Experimental Study of Steel-Concrete Composite Beams Subjected to Negative Bending by Point Loading at Mid-Span Considering Load – Deformation Characteristics

Sylvester O. Osuji[1], Oziegbe I. Oria-Usifo[2]

[1]Associate Professor, Department of Civil Engineering, University of Benin, Benin City, Nigeria
[2]M.E. Structural Engineering Student, Department of Civil Engineering, University of Benin, Benin City, Nigeria

Abstract: **Structural steel-concrete composite beams used for building and bridge constructions are commonly subjected to simultaneous actions of positive and negative bending. However, there are relatively few numbers of experimental tests on this aspect; hence more behavioural researches on composite beam under negative bending are needed. This paper investigates the behaviour of steel-concrete composite beam under negative bending considering the load-deflection, shear connection and concrete-steel interface slip characteristics. Four composite beam specimens were tested under negative bending by carefully turning them upside down before a single point loading was applied at the mid span to simulate the mechanical behaviour of continuous composite beams at the internal supports, while another remaining two beam specimens were subjected to positive bending in order to compare some distinct behaviours between composite beams subjected to negative and positive bending. Experimental results showed that the arrangement of shear connectors influence strength and deformation characteristics of composite beams. The reduction of the stiffness of the shear connectors in the cracked concrete influences the composite beam stiffness and the overall negative plastic moment capacity. However, it was noted that for a composite beam subjected to positive bending the positive plastic moment capacity is effectively utilised. Finite elements models created using SAP 2000 software were used to validate the experimental results trends and vice versa.**

Keywords: **Composite beams, Negative bending, Load-deflection, Slip, Interaction, Finite element, SAP2000.**

1. INTRODUCTION

In most countries, steel-concrete composite structures are commonly used in building and bridge constructions and the technology is gaining more acceptances in Nigeria [1, 2].

Steel and concrete composites results in structures that are both safe and economic. The utilisations of a reduced section and the speed in construction time for composite structures contribute to its economic advantages [2].

Although simply supported composite beams are commonly used in construction, yet continuous composite beams represent an efficient structural method in many structural systems, such as buildings and bridges, due to additional advantages associated with the favourable redistribution of internal forces across the member and the easier satisfaction of serviceability checks. However, the design and analysis of continuous composite beams is rather complicated due to their different behaviour in positive (or sagging) and negative (or hogging) moment regions [3].

Continuous composite beams in high-rise buildings and bridges are characteristically subjected to both positive and negative bending. Although, there are considerable experimental information on the structural behaviour of composite beams under

positive moment yet, experimental research information on the structural behaviour of steel concrete composite beams under negative moment are scarce and studies on the efficiency of shear connection when the slab is under tension are few [4]. There is also the need for first hand experimental results to crosscheck and validate results from finite element computer model analysis on composite beams subjected to bending and vice versa.

This present study is aimed to show the load-deflection, shear connection and interface slip characteristics of steel concrete composite beams subjected to negative bending not forgetting to distinguish some of these characteristics with composite beams subjected to positive bending. Finite element computer models to simulate the composite beam specimen test are being produced and validated by making comparison with the experimental result.

Specification I3.2 of The Specification for Structural Steel Buildings, 2005, by AISC (American Institute of Steel Construction) states that "The negative design flexural strength…shall be determined for the steel section alone…" hence, it is usually seen that the composite action of composite beams subjected to negative bending is commonly mitigated or completely ignored in designs. Therefore, the quest for optimal ways to distribute the forces in a composite section such that it may be useful in regions of negative moments has been on the rise [5, 6]. Consequently, it is imperative to provide more firsthand information for better design and optimal usage of composite beams in construction by engaging in more experimental study on composite beams under negative bending not neglecting those subjected to positive bending.

Based on some hypotheses and assumptions such as: composite beams with partial interaction have lesser ultimate moment of resistance than composite beam with full interaction; slip can occur at the concrete slab and steel beam interface; wide gap of interaction between shear connectors in the concrete member of the composite beam will cause a well distributed yielding of the concrete; the concrete between two subsequent cracks is able to bear tensile stresses, this research was done.

2. LITERATURE REVIEW

Vasdravellis et al (2012) studied the behaviour of steel–concrete composite beams subjected to the combined effects of negative bending and axial compression. Six full-scale tests were conducted on composite beams subjected to negative moment while compression was applied simultaneously. It was observed that only the beam subjected to pure negative bending had a ductile failure mode while the other five beams experienced local buckling [7].

Chen et al (2014) performed negative bending tests and theoretical analysis of eight steel–concrete composite beams specimens, six of which had corroded shear studs. They observed that bending capacity of the corroded beams decreased slightly with increasing corrosion ratio of the studs. The corroded beams also exhibited an obvious decrease in bending rigid stiffness and increase in the slip between the steel beam and the concrete slab [8].

Huie J. P (2009) examined shear connectors at regions of positive and negative moment in composite beams with is an attempt to verify current design methods for composite beams under positive and negative moments with the use of finite element modeling [6].

Pavlovic et al (2013) studied headed shear studs against high-strength bolts in prefabricated composite decks. Their experimental results showed that headed studs and bolted shear connectors have similar shear resistance [9].

Fabbrocino and Pecca (2000) compared the experimental load-deflection relationships with theoretical data calculated assuming linear elastic stress-strain relationships for the materials at serviceability conditions. In their analysis, experimental curves were compared to three straight lines related to the un-cracked stiffness, the fully cracked stiffness of the composite cross section and the stiffness of the steel profile. The load-deflection feature of a simply supported beam under a point load at mid span was described by Orie (2003). Deflection for non-composite and steel-concrete composite beams can be computed using equations 2.1 and 2.2 respectively:

$$\delta = \frac{PL^3}{48EI} \qquad (2.1) \qquad \delta_c = \frac{5PL^3}{384EI_c} \qquad (2.2)$$

where 'P_u' is the ultimate load, 'P' is any load on the load deflection curve, 'E' is the elastic modulus of concrete, 'I' is the moment of inertia of the beam and 'L' is the length of the beam [4, 10].

The deflection of composite beam is however affected by of the degree of shear connection. Though, Adekola in 1974 established in his research that there is a limiting degree of interaction beyond which deflection in not sensibly influenced [11, 12].

Following their tests on composite beams, Vasdravellis et al (2012) developed a detailed nonlinear finite element model, which was validated against the experimental results. The nonlinear spring element was adopted to connect a beam flange node with a slab node at the interface at the same positions where studs were welded to their specimen. Also, Dan et al (2010), performed numerical analysis to evaluate the stress state in composite joints using finite element method. In the first stage of their tests, the SAP 2000 numerical analysis program was used; they obtained the modeling by SHELL finite element type, for the structural steel of the joint. The constitutive elements: structural steel, reinforcements and concrete was taken into account [7, 13].

3. METHODOLOGY

The materials used such as the fine aggregate, coarse aggregate and cement used for the concrete slab are natural river sand, crushed rock granite and ordinary Portland cement respectively all sourced in Edo-state. The steel beam, shear connectors and reinforcements were also locally sourced. The production and testing of the composite beam specimens, concrete and steel material test samples were carried out in the civil engineering and mechanical engineering laboratories / workshops of the University of Benin, Benin-City.

3.1 Experimental Aspects:

3.1.1 Test Specimens Design Description:

Four steel-concrete composite beam specimens were designed for the testing under negative bending. Three of the beams were designed for full shear connection, while the remaining one was designed for partial shear connection. The nomenclatures of beams are Beam specimen A (BsA), Beam specimen B (BsB), Beam specimen C (BsC) and Beam specimen D (BsD). Two additional composite beam specimens namely: Beam specimen E (BsE) and Beam specimen F (BsF) where design for testing under positive bending involving full and partial connection for comparison with those specimens subjected to negative bending. Detailed designs of all six specimens having same cross-sections and lengths but different arrangements of shear connectors are shown in Fig. 1.

Each of the beams consist of a standard 140 x 76 x 10 universal (steel) beam and a 215mm x 85mm concrete deck slab cast to act in composite action with the steel beam. The typical slab was reinforced with 10 mm diameter steel bars. Four steel bars were provided for the longitudinal reinforcements, while, the transverse reinforcing bars were spaced at 130mm c/c all to ensure effective use of the slab in composite action with the steel beam, to resist vertical shear and limit shrinkage cracking. The total length of the beam between supports is 860mm. Shear transfer between the steel and concrete was provided by 10mm diameter x 65mm long steel structural bolts type shear connectors arranged differently for each specimen.

Beam specimen A (BsA) is characterised by 16 shear connectors on a single row, uniformly distributed along beam designed for full connection (studs spacing is 57mm c/c). Beam specimen B (BsB) is characterised by same number of shear connectors (for full connection) spaced @ 57mm c/c as design for specimen A above but in this case, the studs are concentrated at the ends on two rows, in order to realise a wide gap of concrete slab with same interaction force. Beam specimen C (BsC) is also characterised by same 16 shear connectors arranged such that six connectors are concentrated at each ends on two rows spaced @ 57mm c/c, while the four remaining connectors are place in two rows at the mid span of the beam also spaced @ 57mm c/c. This is a variation of Beam specimen B (BsB). Beam specimen D (BsD) is characterised by 6 shear studs spaced 172mm c/c on a single row, uniformly distributed along beam; this design ensured partial connection. Beam specimen E (BsE) is characterised by 18 shear connectors on a single row, uniformly distributed along beam designed for full connection and to be subjected to positive bending (studs spacing is 50mm c/c). Beam specimen F (BsF) is characterised by 8 shear studs spaced 123mm c/c on a single row, uniformly distributed along; this design ensured partial connection for beam specimen to be subjected to positive bending.

3.1.2 Fabrication of Test Specimens:

The steel beams were cut to 1150mm lengths, though the supports positions of the beams were located between 860mm span within which the shear connectors were distributed. Locations of the shear connectors on top of one flange of the steel beams specimens were marked out and the connectors spaced as appropriate for each specimen were then welded appropriately. The ply wood formworks around the steel beams were fabricated and coated with form release oil. The slab reinforcements were then placed as appropriate. All specimens were cast with the steel beam at the bottom of the formwork for the slab. The steel work and the composite beam formwork setup during casting can be seen in Fig. 2. The concrete was batched and mixed using a mix ratio for concrete grade 30 which is 1:1.5:3 with a water/cement ratio of

0.54; a slump of 55mm was achieved. The concrete slabs were then carefully cast and the specimens were compacted using a vibrating table. Curing was allowed to continue for 7 and 28 days as needed.

3.1.3 Material Properties Tests:

Material tests were performed prior to the tests in order to determine the actual strength and stiffness of concrete and steel. All materials were order from the same set; tensile test were carried out on coupons extracted from the steel web, flange and the reinforcing bars in accordance with ASTM A370 -11 [14].

The procedure stipulated in BS 1881 - 108:1983 was used to produce each set of concrete test cubes [15]. As an index test, the particle size distribution of the aggregates used was analysed. While in accordance with the British standard 1881: Part 116: 1983 the compressive strength test on the cubes was carried out after their curing for 7 and 28 days [16]. The resulting values of the material tests and specified characteristic strengths of each material are presented in Table 1.

Table 1   Material Tests Results

	Material strength (N/mm^2)		
	Design	Measured yield	Measured ultimate
Concrete compression (7 days)	n/a	n/a	32.5
Concrete compression (28 days)	30	n/a	39.7
Steel web	235	234	357
Steel flange	235	235	358
Reinforcements bars	475	475	680

3.1.4 Test Setup, Instrumentation and Loading Procedure:

In order to simulate the mechanical behaviour of continuous composite beams at the internal supports, the first four composite beam specimens (BsA, BsB, BsC and BsD) were tested under negative bending by carefully turning them upside down before testing each of them in simply supports configuration. Thereafter, the remaining two beam specimens (BsE and BsF) were tested under positive bending by uprightly subjecting their concrete flanges to the single point mid-span loading.

The test was carried out using a universal testing machine of 600kN

capacity that applies a single force at the mid-span of the beam specimens. Each composite beam specimens were simply supported at its ends, with span of 860mm between the centreline of the supports as shown in the test setup in Fig. 1 and Fig. 3.

Dial gauges were used to monitor global quantities such as displacements and relative slips. A dial gauge was placed at the mid span to measure the mid-span deflection. Two other dial gauges were also located at the end of each beam to measure the end slips i.e. the relative movement between the concrete slab and the steel beam at both ends of each composite beams as shown in Fig. 4.

The load was gradually applied and monitored and recorded using a load cell against the slip at ends and deflections at mid-span recorded using three dial gauges.

Resulting moments on each test beam were also being computed taking into account the equilibrium of the external forces acting on it. The following equation was used to calculate the bending moment:

$$M = \frac{P_v L}{4} + M_{sw} \qquad (3.1)$$

where P_V is the vertical force applied at the centre of the beam, M_{sw} is the moment due to the beam's self weight (about 0.511kNm) and L is the span length of the beam.

3.2   Theoretical Aspects:

3.2.1 Linear Elastic Load- Deflection Computations:

The experimental load-deflection relationships were compared with theoretical data. Theoretical deflections, under appropriate loading interval, were calculated for the un-cracked stiffness, full cracked stiffness of the composite beam and for the steel beam member alone following procedures stipulated in clause 6.1.2 of BS 5950: Part 3: Section 3.1:1990 and simple elastic theory for deflections by assuming linear stress- strain relationships for the materials [11, 17]. Deflections were calculated using the moment of inertia I_c for un-cracked, cracked or steel section in equations 2.2 or 2.1 as the case may be.

3.3 Finite Element Modelling Method:

The model was constructed in order to reproduce the tests on the composite beams. The SAP2000 finite element package was used to carry out the modelling. The applied load was iterated step by step using the Newton-Raphson method. The concrete slab and steel I-beam were modelled using shell elements (SHELL43) with 4-nodes. The element has six degrees of freedom at each node: translations in the nodal x, y, and z directions and rotations about the nodal x, y, and z axes. The deformation shapes are linear in both in-plane directions. The element has plasticity, creep, stress stiffening, large deflection, and large strain capabilities. Nonlinear springs and Link8 truss element was used to model the shear connectors and the steel reinforcement. These elements are capable of plastic deformation.
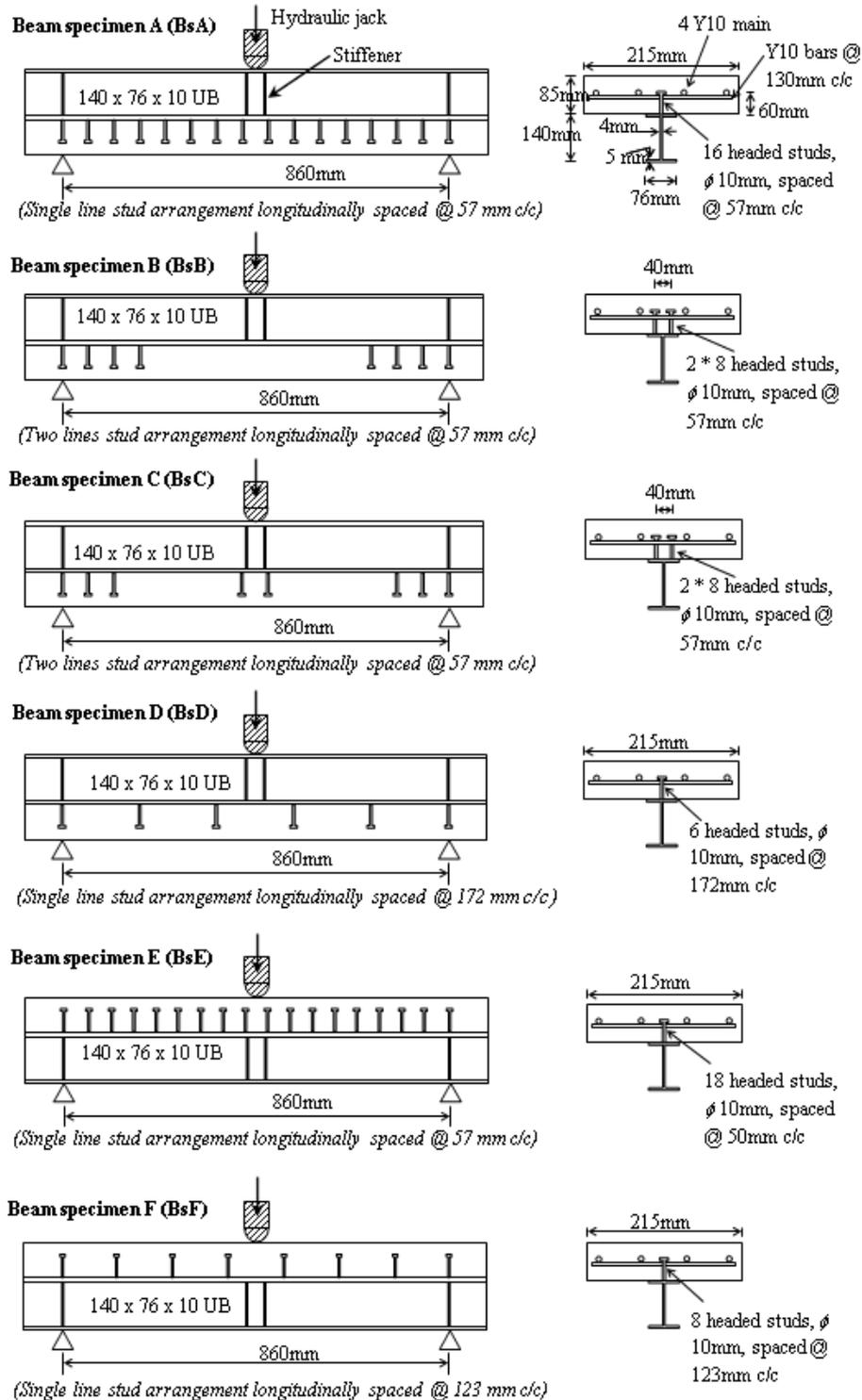

Fig. 1 Loading pattern, arrangements of connectors and cross sections of the six composite beam specimens

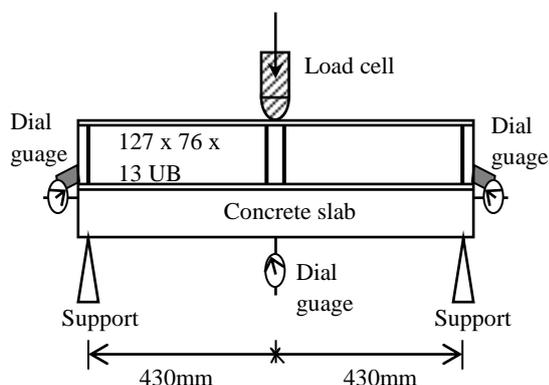**Fig. 2 Formwork, reinforcement, studs welded to steel beam and a layer of concrete being vibrated during casting**

Fig. 3 Test setup for negative bending

Fig. 4 One of the specimens under negative bending test

4. RESULTS AND ANALYSES OF RESULTS

4.1 Experimental Tests Results:

4.1.1 General Observations and Failure Mode:

The major failure mode of the composite beam specimens was the local buckling of the compressive flange, although this was met at the final stages of loading and after the beam has reached its plastic moment capacity. For specimens under the negative bending, local buckling fully developed after the yielding of the reinforcement.

Furthermore several cracks developed at the bottom slab under tension. The distribution, location and extent of these cracks were observed to be dependent on the arrangement of the shear connectors.

For instance, for beam specimen A (BsA), cracking of the concrete slab was first noticeably seen as a single crack at the mid span around a load of about 90kN. The crack continued to enlarge until the ultimate load was reached. For the case of beam specimen B (BsB), the cracking was more evenly distributed as several cracks with smaller widths across the mid span region of the concrete slab. The possible technical explanation for this phenomenon is that the distribution of connectors along the beam influences the zone subjected to the maximum interaction force. The distribution of shear connectors that has been chosen for the beam Type BsB and BsC leads to a relatively higher length of the concrete slab subjected to the maximum interaction level than in the beam Type BsA. The effect of this stress condition is a greater spreading of yielding in the slab. Beam specimen D (BsD) experienced shear connector failure while, for the beam specimens (BsE and BsF) subjected to positive bending, the predominant mode of failure was flexural local buckling failure. The failure regions of some of the specimen are shown in Fig. 5.

(a) Shear connector failure of BsD

(b) Local buckling of the composite beams

Fig. 5 Typical failure modes of the specimens

4.1.2 Load-Deflection Curve:

The experimental vertical load–deflection curves for the specimens BsA to BsF are shown in Figs. 6(a) and 6(b). From the curves, it is observed that the preliminary behaviour of the curves showed approximate linear trends up to the yield points where the various beams stiffness reduced. From figure 6 (a) it is seen that the elastic limits and relative post-elastic behaviour of the load–deflection curves for each of the specimens subjected to negative bending seems to be dependent on the arrangements of the shear connectors.

Beam specimen A (BsA) experienced noticeable yielding at a load of about 90kN, indicating a relatively higher upper yield stress and also it experienced an ultimate load capacity of 119kN. This result is due to the full shear interaction of BsA and also probably due to the stronger restraint of the shear connectors to the steel profile when the uniform distribution along the beam is realized

Although Beam specimen B (BsB) had a reduced upper yield and ultimate load capacity (when compared with BsA), the curve shows an hardening behaviour which increases with increasing load. A greater spreading of yielding in the slab caused by the wider gap of interaction explained the cause of this hardening behaviour phenomenon. Early local buckling of the concrete flange possibly caused by the absence of devices at zones across the mid span BsB which leads to a greater sensitivity to torsional effects for the beam specimen can explain the cause of the reduced upper yield load and ultimate load of BsB.

The curve of BsC is almost perfectly plastic in the post-elastic zone. It has a relatively sharper yield point, though with almost same yield load as BsB which corroborates the greater sensitivity to torsional effects on BsB.

Beam specimen D (BsD) subjected to partial interaction force is shown to be very ductile; though it has a reduced upper yield load of about 50kN, yet it has an ultimate load of 111kN.

In Fig. 6 (b), behaviors of the load–deflection curves for each of the specimens BsE and BsF subjected to positive bending is dependent on the degree of shear interaction. A comparison shows that under full shear connection, the ultimate load of BsE is 75% higher than that of BsA and it is also seen that when subjected to partial connection BsF has an ultimate load which is 51% higher than that of BsD. Beam specimen E (BsE) behaves more perfectly plastic with a higher yield load and ultimate load than Beam specimen F (BsF) which seems to be more ductile.

4.1.3 Normalised Moment-Deflection Behaviour:

Fig. 7 shows plots of the normalised moment versus midspan deflection response of beam specimens BsA, BsD, BsE and BsF.

BsA sustained a maximum force of 119kN, which is equivalent to the achieved moment of 26.01kNm and a maximum midspan deflection equal to 6.5mm. Its normalised moment at this load is 0.9844 as shown in Fig 7, this implies that the experimental moment of resistance of BsA is 98.44% of its resistance predicted by the negative plastic moment analysis. This is a good result, which helps in validating the experimental test, in as much as the full strength capacity of the beam specimen was approximately realised. The 1.56% loss in strength is probably caused by the little reduction in the beam stiffness influenced by the connection deformability after significant the cracking of the concrete slab.

Interestingly, the beam specimen BsE subjected to positive bending Fig 7 shows that the experimental resistance is even 44.36% higher than the resistance predicted by the positive plastic moment analysis. This can attributed to the considerable high strain hardening of steel which is normal when steel is subjected to tension.
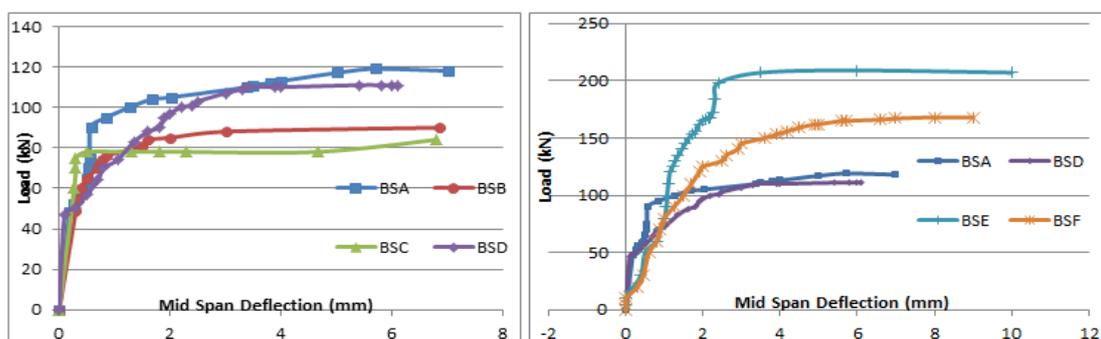
(a) Beam specimens under to negative bending   (b) Comparing positive bending to negative bending

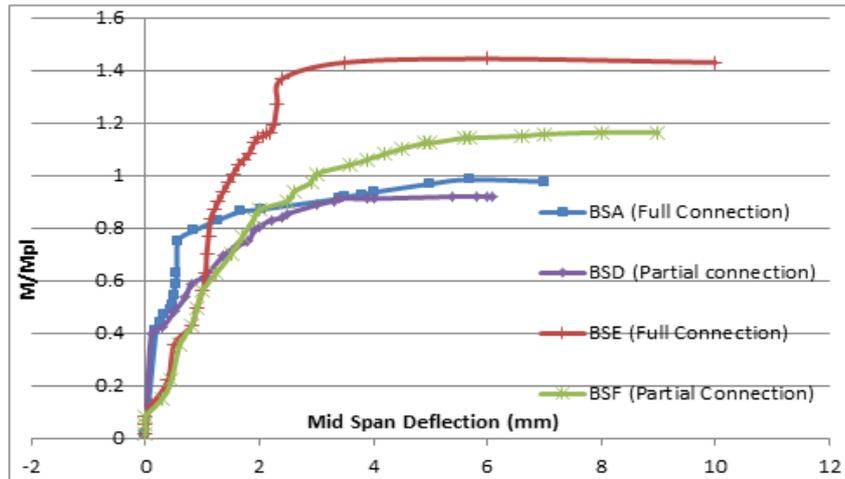**Fig. 6 Experimental Load deflection Curves**

Fig. 7 Normalised moment versus midspan deflection responses of BsA, BsD, BsE and BsF

4.1.4 Load - End Slip Behaviour:

Load - end slip curves for all the tested beam specimens are shown in Fig.8. Observations shows that the local measures of slips at the slab-profile interface are largely influenced by the connecting devices design.

Beam specimen A (BsA) experienced very small interface slip, not exceeding 0.0177 mm which occurred in the final stage of loading, indicating that the shear connection performed well.

Up to the yield point, the load – end slip behaviour of BsB is seen to be similar to BsA in Fig.8, thereafter progressively higher non linear slips set in relation to increasing loads. It is easy to recognize that even if the beams Type BsA and BsB are characterized by the same interaction degree, slips at the interface are quite different beyond the respective specimen-yielding load.

However, when the beam Type BsC is considered, slips are by far greater than in the other beams, regardless of the four shear connectors provided at mid span (which is expected to complete the full interaction design). Interestingly, these connectors placed at the midspan, far apart from the others at the end of the beam specimen is probably the cause of the greater slips experienced; reason being that the uneven distribution of the shear connectors on one half of the beam, posing more shear stress on the fewer shear connectors remaining at the support end of the beam where shear stresses are expected to be predominant. The ultimate beam failure occurred at a slip of about 0.33 mm.

The partial shear connection of beam specimen D is confirmed by the curve BsD in Fig.8. Also the similar trends of BsD and BsC during the early loading stages corroborates, that partial interaction instead of full interaction occurred for BsC. When compared to BsA and BsB, BsD has very larger slips even for low loads. Higher level stress is clearly present in the end connector of BsD, which even caused failure of its end shear connector at a slip of about 0.1mm as seen in Fig.5(a).
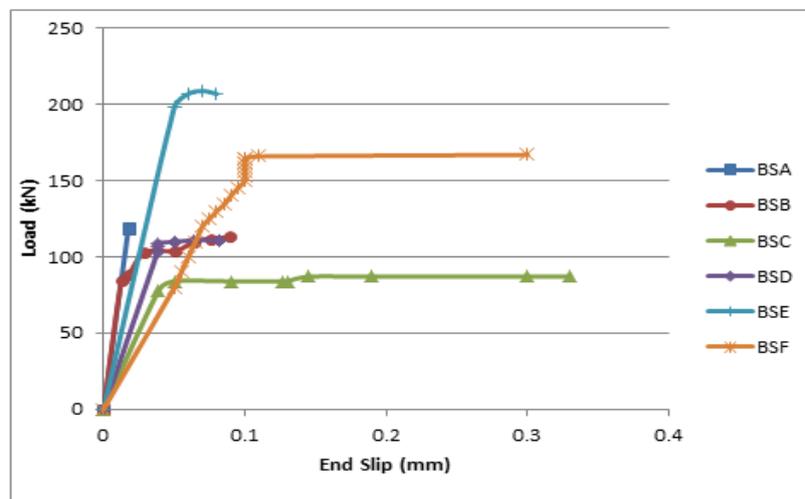

Fig. 8 Load Versus End Slip for each beam specimen

4.2 Experimental – Theoretical Comparison:

To validate the experimental load-deflection curves, an experimental-theoretical load-deflection comparison was performed as seen in Fig.9.

In the linear range of BsA, BsB, BsC and BsD in Fig.9, the theoretical and experimental results match quite well, in which case, the experimental load-deflection curves of all these beam specimens under negative bending fall around the region of theoretical un-cracked and cracked stiffness. This buttresses the increase stiffness of the composite beam specimens, which however slowing or speedily decreases as the case may be, after yielding commences.

Considering the theoretical un-cracked stiffness curve to inhibit the upper bound stiffness while the steel member stiffness the lower bound stiffness, the rate at which the stiffness of the experimental beam specimens decreases in the post-elastic stage, is seen to dependent on the degree of shear connection and the arrangement of the shear connector.

From Fig.9, BsA has highest stiffness after yielding while BsD seems to exhibit a lower stiffness after yielding. This due to the fact that BsA which has its shear connectors uniformly arranged experienced full shear connection, while BsD experienced a good partial interaction. The higher stiffness of the experimental beam curves than the stiffness of the steel member alone confirms the contribution of the concrete slab in tension.
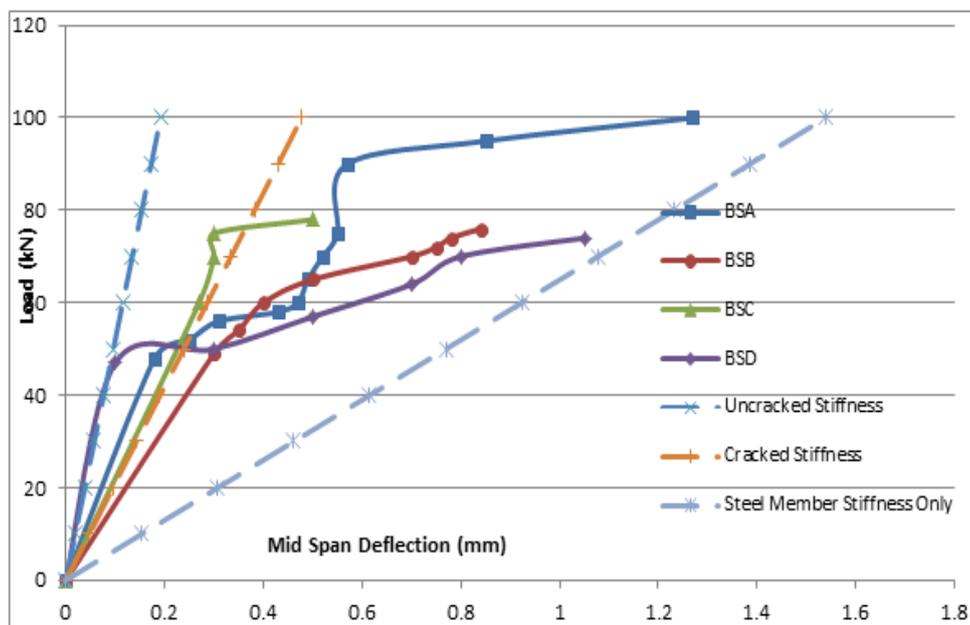

Fig. 9 Experimental-theoretical load-deflection comparison

4.3   Comparison between Finite Element Results and Experimental Results:

Load-deflection plots and load-slip plots obtained from linear static finite element analysis (using SAP2000) for specimen BsA and BsB were compared with the corresponding ones obtained from the experimental results as shown in Figs 10 and 11 respectively. Since linear models resulted, the elastic/ approximate portions of the experimental curves were used in the comparison.

From Fig.10, it can be seen that the initial stiffness of the experimental curves are slightly higher. However, the beam loses that stiffness as the load increases and both experimental and FEM curves became alike for the specimens considered.

In general for the load-deflection behaviour, a good agreement between results of experimental work and finite element results were obtained.

The load-slip comparison curves of Fig.11 show the finite element model for BsB to exhibit a stiffer resistance to slip when compared to that of BsA. Although, this trend is vividly shown in the finite element models than the experimental curves yet, to the extent of the elastic range of BsB it still exist. However, BsA experimental curve reached the highest resistance to slip due to its full shear connection resulting in a larger load capacity, and earlier failure of BsB.
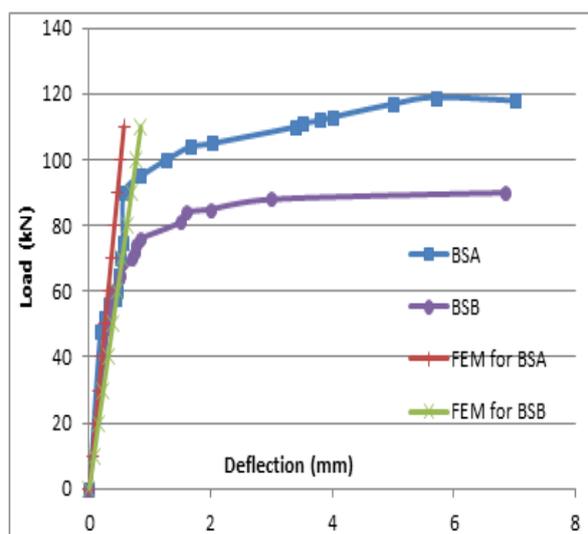
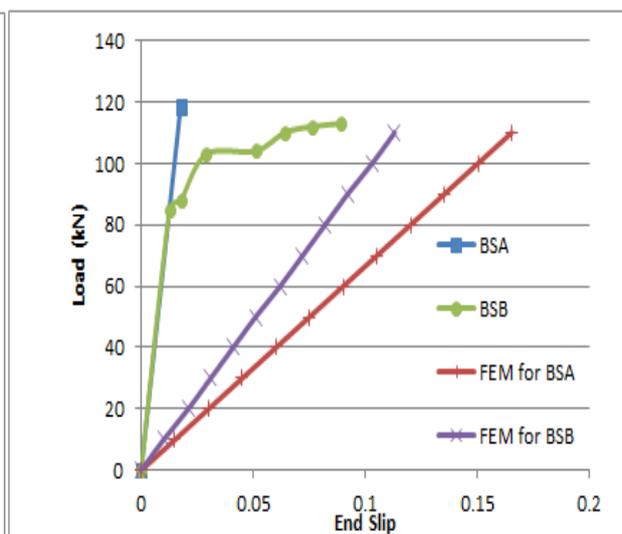| **Fig. 10 Experimental-finite element model load-deflection**	**Fig. 11 Experimental-finite element model load-slip**
Comparison	**comparison**

5. CONCLUSION

This paper mainly described an experimental study on the behaviour of steel-concrete composite beam subjected to negative bending by point loading on the mid-span. Results from the study were corroborated with results from theoretical approaches and finite element modelling. The main conclusions drawn from the study are the following:

- In spite of the fact that the beams specimens A, B, and C were technically designed for full shear connection, their load capacity, deflections and steel-concrete interface slips response was dependent on the order in which the shear connectors were arranged.

- The failure of a beam designed for partial shear connection such as beam specimen D depends strictly on the failure the shear connectors, and as such the slip capacity of the connectors evidently affect behavioural patterns of beams subjected to partial shear connection.

- The yield in the concrete slab at the internal support of a continuous composite beam can be more distributed when the shear connectors are installed far from the support region; however care should be taken not to affect the overall design strength of the beam.

- Concrete slab in tension of a negatively bending composite beam to some reasonably extent (especially before slab yielding) contributes to its strength and stiffness also.

- Composite beams subjected to positive bending more easily attain their design plastic moment capacity than those subjected to negative bending. The reduction of the stiffness of the shear connectors in a cracked concrete, influences the composite beam stiffness and the overall negative plastic moment capacity of beam under negative bending.

Finally, this paper provided the experimental results from a simply supported composite beam model subjected to negative bending, intended to simulate an internal support of a continuous composite beam. The experimental results have shown that the arrangement of shear connectors influence strength and deformation characteristics of composite beams. However, more researches in the future involving a full scale experimental test of continuous composite beam, using more sophisticated instrumentations and testing conditions will be invaluable for better behavioural study of this aspect.

REFERENCES

[1] Tan, E. L. and Uy, B., (2009). *Experimental Study on Straight Composite Beams Subjected to Combined Flexure and Torsion.* Journal of Construction Steel Research, pp 784.

[2] Adekiigbe, A., Bakare, O. T., Awofadeju, A. S. (2014). *Assessment of Steel Framed Buildings in Nigeria- A case Study of Osun School Infrastructure Revamp(O-Schools).* International Journal of Engineering Research and Technology, Vol. 3, pp 1405-1408.

[3] Bavan, M., Baharom, S., Mutalib, A. A., Osman, S. A. (2013). *Numerical Prediction of composite Beam Subjected to Combined Negative Bending and Axial Tension,* Journal of Engineering Science and Technology, Vol. 8, pp 428.

[4] Fabbrocinoa, G. and Pecca, M. (2000). *Experimental tests on steel-concrete composite beams under negative bending.* Research Gate publications, https://www.researchgate.net/publication/231581441, viewed on 03-05-2016, pp 1.

[5] ANSI/AISC 360-05. (2005). *Specification for Structural Steel Buildings.* Chicago, American Institute of Steel Construction, Inc., pp 84, 119.

[6] Huie, J. P. (2009). *Analysis of Shear Connectors at Regions of Positive and Negative Moment in Composite Beams.* An M.Sc Thesis in Civil Engineering. Birmingham, University of Alabama, pp 3, 4.

[7] Vasdravellis, G., Uy, B., Tan, E. L. and Kirkland, B. (2012). *The Effects of Axial Tension on the Hogging–Moment Regions of Composite Beams.* Journal of Construction Steel Research, pp 20 – 33.

[8] Chen, J., Jiang, A., and Jin, W. (2014). *Behavior of Steel-Concrete Composite Beams with Corroded Shear Studs Under Negative Bending Moment.* 4th International Conference on the Durability of Concrete Structures. USA, Purdue University, West Lafayette, pp 20 – 33 .

[9] Pavlovic, M., Spremic, M., Markovic, Z. and Veljkovic, M. (2013). *Headed Shear Studs Versus High-Strength Bolts in Prefabricated Composite Decks.* Research Gate publications, pp 1, https://www.researchgate.net/publication/ 259884259, viewed on 06 – 06 – 2016.

[10] Orie, O., U. (2003). *Flexural Characteristics of Reinforced Hollow-Cored Concrete Beams Point-Loaded at Mid-Span.* An M.Eng Thesis in Civil Engineering. Benin City, Nigeria, University of Benin, pp 22, 23.

[11] Johnson, R. P. (2004). *Composite Structures of Steel and Concrete Beams, slabs, columns and frames for buildings.* United Kingdom, Blackwell Publishing Ltd, pp 2, 22, 24, 35.

[12] Adekola, A. O. (1974). *The Dependence of Shear Lag on Partial Interaction in Composite Beams.* International Journal of Solids and Structures, Elsevier Ltd, Vol. 10(4), pp 389-400.

[13] Dan, D. Stoian, V. and Gyorgy, T. N. (2010). *Theoretical and Experimental Studies Regarding Steel-Concrete Composite Joints.* Department of Civil Engineering. Politehnica University of Timişoara.

[14] ASTM A370-11. (2011). *Standard Test Methods and Definitions for mechanical Testing of Steel Products.* West Conshohocken, PA, ASTM International.

[15] BS 1881 - 108: 1983. *Testing Concrete- Methods for Making Test Cubes from Fresh Concrete.* London, British Standards Institution.

[16] BS 1881 - 116: 1983. *Testing Concrete- Methods for Determination of Compressive Strength of Concrete Cubes.* London, British Standards Institution.

[17] BS 5950- 3- 3.1: 1990. *Structural use of Steelwork in building- Code of practice for design of simple and continuous composite beams.* London, British Standards Institution.