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DUCTILITY PERFORMANCE OF THIN-WALLED COMPOSITE-FILLED (TWFC) BEAM AT INTERNAL SUPPORT

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ABSTRACT

Ductility of reinforced structures is a desirable property where resistance to brittle failure during flexure is required. The most influential parameter towards the ductility value is the ratio of steel area to concrete area. However, the study on ductility performance of TWCF beam is still lacking. This paper aims is to reveal the ductility performance of Thin-Walled Composite-Filled Beam under flexural test. This paper presents a laboratory work which involves three samples of TWCF beams with different strength-enhancement devices. The effects of various modes interface connections are co-related to the generation of shear bond between sheeting and concrete, using both experimental and theoretical results. The strength and failure modes of the beams are found to be dependent on the interface connections. Analytical models for the ductility design of beams are developed and their performance is validated through experimental results using partial shear connection. The beams were designed by considering whether the strength will be governed by buckling or yielding of the steel plate. The analytical ductility model of TWCF beam shows a good agreement with the experimental result.

Keywords: ductility; flexural; partial shear connection; shear bond; strength-enhancement.

1. INTRODUCTION

The use of thin-walled composite sections comprising cold-form open steel box sections with an infill of concrete is an alternative for composite beams (Hossain, 1998). The strength of such beams is limited by the compression buckling capacity of the steel plates at the top of the open box section. Enhancement of strength is possible by stiffening the compression steel plates at the open end of the box section with various modes of interface connections or strength-enhancement devices. Flexural capacity of such beams can be derived based on either yielding or buckling of steel depending on the generated steel-concrete interface shear bond simulating full or partial shear connections. Thin-walled composite sections do not require temporary formwork for in-fill concrete as the steel acts as formwork in the construction stage and as the reinforcement in the serviceability stage. They are simple to fabricate and construct in comparison with conventional reinforced concrete. The in-fill concrete is generally cured quickly and in any case, the load capacity of the steel alone may be relied upon for most construction loads. An investigation of sandwiched composite beams

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with varied plate thickness, stud spacing and length, and concrete strength,was carried out by Oehlers et al.,(1989).The study found that tested beams exhibited three types of failures: flexural (steel yield prior to concrete crushing), horizontal slip (failure of shear connectors) and vertical shear (due to insufficient shear capacity of concrete and studs). The flexural behavior of profiled composite beams is a significant factor to determine the ductility of TWCF beam. Ductility may be defined as the ability to undergo deformations without a substantial reduction in the flexural capacity of the member (Park & Ruitong, 1988). According to Silvestre & Camotim (2002), the deformability is influenced by some factors such as the tensile reinforcement ratio, the amount of longitudinal compressive reinforcement, the amount of lateral tie and the strength of concrete. The ductility of reinforced concrete section could be expressed in the form of the curvature ductility (μ_ϕ):

$$\mu_\phi = \phi_u / \phi_y \tag{1}$$

Where ϕ_u is the curvature at ultimate when the concrete compression strain reaches a specified limiting value, ϕ_y is the curvature when the tension reinforcement first reaches the yield strength. The definition of ϕ_y shows the influence of the yield strength of reinforcement steel on the calculation of μ_ϕ while the definition of ϕ_u reflects the effect of ultimate strain of concrete in compression.

2. THEORETICAL BACKGROUND

For partial shear connection, $N_c \neq N_s$. Consider the equilibrium of forces in concrete in Figure 1.0.

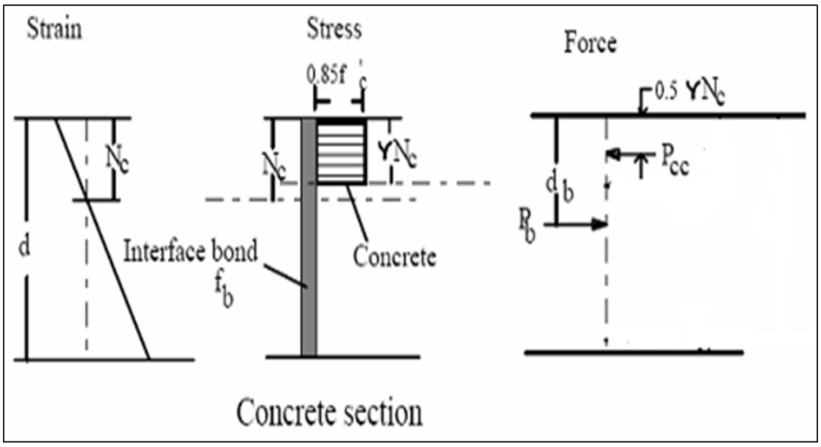

Figure 1: Distribution of forces in the concrete section.

$$P_b = P_{cc}$$

$$P_b = 0.85 f_c' \gamma N_c b_c \tag{2}$$

$$N_c = \frac{P_b}{0.85 f_c' \gamma b_c} \tag{3}$$

The interface bond force of the beam can be written as:

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$$P_b = \sum_o x\, f_b \tag{4}$$

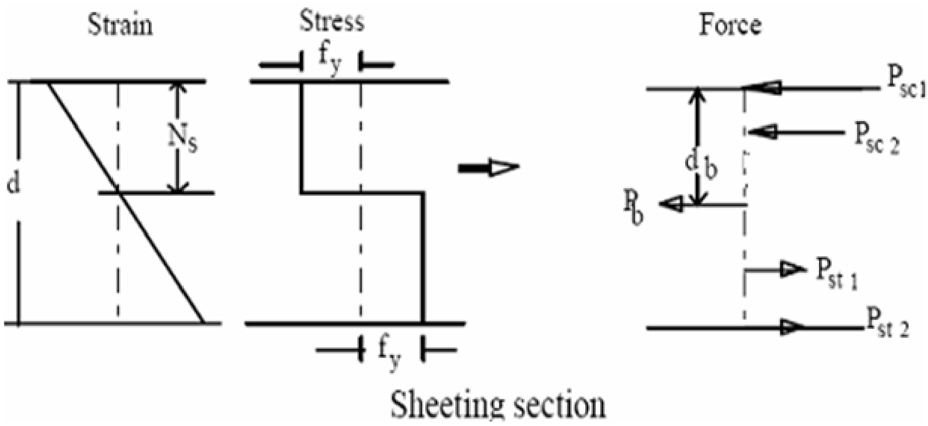

Figure 2: Distribution of forces in the sheeting section.

Considering the equilibrium of forces in sheeting section in the Figure 2:

$$P_b + P_{sc1} + P_{sc2} = P_{st1} + P_{st2}$$

By substituting:

$$P_b + b_s t_s f_{sy} + 2 f_{sy} N_s t_s = 2(d - N_s) f_{sy} t_s + \quad w t_s f_{sy} + 2 s f_{sy} t_s$$

$$N_s = \frac{f_{sy} t_s (2d + w + 2s - b_s) - P_b}{4 t_s f_{sy}} \tag{5}$$

Taking the moment of all the forces about the top fibre of the beam, the moment capacity (***M_u***) is given as follows:

$$M_u = t_s f_{sy}(d^2 + wd + 2s - 2N_s^2)\ 0.425\gamma^2 N_c^2 b_c f_c' \tag{6}$$

When ***w*** = 0

$$M_u = t_s f_{sy}(d^2 + 2sd - 2N_s^2) - 0.425\gamma^2 N_c^2 b_c f_c' \tag{7}$$

However, before TWCF is being designed it is necessary to determine whether the strength is failure prior due to yielding of steel plate (σ_b) or due to the buckling stress (f_{sy}) of steel plate. The equation of buckling stress is given as follows (Thimoshenko and Gere, 1961):

$$\sigma_b = \frac{k_b \pi^2 E_s}{12(1 - v^2)} \left(\frac{t_s}{d}\right)^2 \tag{8}$$

For beams with yielding commencing before buckling, strength should be predicted by using yield stress of steel plate (f_{sy}) in design equations whereas for beams with buckling

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commencing before yielding, strength should be predicted by using buckling stress of steel plate in design equations. The strain diagrams for concrete and steel sheeting need to be developed solely to derive the ductility of TWFC beam for partial shear connection, $N_c \neq N_s$. Consider the strain diagram in Figure 3 and Figure 4 for concrete and steel sheeting respectively.

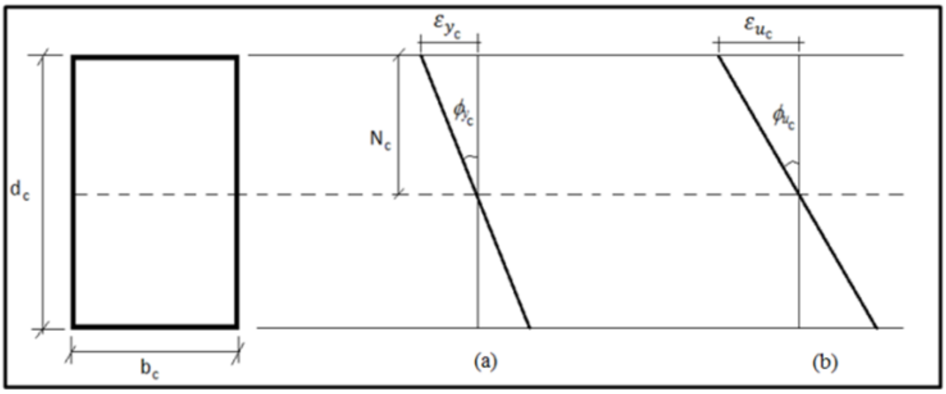

Figure 3: Concrete cross-section: (a) Strain of concrete at yield state (b) Strain of concrete at ultimate state.

From the strain diagram of concrete at yield state:

$$\emptyset_{y_c} = \frac{\boldsymbol{\varepsilon_{y_c}}}{\boldsymbol{N_C}} \tag{9}$$

where from BS 8110-1:1997ε_{yc} for concrete is equal to:

$$2.4\ x\ 10^{-4}\sqrt{\frac{f_{cu}}{\gamma_m}}$$

From the strain diagram of concrete at ultimate state:

$$\emptyset_{u_c} = \frac{\boldsymbol{\varepsilon_{u_c}}}{\boldsymbol{N_C}} \tag{10}$$

By substituting equation (9) & (10) into equation (1) yield the ductility equation for concrete:

$$\boldsymbol{\mu_c} = \frac{\boldsymbol{\varepsilon_{u_c}}}{\boldsymbol{\varepsilon_{y_c}}} \tag{11}$$

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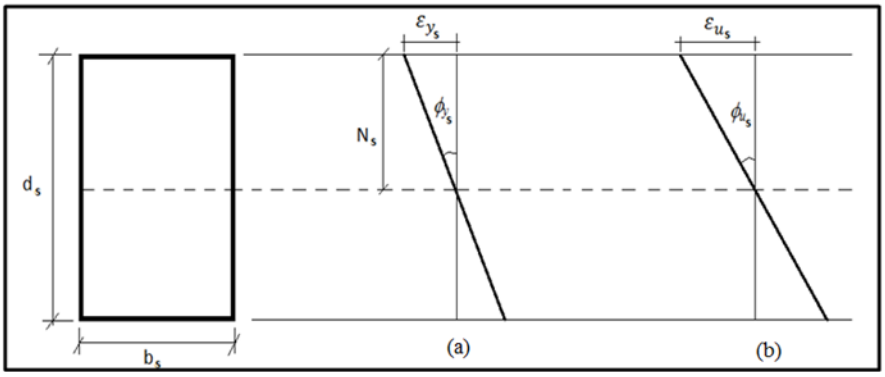

Figure 4: Steel sheeting cross-section: (a) Strain of steel sheeting at yield state (b) Strain of steel sheeting e at ultimate state.

From the strain diagram of steel sheeting at yield state:

$$\emptyset_{y_s} = \frac{\varepsilon_{y_s}}{N_C} \tag{12}$$

From the strain diagram of steel sheeting at ultimate state:

$$\emptyset_{u_s} = \frac{\varepsilon_{u_s}}{N_C} \tag{13}$$

By substituting equation (12) & (13) into equation (1) yield the ductility equation for steel sheeting:

$$\mu_s = \frac{\varepsilon_{u_s}}{\varepsilon_{y_s}} \tag{14}$$

By summing equation (11) & (14) yield the ductility of TWCF beam as follows:

$$\mu = \mu_c + \mu_s = \frac{\varepsilon_{u_c}}{\varepsilon_{y_c}} + \frac{\varepsilon_{u_s}}{\varepsilon_{y_s}} \tag{15}$$

Where:

ε_{uc} is the ultimate compressive strain of concrete which normally considered equal to 0.0035.

ε_{us} is the ultimate tension strain of steel sheeting in the experiment.

ε_{ys} is theyield strain of steel sheeting in the experiment.

3. FINDING FROM PREVIOUS RESEARCH

From past research, a lot of studies that have been conducted dealing with behavior of composite beam such as behavior of thin walled composite sections as structural elements (Hossain, 1998), Composite structures of steel and concrete, Johnson, (1975) and Flexural

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Strength of Profiled Sheeting Beams (Oehlers et al,1989). In fact, the behavior of dodecagonal section double skin concrete-filled steel beam-columns was also studied. Both experimental investigation and finite element analysis were carried out. Load─displacement curves and load─strain curves were obtained. All tested specimens exhibited good ductility(Ju et al.,2016). The flexural behaviour of profiled composite beams was also investigated by Oehlres (1989). He suggested that the flexural strength of such beams could be adversely affected by the local buckling of the sheeting. Davies (1998) and his collaborators have extensively applied the Generalised Beam Theory (GBT) to investigate the buckling behaviour of thin-walled cold-formed steel members and their work has provided a strong contribution towards establishing this theory. The shear bond at the sheet-concrete interface is vital to determine the structural performance of composite beam. According to Patrick (1990), the maximum shear bond stress at the sheet-concrete interface due to mechanical interlock in the form of different types of embossment rolled into the sheet ranges from 0.2 to 0.5 N/mm^2. For plain profile sheeting with no embossments, a value of 0.1N/mm^2 can be used. Analytical models have been developed based on the models proposed by Oehlers (1989) and Hossain (1995) for profiled composite beams taking into consideration of the buckling and failure modes of the experimental beams.These studies revealed that the application of the steel thin plate which replace bar as reinforcement in beam. Based on these studies, the researchers have found that the behavior of composite beam the buckling, steel-concrete interface bond, and strength-enhancement devices are the main factors which influence the strength of the beams. In order to study the flexural behavior of TWCF beams, Hossain (2003) derived the formulas with design consideration as a guideline to design the TWCF beams. Moreover, he has also performed experimental works in order to validate the analytical design model. Hence, this research extends the work of Hossain (2003) who has revealed the performance of TWCF beams as a structural element.

4. INSTRUMENTATIONS, EXPERIMENTAL SET-UP AND TESTING PROCEDURE.

Three samples of TWCF beams (Figure 5.0) have been tested in the laboratory. The details of each sample can be seen in Table 1.0. Static load tests were conducted on three samples of TWCF beams based on instrumentations and experimental set-up as shown in Figure 6.0. To simulate the situation at internal support, the beam was turned upside down and the opening part was allocated at the bottom. This was to ensure that the opening and welded top plate of the beams would be subjected by tension bending. The strain gauges were installed at the require place on the samples. This was followed by putting LVDT at bottom mid-span of the beams as shown in Figure 6.0. Static load test of four point load contact on the beams was undertaken by applying increment of load. All the wires from the strain gauges and LVDT were connected to the data logger and the reading of deflection, strain, and load increment were recorded by computer.

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Table 1: Details of the TWCF beam specimens.

Type of Beam	Compressive Strength of Concrete (N/mm^2)	Yield Stregth of Steel Plate (N/mm^2)	Dimension				
			Length L (mm)	Width b (mm)	Depth d (mm)	Opening o (mm)	Thickness Plate t_s (mm)
Open Top, (OT)	30	250	1100	125	140	45	1.5
Fully Close Welded Top Plate, (FCWTP)	30	250	1100	125	140	-	1.5
Half Close Welded Top Plate, (HCWTP)	30	250	1100	125	140	-	1.5

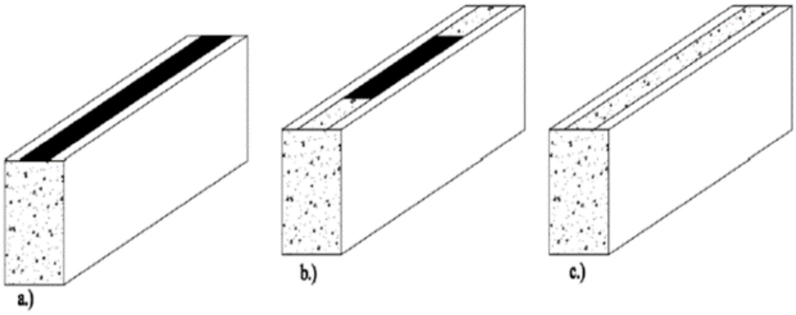
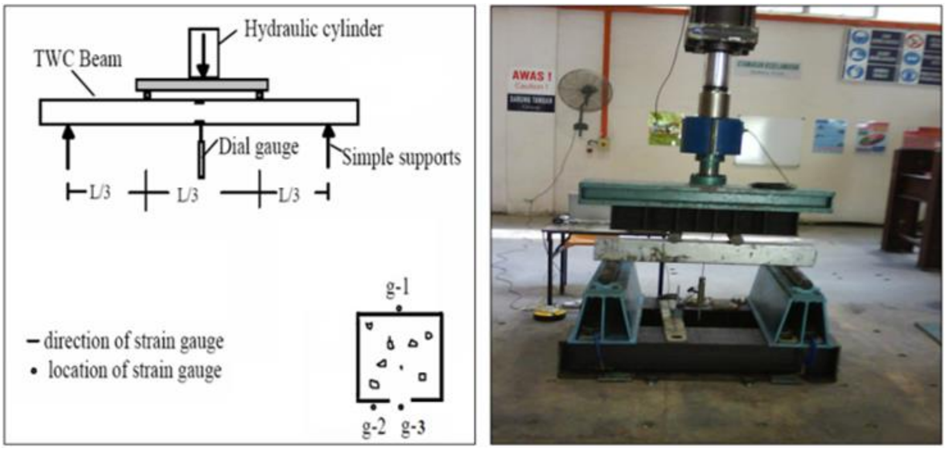


Figure 5: Specimens (a) FCWTP (b) HCWTP (c) OT.



(a) Schematic diagram (b) actual diagram

Figure 6: Experimental set-up.

5. DATA ANALYSIS AND RESULTS

5.1 Load Carrying Capacity and Maximum Deflection

From the experimental work, graph of load against deflection was plotted to determine performance of TWCF beams. Figure 7 shows a plotted graph of load against deflection and load against strain subject to static loading for Open Top sample, Fully Close Welded Top Plate sample and Half Close Welded Top Plate sample respectively. Figure 7, shows that all the three samples were beheaved linearly until they reached the yield point at different load

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values. Open Top sample was yielded at load applied 40kN and both Fully Close Welded Top Plate sample and Half Close Welded Top Plate sample at 60kN. The graph of load against strain, indicated that all analytical models for three samples of TWCF beam could be developed based on the buckling of steel plate.The maximun load carrying capacity for Open Top, Fully Close Welded Top Plate sample and Half Close Welded Top Plate sample was 53kN, 120kN and 100kN respectively. On the other hand the maximun deflection for all the three samples of TWCF beam was within allowable deflection which was 4.5mm while the maximun deflection for Open Top samples was 4.0mm, Fully Close Welded Top Plate sample was 3.51mm and for Half Close Welded Top Plate sample was 2.78mm and maximum deflection was taken at yield point.

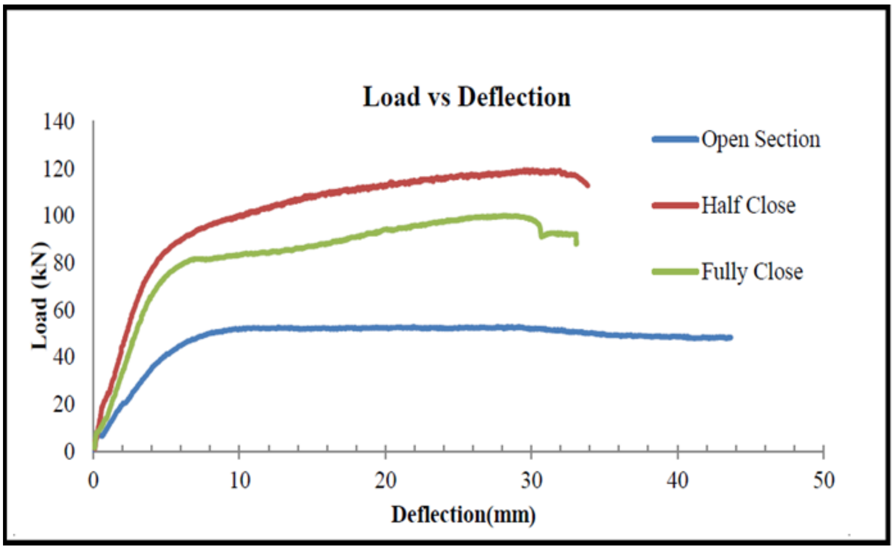

Figure 7: Load vs. Deflection for TWCF samples.

5.2 Ductility of TWCF Beam

From the experimental work, ductility for all three samples of TWCF was determined and shown in Table 2. Ductility value from the experimental work was validated using theoretical ductility that has established in section 2.0 previously. Both values show a reasonable agreement since the ratio is ranging from 0.96 to 0.99. TWCF beam show an increment of ductility value from open to full close of TWCF beam. The HCWTP type shows the most ductile specimen while the OT type is the least ductile one. Thus, the steel sheeting at the top of specimen is able to increase the ductility of TWCF beam. Yet, it is ample to half close the top specimen instead of fully close since there is no significant increase in ductility as shown in Table 2.

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Table 2: Comparison of ductility obtain from experimental work and theoretical.

Type of Beam	Experimental Ductility	Theoretical Ductility	Ratio Theoretical to Experimental Ductility
Open Top, (OT)	4.86	4.7	0.97
Fully Close Welded Top Plate, (FCWTP)	5.60	5.4	0.96
Half Close Welded Top Plate, (HCWTP)	5.83	5.8	0.99

6. CONCLUSION

The experimental result,shows a reasonable agreement with the design equation derived from the research experiment. Ductility behavior of TWCF has the desirable performance required in structural design provision. The analytical ductility model of TWCF beam shows a good agreement with the experimental result. The strength of TWCF beams is limited by buckling capacity of the steel plate at the bottom of the open box section. The strength-enhancement devices used in such beams enhance the strength by stiffening the tension steel plates at the open end of the box section. The effect of strength-enhancement devices on the strength, steel-concrete interface shear bond, buckling of steel plates and failure modes of the beams are identified. The flexural behavior of the TWCF beams in the interior support has been studied through the depicted of the load-deflection response and load-strain relationship. The study confirmed that thin-walled composite beams have great potential to be used in buildings construction.

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NOMENCLATURE

b_s, d, L	width, depth and span of the beams, respectively.
o, s	width of opening, top stripped steel.
t_s	thickness of steel.
v, E_s	Poisson's ratio and Modulus of elasticity of stee,l respectively.
P_{fc}, M_{fc}	Ultimate load and moment for full shear connection, respectively.
P_p, M_p	Ultimate load and moment for partial shear connection, respectively.
P_{exp}, M_{exp}	Experimental load and moment, respectively.
N_c, N_s	Neutral axis position for concrete and steel section, respectively.
f_b, ε	shear bond stress at the interface and degree of edge restrain, respectively.
b_c	width of net concrete section.
P_{cc}, P_b	compressive force due to concrete and force due to shear bond, respectively.

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$\boldsymbol{P_{bfc}}$	bond strength required to achieve full shear connection.
$\boldsymbol{\sigma_b}$, $\boldsymbol{k_b}$	buckling stress and bending buckling coefficient of steel plate, respectively.
$\boldsymbol{f_{sy}}$, $\boldsymbol{f'_c}$, $\boldsymbol{f_{cu}}$	yield stress of steel plate, cylinder concrete strength and cube concrete strength, respectively.

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