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Local and post-local buckling of double skin composite panels

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Double skin composite (DSC) panels are constructed by filling concrete between two steel plates welded with stud shear connectors at a regular spacing. Steel plates in DSC panels used as two-way slabs or shearwalls may buckle locally between stud shear connectors when subjected to in-plane biaxial compression. This paper investigates the local and post-local buckling behaviour of biaxially compressed steel plates restrained by shear connectors and concrete in DSC panels by using the finite element modelling technique. Local buckling coefficients are obtained for steel plates with various aspect ratios, biaxial loading and boundary conditions by incorporating the shear stiffness effect of stud shear connectors. These buckling coefficients can be used to determine limiting width-to-thickness ratios for steel plates and the distribution of stud shear connectors. A geometric and material nonlinear analysis is undertaken to quantify the post-local buckling strength of steel plates under biaxial compression. The initial imperfections of steel plates and nonlinear shear-slip characteristics of stud shear connectors are taken into account in the post-local buckling analysis. Based on numerical results, biaxial strength interaction curves and formulas are developed for the ultimate strength design of steel plates in double skin composite construction.

NOTATION

a length of plate field between shear connectors

b width of plate field between shear connectors

E Young's modulus of elasticity

$E_{0.7}$ secant modulus $E_{0.7} = 0.7E$

k_s shear stiffness of shear connector

k_x	elastic local buckling coefficient in the x direction
k_y	elastic local buckling coefficient in the y direction
Q	longitudinal shear force
Q_u	ultimate shear strength of shear connector
t	thickness of plate
w	lateral deflection
w_0	initial out-of-plane deflection
α	ratio of transverse to longitudinal loading $\alpha = \sigma_y / \sigma_x$
δ	longitudinal slip
η	shape factor of strength interaction curves
ζ	shape factor of strength interaction curves
γ	uniaxial strength factor
ε	strain
ν	Poisson's ratio
σ	stress
σ_0	yield stress or 0.2% proof stress
$\sigma_{0.7}$	stress corresponding to $E_{0.7} = 0.7E$
σ_{xcr}	critical buckling stress in x direction
σ_{ycr}	critical buckling stress in y direction
σ_{xu}	ultimate strength of plate in biaxial compression in x direction
σ_{yu}	ultimate strength of plate in biaxial compression in y direction
σ_x	applied edge stress in x direction
σ_y	applied edge stress in y direction

φ plate aspect ratio, $\varphi = a/b$

1. INTRODUCTION

Double skin composite (DSC) panels are structural elements that comprise of two external steel plates connected to a concrete core by welded stud shear connectors, as shown in Fig. 1. Stud shear connectors resist the longitudinal shear between steel plates and the concrete core, and separation at the interface. This composite system offers many structural and economical advantages over conventional doubly reinforced concrete elements. By filling concrete, the composite panel provides high structural performance in terms of its strength, stiffness and ductility. Steel plates in DSC panels act as permanent formwork and biaxial steel reinforcement for the concrete. The need for plywood formwork and the detailing of steel reinforcing bars is completely eliminated. This significantly reduces the construction time and costs. Moreover, steel skins provide sound waterproofing in marine and freshwater environment. This system was originally developed for use in submerged tube tunnels.[1] Owning to its high structural performance and construction advantages, its potential applications are being extended to nuclear installations, liquid and gas containment structures, military shelters, offshore structures, and shearwalls in buildings.

Experimental behaviour of DSC elements has been investigated by Oduyemi and Wright,[2] and Wright et al.[3] In these experiments, mild steel plates were used to form the skins of composite panels with welded studs as shear connectors. Tests indicated that there were two particular failure modes associated with this composite system. One was the local buckling of steel skins between stud shear connectors when subjected to compression, depending on the plate thickness and stud spacing in two directions. The second was the shear connection failure between steel plates and the concrete core. Wright et al.[4] and Wright and Oduyemi[5] showed that DSC elements could be analysed and designed in accordance with the conventional theories for doubly reinforced concrete elements and composite structures, providing that the local buckling of steel plates and shear connection failure are adequately taken into account. Clubley and Xiao[6] and Bowerman and Pryer[7] reported the behaviour and potential applications of Bi-Steel composite panels produced by Corus (formally British Steel). In Bi-Steel composite panels, stud shear connectors are replaced with welded steel bars.

Local buckling of steel plates in contact with concrete is a unilateral buckling problem in which steel plates are forced by the rigid concrete to buckle in one lateral direction. The restraint of concrete considerably increases the local and post-local buckling strength of steel plates. This beneficial effect has been of increasing concern to researchers. Ge and Usami[8,9] conducted experimental and numerical investigations on the local buckling and strength of concrete-filled steel box columns with and without internal stiffeners. Wright[10,11] studied the local buckling behaviour of steel plates in contact with concrete by using an energy method, and derived limiting width-to-thickness ratios for plates with various boundary conditions. Push-out tests and the finite strip method have been used to quantify the local buckling characteristics of thin steel plates in composite steel-concrete members by Uy and Bradford[12] and Uy.[13] The strength design of composite columns and profiled composite walls incorporating local buckling effects was considered by Uy[14] and Uy et al.[15] Moreover, Liang and Uy[16,17] presented a theoretical study on the post-local buckling strength of steel plates in concrete-filled thin-walled box columns, and proposed effective width models for the design of steel plates in such columns. All these studies focused on the unilateral local buckling of steel plates under uniaxial compression. Little work, however, has been undertaken on the local and post-local buckling behaviour of steel plates in DSC panels under biaxial compression.

The local buckling behaviour of biaxially loaded steel plates that can buckle bilaterally has received considerable attentions. Elastic buckling solutions can be found in the book by Bulson.[18] Little[19] studied the collapse strength of steel plates with geometric imperfections under in-plane biaxial loading using an energy method. The perturbation method has been employed to investigate the post-buckling behaviour of biaxially loaded plates by Williams and Walker.[20] The effect of welding residual stresses in steel plates was taken into account in their study by magnification of the geometric imperfection. Valsgard[21] proposed the biaxial strength formulas for designing steel plates in ship structures based on the results of a nonlinear finite element analysis. This study used the proportional increment of longitudinal and transverse loads in the analysis. Dier and Dowling[22] employed a finite difference approach to generate the strength interaction curves of simply supported steel plates subject to biaxial forces. Geometric imperfections and biaxial residual stresses were considered in the analysis. Solutions by Valsgard[21] and Dier and Dowling[22] were based on the displacement-controlled approach, in which a constant ratio of longitudinal to transverse shortening has been used. Moreover, tests of long rectangular steel plates under biaxial compression have been performed by Bradfield et al.[23] It should be noted that steel plates in these studies were not restrained by the concrete and discrete shear connectors so that they were free to buckle bilaterally.

In this paper, the local and post-local buckling behaviour of biaxially compressed steel plates in DSC panels is investigated by using the finite element code STRAND7.[24] Finite element models, which properly account for the shear-slip characteristics of headed stud shear connectors, material stress-strain behaviour, initial imperfections and boundary conditions, are described. Numerical models are calibrated with results from push-out tests on the local buckling of composite panels. Elastic local buckling coefficients of plates with various boundary conditions are presented. Biaxial strength interaction curves and design formulas are developed for determining the ultimate strength of steel plates in DSC panels.

2. FINITE ELEMENT MODELLING

2.1 General

In the present study, the linear buckling analysis, which is based on the bifurcation buckling theory, is carried out to predict the elastic local buckling coefficients of perfectly flat steel plates under in-plane biaxial compression. The nonlinear analysis, which accounts for the effects of large deformations, pre-and-post buckling displacements, stress stiffening and material yielding, is employed to quantify the post-local buckling interaction strength of plates with initial imperfections. An eight-node quadratic plate element is used in all analyses. The von Mises yield criterion is adopted in the nonlinear analysis to handle the plasticity of a steel plate divided into ten layers through its thickness. A 20 × 20 mesh is used for square plates in the linear buckling analysis, whilst a 10 × 10 mesh is found to be efficient and economic for the nonlinear analysis of square plates. For plates with aspect ratios rather than unity, the mesh in the longitudinal direction is increased.

2.2 Shear-slip model

The shear connection between steel plates and the concrete core has a significant effect on the structural performance of DSC panels in terms of the strength, stiffness and stability. Model Tests on DSC beams and columns under eccentric loads conducted by Wright et al.[3] showed that the most likely failure was by the longitudinal shear or shear bond between the steel plates and concrete core. In DSC panels, stud shear connectors must be provided to resist longitudinal shear forces and slips at the interface. The restraint offered by stud shear connectors considerably improves the stability performance of steel plates in DSC panels. For slender steel plates, local buckling may occur before the failure of stud shear connectors. For stocky steel plates, the shear connectors may fracture before the onset of yielding or plastic local buckling. Moreover, local buckling may couple with shear connection failure, depending on the stud spacing to plate thickness ratio. Therefore, the shear-slip behaviour of stud shear connectors must be incorporated in the buckling analysis in order to yield realistic results.

The shear-slip behaviour of stud shear connectors is usually expressed by shear-slip curves that can be obtained from experimental results (Ollgaard et al.[25]; Oehlers and Coughlan[26]; Liang and Patrick[27]). The analytical model for predicting the shear-slip behaviour of stud shear connectors proposed by Ollgaard et al. is adopted in the present study, and it is expressed by

$$Q = Q_u \left(1 - e^{-18\delta}\right)^{0.4} \tag{1}$$

where Q is the longitudinal shear force, Q_u is the ultimate shear strength of a stud shear connector, and δ is the longitudinal slip. The ultimate shear strength of headed stud shear connectors can be determined in accordance with AS 2327.1.[28]

By using equation (1), a shear-slip curve for 19-mm diameter headed stud shear connectors embedded in concrete with the compressive design strength of 32 N/mm^2 is shown in Fig. 2. In the linear buckling analysis, a stud shear connector is modelled by elastic springs. The tangent modulus of the shear-slip curve generated by equation (1) is taken as the spring stiffness. In the post-local buckling analysis, a spring-type beam element is used to model stud shear connectors, whose nonlinear shear-slip relationship is defined by equation (1).

2.3 Ramberg-Osgood model

The Ramberg-Osgood[29] model is employed in the post-local buckling analysis to define the material stress-strain relationship for steel plates. This model is given by

$$\varepsilon = \frac{\sigma}{E}\left[1 + \frac{3}{7}\left(\frac{\sigma}{\sigma_{0.7}}\right)^n\right] \tag{2}$$

where $\sigma_{0.7}$ is the stress corresponding to $E_{0.7} = 0.7E$, and n is the knee factor that defines the sharpness of the knee in the stress-strain curve. When the knee factor approximates to infinite ($n = \infty$), equation (2) represents the idealised elastic-perfectly-plastic behaviour of steel plates. The knee factor n = 25 is used in the present study to account for the isotropic strain hardening of steel plates. A typical stress-strain curve of a steel plate with the 0.2% proof yield stress of 300 N/mm^2 is shown in Fig. 3.

2.4 Initial imperfections

Initial imperfections of steel plates due to manufacture and construction consist of the out-of-plane deflections and residual stresses. Initial imperfections reduce the strength and stiffness of steel plates, and are thus incorporated in the post-local buckling analysis. The form of initial out-of-plane deflections is taken as the first local buckling mode in the present study. The magnitude of geometric imperfections is an important parameter that affects the buckling results. Different magnitudes of geometric imperfections have been used in the past for the analysis of plates by researcher.[20-22] In the present study, the maximum magnitude of initial geometric imperfections at the plate centre is taken as $w_0 = 0.003b$ for steel plates in DSC panels. A lateral pressure is applied to the plate to induce the initial geometric imperfections, as suggested by Liang and Uy.[17] Residual stresses due to welding of stud shear connectors at discrete positions are less critical in double skin composite panels when compared to continuously welded plate structures. It is assumed that their effects have been incorporated within the geometric imperfections.

2.5 Boundary conditions

In a DSC panel, steel plates are connected to the concrete core by means of welded stud shear connectors at either a uniform spacing or a staggered spacing. The uniform stud spacing is commonly used in DSC panels in practice so that it is studied here. Steel plates in DSC panels are constrained to buckle locally in the unilateral direction when subjected to biaxial compression. In order to determine the maximum stud spacing and the ultimate strength of steel plates, the structural model is considered to be a single plate field between stud shear connectors, as shown in Fig. 4. The edge restraint depends on the stiffness of adjacent plate fields. It could be argued that the edges of the plate field are restrained from rotation by the adjacent plate fields and concrete, but the degree of rotation is not complete as adjacent plate fields in a DSC panel are usually not stiff enough to provide a fully clamped boundary condition. If stud shear connectors are continuously welded to the plate, the edges of the plate field should be assumed as clamped. It is assumed that the edges of the plate field between stud shear connectors, at a worst case, are hinged. This means that the edges between studs can rotate unilaterally but cannot deflect out of the plane. The plate field is welded with stud shear connectors with finite stiffness at its corners. Therefore, the rotations at the corners are constrained whilst their in-plane translations are defined by the shear-slip model. This boundary condition is similar to the simply supported boundary situation with additional restraint offered by studs, and is denoted as S-S-S-S+SC (S = simply supported; SC = shear connectors). The assumption of boundary conditions for plate fields located within a DSC panel results in conservative designs. Clamped edges may exist when the plate field is located at the edge of the panel, which provides restraint against rotations.

3. VALIDATION OF FINITE ELEMENT MODELS

Finite element models developed for the buckling analysis of steel plates restrained by concrete and stud shear connectors are calibrated with existing experimental results in Fig. 5. These push-out tests on the local buckling of mild steel plates under uniaxial compression were conducted by Smith.[30] Specimens were constructed by bolting two steel plates with b = 300 mm and t = 3 mm to a concrete core. The lengths of plates vary from 180 to 450 mm. Each loaded edge of the plate was connected to the concrete block by three 10-mm diameter bolts. The unloaded edges were free to buckle away from the concrete block. The loading was applied to two steel plates only in the test to determine the initial local buckling stress of these plates. In the linear buckling analysis, the bolts are modelled by elastic springs with the stiffness of $k_s = 1.458 \times 10^6$ N/mm determined by the shear-slip model. It can be seen from Fig. 5 that finite element solutions agree well with the experimental results.

4. ELASTIC LOCAL BUCKLING

4.1 Buckling coefficients

The elastic critical bucking stress of a biaxially compressed steel plate depends on the plate aspect ratio (spacing of shear connectors in two directions), the plate thickness, longitudinal and transverse loading and boundary conditions including the restraint of shear connectors. The elastic buckling coefficients of plates can be determined by varying the plate aspect ratios and biaxial loading in the analysis. The elastic buckling coefficient (k_x) in the x direction can then be obtained from the following equation[18]

$$\sigma_{xcr} = \frac{k_x \pi^2 E}{12(1-\nu^2)(b/t)^2} \tag{3}$$

where σ_{xcr} is the elastic critical buckling stress in the x direction. The elastic buckling coefficient in the y direction can be obtained by substituting σ_{ycr} and a in equation (3).

The configurations of plates used in all analyses are b = 500 mm, t = 10 mm, E = 200 kN/mm^2 and ν = 0.3. The shear stiffness $k_s = 4.52 \times 10^6$ N/mm is used for a stud shear connector that resists shear from a single plate field, whilst $\frac{1}{2}k_s$ is used for a stud shear connector that resists shear from two adjacent plate fields. Fig. 6 shows the elastic local buckling coefficients of plates with the S-S-S-S+SC boundary condition. It is seen from Fig. 6 that when the biaxial stress ratio $\alpha \geq 1/3$, the buckling coefficient decreases with an increase in the plate aspect ratio *a/b*. The presence of transverse loading (σ_y) significantly reduces the buckling coefficient of a plate. The initial local buckling stress decreases with an increase in the transverse applied stress. The restraint offered by stud shear connectors also considerably increases the resistance of a plate field against local buckling. It can also be seen from Fig. 6 that a square plate restrained by stud shear connectors and with $\alpha = 1$ has the buckling coefficient of $k_x = k_y = 2.404$, whilst it is only 2.0 for simply supported plates unrestrained by shear connectors.[18]

Buckling coefficients of plates with the C-S-S-S+SC (C = clamped) boundary condition are presented in Fig. 7. It can be observed from Figs. 6 and 7 that the clamped edge considerably increases the buckling critical buckling stress of plates in biaxial compression. Further results for biaxially compressed plates with two clamped adjacent edges (C-C-S-S+SC) are given in

Fig. 8. The clamped edges may cause shortening of the buckling half-wavelength when the deficiency between biaxial applied stresses is large.

4.2 Effect of stud spacing and plate thickness

For plates unrestrained by shear connectors, the plate slenderness *b/t* has no effect on the buckling coefficients. However, for plates in DSC panels, the spacing of stud shear connectors in two directions and the thickness of plates considerably influence the elastic buckling coefficients. Different stud spacing or plate thickness results in different longitudinal shear that is transferred by shear connectors. This complicates the local buckling problem of steel skins in composite panels. The effect of stud spacing and plate thickness on the buckling coefficients of square plates (S-S-S-S+SC) is demonstrated in Fig. 9. It is observed that the buckling coefficient slightly decreases with an increase in both plate width and thickness for the same slenderness of b/t = 50. It should be noted that the plate width b = 500 mm and thickness t = 10 mm have been used in the linear buckling analyses to determine buckling coefficients presented in Section 4.1. These buckling coefficients can be used in design of steel plates with $b \leq 500$ mm and $t \leq 10$ mm in DSC panels.

4.3 Limiting width-to-thickness ratios

Buckling coefficients presented can be used to determine the limiting width-to-thickness ratios for steel plates under biaxial compression in DSC panels. This yield limit is to prevent the elastic local buckling of a steel plate field between stud shear connectors from occurring before steel yielding. The relationship between critical buckling stress components at yield can be expressed by the von Mises yield criterion as

$$\sigma_{xcr}^2 - \sigma_{xcr}\sigma_{ycr} + \sigma_{ycr}^2 = \sigma_0^2 \tag{4}$$

If the material properties E = 200 kN/mm^2 and ν = 0.3, and the plate aspect ratio $\varphi = a/b$ are assumed, the limiting width-to-thickness ratio can be derived by substituting σ_{xcr} and σ_{ycr} into equation (4) as

$$\frac{b}{t}\sqrt{\frac{\sigma_0}{250}} = 26.89\left(k_x^2 - \frac{k_x k_y}{\varphi^2} + \frac{k_y^2}{\varphi^4}\right)^{1/4} \tag{5}$$

For a biaxially compressed square plate with the biaxial stress ratio of $\alpha = 1$, the local buckling coefficient is $k_x = k_y$ = 2.404, as shown in Fig. 5. By using equation (5), the limiting width-to-thickness ratio is obtained as 41.7. If the plate with a thickness of 10 mm is used, the maximum spacing of stud shear connectors in two directions is 417 mm for the steel plate with a yield stress of 250 N/mm^2.

5. POST-LOCAL BUCKLING

5.1 General

The post-local buckling behaviour of biaxially compressed steel plates with the S-S-S-S+SC boundary condition is investigated here. The present study employs the proportional load increment scheme, in which the ratio of the transverse to the longitudinal loading is kept constant. Steel plates (b = 400 mm) with a yield strength of σ_0 = 300 N/mm^2 are studied. The 19-mm diameter headed stud shear connectors are used in DSC panels filled with concrete of a compressive strength of 32 N/mm^2. Due to symmetry, only a quarter of the plate field is modelled. Half of the ultimate shear strength of a stud shear connector is used in equation (1) to account for the effect of the adjacent plate field.

5.2 Load-deflection characteristics

The load-lateral deflection curves of square steel plates under the biaxial compressive stresses of $\alpha = 1$ are presented in Fig. 10. It can be observed that increasing the slenderness of a plate significantly reduces both the stiffness and ultimate strength of the plate in biaxial compression. The effect of plate slenderness on its ultimate strength is further demonstrated in Fig. 11. Plates with higher *b/t* ratios undergo larger lateral deflections to attain their post-local buckling strengths, as shown in Fig. 10. For stocky plates with the b/t ratios of 20 and 40, their ultimate strength is governed by the shear capacity of stud shear connectors or plastic local buckling. Numerical results also indicate that slender plates can develop their full post-local buckling reverse of strength without the fracture of stud shear connectors.

Fig. 12 shows the load-lateral deflection curves of plates with an aspect ratio of *a/b* = 2 and under equal biaxial loading in two directions. It is seen from Figs. 10 and 12 that when the plate *b/t* ratio is greater than 20, the ultimate strength of a biaxially compressed plate decreases with an increase in its aspect ratio. For plates with a slenderness ratio of *b/t* = 20 and different aspect ratios, both plates are able to attain almost the same ultimate strength.

5.3 Biaxial strength interaction curves

For a plate with a specified aspect ratio, slenderness and initial imperfection, its ultimate strength depends on the biaxial loading. By varying the ratio of the transverse to longitudinal loading (α) in the nonlinear finite element analysis, the biaxial strength interaction curve of a plate can be generated. The post-local buckling behaviour of steel plates in biaxial compression can be described by biaxial strength interaction curves, rather than the effective width concept used for plates under uniaxial compression. Fig. 13 shows the biaxial strength interaction curves of square plates with various *b/t* ratios obtained from the results of the nonlinear finite element analysis. It can be observed that the ultimate strength of a biaxially compressed plate decreases with an increase in its *b/t* ratio regardless of the value of the biaxial loading. When the plate slenderness is greater than 20, the presence of the transverse loading (σ_y) reduces the longitudinal ultimate strength of plates (σ_{xu}). For plates with a *b/t* ratio of 20 under the biaxial loading of $\alpha \leq 0.25$, the transverse loading slightly increases the longitudinal ultimate strength of plates. This is because these transverse loads provide restraints to the plate so that stresses within the plate can be redistributed to attain a higher strength. It is also observed from Fig. 13 that when $\alpha = 1$, $\sigma_{xu} = \sigma_{yu}$.

The strength interaction curves of plates with an aspect ratio of *a/b* = 2 is presented in Fig. 14. It can be observed from Fig. 14 that the longitudinal ultimate strength (σ_{xu}) of a steel plate with φ = 2 is higher than its transverse ultimate strength (σ_{yu}). The steel plate with a *b/t* ratio of 20 can attain its yield strength in the longitudinal direction when no transverse compression is applied to the plate. It can be seen from Figs. 13 and 14 that the shapes of the biaxial strength interaction curves of square steel plates are different from those of plates with φ = 2. This means that the shapes of biaxial strength interaction curves depend on the aspect ratios of plates.

5.4 Strength interaction design formulas

The generalisation of a von Mises yield ellipse can be used to develop biaxial strength interaction formulas for design of steel plates in double skin composite panels. The general strength interaction formula is expressed by

$$\left(\frac{\sigma_{xu}}{\sigma_0}\right)^{\zeta} + \eta\left(\frac{\sigma_{xu}\sigma_{yu}}{\sigma_0^2}\right) + \left(\frac{\sigma_{yu}}{\sigma_0}\right)^2 = \gamma \quad (\gamma \leq 1) \tag{6}$$

where the shape factor ζ of the interaction curve depends on the plate aspect ratio and slenderness, η is a function of the plate slenderness, and γ is the uniaxial strength factor. The shape factor η can be used to define any shape of interaction curves from a straight line ($\eta = 2$) to the von Mises ellipse ($\eta = -1$).

For square plates, the shape factor $\zeta = 2$ and the values of η and γ given in Table 1 are found to fit numerical results well. By using equation (6) and parameters in Table 1, a set of biaxial strength design curves for square plates can be generated, as shown in Fig. 15. Parameters that define the shapes of interaction curves for plates with the aspect ratio of *a/b* = 2 can also be obtained by fitting equation (6) to numerical results. However, it is observed from Fig. 13 that the shapes of these curves are quite different. Thus, different shape factors of ζ may be used to define the shapes of interaction curves for plates with an aspect ratio of *a/b* = 2 and with different slenderness.

6. CONCLUSIONS

This paper has presented the unilateral local and post-local buckling behaviour of biaxially compressed steel plates restrained by concrete and stud shear connectors in DSC panels. The numerical model developed has allowed for the shear-slip characteristics of stud shear connectors, material yielding, initial imperfections and boundary conditions including restraints by concrete and shear connectors to be taken into account in the buckling analysis. Elastic local buckling coefficients have been obtained for biaxially compressed steel plates with various aspect ratios, biaxial loading and boundary conditions. The load-deflection performance of plates with various width-to-thickness ratios has been reported. Biaxial strength interaction curves have been generated by using the proportional load increment scheme for square plates and plates with an aspect ratio of *a/b* = 2. Biaxial strength interaction design formulas have also been proposed for the design of square plates with various slenderness ratios.

This study indicates that the buckling coefficient of a plate restrained by shear connectors with finite shear stiffness decreases slightly with an increase in both the stud spacing and plate thickness. Elastic buckling coefficients presented can be used to determine limiting width-to-thickness ratios for steel skins and the spacing of stud shear connectors in two directions. Numerical investigations showed that the ultimate strength of stocky plates was governed by shear stiffness of stud shear connectors or plastic local buckling. The post-local buckling strength of slender plates can be fully developed without the shearing failure of stud shear connectors. Any significant transverse loading significantly reduces the longitudinal ultimate strength of biaxially compressed steel plates. Biaxial strength interaction curves can be generated by varying the ratio of transverse to longitudinal loading in the post-local buckling analysis. It has been demonstrated that the post-local buckling behaviour of steel plates in biaxial compression can be described by biaxial strength interaction curves and formulas. Numerical models developed herein can be extended to other design situations in composite steel-concrete construction. Further research could include the effects of in-plane shear stresses on the local buckling and ultimate strength of DSC panels under combined states of stresses.

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Figures and Tables

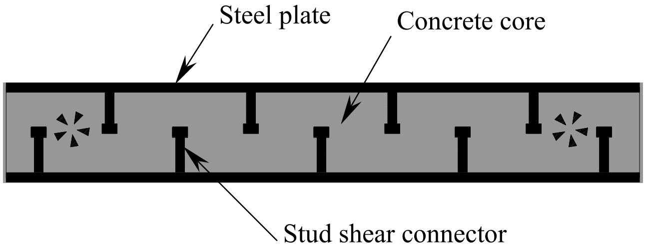

Fig. 1. Cross-section of double skin composite panel

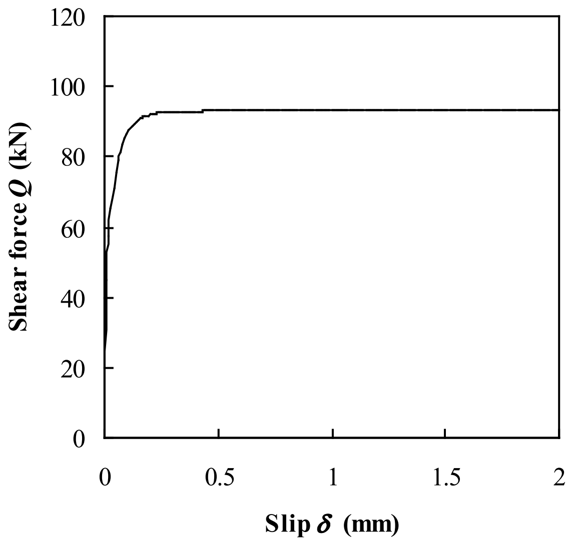

Fig. 2. Shear-slip curve for stud shear connectors

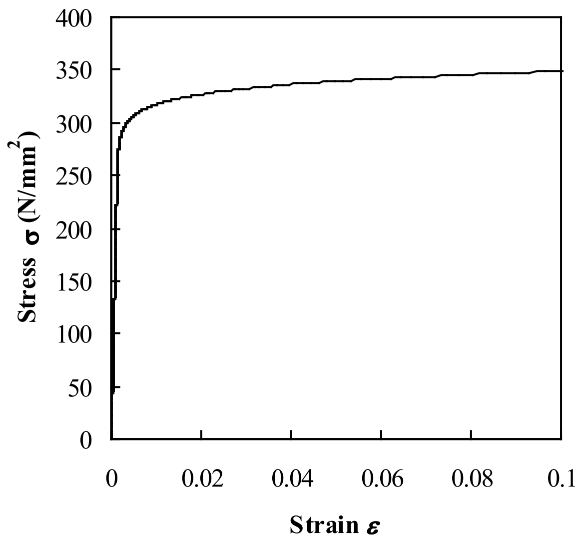

Fig. 3. Stress-strain curve based on Ramberg-Osgood model

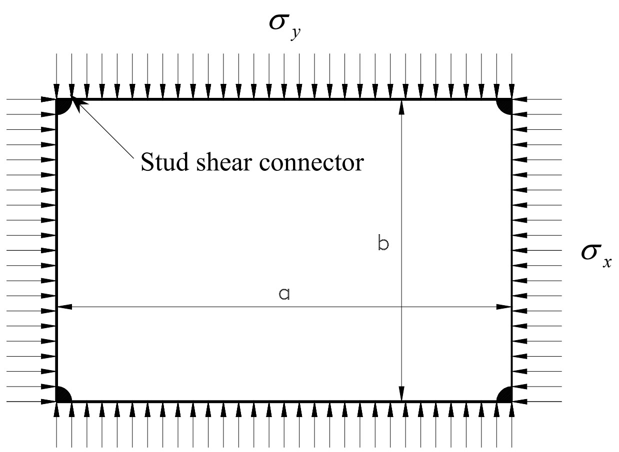

Fig. 4. Single plate field restrained by stud shear connectors

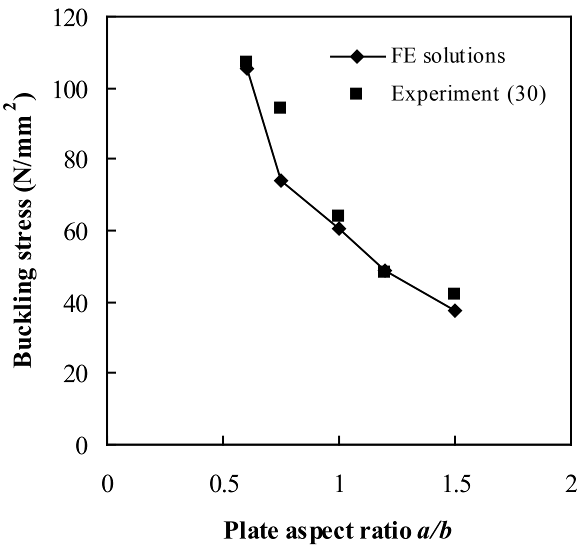

Fig. 5. Comparison of FE solutions with experimental results

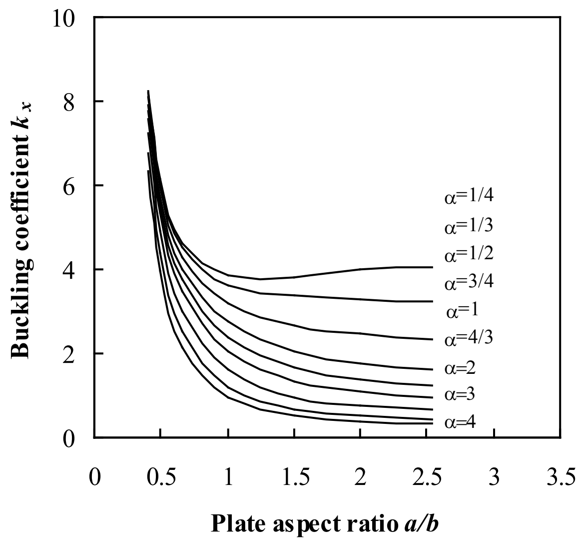

Fig. 6. Buckling coefficients of plates under biaxial compression (S-S-S-S+SC)

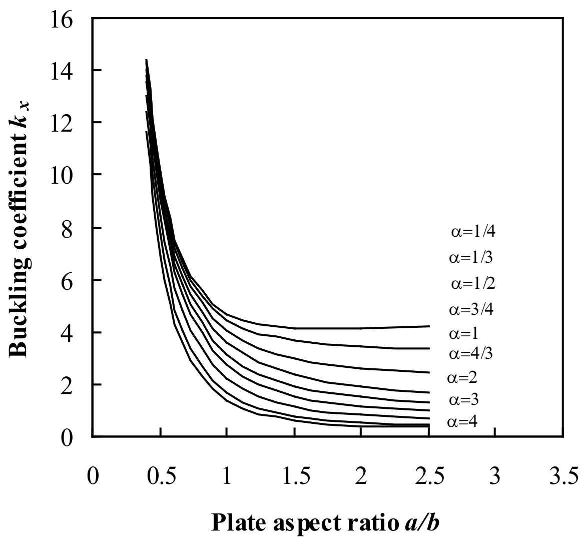

Fig. 7. Buckling coefficients of plates under biaxial compression (C-S-S-S+SC)

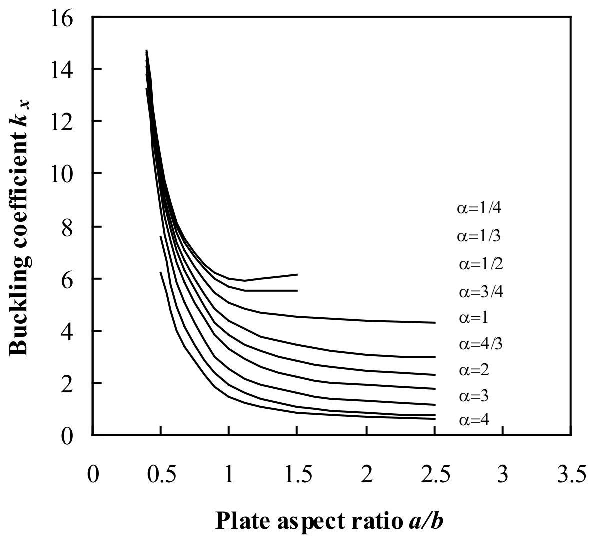

Fig. 8. Buckling coefficients of plates under biaxial compression (C-C-S-S+SC)

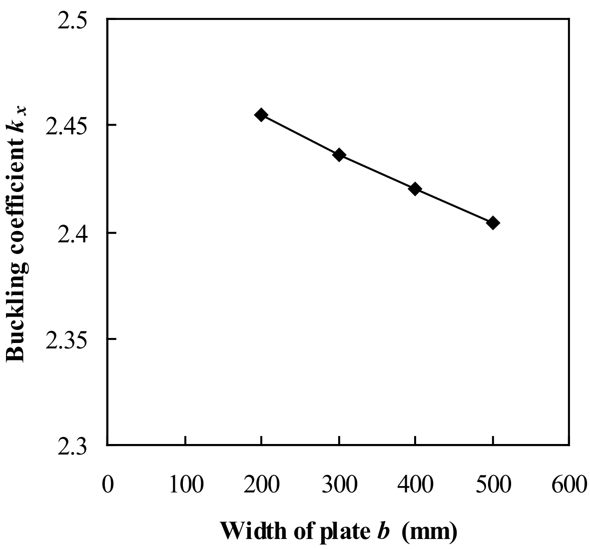

Fig. 9. Effect of stud spacing and plate thickness, a/b = 1, $\alpha = 1$

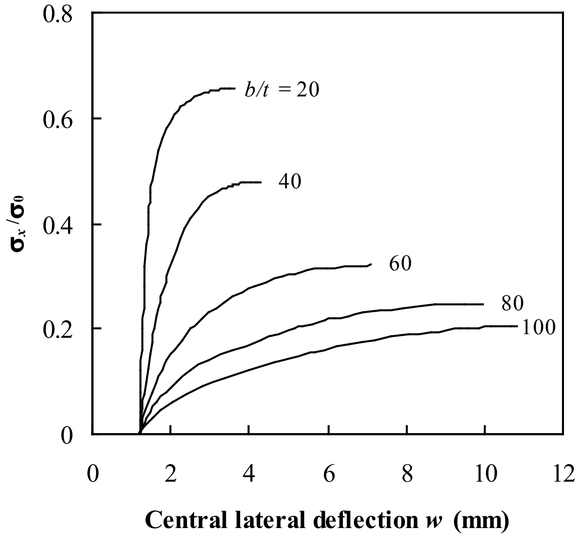

Fig. 10. Load-deflection curves of square plates, $\alpha = 1$

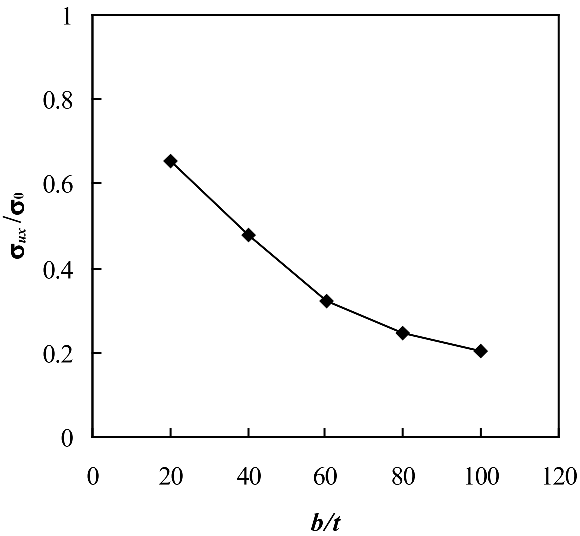

Fig. 11. Effect of slenderness on ultimate strength of square plates, $\alpha = 1$

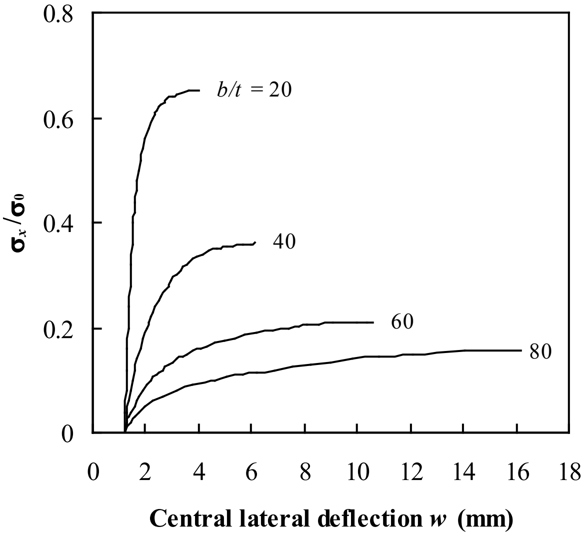

Fig. 12. Load-deflection curves of plates, *a/b* = 2, $\alpha = 1$

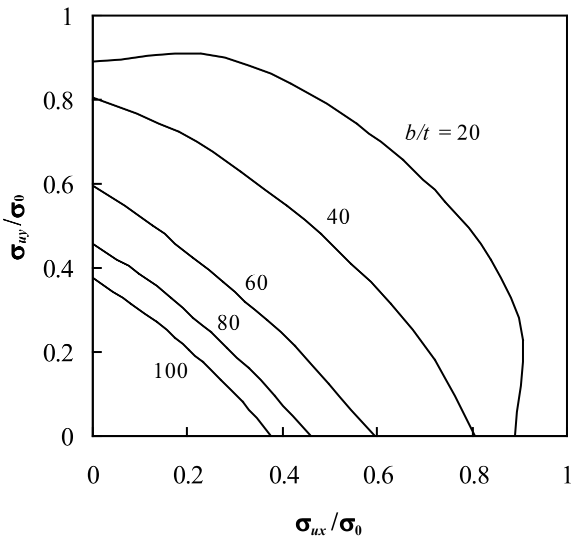

Fig. 13. Biaxial strength interaction curves of square plates, *a/b* = 1

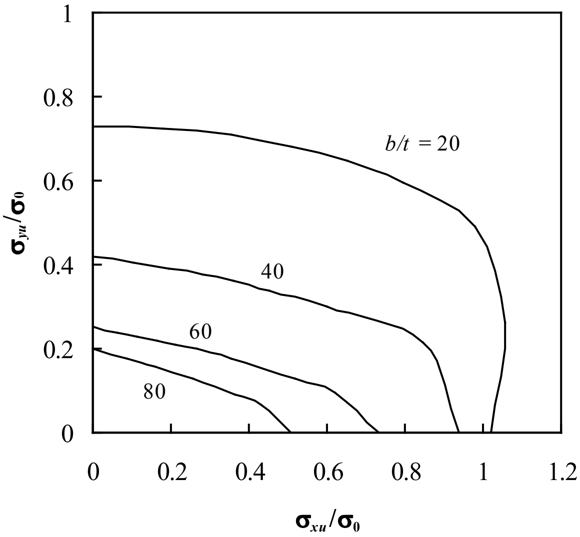

Fig. 14. Biaxial strength interaction curves of plates, *a/b* = 2

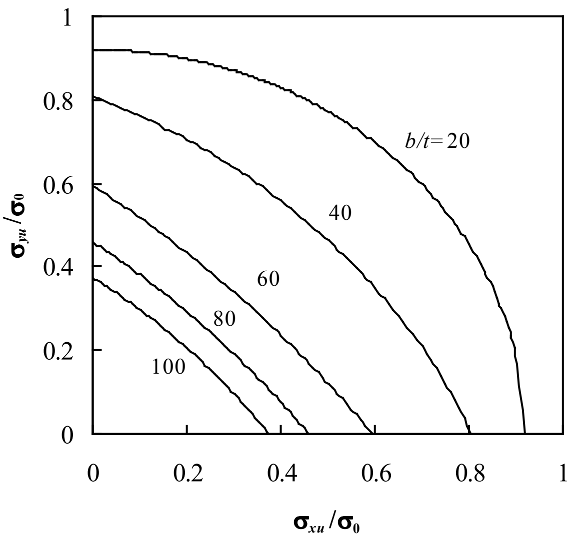

Fig. 15. Biaxial strength interaction curves of square plates generated by formulas

Table 1. Parameters of strength interaction formulas for square plates, $\zeta = 2$

b/t	η	γ
20	0	0.846
40	0.8	0.65
60	1.45	0.353
80	1.47	0.211
100	1.4	0.14