

Behavior of steel-concrete-steel sandwich structures with lightweight cement composite and novel shear connectors

K.M.A. Sohel*, J.Y. Richard Liew, J.B Yan, M.H. Zhang and K.S. Chia
Department of Civil & Environmental Engineering, National University of Singapore,
BLK E1A, #07-03, One Engineering Drive 2, Singapore 117576,

Abstract

In Steel-Concrete-Steel (SCS) sandwich structure, mechanical shear connectors are commonly used to transfer longitudinal shear forces across the steel–concrete interface. In this paper, novel shear connectors such as J-hook and cable shear connectors are proposed and their performance to achieve composite strength of SCS sandwich structures is investigated. The use of these connectors together with ultra-lightweight cement composite core reduces the overall weight of SCS sandwich system making it suitable for the construction of marine and offshore structures. Static tests were carried out on SCS sandwich beams with J-hook, cable shear connectors and headed studs. Their ultimate strengths were reported and their respective failure modes were discussed. An analytical method to predict the ultimate strength of the Steel-Concrete-Steel sandwich beams with various types of shear connectors was developed and its accuracy was ascertained by comparing with the test results. Design recommendations are made on minimum connector spacing to prevent shear cracking of concrete core and local buckling of face plates.

Key words: Cement composite; Steel-Concrete-Steel; Sandwich structure; Shear connector, J-hook; Ultra lightweight.

*Corresponding author. Tel. +65-65163516; Fax: +65-67791635. E-mail address: ceekmas@nus.edu.sg; kmasohel@yahoo.com

1. Introduction

Steel-Concrete-Steel (SCS) sandwich comprises a central concrete core which is sandwiched between two steel skins to form a composite unit whose behavior is greatly influenced by the interfacial bond between the two materials. During the past 30 years there have been many research and development in SCS sandwich construction. Cohesive bonding material (e.g. epoxy) and different types of mechanical shear connectors such as headed stud, angle connector, Bi-Steel connector, plate connectors, and bi-directional corrugated-strip-core (CSC) system) were proposed to bond the steel plate and the concrete core as illustrated in Fig.1.

Solomon *et al.* [1] appraised the form of steel-concrete-steel (SCS) sandwich as a potential structural form to reduce the weight of roadway slab on medium and long span composite bridges. In their study, the precast concrete slab was bonded to steel face plates with the aid of epoxy. Tomlinson *et al.* [2] proposed double skin composite (DSC) or SCS sandwich system with shear studs for immersed tube tunnel application under Conwy River. In 1990s, steel construction institute [3, 4] issued two design guidelines for the application of SCS sandwich construction. In these applications, the shear transfer between steel face plates and concrete core relied on the overlapped headed shear studs. The overlap length and close spacing of studs could lead to difficulty in concreting or grouting [5]. Furthermore, the research by Liew *et al.* [6] showed that the shock wave generated by impact and blast loads tend to push the face plate out from the core leading to tensile separation as shown in Fig. 2. Therefore, the structural integrity of the sandwich structure could be compromised. Similar research was conducted on SCS beams with angle connector to resist drop weight impact [7]. Separation of the face plates was observed after the impact test as shown in Fig. 3. A proprietary product named Bi-Steel as shown in Fig. 4 was developed. This technology involved a through core thickness transverse bar to be friction welded by rotating the steel bar

to generate enough heat at the two ends hence forging the bar and steel plates together [8, 9]. The key advantages of SCS panels made of Bi-Steel connectors are their high strength and speed of construction. However, the thickness of the prefabricated panel is limited by the welding machine in which the panel depth should be between 200 mm and 700 mm with face plate thickness between 5 mm and 20 mm. The bar diameter is fixed at 25 mm with minimum spacing of 200 mm. Thus the Bi-Steel SCS panel cannot be used for slim deck with thickness less than 200 mm. Sandwich system with Bi-directional corrugated-strip-core [10] [see Fig. 1e] has been proposed but it is still in conceptual form. For larger panels, welding of the corrugated strip to the both steel face plates may not be possible using the conventional welding methods. Plate connectors [11], as shown in Fig. 1d, are not suitable for lightweight slim deck system due to higher depth requirement to facilitate the welding of the plate connectors to the steel face plate. Another concept of sandwich structure is axially restrained non-composite sandwich structures for protective barrier introduced by Remennikov et al. [12]. No shear connector was introduced as this system utilized tensile membrane action of steel face plate to resist blast load when undergoing large deformation.

For weight sensitive marine and offshore structures, the core thickness shall be optimized. In addition, to minimize the dependence on the welding equipment and to introduce flexibility in construction and repair, novel shear connectors and associated construction methods were proposed by Liew *et al* [13]. J-hook connectors may be manufactured by bending reinforcing bar or by forging method. They can be produced with various diameters and heights to suit the strength and deck height requirements. For slim decking with thin core, the diameter of the J-hook connector is limited by its bending radius. The J-hook connectors can be welded to the face plate by using commercially available shear stud arc welding device as shown in Fig. 5a. The J-hook shear connectors can be designed to achieve composite action between the

steel face plates and concrete core under normal design loads. They also provide restraint to the face plate to prevent local buckling. Under extreme loads due to blast and impact, the J-hook connectors prevent separation of the steel face plates and ensure both face plates are inter-locked with the central core material. As shown in Fig. 5b, two steel plates with J-hook connectors are assembled to form the outer skeleton of the SCS sandwich panel. Liew *et al* [6, 13] studied both the static and low velocity impact resistance of the proposed SCS sandwich structures and test evidence showed that such novel SCS sandwich system was suitable for applications where high stiffness and damage tolerance were expected.

Although the SCS sandwich construction was originated in civil/structural engineering application, it has been further researched and developed for shipbuilding/offshore applications. Recent research [14, 15] is to extend the use of SCS sandwiches in floating and offshore structures by reducing overall weight of the sandwich system. This may be achieved by introducing structural lightweight concrete or cement composite, reducing the core depth and using effective shear connectors to produce slim deck systems.

Considering the existing SCS systems, three types of shear connectors for SCS sandwiches were investigated experimentally in this research. Lightweight concrete (LWC) and high performance ultra lightweight cement composite (ULCC) materials were used as core material. A series of tests were carried out to evaluate the performance of the SCS sandwich beams with different types of shear connectors subject to static load. The test results were used to validate the analytical methods.

2. Novel Connectors for Sandwich Structures

In sandwich structures, mechanical connectors are required to provide effective bond between the steel face plates and the concrete core. The connectors should be designed to satisfy three basic requirements: (1) providing interface slipping resistance, (2) preventing complete ‘pull-out’ from the concrete core, and (3) enhancing the cross section shear resistance to resist vertical load. Several new types of connectors are proposed for the SCS composite structures. They are (a) Angle-Steel bar-Angle (ASA); (b) Angle-T channel (AT); (c) Angle-Steel hoop-Angle (AHA); (d) Angle-C channel-Angle (ACA); (e) U connector-Steel bar-U connector (USU); (f) Angle-I beam-Angle (AIA); (g) Angle-Angle (AA); (h) Root connectors (RC); (i) U connector-Steel Cable-U connector (UCU). The proposed new types of connectors together with the J-hook connectors developed earlier by the authors [13] are shown in Fig. 6.

The angle connectors or the ‘U’ shape connectors are welded to the exterior steel plates to provide interface slipping resistance. The inserted steel bar (used in ‘ASA’ and ‘USU’), steel hoops (used in ‘AHA’), C channel (used in ‘ACA’), ‘I’ beam (used in ‘AIA’) and the steel cables (used in ‘UCU’), all serve the same function which is to link the two face steel plates preventing them from tensile separation and to provide bond enhancement between the concrete core and face plates. These connectors have their own merits in term of ease of installation and ability to withstand extreme loads without loss of structural integrity. The cable and U-shaped connectors, as shown in Fig. 6i, require the least steel consumption and they are relatively easy to install in a slim sandwich panel. Therefore, their effects on the ultimate strength behavior of the sandwich beams are evaluated and compared with sandwich beams with J-hook connectors and headed stud connectors in the following sections.

3. Experimental Investigations

A series of quasi-static tests was carried out to investigate the performance of lightweight SCS sandwich beams with cable, J-hook and headed studs connectors. Two beams with cable connectors (UCU), three beam with J-hook Connectors and two beams with headed stud were tested. Concentrated load was applied at the mid-span of first three simply supported beams (3 point loading) with a shear span to thickness ratio of 5. The core material for the specimens with cable connectors was lightweight aggregate concrete but using different orientation of U-connectors and steel cables. Shank diameter of the U connectors was 10 mm and diameter of the cable was 6 mm. Overall height of the U-connectors was 34 mm. Details of the beams with cable connectors are shown in Fig. 7. For comparison, one beam with J-hook connectors (specimen J4) was tested under 3 point loading. The other five beams with J-hook shear connectors and headed studs were tested under four point loading as shown in Fig. 8. Although the geometry of the J-hooks and headed studs connectors was different, the steel consumption per pair of connectors was almost identical. In addition, the diameter of the headed studs in specimens B6 and B7 was 13 mm, while the diameter of the J-hooks in specimens J6 and J7 was 12 mm. The yield and ultimate strength of the headed studs were 500 MPa and 550 MPa respectively, while these values for J-hook connectors were 350 MPa and 450 MPa respectively. The flexural capacity of the sandwich beams with J-hook connectors were compared those with headed shear stud.

Steel coupon tests and concrete compression tests were conducted separately to obtain the mechanical properties of the steel face plate and the sandwich core materials. The test specimens were casted and cured using the wet sacks. Concrete cylinders were prepared and tested on the same day of the beam test to obtain the compressive strength and secant modulus of the concrete. Tensile test was also carried out on the steel cable to obtain its

tensile capacity. The details of the test specimens and basic material properties of the lightweight concrete and ultra-lightweight cement composite are summarized in Tables 1 and 2.

Two different types of lightweight core were used in SCS sandwich beams. One was lightweight concrete with density = 1430 kg/m^3 , cylinder compressive strength = 26 MPa and modulus of elasticity = 12.0 GPa. Ordinary Portland cement and expanded clay type of lightweight aggregate (LWA) with average particle density of 1000 kg/m^3 were used to produce the lightweight concrete. The maximum size of the LWA was 8 mm. The second type of core material did not contain any coarse aggregate. It was a type of fibre-reinforced ultra lightweight cement composite (ULCC) having a 28-day compressive strength above 60 MPa and density 1440 kg/m^3 [16, 17, 18]. Compared with typical concrete with similar strength and normal density of 2400 kg/m^3 , the ULCC had a high specific strength (strength-to-density ratio) above 40 versus 25 for the former. Besides a 40% weight reduction from conventional concrete, the ULCC exhibited ultimate tensile and flexural strengths comparable with conventional concrete of similar strength. The ULCC contained microspheres as filler, otherwise known as cenosphere and polyvinyl alcohol (PVA) fibers with an average length of 6 mm and a diameter of $27 \mu\text{m}$. The fibres had a tensile strength of 1600 MPa, an elastic modulus of 39 GPa, an elongation of 7%, and a specific gravity of 1.30. The cenosphere is a hollow alumino-silicate sphere obtained as by-product from ash ponds of coal combustion at thermal power plants, which is formed from cooling and solidifying of inorganic molten minute coal residues around a trapped gas (usually CO_2 and N_2). Due to its hollow structure, cenosphere has low particle density typically ranging from 600 to 900 kg/m^3 . Particle size is usually between $100 - 300 \mu\text{m}$ with top size at $\sim 600 \mu\text{m}$. Typically, a lightweight aggregate concrete with 28-day compressive strength of 40 MPa or greater is

considered as high strength concrete. The ULCC has 28-day compressive strength above 60 MPa and may be regarded as a type of high-strength lightweight cement composite but technically it is not classified as a concrete due to absence of coarse aggregate. The mechanical properties of ULCC and LWC are given in Table 2.

4. Test Results

The beams were subjected to either three-point loading or four-point loading until fail. The load was applied in displacement controlled mode with a speed of 0.05 mm/min. The applied load versus central deflection curves are shown in Figs. 9 and 11. The deformed shapes and the cracks developed in the concrete under the ultimate deflection are shown in Figs. 10 and 12. A summary of the test results are presented in Table 3.

4.1 Basic Structural Behavior of the sandwich beams with U-cable and J-hook connectors

Beams A and B with U-cable connectors showed similar deflection and ultimate load behavior under flexural load, as shown in Fig. 9a. In beam A, the first crack appeared at 45% of ultimate load with a corresponding deflection of 2.1 mm. At 68% of ultimate load level, four noticeable shear cracks occurred in the vicinity of the loading point. The ultimate load was reached at 58.3 kN with a mid-span deflection of 21.3 mm. In the post peak region, several drops and rises in the load-deflection curves were observed. This was due to the failure of the steel cable which has multiple strands; the failure of each strand led to sudden change in the load displacement curve. For beam B, first flexural crack occurred at about 25% of the ultimate load. The load continued to increase until the maximum value of 53.4 kN at a deflection of 15 mm. The load deflection curve then exhibited a plateau within a deflection range between 15 mm and 33 mm. Abrupt load reductions were also observed at the post peak region of the curve.

Fig. 9b compares the load deflection behavior of beams J4 and J5 which were subject to three-point loading and four-point loading, respectively. The elastic load displacement response of these two beams were close to each other even their face plate thickness were slightly different. The ultimate loads of beams J4 and J5 were governed by welding failure of the J-hook connectors and yielding of the tension steel plate, respectively. Although the maximum loads of beams A, B, J4 and J5 were almost the same, beams A and B exhibited softer load-displacement behavior due to the flexibility of the cable in resisting vertical loads as compared to beams J4 and J5 with interlocked J-hook connectors.

The deformed patterns of the shorter shear span beams at various stage of loading are shown in Fig. 10. As the applied load increased to a certain level, inclined cracks bridging the load points and the bottom face plate were developed. More cracks linking the location of the applied load and supports were observed as the load was increased. The inclined angle of the shear cracks was approximately 45° near the loading point and had a tendency to decrease near the support. Cracks in the beams with cable connectors were almost symmetric at both sides of the load point as shown in Fig. 10a - b. After testing, it was observed that the cables inside the sandwich beams were snapped. For the beams with J-hook connectors, the cracks were not symmetric. Failure occurred at one side of the beam (Fig. 10c- d). This might be the cause of premature welding failure of the J-hook connectors.

4.2 Behaviour of Sandwich beams with ULCC core

The load versus mid-span deflection curves of the sandwich composite beams with ULCC core are shown in Fig. 11. Beams B6, B7, J6 and J7 with ULCC core exhibited some degree of ductility before reaching their maximum loads. The beams with either headed studs or J-hook connectors and with the same shear span have similar load-displacement behaviour up

to the peak load. The ultimate loads of the beams with headed stud were 5% to 13% higher than those of the beams with J-hook connectors. The slight difference in ultimate loads was due to the fact that the ultimate strength of the headed stud were higher than those of the J-hook connectors (see section 3) since the moment capacities of the beams were governed by the shear connectors failure as discussed below.

4.3 Failure modes

Two principal failure modes were observed from the tests; they are shear failure and flexural failure. Beams failed in shear tend to exhibit inclined shear cracks in the core material. However, for the beams failed in flexural mode, vertical cracks were observed and they were well distributed along the beam's length.

Due to the applied point load, the beams were subject to combined action of shear and bending moment. As the shear span increased, the failure tended to be governed by flexure; whereas for shorter shear span, failure was governed by shear. The flexural capacity of the sandwich beam was governed by the number of shear connectors since they might not be enough to achieve full composite action. The beams failed in flexure showed ductile behaviour due to the yielding of the steel face plate in tension. The beams failed in shear showed non ductile behaviour due to concrete cracking and straightening of J-hooks as shown by the sudden change in the load displacement curves (Fig. 9).

For beams J6 and B6, the load deflection behaviour was initially characterised by the yielding of the shear connectors. Final failure occurred suddenly due to shear failure of the core resulting in a sudden drop of load as shown in Fig. 11. Diagonal shear cracks in the core were

visible between the load points and end supports as shown in Fig. 12a and c. Beams J7 and B7 failed in typical flexural failure mode as observed in Fig.12 b and d.

From the experimental observation, it is clear that the flexural behaviour of the sandwiches with J-hook connectors and headed stud are similar and the moment capacities are also comparable. As the design formula for sandwich panel with headed studs are well established, similar design formulas can be developed for sandwiches with J-hook connectors. One of the advantages of using J-hooks over headed studs is the inter-locking between steel face plates by the coupling of the J-hooks, which is not possible in case of headed studs. This inter-locking system helps to prevent separation during casting of concrete and it may enhance the vertical shear resistance of the section. The force transfer mechanism can be explained by a truss analogy as shown in Fig. 13. The J-hook connectors welded to the top and bottom plates act as vertical tension members, and the inclined compressive forces are resisted by the virtual concrete struts as shown in Fig. 13. To prevent the premature shear failure in the concrete core, the connector spacing should not be more than the thickness of the concrete core.

5. Analysis of SCS sandwich structures under static load

The potential failure modes of SCS panels are (1) tensile yielding of the face steel plate, (2) crushing of diagonal concrete strut in compression, and (3) buckling of the steel plate in compression. The ultimate resistance of the SCS sandwich beam may be estimated based on the most critical failure mode from those shown in Fig. 13.

5.1 Flexural resistance of SCS sandwich beams

The plastic moment resistance of a fully composite SCS sandwich section can be determined by assuming a rectangular plastic stress block of depth x_c for the concrete and the tension and compression stress blocks for the face plates as shown in Fig. 14. The concrete beneath the plastic neutral axis (PNA) is assumed to be cracked. The forces in the steel plates depend on the material yield strength and shear strength of the connectors in resisting interfacial shear stresses between the steel plate and the concrete core. It is also assumed that sufficient shear connectors are provided to prevent local buckling of the compression steel plate.

The nominal compressive force in concrete (N_{cu}) is given by:

$$N_{cu} = \frac{0.85f_c}{\gamma_c} bx_c \quad (1)$$

where b , f_c and γ_c are the beam width, concrete cylinder strength and partial safety factor for concrete respectively. By equating the tensile and compressive forces in the section, the depth of the plastic neutral axis can be obtained:

$$N_c + N_{cu} = N_t \quad (2)$$

Letting $N_c = \sigma_y bt_c$, $N_t = \sigma_y bt_t$ and N_{cu} from Eq.1 into Eq. 2,

$$x_c = 1.176\gamma_c\sigma_y(t_t - t_c)/f_c \quad (3)$$

where $\gamma_c=1.5$ as recommended by Eurocode 2 (2004) [19] for design purpose. N_c and N_t are the compressive and tensile resistance of the top and bottom steel plates, respectively.

By taking moments about the centre of the compression steel plate, the plastic moment of resistance of the sandwich section is

$$M_{pl} = \sigma_y bt_t \left(h_c + \frac{t_c}{2} + \frac{t_t}{2} \right) - \frac{0.85f_c bx_c}{\gamma_c} \left(0.5x_c + \frac{t_c}{2} \right) \quad (4)$$

If the steel plates are of equal thickness and strength, the SCS sandwich beams can be treated as an under reinforced concrete beam. The SCS sandwich beam will deflect extensively and develop extensive and wide cracks in the final loading [8, 9]. After yielding of tension steel plate, the cracking of the concrete will continue to rise towards the compression steel plate. In this case, the strain at the bottom plate is very large compared to top steel plate. The moment capacity of the beam is reached when the neutral axis moves near to the lower surface of the compression plate (i.e. $x \approx 0$) and the bottom plate is fully yielded. Therefore, in case of $t_c = t_t = t$, the moment of resistance of the sandwich section becomes

$$M_{pl} = N_t (h_c + t) \quad (6)$$

For fully composite beam, $N_t = \sigma_y b t_t$, in which σ_y is the yield strength of the steel plate, the number of shear connectors required to achieve full composite action should be $n_s = \sigma_y b t_t / (\kappa P_R)$ in which κ is the reduction factor for concrete. Therefore, Eq. 6 becomes:

$$M_{pl} = \sigma_y b t_t (h_c + t) \quad (7)$$

If full composite cannot be achieved, the beam should be designed for partially composite and the moment resistance has to be reduced correspondingly. For partially composite beam,

$$N_t = n_p (\kappa P_R) \quad (8)$$

in which n_p is the actual number of connectors provided between the points of zero and maximum moment. Therefore, Eq. 6 can be written as:

$$M_{pl} = n_p (\kappa P_R) (h_c + t) \quad (9)$$

For SCS sandwich beam, it should be ensured that a 45° shear cracking of concrete can be resisted by at least one pair of shear connectors. For beams with shallow depth and lightweight concrete core, it is recommended that the spacing of the shear connector (S_s)

should be at least equal to the core thickness to provide effective resistance against shear cracking of concrete core.

5.2 Resistance to local buckling of the steel face plate

Local buckling of the steel face plate in the compression depends on the spacing of the shear connectors. The spacing of the connectors may be derived from the critical buckling load of a plate fixed at both ends:

$$\frac{S_s}{t_c} = \sqrt{\frac{\pi^2 E}{3(1-\nu^2)\sigma_{cr}}} \quad (10)$$

where S_s and t_c are the longitudinal spacing of the shear connectors and thickness of the steel face plate, respectively, in the compression region. Assuming, $E = 210 \times 10^3 \text{ N/mm}^2$, $\sigma_{cr} = \sigma_y$ and $\nu = 0.3$, the spacing of the connectors should be governed by

$$\begin{aligned} \frac{S_s}{t_c} &\leq 52 \quad \text{for S275 steel and} \\ \frac{S_s}{t_c} &\leq 46 \quad \text{for S355 steel} \end{aligned} \quad (11)$$

This limiting spacing for J-hook connectors should be maintained assuming no chemical and frictional bonds exist between the concrete core and steel face plates. For structural efficiency and to avoid local buckling of the face plate, full composite interaction is required between the core and the steel face plates.

6. Comparison of analytical predictions with test results

The maximum flexural capacities of the SCS sandwich beams predicted using the analytical methods described in Section 5 by assuming partial safety factor $\gamma_c = 1.0$ are compared with the test results as shown in Table 4. The shear capacity of the J-hook connectors, P_R , shown

in Table 4 was obtained from push-out tests conducted by Liew and Sohel [13] and Yan *et al.* [20]. In the calculation of N_t and M_{pl} in Table 4, $0.9P_R$ should be used to account for flexural cracking in the concrete core.

The shear capacity of the headed stud may be calculated using the Eurocode 4: Part 1-1 approach [21],

$$P_R = \min \left(0.8\sigma_u \frac{\pi d^2}{4\gamma_v}, 0.29\alpha d^2 \sqrt{f_{ck} E_c} / \gamma_v \right) \quad (12)$$

where d = diameter of the stud shank; σ_u = ultimate tensile strength of the stud (≤ 500 MPa); f_{ck} = characteristic cylinder strength of concrete; E_{cm} = secant modulus of concrete; $\alpha = 0.2(h_s/d + 1)$ for $3 \leq h_s/d \leq 4$ or $\alpha = 1.0$ for $h_s/d \geq 4$; h_s = overall height of the stud. The partial safety factor γ_v is 1.25, but for comparison purpose $\gamma_v = 1.0$ is used.

In case of beams with cable connectors, the shear capacity of the U-connectors can also be predicted by using Eq.12. The minimum height of the U-connector should be three times of the bar diameter (i.e. $3 \leq h_s/d$). The vertical shear force is resisted by the steel cables connecting the top and bottom U-connectors. The vertical shear resistance of the cable (V_{sc}) can be written as

$$V_{sc} = T \sin \theta \quad (13)$$

where T is the tensile resistance of the cable and θ is the inclined angle of the cable with respect to the horizontal axis as shown in Fig. 15.

The comparison of the predicted loads and test results are shown in Table 4. Beams J7 and B7 were designed for tension plate failure due to flexure. The ratio of the experimental ultimate load (P_{pl-exp}) to the predicted load (P_{pl}) by plastic theory is 1.01 and 1.06,

respectively. Beams J5 and J6 were designed for partial composite action, in which the maximum load was governed by the shear connector failure. The ratio P_{pl-exp} / P_{pl} of these two beams are 1.04 and 1.00 respectively. J4 was also a partial composite beam but the failure was due to premature weld failure of the J-hook connector. In case of beam B6, the shear capacity of the studs ($0.9n_p P_R$) and tensile capacity of the steel plate ($\sigma_y b t_t$) were almost identical at the load point. Either tensile yielding of the bottom plate or shear failure of the studs was expected. But, finally the beam failed in shear. For this reason, full flexural capacity was not developed in this beam and the ratio P_{pl-exp} / P_{pl} is 0.87 for this beam.

7. Conclusions

Novel shear connectors such as J-hooks and U connectors with interlinked cables (UCU) have been proposed for the Steel-Concrete-Steel (SCS) composite structures to enhance the interfacial bond between the face plate and the internal core.

A series of tests was carried out on composite beams with UCU connectors and J-hooks. Test results showed that the cable connectors (UCU) could enhance the interfacial bond between the steel face plates and the internal core as well as increase the vertical shear resistance of the beams. However, the elastic stiffness of the SCS beam with UCU connectors was less than the beam with J-hook connectors. This is because the cable is more flexible to resist the vertical shear force than that of J-hook connectors.

Further tests were carried out on SCS sandwich composite beams in-filled with ultra lightweight cement composite (ULCC). Both the conventional headed shear studs and J-hook connectors were used in the beam tests to compare their load deflection behaviour and ultimate strength. Test results showed that sandwich beams with J-hook connectors exhibited

similar load deflection and ultimate strength behaviour as compared to conventional sandwich beams with headed shear studs. Therefore, Eurocode 4 design method for conventional headed studs may be extended with slight modification to J-hook connectors for beams subject to quasi-static loads.

The ultimate strength behaviour of the sandwich beams with ULCC is similar to those with normal concrete, if brittle failures due to weld toe failure of connectors can be avoided. Therefore, Eurocode 4 for ultimate strength design method may be extended to sandwich beams with ULCC. The moment capacity of the SCS sandwich beam may be predicted using the plastic stress block methods in Eurocode 4. In general, the predicted results agreed well with the test results with error up to 6% except one specimen in which the predicted result is lower than the test result by 7%. It is recommended that the shear capacity of the J-hook connectors should be reduced by 0.9 to account for the lower bearing strength in the ultra-lightweight cement composite core.

ACKNOWLEDGMENT

The research herein was funded by Singapore Defense Technology Agency and National University of Singapore on a project “Sandwich composite systems with ultra lightweight cement based core for applications in extreme environment” (Project No. 0921420044). The authors gratefully express their gratitude for the support.

REFERENCES

- [1] Solomon SK, Smith DW Cusens AR. Flexural tests of steel-concrete-steel sandwiches. Magazine of Concrete Research 1976; 28(94):13-20.
- [2] Tomlinson M, Tomlinson A, Chapman M, Wright HD and Jefferson AD. Shell composite construction for shallow draft immersed tube tunnels. ICE International Conference on Immersed Tube Tunnel Techniques, Manchester, UK, April 1989.
- [3] Narayanan R, Roberts TM, Naji FJ. Design Guide for Steel-Concrete-steel Sandwich Construction. Volume 1: General Principles and Rules for Basic Elements. The steel Construction Institute, 1994. Ascot, Berkshire, UK.
- [4] Narayanan R, Bowerman H, and Naji F. Application guidelines for steel-concrete-steel sandwich Construction: 1: Immersed tube tunnels. Steel Construction Inst., Ascot (United Kingdom), 1997.
- [5] Shanmugam NE, Kumar G, Thevendran V. Finite element modelling of double skin composite slabs. Finite Elements in Analysis and Design 2002; 38(7):579-599.
- [6] Liew JYR., Koh CG, and Sohel KMA. Impact tests on steel-concrete-steel sandwich beams with lightweight concrete core. Engineering Structures 2009, 31(9), 2045-2059.
- [7] Sohel KMA, Liew JYR, Alwis WAM, Paramasivam P. Experimental investigation of low-velocity impact characteristics of steel-concrete-steel sandwich beams. Steel and Composite Structures: An International Journal 2003; 3(4):289-306.
- [8] Bowerman H, Gough M, King C. Bi-Steel Design & Construction Guide. Scunthorpe: British Steel Ltd. 1999.
- [9] McKinley B, Boswell LF. Behaviour of double skin composite construction. Journal of Constructional Steel Research 2002; 58(10):1347-1359.
- [10] Leekitwattana M, Boyd SW, Sheno RA. Evaluation of the transverse shear stiffness of a steel bi-directional corrugated-strip-core sandwich beam. Journal of Constructional Steel Research 2011; 67(2):248-254.
- [11] Gowda SS, Hassien P. Improvement of shear capacity of composite Structures. International Journal of Offshore and Polar Engineering 1991; 1(4):324-326.
- [12] Remennikov, A., Kong, S.Y., Uy, B. Impact resistance of non-composite axially restrained steel-concrete-steel sandwich panels. The 5th Civil Engineering Conference in the Asian Region and Australasian Structural Engineering Conference 2010, 8-12 August 2010, Sydney, Australia, 2010
- [13] Liew JYR, Sohel KMA. Lightweight Steel-Concrete-Steel Sandwich System with J-hook Connectors. Engineering Structures 2009; 31(5):1166-1178.

- [14] Weitzenböck JR, Grafton T. Assessment of the INCA Steel-concrete-steel sandwich technology- A public report. DNV, Det Norske Veritas, NO-1322 Høvik, Norway, 2010.
- [15] Grafton TJ and Weitzenböck JR. Steel-concrete-steel sandwich structures in ship and offshore engineering. *Advances in Marine Structures*—Guedes Soares & Fricke (eds), 549-558, Taylor & Francis Group, London, 2011.
- [16] Dong Wei. Mechanical Properties of Fiber Reinforced Ultra Lightweight Cement Composite. B. Eng Thesis, Civil Engineering Department, National University of Singapore 2010.
- [17] Chia KS, Zhang MH, Liew JYR. High-strength ultra lightweight cement composite – material properties. *Proceedings of 9th International Symposium on High Performance Concrete - Design, Verification & Utilization*, 9 - 11 Aug 2011, Rotorua, New Zealand, 2011.
- [18] Wang JY, Zhang MH, Li W, Chia KS, Liew JYR. Stability of cenospheres in lightweight cement composites in terms of alkali–silica reaction. *Cement and Concrete Research* 2012, 42(5):721–727.
- [19] Eurocode 2: design of concrete structures - part 1-1: general rules and rules for buildings. BS EN 1992-1-1:2004.
- [20] Yan JB, Liew JYR, Soheli KMA, Zhang MH and Chia KS. Strength of J-hook connectors subjected to combination of axial and shear load. *The 2011 World Congress on Advances in Structural Engineering and Mechanics (ASEM'11)*, 238-239. Sep, 2011, Seoul, Korea
- [21] Eurocode 4: Design of composite steel and concrete structures - Part 1.1: General rules and rules for buildings. BS EN 1994-1-1:2004.

Table 1
Details of the beam specimens and connectors

Beam name	Connector type	t (mm)	σ_y (MPa)	S (mm)	h_c (mm)	Span (mm)	d (mm)	Core type	f_c (MPa)	ρ_c (kg/m ³)	Loading
A	Cable*	5.9	275	100	100	1000	10	LWC	26	1430	3 point
B	Cable*	5.9	275	100	100	1000	10	LWC	26	1430	3 point
J4	J-hook	4.0	275	100	80	1000	10	LWC	26	1430	3 point
J5	J-hook	5.8	305	100	80	1100	10	LWC	26	1430	4 point
J6	J-hook	5.8	305	100	100	1100	12	ULCC	60	1440	4 point
J7	J-hook	5.8	305	100	100	1600	12	ULCC	60	1440	4 point
B6	Stud	5.8	305	100	100	1100	13	ULCC	60	1440	4 point
B7	Stud	5.8	305	100	100	1600	13	ULCC	60	1440	4 point

f_c = cylinder compressive strength of the cementitious core; σ_y = yield strength of the steel face plate; t = steel plate thickness; S = spacing of the shear connectors; d = shank diameter of the shear connectors; h_c = core thickness; ρ_c = density of the cementitious core.

* diameter of the cable = 6 mm and tensile strength of the cable = 10.3 kN

Table 2
Basic material properties of ULCC and LWC at age 28-day

Material property	ULCC	LWC
Density after de-mould (kg/m ³)	1440.00	1430.00
Compressive strength, cube f_{cu} (MPa)	64.00	28.00
Compressive strength, cylinder f_c' (MPa)	64.60	26.00
Ratio of f_c'/f_{cu}	1.01	0.93
Splitting tensile strength (MPa)	4.40	2.10
Flexural strength (MPa)	6.70	3.77
Static modulus of elasticity (GPa)	16.00	12.00
Static Poisson's ratio	0.25	0.23

Table 3
Test results and failure modes

Beam name	Connector type	Loading	Ultimate load, P_{pl-exp} (kN)	Plastic moment, M_{pl} (kN-m)	Failure modes
A	Cable	3 point	59	14.8	Core shear
B	Cable	3 point	54	13.5	Core shear
J4	J-hook	3 point	56	14.0	Connector welding failure
J5	J-hook	4 point	70	12.8	Connector shear failure
J6	J-hook	4 point	165	30.5	Connector shear failure
J7	J-hook	4 point	122	37.8	Tensile yielding of bottom plate
B6	Stud	4 point	175	32.4	Core shear
B7	Stud	4 point	128	39.7	Tensile yielding of bottom plate

Table 4
Comparisons between theoretical predictions and test results of sandwich beams

Beam	t (mm)	h_c (mm)	d (mm)	Loading	L_s (mm)	P_R (kN)	n_p	N_t (kN)	M_{pl} (kN-m)	P_{pl} (kN)	P_{pl-exp} (kN)	P_{pl-exp} / P_{pl}
J4	4.0	80	10	3 point	500	20.0	10	180.0	15.1	60.5	56	0.93
J5	5.8	80	10	4 point	367	20.0	8	144.0	12.4	67.3	70	1.04
J6	5.8	100	12	4 point	370	42.1	8	303.1	30.5	164.7	165	1.00
J7	5.8	100	12	4 point	620	42.1	12	353.8	37.4	120.7	122	1.01
B6	5.8	100	13	4 point	370	49.5	8	353.8	37.4	202.2	175	0.87
B7	5.8	100	13	4 point	620	49.5	12	353.8	37.4	120.7	128	1.06

h_c = core thickness; t = steel plate thickness; d = shank diameter of the shear connectors; P_R = Shear connector capacity; P_{pl} = calculated ultimate load; P_{pl-exp} = experimental ultimate load; L_s = shear span; n_p = number of shear connectors between zero to maximum moment; N_t = force in the tensile steel plate;

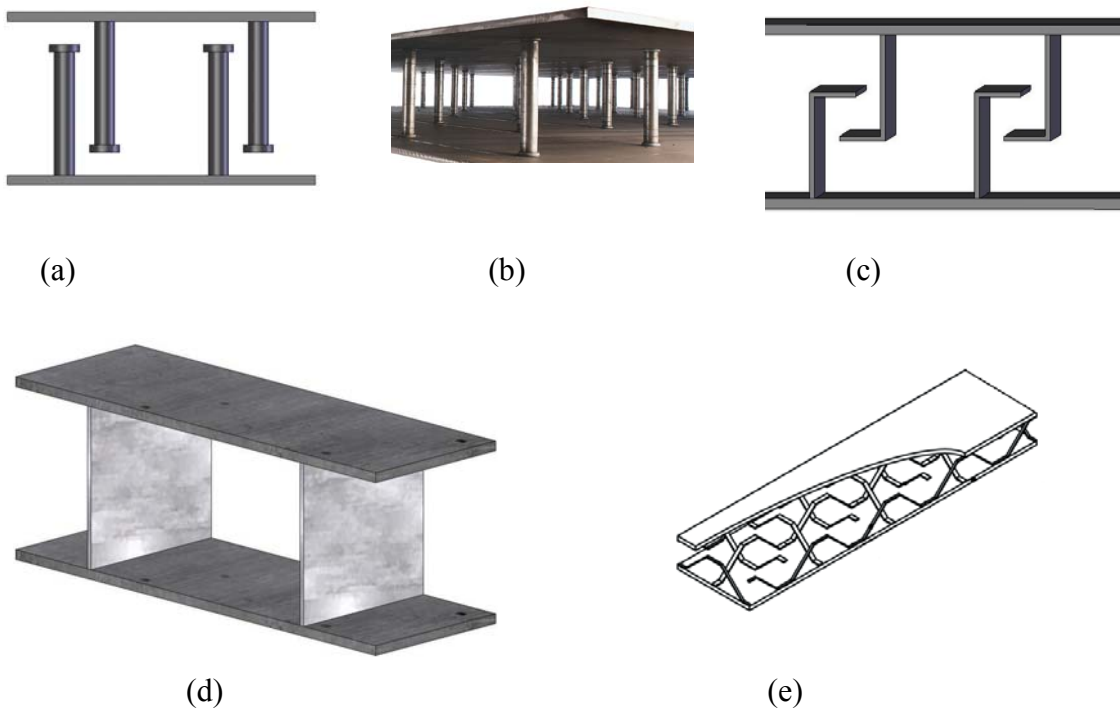


Fig. 1 Different types of mechanical shear connectors used in SCS structure (a) Headed shear stud connector; (b) Bi-steel connectors; (c) angle shear connectors; (d) plate connectors; (e) bi-directional CSC system [10].

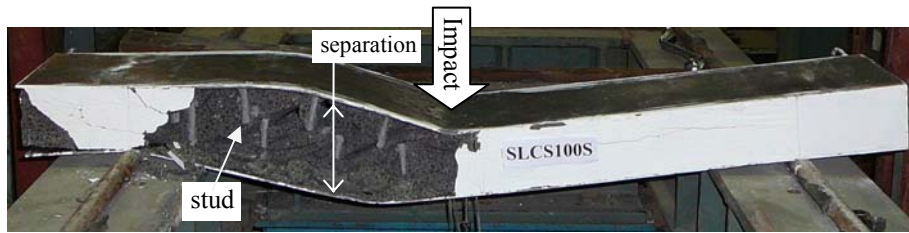


Fig. 2 Separation of face plates from core when SCS beam with overlapped shear studs subject to impact

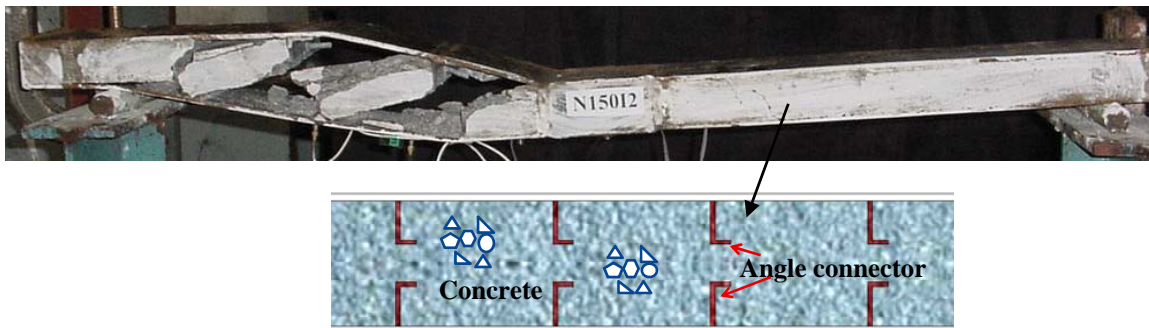


Fig. 3 Separation of face plate from core when the sandwich beams with angle connector subjected to impact

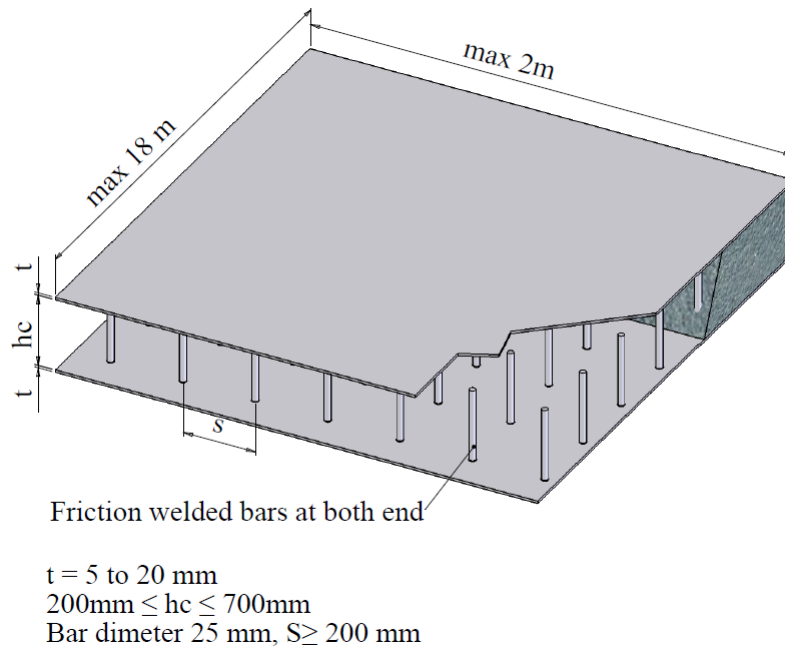
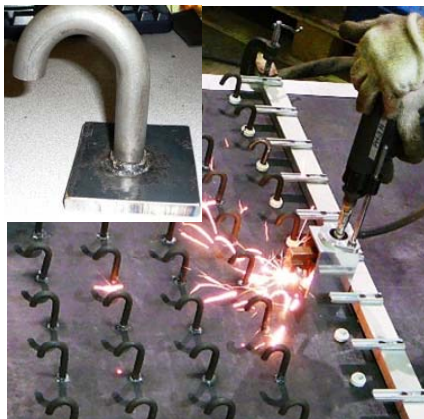


Fig. 4 Bi-Steel panel by Corus

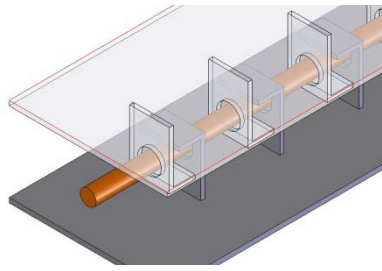


(a)

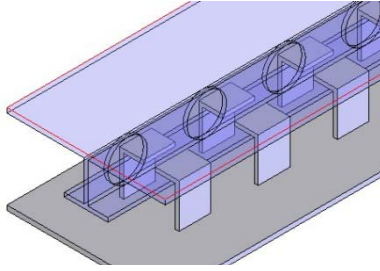


(b)

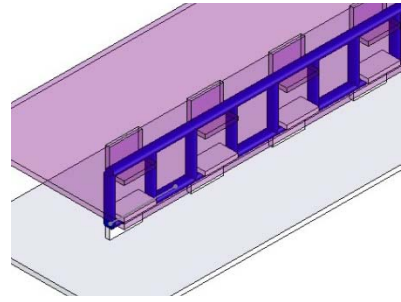
Fig. 5 (a) Automatic welding of J-hook connector; (b) Assembly of SCS panels with J-hook connectors



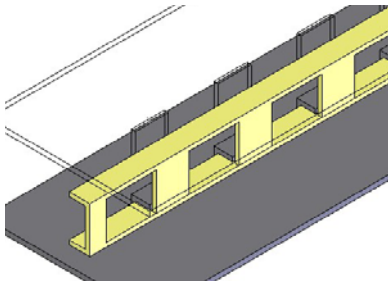
(a) ASA



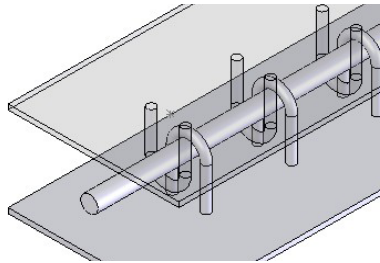
(b) AT



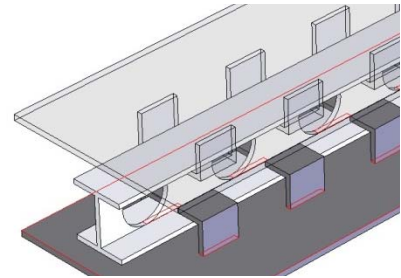
(c) AHA



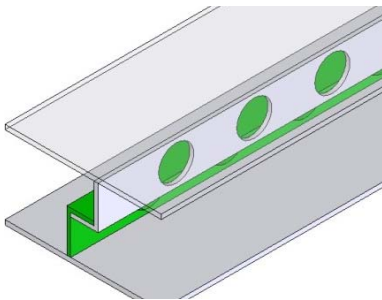
(d) ACA



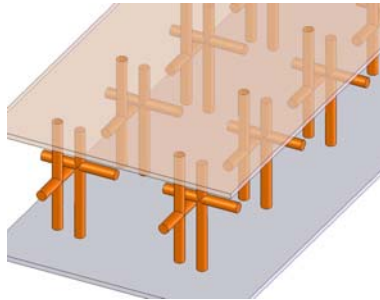
(e) USU



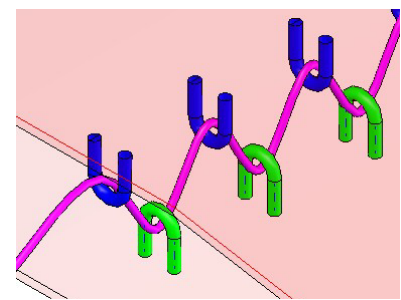
(f) AIA



(g) AA

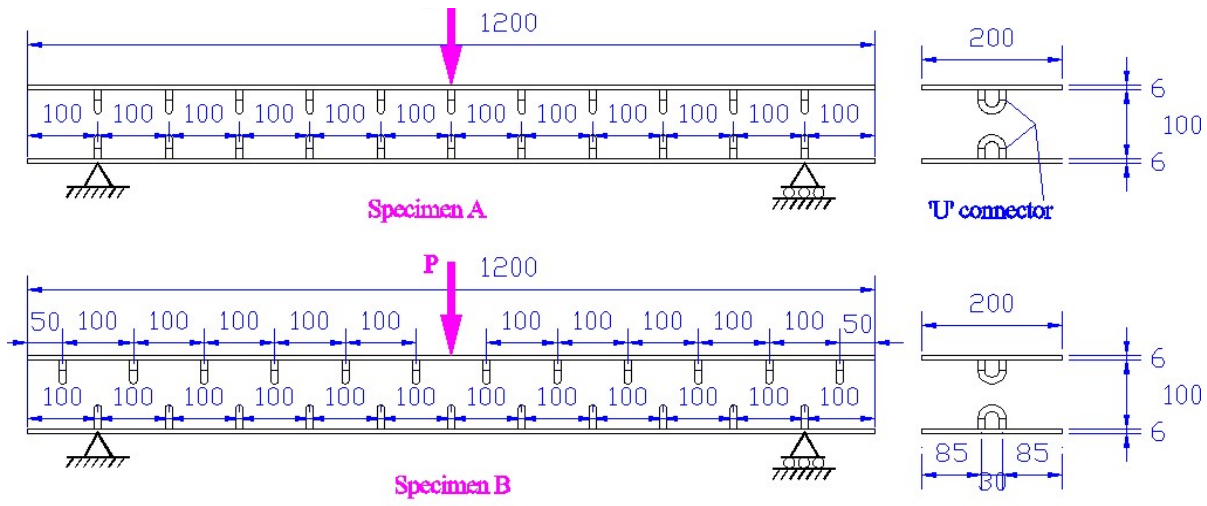


(h) RC



(i) UCU

Fig. 6 Proposed mechanical connectors used in SCS panel: (a) Angle-Steel bar-Angle (ASA); (b) Angle-T channel (AT); (c) Angle-Steel hoop-Angle (AHA); (d) Angle-C channel-Angle (ACA); (e) U connector-Steel bar-U connector (USU); (f) Angle-I beam-Angle (AIA); (g) Angle-Angle (AA); (h) Root connector (RC); (i) U connector-Steel Cable-U connector (UCU).



(a) Schematic diagram of the SCS beam showing the U-connector position



Beam A



Beam B

(b) SCS sandwich beams UCU connectors of different cable arrangements

Fig. 7 SCS beams with U connector and cables

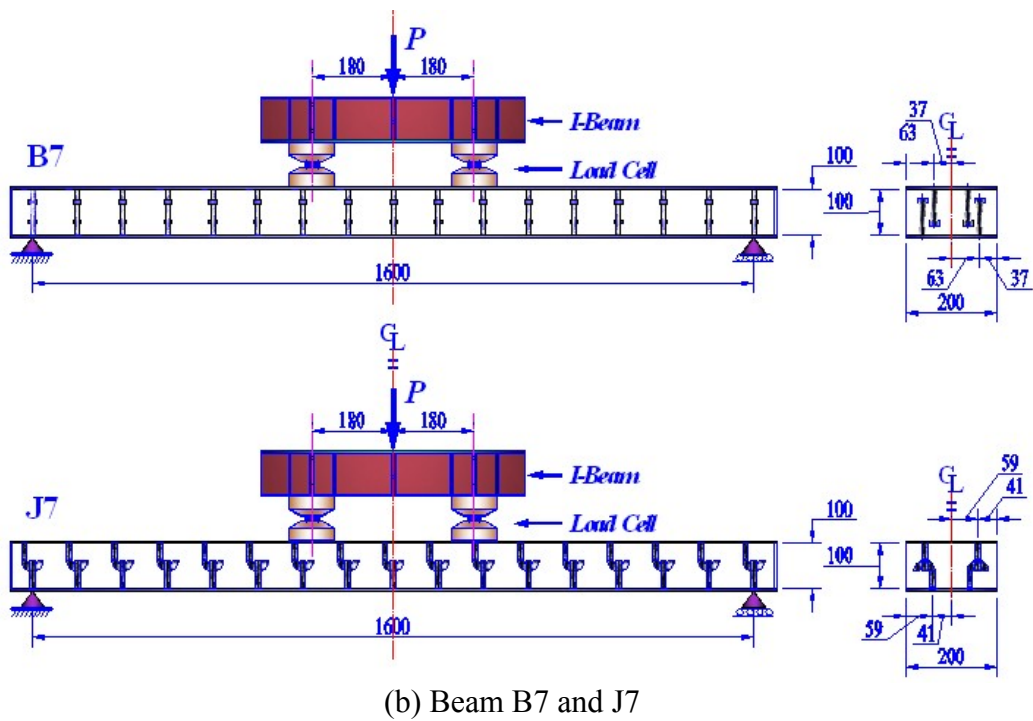
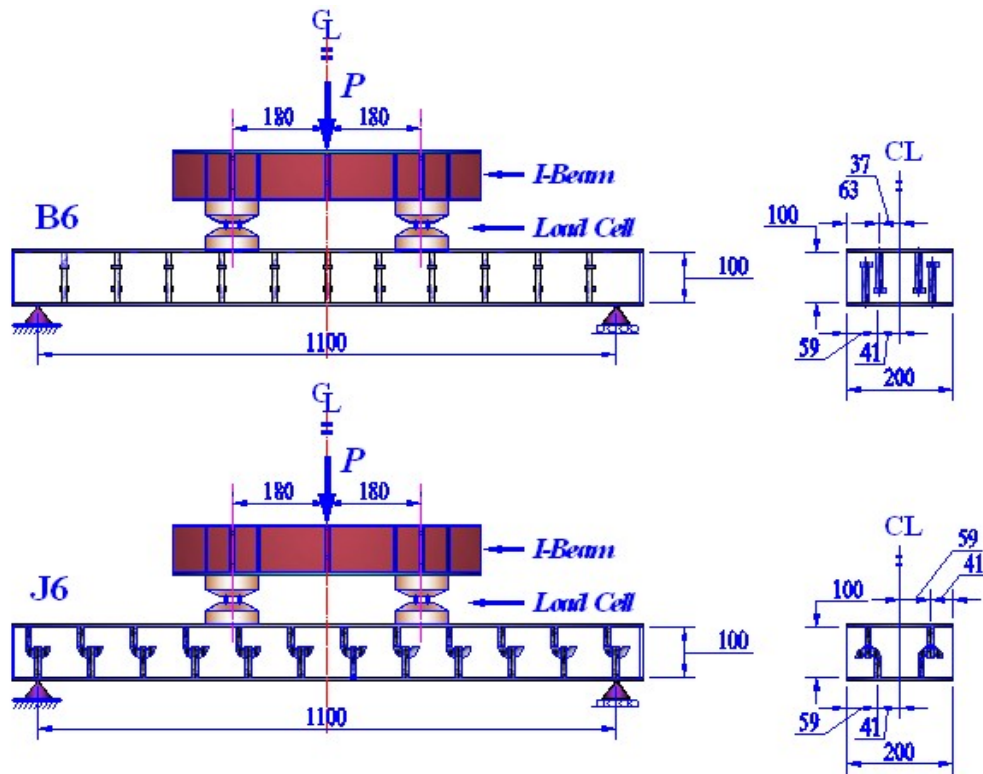


Fig. 8 Beams with headed stud and J-hook connectors

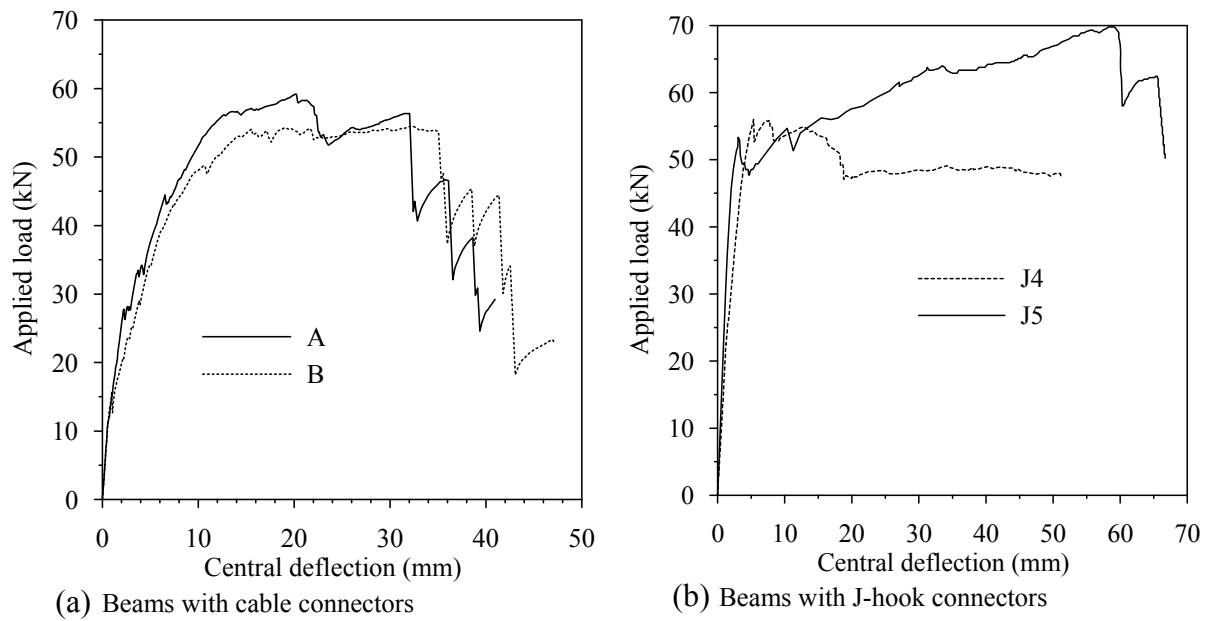


Fig. 9 Load deflection curves of beams A, B, J4 and J5

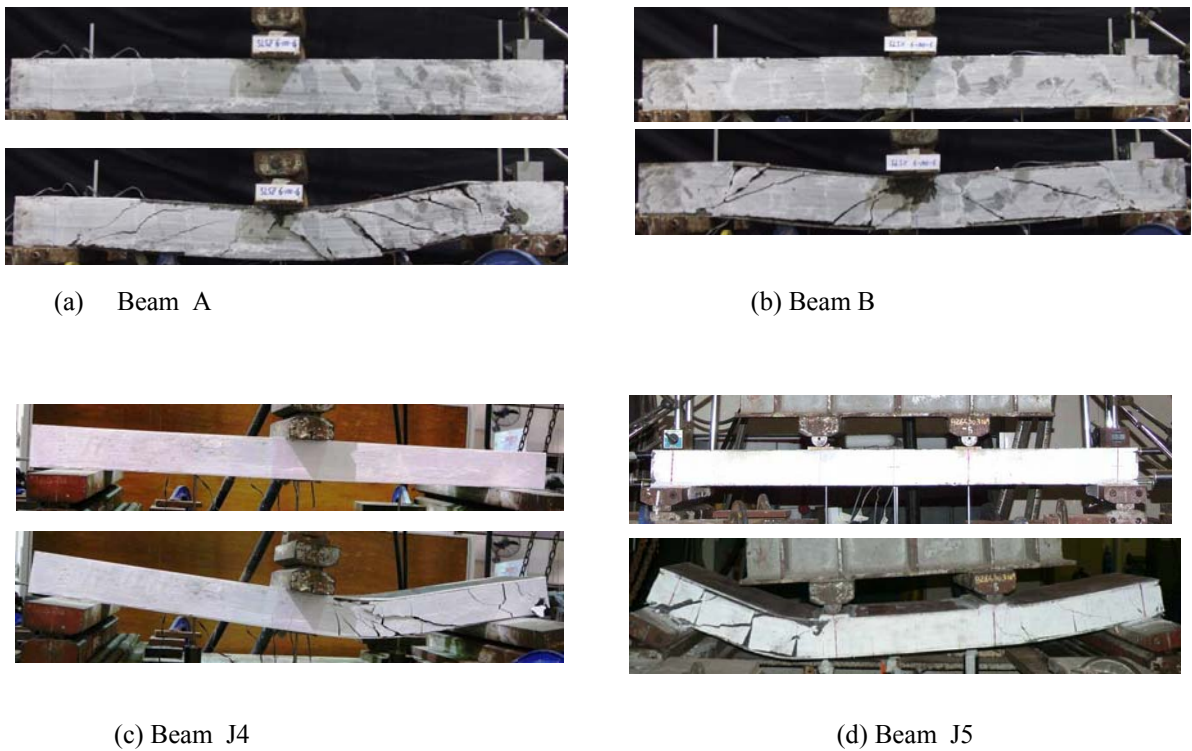


Fig.10 Failure modes of the beams specimens A, B, J4 and J5

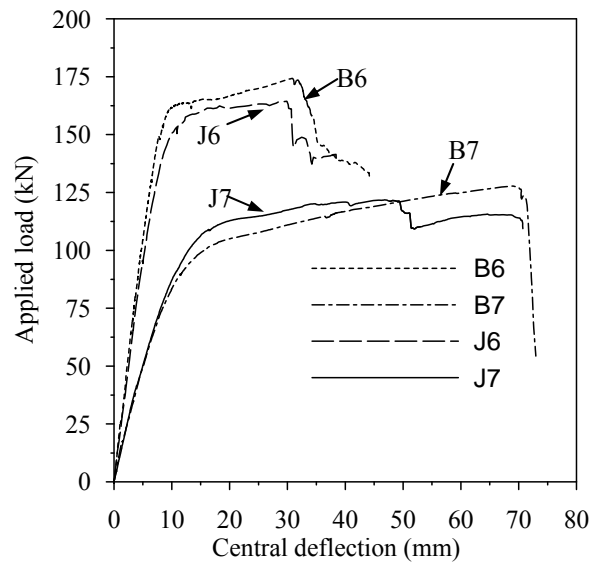


Fig. 11 Load deflection curves of sandwich beams with ULCC core

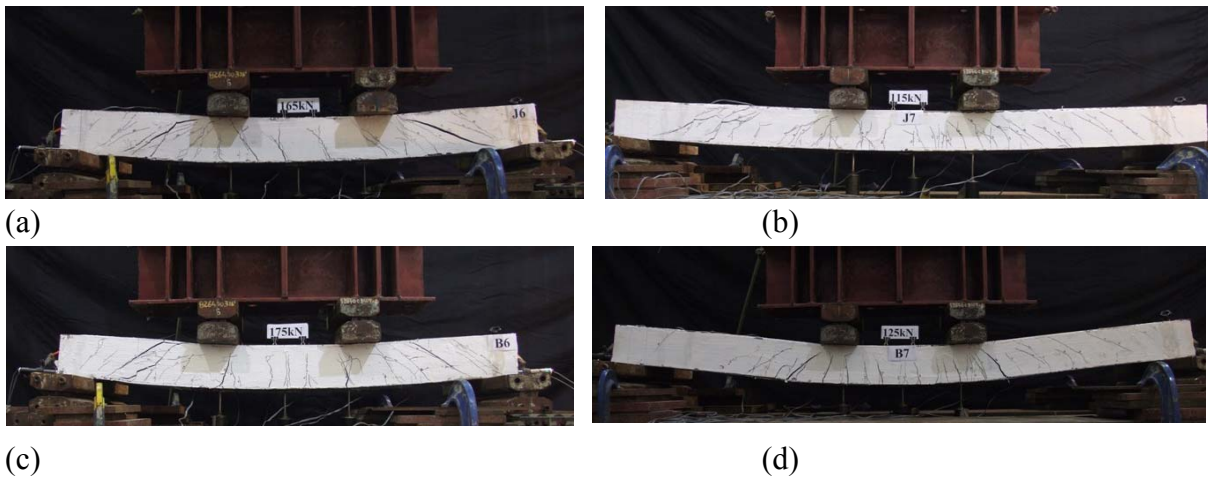


Fig. 12 Pictures at final failure of the beams with ULCC core (a) J6, (b) J7, (c) B6, and (d) B7

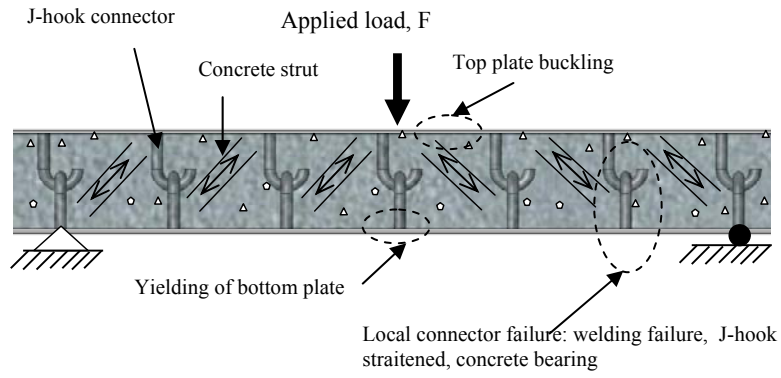


Fig. 13 Load transfer mechanism and failure modes of SCS sandwich beam with J-hook connectors

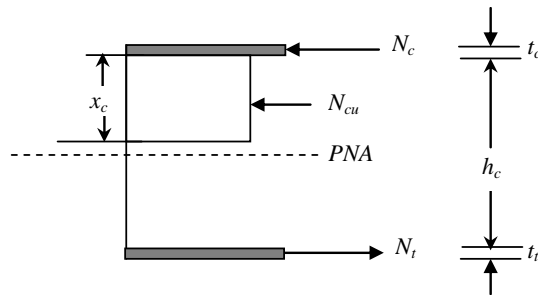


Fig. 14 Force distribution in the section at fully plastic stage

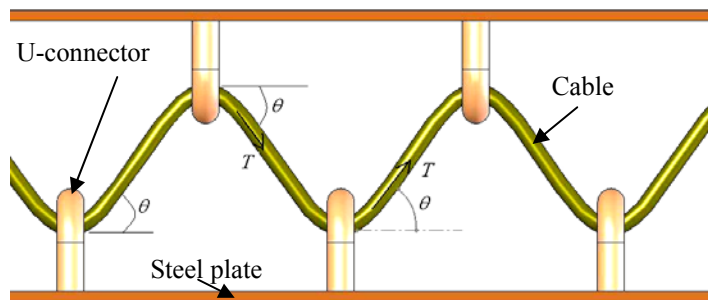


Fig. 15 Force acting on cable in SCS sandwich with UCU connectors