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Buckling behavior of double-skin composite walls: An experimental and modeling study

Yue Yang, Jingbo Liu, Jiansheng Fan *

Key Laboratory of Civil Engineering Safety and Durability of China Education Ministry, Department of Civil Engineering, Tsinghua University, Beijing 100084, China

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ABSTRACT

Double-skin composite (DSC) panels can offer high strength and robustness while improving the convenience of construction, with great potential for application in high rise buildings and nuclear power plants. In DSC panels, the stability of the outer surface steel plates are governed by the constraints of the in-fill concrete and the discrete shear connectors, i.e., the ratio of connector spacing (B) and surface steel plate thickness (t). In this paper, tests were performed on 10 specimens to assess the buckling behavior of DSC panels. The arrangement of the shear studs and the B/t ratio were varied in the tests. The results show that the arrangement and spacing of the shear studs can considerably influence the buckling shapes and loading capacity of the steel plates. Three-dimensional finite element (FE) models were developed to simulate the behavior of DSC panels subject to compression, and the FE results were found to be in good agreement with the observed buckling behavior during tests. A theoretical model based on Euler's equation was also proposed to predict the buckling stress of steel plates, and it showed reasonable agreement with the experimental measurements and FE results. The formula proposed in this paper can be used for determining the number or spacing of shear studs in DSC walls.

1. Introduction

DSC panels consist of two steel plates and in-fill concrete, combined to work compositely using mechanical connectors such as shear studs or tie bars. With the constraints of the in-fill concrete and the regularly spaced connectors, the stability of the surface steel plate is enhanced greatly [1]. Moreover, the brittleness of the concrete is also improved. DSC structures also exhibit superior characteristics in impermeability and impact resistance [2,3]. DSC panels provide high efficiency in construction practice with the steel plates prefabricated in factory and assembled on-site. Compared with reinforced concrete structures, formwork is avoided. Given these advantages, DSC structures demonstrate notable competitiveness in under-sea tunnels, nuclear power plants, and high-rising buildings [4,5].

When DSC panels sustain axial compression or out-of-plane bending, the surface steel plate between connectors is constrained by the rigid concrete on one side and may buckle outwards, reducing the load capacity and stability of the structure. The ratio of the spacing between connectors (B) and the steel plate thickness (t) is the key factor governing the buckling of the steel skins. Akiama et al. [6] proposed the buckling stress of a surface plate according to test results as follows:

$$\sigma_{cr} = \frac{\pi^2 E_s}{12K^2 (B/t)^2} \tag{1}$$

where K is the effective length factor and equals 0.7.

Let $\sigma_{cr} = f_y$, then the critical B/t ratio for buckling before yielding can be derived as:

$$\frac{B}{t} \le 1.30\sqrt{\frac{E_s}{f_y}} \tag{2a}$$

Using the principle of virtual work, Wright [7] established the formula for buckling stress of a steel plate constrained by concrete on one side. By assuming the buckling shape modes, the critical B/t ratio is given as:

$$\frac{B}{t} \le 1.90\sqrt{\frac{E_s}{f_y}} \tag{2b}$$

Many experiments have been performed to study DSC walls [8–11], but only limited measurements have focused on buckling behavior

* Corresponding author.
E-mail address: fanjsh@tsinghua.edu.cn (J. Fan).

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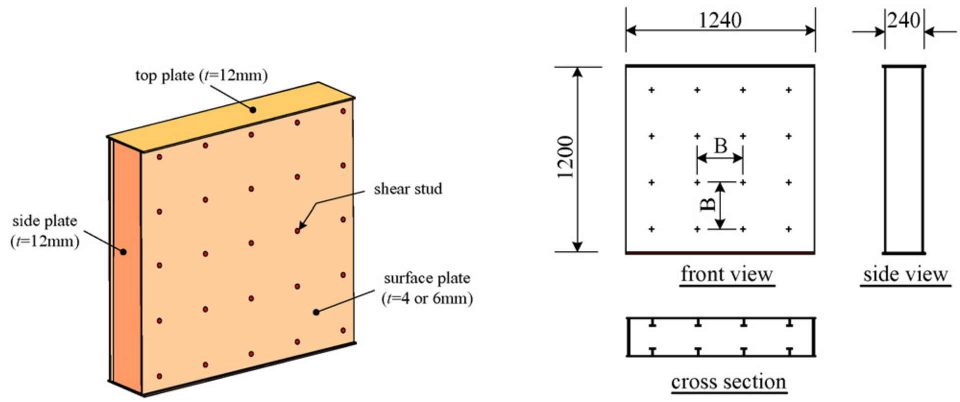

Fig. 1. Double skin composite panel specimens.

[12–15]. The parameters studied in such experiments include the B/t ratio and steel strength. Kai et al. [16] developed finite element (FE) models of DSC panels and calibrated these models with laboratory test results reported in the literature [12]. Based on a parametric study, the critical B/t ratio is given as:

$$\frac{B}{t} \le 1.0\sqrt{\frac{E_s}{f_y}} \tag{2c}$$

On the basis of the stationary potential principal, Nie and Li [17] proposed the local buckling stress for the surface steel plate, with the critical B/t ratio given as:

$$\frac{B}{t} \le 1.03\sqrt{\frac{E_s}{f_y}} \tag{2d}$$

The above formulas have similar expressions but different coefficient values. The energy methods provide meaningful physical models but their accuracy depends on the assumed boundary conditions and buckling shape functions. The empirical method based on experimental results can account for the influence of initial imperfections and the welding residual stress, but the accuracy depends on the number of laboratory tests, which are expensive and time-consuming. In most previous experiments on DSC panels, the B/t ratio was less than 50, but results with larger B/t values are also valuable to understand elastic buckling. Further, the connector arrangement also affect the buckling behavior of surface plates, and the effectiveness of such arrangements needs to be tested and evaluated.

Apart from experiments, many numerical methods have been adopted to study the unilateral buckling of steel plates [18,19]. In these studies, initial imperfections, slip at the steel-concrete interfaces, and support conditions have been investigated, and the conclusions compared with theoretical and experimental results.

In this paper, a series of experimental tests was conducted on DSC walls subjected to axial forces. FE models of the DSC specimens were also developed and calibrated with the test data. The objectives of this study were 1) to provide experimental data and FE analysis results for steel plate buckling behavior by considering the influences of connector arrangement, B/t ratio, tensile stiffness of shear studs, and initial imperfections and 2) to propose more reasonable limits to the B/t ratios.

2. Test specimens

Ten DSC panel specimens were designed and tested. All the specimens had the same geometry: width = 1200 mm, height = 1200 mm, and total thickness (including surface steel plate) = 240 mm. The three parameters considered in the experiments were 1) surface plate thickness, 2) stud spacing, and 3) the arrangement of connectors. The thickness of the surface steel plates was 4 mm and 6 mm, respectively. Shear studs with diameter 5 mm and height 35 mm were welded onto the 4 mm surface steel plates, and studs with diameter 10 mm and height 75 mm were welded onto the 6 mm plates. The top plate, side plate and bottom plate welded around the surface plate had thickness 12 mm for all specimens. Fig. 1 shows a schematic of a typical test wall. Details of the specimens are summarized in Table 1 in which B/t is calculated with the studs spacing in the vertical (or loading) direction. (See Fig. 2.)

The steel plates in the specimens were made of Q345B steel (equivalent to American A36 steel plates). The elastic modulus of the steel was 2.06×10^5 MPa. The characteristics of the steel used for the specimens are shown in Table 2, where f_y is the lower yielding strength. The in-fill concrete in the DSC specimens was of grade C40, with a nominal cubic compressive strength of 40 MPa and actual cubic compressive strength f_{cu} listed in Table 1.

The instrumentation employed in the experiments included strain gauges placed on the surface of the steel plates and linear variable differential transformers (LVDTs) placed vertically on the side of the specimen panels. No attempt was made to measure the horizontal separation and slip between the steel plate and the in-fill concrete.

3. Loading process and failure mode

The specimens were tested using 20,000 kN loading equipment in the Structural Engineering Lab of Tsinghua University. First, the tests were performed under force control with a force increment of 500 kN. When the specimens reached the yield point, the loading process changed to displacement control. The loading process was stopped after the load decreased noticeably as a result of the buckling of the steel plate and crushing of the concrete.

Table 1
Specimen parameters.

Specimen no.	Steel thickness t (mm)	Stud spacing B (mm)	B/t ratio	Stud arrangement	f_{cu} (MPa)
DSC4-150	4	150	37.5	Square	43.3
DSC4-200	4	200	50.0	Square	35.9
DSC4-250	4	250	62.5	Square	42.2
DSC4-300	4	300	75.0	Square	39.6
DSC4-150/300	4	150/300	75	Vertical rectangle	34.5
DSC4-300/150	4	300/150	37.5	Horizontal rectangle	35.5
DSC4-300X	4	300	–	Staggered	39.8
DSC6-240	6	240	40.0	Square	42.5
DSC6-300	6	300	50.0	Square	37.1
DSC6-360	6	360	60.0	Square	39.7

Note: f_{cu} = concrete cubic compressive strength determined by 150 mm × 150 mm × 150 mm specimens.