1. Introduction

CFT columns have been increasingly used in North America and Asia. A typical CFT column consists of four steel plates infilled with concrete. The composite action of the steel and concrete provides excellent strength and stiffness for the column. In a CFT column, steel plates under compression can only buckle locally outward owing to the restraints of the infilled concrete. This buckling mode results in a considerable increase in the critical local buckling strength of the steel plates as well as the load-bearing capacity of the column. Meanwhile, the steel plates offer confinement to the concrete, which enhances the concrete performance and thus, increasing the stability and strength of the column as a system.

The steel plate in CFT column can be modeled as an isotropic plate which is rotationally restrained along the two loaded and two unloaded edges and the rotational stiffness of the elastic restraint at the four edges is introduced to account for the flexibility of the conjunctions. As a result of different loading states and rotational restraints provided by concrete, the loaded and unloaded edges of the steel plate can be assumed as simply supported or clamped.

The fact that the CFT column has steel plates restrained by concrete and the availability of high strength structural steel leads to the application of thin steel plates in CFT columns. However, this gives rise to the local instability problem of thin steel plates under axial loadings, which is common in CFT columns.

A number of research have been undertaken on the local buckling of plates with various edge restraints and under different loadings. It is well recognized that numerical methods such as the finite element method, \(^1,^2\) finite difference method, \(^3\) and finite strip method \(^4\) could provide explicit analytical solutions for instability analysis of plates. However, it is computationally complicated, time consuming and tedious for broad use in design as several iterations may be necessary to ensure the reasonable solution.

As an alternative to the numerical approach, the classical Rayleigh-Ritz method has been adopted by many researchers to obtain the local buckling solution of plates. In applying the variational formulation of this approach, Qiao and his research group \(^5-^8\) conducted extensive analyses to develop the explicit local buckling of composite plates with rotational restraint under uniaxial compression, uniform shear, and biaxial loading, and fully restrained under combined linearly varying axial and in-plane shear loadings. Kang and Leissa \(^9\) proposed an exact buckling solution of plates subjected to linearly varying normal stresses and with...
two opposite edges simply supported, and the other two
dges may be clamped, simply supported, free, or elasti-
cally supported. Zhong and Gu\(^{10}\) also presented an exact
buckling solution for simply supported symmetrical cross-
ply rectangular plates subjected to linearly varying loadings.
Mittelstedt\(^{11}\) presented an energy-based approach to obtain
the local buckling loads of blade-stringer-stiffened and
compressively loaded orthotropic composite plates. Seif and
Kabir\(^{12}\) developed an analytical approach for both sym-
metric and anti-symmetric local buckling of thin-plate in finite
sizes and with a center crack under tension, the solution of
which was based on the principle of minimum total potential
energy. Recently, Galerkin method, which is a powerful
numerical solution approach in solving differential equa-
tions, was used by Jaberzadeh and Azhari\(^{13}\) to analyze the
elastic and inelastic local buckling of flat rectangular plates
with centerline boundary conditions under non-uniform in-
plane compression and shear stress.

Similar to the local buckling problems of plates, a special
case about the local buckling of steel plates in steel-concrete
composite members has been studied by a few researchers.
Many of the studies were based on the numerical approach.
Uy and Bradford\(^{14}\) proposed a modified semi-analytical
finite strip method to study the elastic local buckling behav-
or of steel plates in composite steel-concrete members.
Liang and Uy\(^{15}\) studied the local and post-local buckling
of steel plates in concrete-filled tubular columns under axial
compression by finite element method and proposed
effective width formulas for the steel plates. Furthermore,
Liang et al.\(^{16,17}\) studied the local buckling strength of steel
plates in double skin composite panels and concrete-filled
thin-walled steel tubular beam-columns by using finite
element method. On the other hand, some were based on
energy method. Wright\(^{18}\) described an analytical model for
evaluating local buckling of a variety of sections where the
steel plates was in contact with a rigid medium. Recently,
Cai and Long\(^{19}\) and Long et al.\(^{20}\) applied Rayleigh-Ritz
method to obtain the solution of elastic buckling of steel
plates in rectangular concrete-filled steel tubular columns
with binding bars under axial and eccentric compressions.
Arabzade et al.\(^{21}\) used theoretical models to obtain the
elastic buckling coefficients of steel plates in composite steel
plate walls with various aspect ratios under shear loading
based on Rayleigh-Ritz method.

However, most of the studies on the local buckling of
steel plates in contact with concrete reported in the litera-
ture were concerned with the cases where the four edges
of the steel plate were assumed to be clamped or simply
supported. It should be acknowledged that the four edges
of the steel plate in CFT columns were restrained to some
extent by the surrounded components, but the restraints are
normally not stiff enough to be regarded as clamped. The
partial restraint has a pronounced impact on the local buck-
l ing behavior of CFT columns under out-of-plane compres-
sion. Therefore, the analytical solution of the CFT columns
elastically restrained along four edges is needed in order to
predict accurately the local buckling response of steel plates
in CFT columns.

This paper presents a theoretical study on the local buck-
l ing response of steel plates in CFT columns based on the
Rayleigh-Ritz method. Two opposite edges are subjected to

2. Analytical Formulation

2.1. Yield Line Model for Orthotropic Problem

The Rayleigh-Ritz method was first adopted by Timoshenko and Gere\(^{22}\) to obtain the solution of stability problems of the plate. The analysis in this paper is based
on this method, which also resembles the method used by
several previous researchers such as Mittelstedt\(^{11}\) and Cai
and Long.\(^{19}\) Figure 1 shows a steel plate in CFT column
subjected to axial compression \(N_x\) along the \(x\)-direction.

According to classical theory of elastic stability,\(^{22}\) in
the calculation of critical values of forces applied in the
middle plane of a plate at which the flat form of equilibrium
becomes unstable, the governing differential equation of
the deflection surface for the buckled plate is given by the
following equation

\[
\frac{\partial^4 w}{\partial x^4} + 2 \frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} = \frac{1}{D} \left( N_x \frac{\partial^2 w}{\partial x^2} + N_y \frac{\partial^2 w}{\partial y^2} + 2 N_{xy} \frac{\partial^2 w}{\partial x \partial y} \right)
\]

where \(D\) is called the flexural rigidity of the plate and is
given as,

\[
D = \frac{E t^3}{12(1 - v^2)}
\]

Using the energy method to investigate the local buckling
of the elastically restrained steel plate subjected to one axial
in-plane compression, the total potential energy of the steel
plate \(U\) is the summation of the strain energy stored in
the plate associate with the thin plate deforming \((U_t)\), the
strain energy associate with the elastic restraint along the
rotationally restrained boundary of the plate \((U_f)\), and the
energy associate with the external loads \((U_f)\). It can be mathematically expressed by,

Fig. 1. Analytical model of elastically rotationally restrained steel plate.
\[ \Pi = U + V + U_t \] .......................... (3)

The strain energy stored in the steel plate \( U \) is given as

\[ U = \frac{D}{2} \iint_{\Omega} \left( \frac{\partial^2 w}{\partial x^2} + \frac{\partial^2 w}{\partial y^2} \right)^2 - 2(1-\nu) \left( \frac{\partial^2 w}{\partial x^2} \frac{\partial^2 w}{\partial y^2} - \frac{\partial^2 w}{\partial x \partial y} \right)^2 \right) \, dx \, dy \] .......................... (4)

where \( \Omega \) is the area of the steel plate.

Assuming that the steel plate is elastically restrained along four edges with the elastic rotational restraint stiffness coefficients \( k_i \) along the two opposite loaded edges and \( k_s \) along the two opposite unloaded edges, the analytical model of the steel plate in CFT column in indicated in Fig. 1. For steel plate with rotational restraints distributed along the four edges, the strain energy \( U_t \) stored in the equivalent elastic rotational springs is expressed as

\[ U_t = \frac{1}{2} \int_{\Gamma_y} k_s \left( \frac{\partial w}{\partial y} \right)^2 \, dx + \frac{1}{2} \int_{\Gamma_x} k_s \left( \frac{\partial w}{\partial x} \right)^2 \, dy \] .......................... (5)

\[ + \frac{1}{2} \int_{\Gamma_y} k_i \left( \frac{\partial w}{\partial y} \right)^2 \, dx + \frac{1}{2} \int_{\Gamma_y} k_i \left( \frac{\partial w}{\partial y} \right)^2 \, dy \]

where \( k_i \) is the rotational stiffness of the elastic restraint at the loaded edges of \( x = 0 \) and \( \alpha \) (see Fig. 1) and \( \Gamma_y \) is along the width of the plate (\( \Gamma_y = 0 \) to \( b \)); \( k_s \) is the rotational stiffness of the elastic restraint at the unloaded edges of \( y = 0 \) and \( \beta \) (see Fig. 1) and \( \Gamma_x \) is along the length of the plate (\( \Gamma_x = 0 \) to \( a \)).

The energy associated with the external loads \( U_t \) can be written as

\[ V = \frac{1}{2} \iint_{\Omega} \left[ N_x \left( \frac{\partial w}{\partial x} \right)^2 + N_y \left( \frac{\partial w}{\partial y} \right)^2 + 2N_x \frac{\partial w}{\partial x} \frac{\partial w}{\partial y} \right] \, dx \, dy \] .......................... (6)

where \( N_x \) is the axial force per unit length at the boundary of \( x = 0 \) and \( a \); \( N_y \) is the axial force per unit length at the boundary of \( y = 0 \) and \( b \); and \( N_x \) is the force per unit in the transverse direction.

For the CFT columns subjected to axial compression, the boundary stress of the steel plate is \( \sigma_x = \tau_y = 0 \), and thus

\[ N_x = -\sigma_x t \] .......................... (7)
\[ N_y = -\sigma_y t = 0 \] .......................... (8)
\[ N_{xy} = -\tau_{xy} t = 0 \] .......................... (9)

### 2.2. Displacement Shape Function

It is of significant importance to choose the suitable form of the plate out-of-plane buckling displacement function \( w \) in order to solve the eigenvalue problem. In this research, a unique plate buckled displacement shape was adopted to obtain the explicit analytical solution for local buckling of the steel plate in CFT column under uniform in-plane compression in the \( x \) direction, as shown in Fig. 1.

The steel plate in composite wall only buckles outwards due to the restraint of the infilled concrete which prevents the plate buckling inwards. Assuming that the displacement shape in the \( x \) direction is a combine sine and cosine function and the deflection in the \( y \) direction is a biquadratic function, the displacement shape function can be expressed as

\[ w(x,y) = C \left[ \frac{y}{b} + \phi_1 \left( \frac{y}{b} \right)^2 + \phi_2 \left( \frac{y}{b} \right)^3 + \phi_3 \left( \frac{y}{b} \right)^4 \right] \times \left[ (1-\omega)\sin \frac{\pi x}{a} + \omega \left( 1-\cos \frac{2\pi x}{a} \right) \right] \] .......................... (10)

where \( C \) is a constant, and \( \phi_1, \phi_2, \phi_3, \) and \( \omega \) are the constants to be determined which should satisfy both the boundary conditions and the requirement of compatibility.

As shown in Fig. 1, the boundary conditions of the plate elastically restrained against rotation at loaded and unloaded edges are given by

\[ w(0,y) = w(a,y) = 0 \] .......................... (11a)
\[ M_x(0,y) = -D \left( \frac{\partial^2 w}{\partial x^2} \right)_{y=0} = -k_s \left( \frac{\partial w}{\partial x} \right)_{x=0} \] .......................... (11b)
\[ M_x(a,y) = -D \left( \frac{\partial^2 w}{\partial x^2} \right)_{y=a} = k_s \left( \frac{\partial w}{\partial x} \right)_{x=a} \] .......................... (11c)
\[ w(x,0) = w(x,b) = 0 \] .......................... (12a)
\[ M_x(x,0) = -D \left( \frac{\partial^2 w}{\partial x^2} \right)_{y=0} = k_s \left( \frac{\partial w}{\partial y} \right)_{y=0} \] .......................... (12b)
\[ M_x(x,b) = -D \left( \frac{\partial^2 w}{\partial x^2} \right)_{y=b} = -k_s \left( \frac{\partial w}{\partial y} \right)_{y=b} \] .......................... (12c)

Substituting the derivative function of Eq. (10) with respect to \( y \) into Eq. (11) gives the weight constant \( \omega \) in terms of the rotational stiffness of the elastic restraint \( k_i \) at the loaded edges:

\[ \omega = \frac{k_i a}{k_s a + 4\pi D} \] .......................... (13)

Substituting the first-order partial derivative and the second-order partial derivative of Eq. (10) with respective to \( y \) into Eq. (12), the unknown constants \( \phi_1, \phi_2, \phi_3 \) can be solved in terms of the elastic rotational restraint stiffness \( k_s \) along the two opposite unloaded edges as

\[ \phi_1 = \frac{k_s b}{2D} \] .......................... (14a)
\[ \phi_2 = -\frac{2D + k_s b}{D} \] .......................... (14b)
\[ \phi_3 = \frac{2D + k_s b}{2D} \] .......................... (14c)

By substituting Eq. (10) into Eqs. (4), (5), and (6), integrating and summing them according to Eq. (3), the total
potential energy of the steel plate is given as
\[
\Pi = \frac{D}{2} \left[ \frac{b}{a} \pi C^2 a^4 A_1 B_1 + \frac{a c^2}{b^2} A_2 B_2 + 2(1-v) \frac{\pi C^2}{ab} A_3 B_3 - 2v \frac{\pi C^2}{ab} A_4 B_4 \right] + \frac{\sigma_t b \pi C^2}{2a} A_5 B_5 + \frac{k a c^2}{2 b^2} \left( 1 + A_2^2 \right) B_6 + \frac{k_b b \pi C^2 (1-\omega)^2}{a^2} A_6
\]

where \(A_1, A_2, A_3, A_4, A_5, A_6, B_1, B_2, B_3, B_4, \) and \(B_5\) are defined as
\[
A_1 = \frac{1}{3} \phi_1 + \frac{1}{3} \phi_2 + \frac{2}{3} \phi_3 + \frac{4}{5} \phi_4 + \frac{2}{7} \phi_5 + \frac{2}{9} \phi_6
\]
\[
A_2 = 4 \phi_1^2 + 12 \phi_2^2 + \frac{144}{5} \phi_3^2 + 12 \phi_4 \phi_5 + 16 \phi_6 \phi_7 \ldots
\]
\[
A_3 = 1 + 2 \phi_2 + 2 \phi_3 + \frac{4}{3} \phi_4 + \frac{9}{5} \phi_5 + \frac{16}{7} \phi_6
\]
\[
+ 3 \phi_7 \phi_8 + 16 \phi_9 \phi_10 \ldots
\]
\[
A_4 = \phi_1 + 2 \phi_2 + 3 \phi_3 + \frac{2}{3} \phi_4 + \frac{6}{5} \phi_5 + \frac{12}{7} \phi_6
\]
\[
+ 2 \phi_7 \phi_8 + \frac{14}{5} \phi_9 \phi_10 + 3 \phi_11 \phi_12 \ldots
\]
\[
A_5 = 1 + 2 \phi_2 + 3 \phi_3 + 4 \phi_4 \ldots
\]
\[
A_6 = \frac{1}{3} \phi_1 + \frac{1}{2} \phi_2 + \frac{2}{5} \phi_3 + \frac{3}{5} \phi_4 + \frac{1}{3} \phi_5 + \frac{2}{7} \phi_6
\]
\[
+ \frac{1}{4} \phi_7 \phi_8 + \frac{1}{5} \phi_9 \phi_10 + \frac{1}{9} \phi_11 \phi_12 \ldots
\]
\[
B_1 = \frac{\pi (1-\omega)^2}{2} + 8 \pi \omega^2 - \frac{16 \omega (1-\omega)}{3}
\]
\[
B_2 = \frac{(1-\omega)^2}{2} + 3 \omega^2 + \frac{14 \omega (1-\omega)}{3}
\]
\[
B_3 = \frac{\pi (1-\omega)^2}{2} + 2 \pi \omega^2 - \frac{8 \omega (1-\omega)}{3}
\]
\[
B_4 = \frac{\pi (1-\omega)^2}{2} + 2 \pi \omega^2 + \frac{16 \omega (1-\omega)}{3}
\]
\[
B_5 = \frac{(1-\omega)^2}{2} + 3 \omega^2 + \frac{4 \omega (1-\omega)}{\pi}
\]

The first polynomial in Eq. (15) represents the change in strain energy due to the deforming of the plate, the second represents the work done by the applied loads, and the third and the fourth represent the energy associated with the elastic rotational restraint at the unloaded and loaded edges, respectively. Note that \(k_i = 0\) or \(k_\infty = 0\) stands for the loaded or unloaded edges being simply supported; while \(k_i \to \infty\) or \(k_\infty \to \infty\) represents loaded or unloaded edges being clamped. Any values of \(k_i\) or \(k_\infty\) between the two extreme conditions corresponds to the boundary edges being elastically restrained.

### 2.3. Explicit Solution

According to the principle of minimum potential energy, the value of \(\sigma_{t,\infty}\) can be obtained by the condition of minimizing the value of \(\Pi\) by taking a partial derivative of Eq. (15) with respect to \(C\) and setting the derivative equal to zero:
\[
\frac{\partial \Pi}{\partial C} = 0
\]

Substituting Eq. (15) into (17) and Rearranging gives
\[
\sigma_{t,\infty} = \frac{\pi^2 D}{b^2} \left[ \frac{B_1}{y^2 B_4} + \gamma^2 A_2 B_2 + \frac{2(1-v)A_3 B_3 - 2v A_4 B_4}{\pi^2 A_1 B_1} \right] \left[ \frac{2 \lambda_2 y^2 (1+ A_3^2) B_5}{\pi^3 A_1 B_4} + \frac{4 \lambda_5 (1-\omega)^2 A_6}{y^2 \pi A_1 B_4} \right]
\]

where \(\gamma\) is the half-wave parameter or so called aspect ratio and is defined as \(\gamma = a/b\); \(k\) is elastic local buckling coefficient of the steel plate as given in Eq. (19); \(\lambda_2\) and \(\lambda_5\) are elastically restraining factors of loaded and unloaded edges, respectively, as defined in Eqs. (20)–(21).
\[
k = \frac{B_1}{y^2 B_4} + \gamma^2 A_2 B_2 + \frac{2(1-v)A_3 B_3 - 2v A_4 B_4}{\pi^2 A_1 B_1} \left[ \frac{2 \lambda_2 y^2 (1+ A_3^2) B_5}{\pi^3 A_1 B_4} + \frac{4 \lambda_5 (1-\omega)^2 A_6}{y^2 \pi A_1 B_4} \right]
\]

By taking a partial derivative of Eq. (19) with respect to \(\gamma\) the minimum of Eq. (19) can be found when the critical aspect ratio \(\gamma_c\) is given by
\[
\gamma_c = \frac{1}{2D} \left[ \frac{\pi^3 A_1 B_1 + 4 \lambda_2 \pi^2 (1+ A_3^2) B_5}{A_2 B_2 + 2 \lambda_5 (1+ A_3^2) B_5} \right]^{\frac{1}{2}}
\]

Substituting Eq. (22) into Eq. (19), the critical elastic local buckling coefficient \(k_{cr}\) can be obtained. When the steel plate is relatively long (e.g., \(\gamma \geq \gamma_c\)), which is generally the case for steel plate in CFT column, the critical local buckling stress of the steel plate \((\sigma_{t,cr})\) elastically restrained against rotation at loaded and unloaded edges can be determined by substituting \(k_{cr}\) and \(D\) (Eq. (2)) into Eq. (18) as
While for some rare case with relatively short-span plates (e.g., \( \gamma < \gamma_c \)), the critical local buckling stress should be calculated directly by employing Eq. (18).

3. Validity of the Explicit Solution

In this section, the explicit formulas for two special cases which are commonly used in the practical CFT column design and analysis are first discussed and compared with previous theory and data. A more general case was then presented and the accuracy of the explicit local buckling solution based on the energy method was validated against experimental results.

3.1. Case 1: Clamped at Loaded Edge and Simply-supported at Unloaded Edges

When the elastic rotational restraint stiffness \( k_r \rightarrow \infty \), it results in \( \omega = 1 \) from Eq. (13), \( \lambda_c = 0 \) from Eq. (20), \( \lambda_c = 0 \) from Eq. (21), and \( \gamma_c = 1.519 \) from Eq. (22). This case is equivalent to the condition that the steel plate is clamped at the loaded edges and simply supported at the unloaded edges. Substituting \( \gamma_c = 1.519 \) into Eq. (19) gives the value \( k_{cr} = 5.467 \). Then the critical local buckling stress is

\[
\sigma_{cr} = \frac{5.467 \pi^2 E}{12 \left(1 - \nu^2\right) \left(b / t\right)^2} \text{ (24)}
\]

This result is close to \( k_{cr} = 5.6 \) suggested and used by Uy and Bradford\(^{14} \) based on finite strip method.

3.2. Case 2: Clamped at All Edges

When the elastic rotational restraint stiffness \( k_r \rightarrow \infty \) and \( k_c \rightarrow \infty \), it results in \( \omega = 1 \), \( \lambda_c \rightarrow \infty \), \( \lambda_r \rightarrow \infty \), and \( \gamma_c = 1.008 \). This case is equivalent to the condition that the steel plate is clamped at the loaded and unloaded edges. Substituting \( \gamma_c = 1.008 \) into Eq. (19) gives the value \( k_{cr} = 10.311 \). Then the critical local buckling stress is

\[
\sigma_{cr} = \frac{10.311 \pi^2 E}{12 \left(1 - \nu^2\right) \left(b / t\right)^2} \text{ (25)}
\]

This result is close to \( k_{cr} = 10.31 \) which was proposed by Uy and Bradford\(^{14} \), \( k_{cr} = 9.99 \) recommended by Bridge and O’Shea\(^{23} \) based on finite strip analysis, and \( k_{cr} = 9.81 \) obtained by Liang and Uy\(^{15} \) and Liang \textit{et al.}\(^{17} \) by a linear finite element buckling analysis of plates based on the bifurcation buckling theory.

3.3. Case 3: Clamped at Loaded Edges and Elastically-restrained at Unloaded Edges

For a more general case where the CFT column is under axial compression in practical engineering application, the loaded edges of the steel plate can be regarded as be clamped while the unloaded edges is more exactly to consider elastically rotationally restraint, \( i.e., \), it can be assumed that \( k_c \rightarrow \infty \). In this case and referring back to Eq. (19), the most important issue to deal with is to find out the most suitable solutions for \( \lambda_r \). Bleich\(^{24} \) proposed an equation to predict the elastically restraining factor for steel plate in box steel tube without infilled concrete as shown in Eqs. (26)–(28).

\[
\lambda_r = \left( \frac{t_c}{t_f} \right)^3 \frac{r}{\rho} \text{ (26)}
\]

\[
r = 1 - \left( \frac{t_r b_r}{t_w b_w} \right)^2 \text{ (27)}
\]

\[
\rho = \frac{1}{\pi} \tanh \left( \frac{\pi b_w}{4b_r} \right) \left[ 1 + \pi b_w / 2b_r \sinh \left( \pi b_w / 2b_r \right) \right] \text{ (28)}
\]

where \( b_r \) and \( t_f \) are the width and thickness of the calculated steel plate, respectively; \( b_w \) and \( t_w \) are the width and thickness of the adjacent steel plate, respectively. Bleich\(^{24} \) suggested values of \( k_{cr} = 4.0 \) and \( k_{cr} = 6.97 \) for steel plate with loaded edges clamped and unloaded edges simply supported and clamped, respectively. Note that in Bleich’s research, the steel plate was not restrained by the concrete. Comparing these results with the critical elastic local buckling coefficient for steel plate infilled by concrete (case 1 and case 2 in this paper), it can be observed that the critical local buckling stress increases by around 37% to 50% due to the presence of concrete. Therefore, in order to account for this effect, a reduction factor of 0.5 is considered to reduce \( \gamma \). It should be mentioned that a larger value (0.5 rather than 0.37) was selected due to the fact that this leads to the predicted value of \( \sigma_{cr} \) on the conservative side as can be seen from Eqs. (19), (23) and (26). Consequently, Eq. (27) is modified as

\[
r' = 1 - \beta \left( \frac{t_r b_r}{t_w b_w} \right)^2 \text{ (29)}
\]

where \( \beta = 0.5 \) is the reduction factor used for considering the beneficial restraining effects offered by concrete.

In order to validate the accuracy of the proposed formulas, the theoretical results from the equations proposed above are compared with the experimental data obtained by Uy\(^{25,26} \) and Mo \textit{et al.}\(^{27} \) as shown in Table 1. The theoretical value of \( \sigma_{cr} \) was obtained according to Eq. (23), where \( k_{cr} \) was calculated from Eq. (19) and \( \lambda_c \) was determined from Bleich’s formula of Eq. (26) including modified \( \gamma \). It should be note that the data of cases where the yielding occurs before the local buckling of steel plate have been excluded in Table 1.

Uy\(^{25} \) conducted an extensive set of experiments for the local buckling response of steel plates restrained by infilled concrete. The tests included five specimens (with concrete) which are identified in Table 1. The experiments were undertaken in a 5 000 kN capacity compression testing machine. The composite section was designed to bear the load uniformly. The local buckling half-wavelengths were measured throughout the tests. The results demonstrated that the measured half-wavelengths for all the tested specimens were initially equal to the width of the steel plate.
The proposed solution was first discussed based on two extreme cases where the steel plate was clamped at the loaded edges and simply supported or clamped at the unloaded edges. The predicted values of elastic local buckling coefficient agrees well with the numerical results by various researchers. A more general case was then presented and comparisons made between the proposed formulas and extensive experimental data show that the model provides remarkably accurate predictions of the local buckling stresses that are suitable for use. Furthermore, the increase in the width to thickness ratio results in a reduction in the critical buckling stress. The impact of thickness to width ratio of the steel plate becomes significant for the cases where the local buckling occurs before the yielding. The work in this paper provide basis for the capacity design of CFT column under axial compression.

Acknowledgments
This work is sponsored by Jiangsu Provincial Natural Science Foundation of China (Grant No. BK20170685), A Project Funded by the Priority Academic Program Development of Jiangsu Higher Education Institutions (PAPD). The authors also appreciate the idea on development of displacement function by Dr. Luyang Shan and Prof. Pizhong Qiao at Washington State University, which provides solid basis for the development of this paper.

Notation

\( a \): a half buckling wave length  
\( b \): width of steel plate  
\( b, b \): width of the calculated steel plate and adjacent steel plate, respectively  
\( C \): A constant  
\( D \): flexural rigidity of the plate  
\( E \): elastic modulus of steel plate  
\( k \): elastic local buckling coefficient of the steel plate  
\( k_c \): critical elastic local buckling coefficient of the steel plate  
\( k_r \): rotational stiffness of the elastic restraint at the

Table 1. Comparison of experiments by other researchers and theoretical results.

| Specimen No. | \( b \) (mm) | \( t_1 \) (mm) | \( t_2 \) (mm) | \( \sigma_{e,cr} \) (MPa) | \( \sigma_{p,cr} \) (MPa) | \( \sigma_{e,cr}/\sigma_{p,cr} \) | Reference |
|--------------|--------------|---------------|---------------|--------------------------|--------------------------|-----------------------------|-----------|
| LB7          | 240          | 3             | 3             | 200                      | 197                      | 0.985                        | Uy (1998)     |
| LB9          | 300          | 3             | 3             | 120                      | 126                      | 1.050                        |           |
| FB1          | 360          | 3             | 3             | 93.4                     | 88                       | 0.942                        |           |
| FB2          | 420          | 3             | 3             | 79.9                     | 64                       | 0.801                        |           |
| FB3          | 480          | 3             | 3             | 43.7                     | 49                       | 1.121                        |           |
| FB4          | 540          | 3             | 3             | 38.8                     | 39                       | 1.005                        |           |
| SCC2         | 200          | 3             | 3             | 246                      | 283                      | 1.150                        | Mo et al. (2004) |
| SCC3         | 200          | 3             | 2             | 191                      | 221                      | 1.157                        |           |
| SCC4         | 200          | 2             | 5             | 166                      | 183                      | 1.102                        |           |
| SCC6         | 200          | 2             | 2             | 118                      | 126                      | 1.068                        |           |
| Average      | 253.346      | 2.77          | 2.77          | 217.236                  | 236.866                  | 1.034                        |           |
| Standard deviation | 0.104 |           |               |                          |                          |                               |           |

*Note: \( t_1 \) and \( t_2 \) = thickness of steel plate at the loaded and unloaded sides, respectively; \( \sigma_{e,cr} \) = local buckling strength from experimental results; \( \sigma_{p,cr} \) = local buckling strength determined from analytical model.
loaded edges of $x = 0$ and $a$

$k_y$: rotational stiffness of the elastic restraint at the unloaded edges of $y = 0$ and $b$

$N_x$: surface force in the normal direction along x-axis

$N_y$: surface force in the normal direction along y-axis

$N_{xy}$: surface force in the transverse direction

$t$: thickness of steel plate

$t_1$, $t_2$: thickness of steel plate at the loaded and unloaded sides, respectively

$t_0$: thickness of the calculated steel plate and adjacent steel plate, respectively

$U$: strain energy stored in the plate associate with the thin plate deforming

$U_I$: work done by the external loads

$V$: strain energy stored in elastic restraint edges

$\sigma_{cr}$: critical local buckling stress of the steel plate

$\sigma_{cr, ex}$: measured local buckling stress from the experiments

$\sigma_{cr, pr}$: predicted local buckling stress based on the proposed formulas

$\sigma_x$, $\sigma_y$: normal stress in the $x$ and $y$ direction, respectively

$\tau_{xy}$: shear stress in the steel plate

$w$: deflection function

$\nu$: Poisson’s ratio of steel ($\nu = 0.3$)

$\Pi$: total potential energy of the steel plate

$\Omega$: area of the plate

$\Gamma_x$: along the length of the plate ($\Gamma_x = 0$ to $a$)

$\Gamma_y$: along the width of the plate ($\Gamma_y$ to $b$)

$\phi_x$, $\phi_y$, $\phi_z$: unknown constants representing the deflection along the $y$ direction

$\alpha$: weight constant representing the deflection shape along the $x$ direction

$\varepsilon$: measured local buckling strain from the tests

$\beta$: reduction factor used for considering the beneficial restraining effects offered by concrete

$\gamma$: aspect ratio ($\gamma = a/b$)

$\gamma_c$: critical aspect ratio

$\lambda_{cr}$, $\lambda_{y}$: elastically restraining factors of loaded and unloaded edges, respectively

REFERENCES

1) S. Iwamoto, S. Ochiai and H. Okuda: *ISIJ Int.*, 49 (2009), 139.
2) S. Iwamoto, S. Ochiai, H. Okuda and T. Inoue: *ISIJ Int.*, 47 (2007), 930.
3) J. D. Rodrigues, C. M. C. Roque, A. J. M. Ferreira, E. Carrera and M. Cinefra: *Compos. Struct.*, 93 (2011), 1613.
4) N. Jafari, M. Azhari and A. Heidarpour: *Struct. Eng. Mech.*, 40 (2011), 783.
5) P. Qiao and L. Shan: *Int. J. Struct. Stab. Dyn.*, 7 (2007), 487.
6) L. Shan and P. Qiao: *Eng. Struct.*, 30 (2008), 126.
7) P. Qiao and X. Huo: *Compos. Struct.*, 93 (2011), 2785.
8) P. Qiao, F. Chen, J. Xu and L. Lu: *J. Struct. Eng.*, 139 (2013), 936.
9) J. H. Kang and A. W. Leissa: *Int. J. Solids Struct.*, 42 (2005), 4220.
10) H. Zhong and C. Gu: *Compos. Struct.*, 80 (2007), 42.
11) C. Mittelstedt: *Thin-Walled Struct.*, 45 (2007), 371.
12) A. E. Seif and M. Z. Kabir: *Lat. Am. J. Solids Struct.*, 12 (2015), 2078.
13) E. Jaberzadeh and M. Azhari: *Appl. Math. Model.*, 33 (2009), 1874.
14) B. Uy and M. A. Bradford: *Eng. Struct.*, 18 (1996), 193.
15) Q. Q. Liang and B. Uy: *Comput. Struct.*, 75 (2000), 479.
16) Q. Q. Liang, B. Uy, H. D. Wright and M. A. Bradford: *J. Eng. Struct.*, 130 (2004), 443.
17) Q. Q. Liang, B. Uy and J. Y. R. Liew: *J. Constr. Steel Res.*, 63 (2007), 396.
18) H. D. Wright: *Struct. Eng.*, 71 (1993), 209.
19) J. Cai and Y. L. Long: *J. Constr. Steel Res.*, 65 (2009), 965.
20) Y. L. Long, J. Wan and J. Cai: *J. Constr. Steel Res.*, 120 (2016), 70.
21) A. Arabzade, H. Moharami and A. Ayazi: *Sci. Iran.*, 18 (2011), 9.
22) P. Qiao and L. Shan: *Appl. Math. Model.*, 33 (2009), 1874.
23) R. Q. Bridge and M. D. O’Shea: *J. Constr. Steel Res.*, 47 (1998), 73.
24) F. Bleich: *Buckling Strength of Metal Structures*, McGraw-Hill, New York, (1961), 348.
25) B. Uy: *J. Constr. Steel Res.*, 47 (1998), 47.
26) B. Uy: *J. Struct. Eng.*, 127 (2001), 66.
27) S. Mo, R. Zhao and X. Zhong: *J. Hunan Univ. Sci. Technol. (Nat. Sci. Ed.),* 19 (2004), 43.