Experimental Behavior of Steel-Concrete-Steel Sandwich Beams with Truss Configuration of Shear Connectors

Dr.Thaar Saud Salman Al-Gasham
Lecturer
College of Engineering - University of Wasit
E-mail:dr.thaaralgasham@yahoo.com

ABSTRACT

This paper presents experimentally a new configuration of shear connector for Steel-Concrete-Steel (SCS) sandwich beams that is derived from truss configuration. It consists of vertical and inclined shear connectors welded together and to cover steel plates infilled with concrete. Nine simply supported SCS beams were tested until the failure under a concentrated central load (three-point bending). The beams were similar in length (1100mm), width (100mm), and the top plate thickness (4mm). The test parameters were; beam thickness (150, 200, 250, and 300mm), the bottom plate thickness (4, and 6mm), the diameter of the shear connectors (10, 12, and 16mm), and the connector spacing (100, 200, and 250mm). The test results showed that the stiffness of SCS beam augmented with the increase in beam thickness, lower plate thickness, and connector diameter while it decreased with increasing the connector spacing. The ultimate load capacity of the SCS beams increased to 72.2% and 42.1% by enlarging the beam thickness and connector diameter to 100% and 60%, respectively. Increasing the connector spacing of 150% led to a considerable reduction in the ultimate load reached to 68.4%. Finally, the ultimate strength was not affected by augmenting the bottom plate thickness up to 50%.

Keywords; sandwich beam, SCS beam, shear connectors, steel plate, concrete core, truss configuration

السلوك التجريبي للعتبـات السندوجية حديد-خرسانة-حديد مع روابط قصية بهيئة مسنم

د. ثائر سعود سلمان الغشام
كلية الهندسة / جامعة واسط

الخلاصة

في هذا البحث تم تقديم هيئة جديدة(هيئة المسنم) للروابط القصية في العتبات السندوجية بشكل تجريبي . هذه الهيئة تتكون من روابط قصيه عمودية ومائلة مرتبطة مع بعضها ومع الصفائح الحديدية بواسطه اللحام مملوءه بالخرسانة. تم فحص تسع عتبات بسيطة الاسناد الى حد الفشل تحت تأثير حمل مركز مسلط في منتصف العتبة. هذه العتبات متشابهة في الطول (1100 ملم) و العرض (100ملم) وسمك الصفيحة العلوية (4ملم) . متغيرات الفحص تضمنت سمك العتبات (150 و 200 و 250 و 300 ملم) وسمك الصفيحة السفلية (4 و 6 ملم) وقطر الروابط القصية (10 و 12و 16 ملم) ومسافات الروابط القصية (100 و 200 و 250 ملم) . نتائج الفحص بينت , أن صلابة العتبات تزداد مع زيادة كلاً من سمك العتبة وسمك الصفيحة السفلية وقطر الروابط القصية في حين أنها تقل مع إزدياد المسافات بين الروابط القصية . أن الحمل الاقصى للعتبات قد أزداد بحدود 72.2 % و 42.1% عند ازدياد سمك العتبة وقطر الرابط القصي الى 100% و 60% على التوالي. زيادة مسافات الروابط القصية الى 150% أدى الى نقصان كبير في الحمل الاقصى وصل الى 68.4% . اخيراً الحمل الاقصى لم يتأثير بزيادة سمك الصفيحة السفلية بحدود 50%.

الكلمات المفتاحية: العتبة السندوجية , الروابط القصية, الصفيحة الحديدية, اللب الخرساني, هيئة المسنم.

1. INTRODUCTION

Steel-Concrete-Steel (SCS) beams are a comparatively modern system of construction composed of two relatively thin cover plates and a core of plain concrete sandwiched between them. The forces between the concrete core and the cover plates are transferred by shear connectors. Thus, the structural behavior of SCS sandwich beams to be influenced significantly by shear connectors' efficiency, **Anandavalli, et al., 2013**. The first use of SCS construction was in a submerged tube tunnel for Conway River in Cardiff, UK. The SCS system sometimes is known as Double skin Composite (DSC) construction.

The SCS sandwich beams are considered more economical than the beams with ordinary reinforcement due to the replacement of both conventional reinforcement and permanent formwork by external plates and second, the exterior plates and shear connectors are easy to fabricate at the site. Therefore, their cost is relatively low compared with the high cost for detailing, bending and fixing of the conventional bar reinforcement. The cover plates improve the water tightness of SCS sandwich constructions. The SCS beams have an ability to endure large deformations without cracking because of their ductility and energy absorption are relatively high, **Shanmugam and Kumar, 2005**.

Various forms of shear connectors are used in the SCS sandwich constructions. The common forms are through-through connectors and headed stud connectors. In conventional headed stud connectors, the pullout strength of studs influences the resistance of cover plates against tensile separation, **Wright, et al., 1991**.

In Bi-Steel SCS system, an array of transverse bars welded into surface plates is used as shear connectors. Bars have regular patterns with close spacing. **Roberts et al., 1996**, tested a series of SCS sandwich beams under two or four-point loads with a range of the span to depth ratios. The tests showed that the yield and slip in tension plates caused the primary failure modes. **Xie, et al., 2007**, tested eighteen SCS beams with Bi-steel connectors. The tests showed four types of failure mode; concrete shear failure, bar tension failure, bar shear failure, and tension plate yielding.

In **2008, Foundoukos, et al**., presented experimental and analytical studies to evaluate the static and fatigue responses of Bi-Steel SCS beams. A truss model with tapering web compression was developed to determine the forces inside the beams. The predictions of the model were conservative comparing with the test results. J-hook connectors were used by **Liew and Sohel, 2009**, to fix the cover plates on their position with the light weight concrete core. Push-out tests observed that the abilities of J-hook connectors to transfer the shear force were larger than that of the conventional headed stud connector. **Chu, et al., 2013**, tested eight SCS beams with channel steel connectors. Angle steel was used to connect the surface plates with channel steel. Most of the beams experienced ductile failure where the tension steel plates yielded. Numerical simulation using the finite-element approach was presented in 2013 by **Anandavalli, et al.,** to study the static behavior of SCS beams. In this study, two new configurations of Bi-directionally connectors were introduced. The results indicated that Bi-directionally inclined connectors were more ductile than through-through connectors while the ultimate load remained same.

In this paper, a new shape of shear connectors (truss configuration) was proposed. The static behavior of SCS sandwich beams with truss configuration connectors under three-point loads was studied experimentally. The tests focused on the influences of beam thickness, tensile plate thickness, diameter and spacing of shear connectors.

2. SPECIMENS DESIGN

A total of nine SCS sandwich beams with truss configuration of shear connectors were fabricated to exhibit different failure modes. All beams were conformable in length and width that were 1100mm and 100mm, respectively. The thickness of top cover plates was 4mm and kept constant for all specimens. The other geometric properties were varied to study their effect on the static behavior of SCS sandwich beams. They included; beam thickness (150, 200, 250, and 300mm), bottom plate thickness (4 and 6 mm), connector diameter (10, 12, and 16mm) and connector spacing (100, 200, and 250mm).

The reference beam (R) was constructed with thickness of 200mm, lower plate (tensile plate) thickness of 4mm, and shear connectors' diameter and spacing of 10 mm and 100mm, respectively. The remaining eight specimens are named by codes composed of three capital letters followed by a number. The first two letters are (RE); they mean that specimens are geometrically similar to the reference beam except for one property. The third letter refers to this property as follows (H=beam thickness, P=bottom plate thickness, D=connector diameter, and S=connector spacing). Finally, the number refers to the value of property in mm. The details of test specimens are listed in **Table 1**.

The truss shear connectors consisted of vertical and inclined members equally spaced over the length of SCS sandwich beams, as shown in **Figs. 1 and 2**. They were deformed bars and welded to the cover plates using a welding gun. All beams contained two rows of shear connectors spaced at 60mm center to center in the width direction. **Table 2** shows the properties of the shear connectors and the cover plates.

A normal-weight concrete was used as a core sandwiched between the two external plates. The core composed of normal Portland cement, well-graded crushed aggregates of 10 mm maximum size, and washed sand. The mix design proportions by weight were 1 (cement): 1.77 (sand): 2.22 (gravel) with 0.52 water cement ratio. All three specimens were cast in one batch. The compressive concrete strength of each batch was determined by taking the average strength of three cubes with the side length of 150mm as summarized in **Table 1**.

A mechanical mixer was used to produce the concrete. The mixing operations were performed according to the procedure of ASTM C192-1995, where the course and fine aggregates with two-thirds of required water were blended first for one minute, then the cement with the rest water was added and mixed for three minutes followed by three minutes rest period. The specimens were placed inside the plywood molds; these molds were lightly oiled. After mixing, the concrete was poured into molds by three layers. For each layer, well compacting was ensured using vibrator tables, especially in the regions of shear connectors and corners. After removing the molds, the specimens were covered with nylon sheets and kept wet for twenty-eight days.

3. TEST SET UP

The specimens were simply supported and tested under a centrally concentrated load (three-point bending) as shown in **Fig. 3**. The supported length for all beams was 1000mm. A universal machine of 150-ton capacity was employed to apply the load gradually until the failure of the specimen. At each load increment, the initiation and propagation of cracks were carefully examined and marked, and the central deflection of the beams was recorded using a digital dial gauge of 0.01mm accuracy. At the end of the test, both the ultimate strength and the failure mode were determined. The tests were performed at the concrete laboratory at the Engineering College of Wasit University.

4. TEST RESULTS

4.1 Failure Modes

The nine specimens displayed various failure modes depending on their geometric properties as illustrated in **Fig.4**. For all beams, the first flexural cracks were initiated at loads of 25-50kN in the middle third of the beams as listed in **Table 3**.

In the reference specimen R with 200mm thick, the flexural cracks were little and did not cause the failure. The failure mode was characterized by the shear connector failure at the left end.

The specimen REH150 of 150mm thickness failed in the flexural mode. In which, the first flexural crack was initiated at the mid-span. As the applied load increased, this crack widened and moved up towards the top of the beam as well as the formation of other flexural cracks in the middle third of the beam. At failure load, the concrete crushing and the buckling in the upper steel plate were noticed at the top of the beam.

The flexural-shear failure was observed in the specimen REH250 (250 mm thick), where a large number of flexural and diagonal shear cracks developed and spread throughout the length of the beam.

In the beam REH300 with the largest thickness of 300mm, two inclined shear cracks occurred in the right third of the beam and extended to the compression face resulting in shear failure of the specimen.

The shear connector diameter has a great effect on the failure modes, where the specimen RED12 (connector diameter=12mm) showed the similar failure mode to the reference beam R (connector diameter=10mm). However, increasing the connector diameter to 16mm as in the beam RED16, changed the failure mode from connector failure to the flexural failure.

Furthermore, the Shear connector failure was observed in the specimens RES200 and RES250, in which the connectors were spaced at 200 and 250mm, respectively. Finally, the failure mode was not influenced by enlarging the tensile plate thickness where specimen REP6 with the thicker plate (6mm) failed in the same mode of the reference beam R with the thinner plate (4mm).

4.2 Ultimate Strength

The ultimate loads for all specimens are summarized in **Table 3**. In order to investigate the effect of the beam thickness on the load carrying capacity, the test results of specimens REH150, R, REH250, and REH300 are compared in **Fig. 5**. They were constructed with the thickness of 150, 200, 250, and 300mm, respectively. The other properties of them were kept constant. The test results showed that a marginal decrease in the failure load of specimen REH150, about 5.3%, compared with the reference specimen R due to change the mode of failure from flexural to the shear connector failure. Beyond this, the failure load increased rapidly to about 57.9% and 63.2% for specimens of thickness 250 and 300mm with respect to the reference beam R, respectively, because they did not experience the connector failure. However, the shear failure of the beam REH300 caused a little increase in the ultimate strength about 3.3% compared with the specimen REH250 that failed in combined flexural-shear.

Increasing the diameter of connectors enlarged the contact area between the shear connectors and the cover plates and enhanced the bonding between these plates and the concrete core, this reflected at the failure load as plotted in **Fig.6**. The ultimate loads of specimens RED12 and RED16 were 31.6% and 42.1% greater than that of the reference specimen R, respectively.

Fig.7 shows the inverse relationship between the connectors spacing and the failure load. The ultimate loads dropped by about 31.6% and 68.4% when the spacing of connectors increased from 100 mm to 200 and 250mm, respectively.

Finally, enlarging the thickness of tensile steel plate (bottom plate) from 4 to 6mm did not influence the failure load, where both specimens R and REP6 collapsed at the load of 95kN since they showed the connector failure.

4.3 Load-Deflection Response

Since the deflection is a one term of the serviceability measurements, the deflections for all specimens are compared at a service load of the reference specimen R that is 66.5 kN representing 70% of its ultimate load.

Fig.8 shows the effect of the beam thickness on the load-deflection responses of SCS sandwich beams. It is clear that the stiffness of the beam raised with increasing the beam thickness due to the increment in the moment of inertia of the beams. At the load of 66.5 kN, the recorded deflection for the smallest specimen REH150 was 30.8% larger than that of the specimen R. Whereas the larger specimens REH250 and REH300 displayed major reductions in deflection relative to the reference beam at the service load, which were 60.6% and 63.6%, respectively.

The load-deflection behaviors for specimens, constructed with connector diameters of; 10mm (R), 12mm (RED12), and 16mm (RED16), are plotted in **Fig.9**. The three specimens showed an extremely identical response until occurring the yielding in the tensile steel plate (second point of deviation the curve of load-deflection). The flexural plateau is observed clearly in the response of specimen RED16 because it failed in flexure. However, small reductions of 10.1% and 15.7% were recorded in the deflections of specimens RED12 and RDE16 with regard of specimen R at the service load, respectively.

Since the connection between the steel plates and the shear connector, and the bonding between the plates and the concrete core were weakened significantly by increasing the spacing of the shear connectors more than 100mm, specimens RES200 and RES250 behaved like an unreinforced beam where their responses were approximately linear as shown in **Fig.10**. Both specimens failed at loads less than the service load of the specimen R.

Increase the tensile steel plate thickness enhanced the stiffness of the SCS sandwiched beam, especially after initiating of the first crack as illustrated in **Fig.11**. At the service load, the measured deflection of the specimen REP6 (plate thickness= 6mm) was 66.7% smaller than that of specimen R with the plate thickness of 4mm.

5. CONCLUSIONS

The main conclusions of the presented experimental program are summarized as follows:

1. All specimens with 200mm thick, except for specimen RED16 (connector diameter=16mm), experienced the connector failure. The specimen RED16 failed in the flexure. Also, the flexural failure was observed at specimen REH150 with thickness of 150mm .The specimen REH250 of beam thickness 250 mm displayed flexural-shear failure. Finally, the shear failure was exhibited by specimen REH300 (beam thickness=300mm).
2. The ultimate strength and the stiffness of the SCS sandwich beams improved by increasing their thickness. A considerable increase in the failure load, about 72.2%, was observed when the thickness of the beam increased to double.

3. Augmenting the diameter of the shear connector had a negligible effect on the beam stiffness before yielding of the tensile steel plate. After that, the stiffness enhanced with raising the connector diameter. However, increasing the connector diameter to 60% caused an increment in the ultimate load about 42.1%.
4. Increase the connector spacing influences adversely the ultimate capacity and stiffness of the SCS beams. A great reduction in the failure load reached to 68.4% was noticed as spacing increased by 150%.
5. Enlarging the thickness of the tensile plate thickness by 50% increased only the stiffness of the SCS beams, especially after occurring the first crack. The load carrying capacity did not affect.

REFERNCES

➢ American Specification for Test and Materials, *Making and Curing Concrete Test Specimens in the Laboratory*, C192-1995.

➢ Anandavalli, N., Rajasankkar, J., Parkash, A., and Sivaprasad, B., 2013, *Static Response of Steel-Concrete-Steel Sandwich Beam with Bi-Directionally Inclined Connections*, American Journal of Civil Engineering and Architecture, Vol.1, No.1, PP.15-20.

➢ Chu, M., Song, X., and Ge, H., 2013, *Structural Performance of Steel-Concrete-Steel Sandwich Composite Beams with Channel Steel Connectors*, 22nd Conference on Structural Mechanics in Reactor Technology, San Francisco, California, USA-August 18-23, Division X.

➢ Foundoukos, N., Xie, M., and Chapman, J.C., 2008, *Behavior and Design of Steel-Concrete-Steel Sandwich Construction, Advanced Steel Construction*, Vol.4, No.2, PP. 123-133.

➢ Liew, J. Y. R., and Sohel, K. M. A., 2009, *Lightweight Steel-Concrete-Steel Sandwich System with J-hook Connectors*, Engineering Structures, Vol.31, No.9, PP.1166-1178.

➢ Roberts, T. M., Edwards, D. N., and Narayanan, R., 1996, *Testing and Analysis of Steel-Concrete-Steel Sandwich Beams*, Journal of Construction Steel Research, Vol.38, No.3, PP.257-279.

➢ Shanmugam, N. E., and Kumar, G., 2005, *Behavior of Double Skin Composite Slab-an Experimental Study,Steel Structures*,Vol.5, No.5, PP.431-440.

➢ Wright, H. D., Oduyemi, T. O., and Evans, H. R., 1991, *The Design of Double Skin Composite Elements*, Journal of Constructional Steel Research, Vol.19, No.2, PP.111-132.

➢ Xie, M., Foundoukos, N., and Chapman, J.C., 2007, *Static Test on Steel-Concrete-Steel Sandwich Beams*, Journal of Construction Steel Research, Vol.63, No.5, PP.735-750.

Table 1. Details of test specimens.

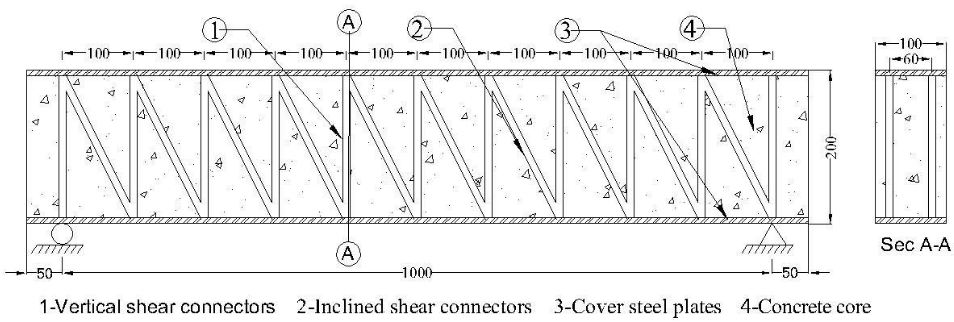
Specimen designation	Beam thickness (mm)	Bottom plate thickness (mm)	Connectors' diameter (mm)	Connectors' spacing (mm)	Concrete compressive strength (MPa)
R	200	4	10	100	34.37
REH150	150	4	10	100	34.37
REH250	250	4	10	100	34.37
REH300	300	4	10	100	33.96
REP6	200	6	10	100	33.96
RED12	200	4	12	100	33.96
RED16	200	4	16	100	33.88
RES200	200	4	10	200	33.88
RES250	200	4	10	250	33.88

Table 2. Properties of steel bars and cover plates.

<table>
<tr><td rowspan="4">Shear connectors</td><td>Nominal diameter (mm)</td><td>Yield stress (MPa)</td><td>Ultimate stress (MPa)</td></tr>
<tr><td>10</td><td>436</td><td>556</td></tr>
<tr><td>12</td><td>482</td><td>571</td></tr>
<tr><td>16</td><td>520</td><td>618</td></tr>
<tr><td rowspan="3">Cover plates</td><td>Plate thickness (mm)</td><td>Yield stress (MPa)</td><td>Ultimate stress (MPa)</td></tr>
<tr><td>4</td><td>245</td><td>369</td></tr>
<tr><td>6</td><td>251</td><td>384</td></tr>
</table>

Table 3. First cracking and ultimate loads of tested specimens.

Specimen designation	First cracking load (kN)	Ultimate load (kN)	Failure mode
R	30	95	Shear connector
REH150	25	90	Flexural
REH250	40	150	Flexural-shear
REH300	50	155	Shear
REP6	40	95	Shear connector
RED12	30	125	Shear connector
RED16	30	135	Flexural
RES200	25	65	Shear connector
RES250	25	30	Shear connector



All dimensions are in mm

Figure 1. Experimental details of reference specimen (R).

Figure 2. Typical cover plates welded with shear connectors.

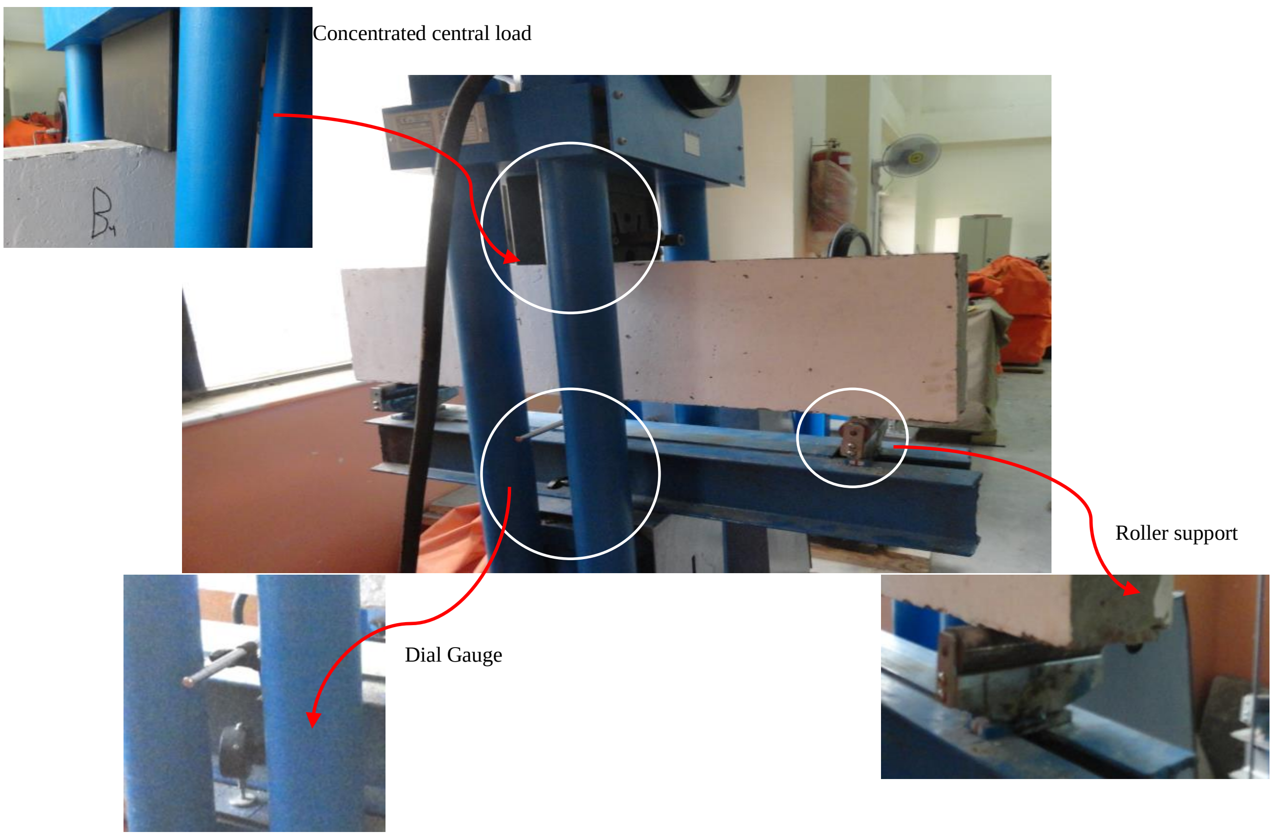
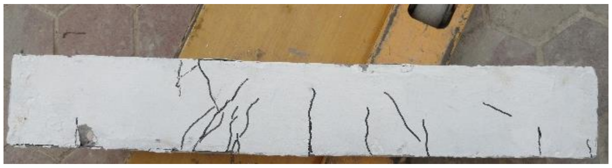


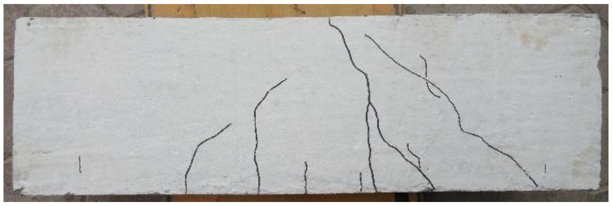
Figure 3. Photograph of the testing setup.



(a) Specimen REH150

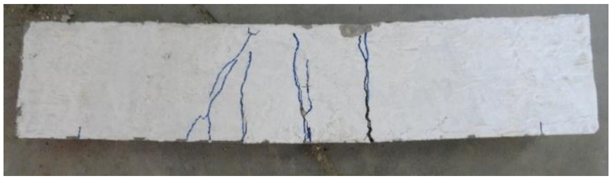
(b) Specimen R

(c) Specimen REH250



(d) Specimen REH300

(e) Specimen RED12



(f) Specimen RED16

(g) Specimen RES200

(h) Specimen RES250

(i) Specimen REP6

Figure 4. Crack patterns for nine tested specimens.

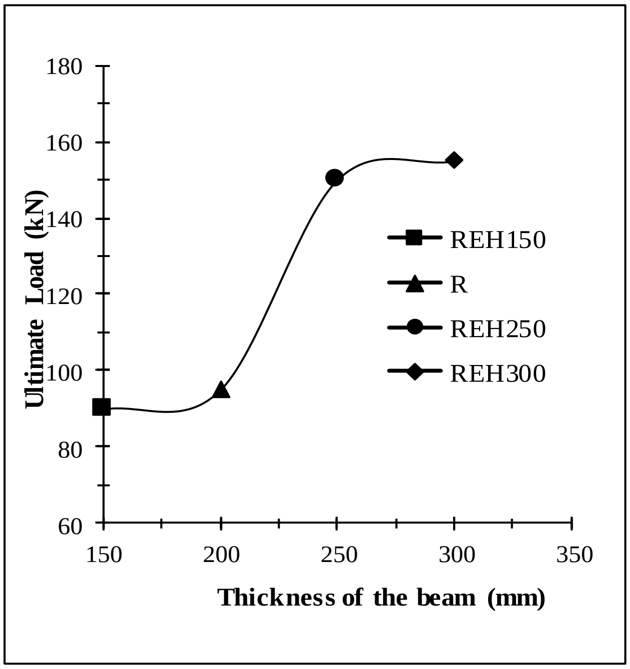


Figure 5. Ultimate load versus thickness of the beam.

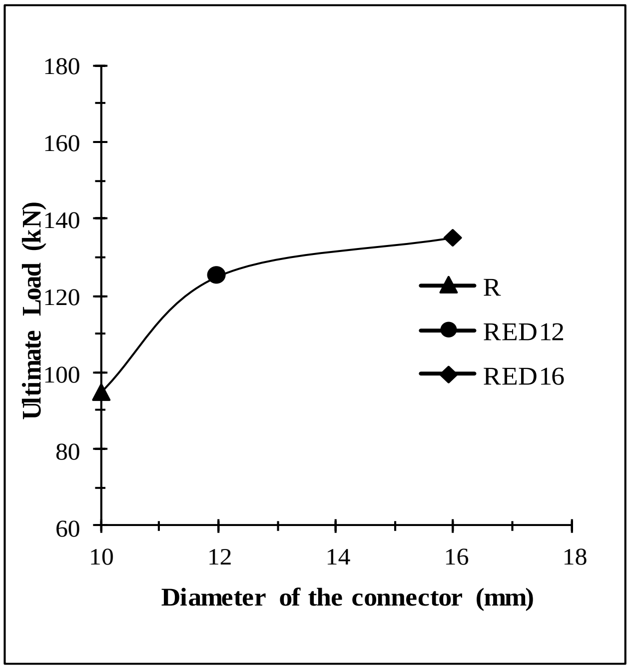


Figure 6. Ultimate load versus diameter of the connector.

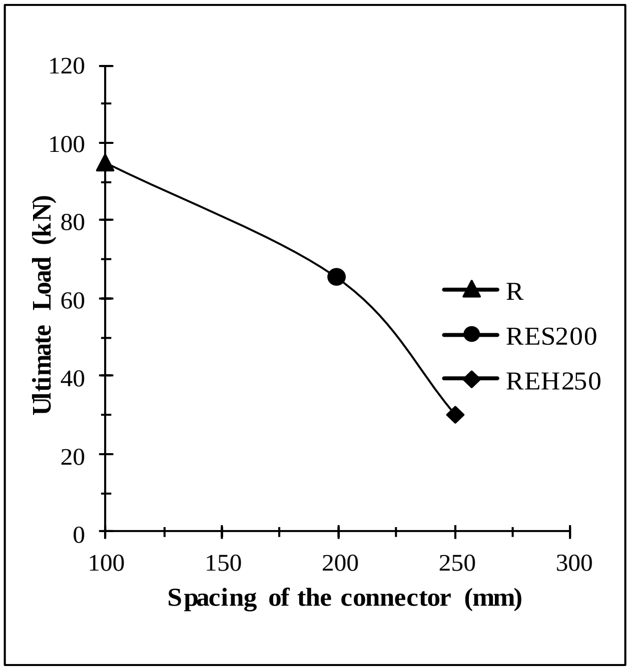


Figure 7. Ultimate load versus spacing of the connector.

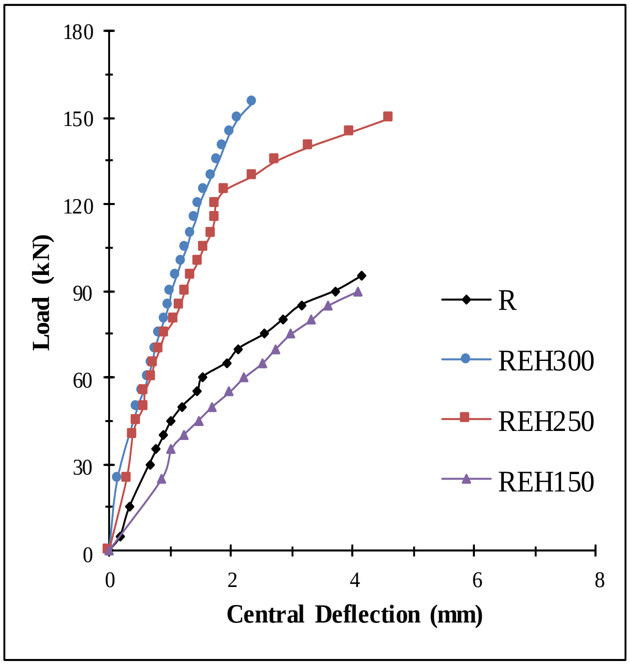


Figure 8. Load-deflection response of specimens with different beam thickness.

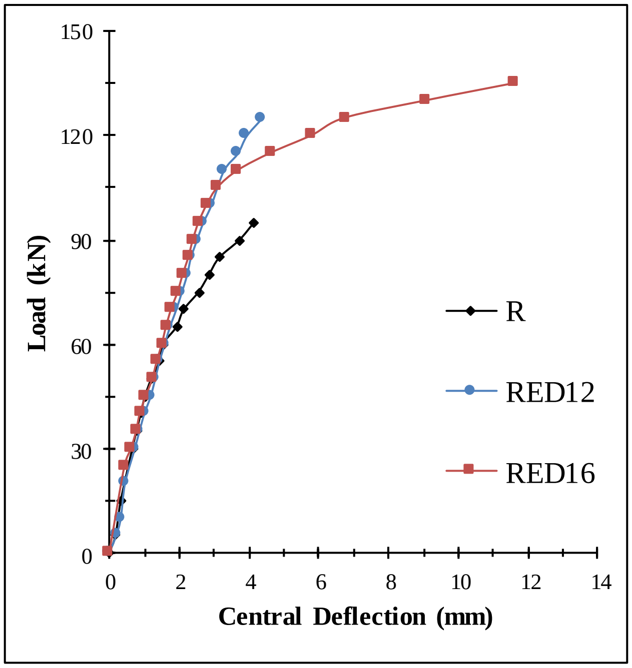


Figure 9. Load-deflection response of specimens with different connector diameter.

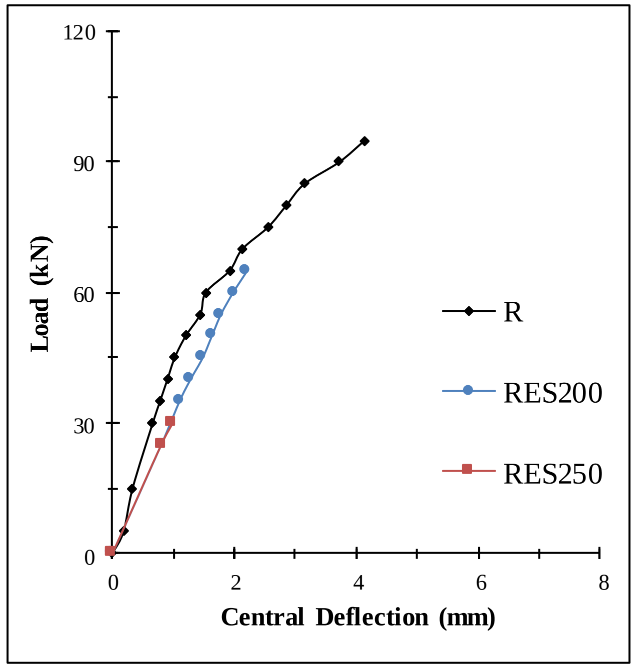


Figure 10. Load-deflection response of specimens with different connector spacing.

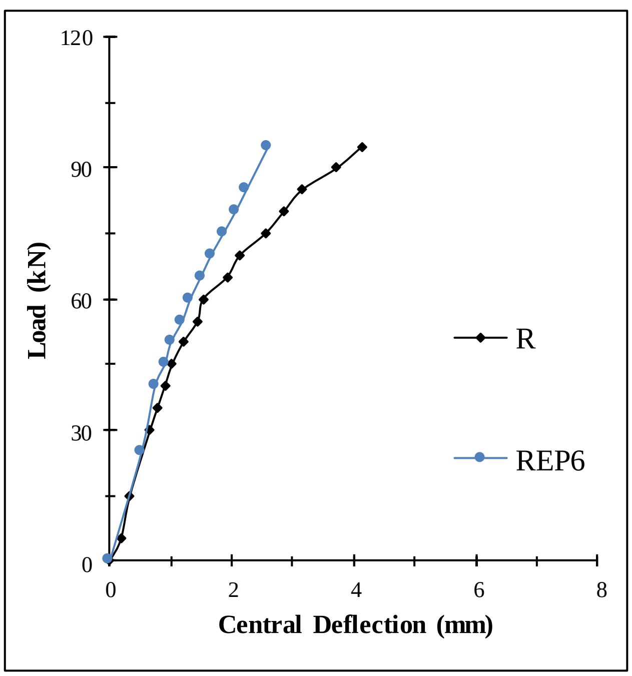


Figure 11. Load-deflection response of specimens with different tensile steel plate thickness.