengthened specimens is smaller, the splitting shear strength of FRP-tube walls or the wrapped FRP fabrics layers at the ends of FRP tubes are larger, the local buckling at the ends of the steel member may occur. Local buckling often occurs at the ends of the steel member near the transition region (Figure 2(d)). The theoretical analysis on the failure modes of the critical sections has indicated that the composite columns are equivalent to a steel column with a length of \( 2l_0 \), which neglects the influence of the strengthened composite section, namely, the middle of the strengthened composite column with a length of \( L \) (Figure 1). Again, the end-restrained conditions for both ends of the analytical specimen are the same as the weaker restrained ends of the composite columns.

The maximum compressive stress \( \sigma_{\text{max}} \) can be defined based on the yield strength \( f_y \) of the steel and in consideration of the second-order bending and initial imperfection (out-of-straightness) of the material [15]:
\[
\sigma_{\text{max}} = \frac{P}{A_s} + \frac{p}{W_s} \frac{w_{0,\text{max}}}{1 - (P/P_{E2})} \leq f_y,
\]
where \( W_s \) is the section modulus of the steel member, \( A_s \) is the cross-sectional area of the steel member, \( w_{0,\text{max}} \) is the initial imperfection of the steel column with a length of \( 2l_0 \), and \( P_{E2} \) is the Euler load corresponding to the steel column with a length of \( 2l_0 \) such that
\[
P_{E2} = \frac{\pi^2 \cdot E_s I_s}{\mu^2 \cdot (2l_0)^2}.
\]

Equation (10) is a quadratic equation about \( P \), which is defined as
\[
P = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a},
\]
where \( a, b, \) and \( c \) define the coefficients of a quadratic equation such that
\[
a = W_s,
b = -\left( \frac{f_y \cdot A_s \cdot W_s + A_s \cdot w_{0,\text{max}} + W_s}{P_{E2}} \right),
c = f_y \cdot A_s \cdot W_s.
\]

3.3. Global Buckling. When the nominal slenderness ratio of strengthened specimens is larger, the global buckling failure mode may occur. Previous studies have defined an energy method to define the global buckling load for columns with variable cross-sections [11, 13]. However, this energy method required complex definite integrals, and the calculated results were quite different from experimental results. Therefore, Hermite’s difference method with a variable step length [16] was used to analytically formulate the global buckling capacities for such strengthened composite columns. Liu and Duan [16] established the second-order derivative Hermite’s difference equation with a variable step length (Figure 6) as follows:

\[
a w_{i-1} - (1 + a) w_i + w_{i+1} = \frac{h^2}{12} \left( \alpha^3 + 4\alpha^2 + 4\alpha + 1 \right) w_i^{11} + \left( \alpha^2 + \alpha - 1 \right) w_i^{10} = 0(h^3).
\]

Different end-restrained conditions may be observed at both ends of a real strengthened composite column. For the sake of simplification, the present study first assumed the lateral restraints at both ends follow pin-pin end conditions (Figure 7). In addition, the theoretical global buckling loads for the pin-pin end condition can be corrected by a magnification factor \( K_m \) [11] such that
\[
K_m = \frac{1}{\mu^2}.
\]

The stiffness distribution and analysis model of the strengthened composite column hinged at both ends were characterized based on the transition length and section stiffness simplification analysis above (Figure 7). The whole column can be divided into 5 segments symmetrically, in which points \( i = 0 \) and \( i = 5 \) are at both ends of the composite column, \( i = 1 \) and \( i = 4 \) are in the region where the stiffness is equal to \( E_s I_s \), and \( i = 2 \) and \( i = 3 \) are in the middle
Equation (15) can then be transformed as

\[ \frac{P}{E_i I_i} w_i'' = -w_i, \quad (i = 1 \sim 5). \]  \hspace{1cm} (16)

For the \( i = 1 \) point, \( h = l_{x1} \) and \( \alpha = (h_1/h) = (l_{x1}/l_{h1}) \). By substituting the values of \( h, \alpha, w_{i-1}, w_i, \) and \( w_{i+1} \) from equations (16) into (12), which can then be combined with equation (14), an equation about \( w_1 \) and \( w_2 \) can then be defined as follows:

\[ (a_1 P + b_1)w_1 + (c_1 P + d_1)w_2 = 0, \]  \hspace{1cm} (17)

where \( a_1, b_1, c_1, \) and \( d_1 \) are the coefficients in equation (17), which are constants and depend on \( E_w I_w, E_s I_s, l \) (the total length of specimens), and \( \alpha \) at the \( i = 1 \) point.

Once again, for \( i = 2 \), equation (18) about \( w_1 \) and \( w_2 \) can be expressed as follows:

\[ (a_2 P + b_2)w_1 + (c_2 P + d_2)w_2 = 0, \]  \hspace{1cm} (18)

where \( a_2, b_2, c_2, \) and \( d_2 \) are the coefficients in equation (18), which are also constants, but the coefficients depend on \( E_w I_w, E_s I_s, l \), and \( \alpha \) at the \( i = 2 \) point.

If \( w_1 \) and \( w_2 \) are nonzero, \( P \) must make the determinant of the coefficient matrix of equations (17) and (18) zero such that

\[ \begin{vmatrix} (a_1 P + b_1) & (c_1 P + d_1) \\ (a_2 P + b_2) & (c_2 P + d_2) \end{vmatrix} = 0. \]  \hspace{1cm} (19)

Equation (19) is a quadratic equation with one unknown \( P \). Solving equation (19), the theoretical global buckling load \( P \) can be determined for a composite column hinged at both ends. For other boundary conditions, the calculating global buckling load \( P \) can be corrected by a magnification factor \( K_m \) (equation (13)).

4. Comparison and Validation

The geometrical configurations of all of the tested specimens are presented in Figure 1 with the geometrical parameters given in Table 1 [13] at an imperfection factor \( k \) of 200 (i.e., out-of-straightness is 1/200 of the specimen length). The transition length \( L_T \) is 70 mm according to [11]. The elastic moduli of the steel, FRP tube, and expansive mortar are 204.2 GPa, 36.4 GPa, and 23.1 GPa (see Table 2), respectively. The splitting shear strength \( f_s \) of the FRP-tube wall and the yield strength \( f_y \) of steel are 26.7 MPa and 362.0 MPa, respectively. The length coefficient \( \mu \) for the pin-pin, pin-fix, and fix-fix end conditions are 1, 0.7, and 0.5, respectively.

Table 4 presents a comparison between the theoretical ultimate load-carrying capacities and the experimental and theoretical results from [13] for the FRP-mortar-steel composite columns under axial compression. FRP fabrics rupture occurred in C105-2 specimen; based on the above analysis, this specimen is not discussed. Because of a machine malfunction, the 1105-5 specimen is also not discussed. Feng et al. [13] only consider the global buckling failure (for other failure modes, there is no in-depth theoretical analysis, and no calculation method of ultimate bearing capacity is given), and a great difference can be observed between the theoretical ultimate load-carrying...
capacity and the experimental results, except for the C175-5 specimen. The theoretical results in Table 4, which are based on the proposed method, are compared well with the experimental results. With the exception of a 20.2% difference for the C140-5 specimen, the maximum differences of all of the other specimens were less than 13%.

Moreover, the predicted possible failure modes for the different specimens, which can be judged by the minimum ultimate load-carrying capacities of several failure modes, were consistent with the experimental observations. Local buckling failure is likely to occur in specimens with smaller nominal slenderness ratios. In comparison, specimens with larger nominal slenderness ratios are likely to exhibit global buckling failure. Specimens with medium nominal slenderness ratios may show FRP-tube splitting or local buckling failure.

5. Conclusions

The present study introduced ultimate design approaches for FRP-mortar-steel composite columns under axial compression. Possible failure modes, such as FRP-tube splitting, local buckling, and global buckling, were identified and their corresponding ultimate capacities were theoretically derived to define the minimal capacity required to suggest potential failure modes.

Specimens with smaller nominal slenderness ratios were most likely to exhibit local buckling at the ends of their steel members near the transition region. The ultimate capacities are dependent upon a few design parameters, including the cross section parameters of the steel members, the length of steel outside the FRP tubes at both ends, the imperfection factor (such as out-of-straightness), and the yield strength of steel.

Specimens with larger nominal slenderness ratios were most likely to exhibit global buckling failure. The ultimate capacities may be affected by several parameters, such as the section stiffness and length of the different composite column sections (such as the strengthened and non-strengthened zones) and the lateral restraints at both ends of the composite columns.

FRP-tube splitting failure may occur in medium nominal slenderness ratio specimens such that the ultimate capacities are mainly dominated by the length of the steel outside of the FRP tubes at both ends, the transition length, the section stiffness of the steel members, the imperfection factor, the thickness and splitting shear strength of the FRP-tube wall, and the lateral restraints at both ends of the composite columns.

Data Availability

The data used to support the findings of this study are included within the article.

Conflicts of Interest

The authors declare that there are no conflicts of interest.

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