Al-Nahrain Journal for Engineering Sciences (NJES) Vol.21 No.3, 2018 pp.393-404

Experimental Investigation on Behavior of Composite Open Web Steel Joists

Ali Farhan Hadeed
Building and Construction Technology Eng. Dep.
Al-Esra'a University College, Baghdad, IRAQ
engalifarhan@yahoo.com

Laith Khalid Al-Hadithy
Civil Engineering Dep.
Al-Nahrain University, Baghdad, IRAQ
lthadithy@yahoo.com

Riyadh J. Aziz
Architecture Dep.
Al- Esra'a University College, Baghdad, IRAQ
riyadhaziz48@yahoo.com

Received: *26-Oct.-2017* ***Revised:*** *03-Jan.-2018* ***Accepted:*** *27-June-2018*
http://doi.org/10.29194/NJES.21030393

Abstract

The composite opened web steel joist supported floor systems have been common for many years. It is economic and has light weight and can embed the electrical conduit, ductwork and piping, eliminating the need for these to pass under the member, consequently eliminate the height between floors. In order to study the joist strength capacity under the various conditions, it had been fabricated seven joists composed of the steel and concrete slab connected to the top chord by shear connectors (headed studs). These joist have 2820 mm length c/c of the supports and 235 mm overall depth. In the present study, six variable parameters are adopted (Studs distribution, Degree of shear connection, Degree of the web inclination, Shape of the web, Density of concrete for slab and length of the shear connector). The test results exhibited that minimum strength capacity was 160kN for light weight joist and maximum capacity was 225kN for joist of long shear connectors at failure. The results were compared by ultimate flexural model by Azmi.

Keywords: Composite, Open Web, Steel Joists, Analytical Modeling

Notations

$F_{y=}$ Bottom chord yield stress taken from tensile coupon results, MPa
A_s = Cross sectional area of bottom chord, mm^2
A_{sc}= Cross –sectional area of stud shear connector, mm^2
b_a= Average rib width, in, Lawson study
D_p= Profile height, in, Lawson study
d= Shear stud diameter, mm, Eurocode equation
E_c= Elastic modulus of concrete, MPa, AISC equation
E_{cm}= Elastic modulus of concrete, MPa, Eurocode equation
f'_c= Cylinder compressive strength of the concrete, MPa
f_{ck}= Characteristic cylinder compressive strength of the concrete, MPa, Euro code equation
f_u= Ultimate stud strength, Mpa , Euro code equation
F_u= Ultimate stud strength, MPa, AISC equation
$F_{y=}$ Bottom chord yield stress taken from tensile coupon results, MPa
H= Height of shear connector, in, Lawson study
$h_{r\,=}$ Nominal rib height, mm, AISC equation
$H_{r=}$ Length or height of stud, mm, AISC equation
N= Number of shear connectors per rip, Lawson study
$N_{r=}$ Number of stud connectors in one rib, AISC equation
Q= Shear capacity of single shear stud, kN, Eurocode equation
Q_n= Nominal stud shear resistance, kN AISC equation
r_p= Stud reduction factor for metal deck, Lawson study
Rp=Stud reduction factor for metal deck, AISC equation
T_y=Bottom chord yield force taken from tensile coupon results, KN
V'= Total horizontal shear force
w_r= Average width of concrete rib, mm, AISC equation

Abbreviations

COWSJ=Composite opened web steel joist
CSJs=Composite steel joists

1 Composite Structure

In the composite structure, concrete is efficient in compression and steel in tension. Steel components are relatively thin and prone to buckling, concrete can restrain these against buckling, and concrete also gives protection against corrosion and provides thermal insulation at high temperature, finally steel brings ductility into the structure [1, 2].

The term composite joist (CJ Series) refers to opened web, parallel chord, load carrying members utilizing hot-rolled or cold-formed steel, including cold-formed steel whose yield strength has been attained by cold working, suitable for the direct support of floors of one-way floor or roof systems[4]. Full connection using shear connectors between the joist top chord and overlying concrete slab allows the steel joist and concrete slab to act together as an integral unit after the concrete has adequately been cured [3,4]. Its main advantage is the more efficient and stiffer composite design makes it possible to support a given load with a shallower joist [4]. The major limitation according to the Steel Joist Institute (SJI) [4] is, the span of a standard composite joist shall be from 12 to 30 times the depth of the steel joist.

The distinguishing feature of the composite joist system as compared to a non composite system is the presence of shear connectors. The quantity of shear connectors provided is denoted by **"shear connectors per half – span"** or **"shear connectors per shear span"**, the total number of connectors per member being twice this amount. When used in this study the terms" **under connected**" and **"over connected** " will refer to how the shear connection force compares to the yield force of the primary tension resisting component [5].

2 Experimental Work

2.1 Experimental Program

Degree of connection; either over or under shear connection, based on $(\sum Q_n / T_y)$, which is less than one for under connection or more than one for over connection. Individual connector shear strength (Q_n) must be computed according to the steel manual AISC2005[6].The number of shear connectors at balance state can be estimated by dividing the total horizontal shear force (V') by (Q_n).The total horizontal shear force (V') is the lowest value of the limit states of concrete crushing$(0.85f'_c\ A_c)$ and tensile yielding of the bottom chord $(A_s\ F_y = T_y)$ according to steel manual,AISC,2005 [6]. (T_y) is the bottom chord yield force taken from laboratory test result. In this study 6.2 studs need for balancing, so ten studs were used for over connection (62%) and five for under connection (19%). Six main variables were adopted and seven steel joists were constructed to investigate the effects of each variable. ***The first*** joist was considered as the *reference one*, with slab of normal weight concrete(NWC) ,its strength 27 MPa, steel angle web with 45° inclination, over shear connection with short height (30 mm) ,headed studs uniformly distributed ***The second*** had *non-uniform stud distribution*. *Under Shear connection* was implemented in the ***third joist***, *while 34° inclination of web angle* was the variable of the ***fourth one***. ***The fifth*** was distinguished by variable *Ø 25mm rounded bar web member*. The variable of *Light weight slab concrete* (LWC) of 26.5MPa was used in manufacturing the ***sixth composite joist***. ***The seventh*** had *long headed stud* (50 mm) .all previous details are given in table1.

Table1 Details of tested joists with their parameters

Joist Designation	**Slab conc. Type**	**Web details**		**Shear connector (headed stud)**			
		Shape	Inclination	Length	Mode of distribution	Degree of connection	Studs No. Per half span
CSJ-1(B1)	NWC	Double Angle	45°	(short)	Uniformly	Over connection	10
CSJ-2(B2)	NWC	Double Angle	45°	(short)	Non-Uniformly*	Over connection	10
CSJ-3(B3)	NWC	Double Angle	45°	(short)	Uniformly	Under Connection*	5
CSJ-4(B4)	NWC	Double Angle	34°*	(short)	Uniformly	Over connection	10
CSJ-5(B5)	NWC	Rounded Bar *	45°	(short)	Uniformly	Over connection	10
CSJ-6(B6)	LWC*	Double Angle	45°	(short)	Uniformly	Over connection	10
CSJ-7(B7)	NWC	Double Angle	45°	(long)*	Uniformly	Over connection	10

2.2 Push out Test

Three push out test specimens including 4 studs, Φ10mm for each were fabricated according to the BS5400-part 5 [7]. Each of them was set up under the hydraulic jack of the 3000 kN capacity MFL machine. A dial gauge of 0.01mm accuracy was fixed at web of steel I - section beam with level of headed stud immersed in the concrete slab. Plate 1 depicted the actual setup of the specimen while Fig.1 represented the standard of push out test.

Plate 1 Setup for the push-out test

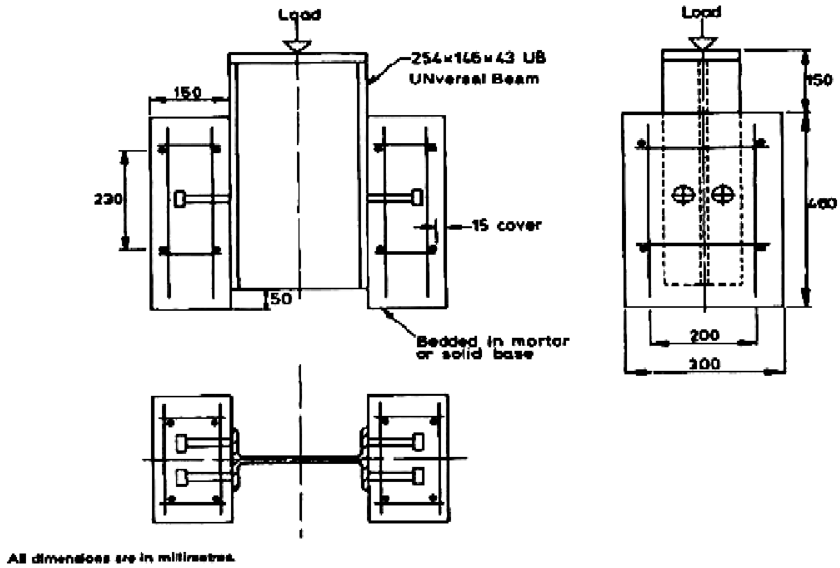

Figure 1: Standard of push- out test

2.3 Joists Construction and Instrumentations

Typically, each of the seven test specimens consisted of single simply supported composite joist effectually joined by ten mechanical shear connectors to 60 mm-thick cast-in- place concrete slab through the corrugated steel deck of gauge 20 (0.9mm thick). Each joist had been constructed with clear span of 2820mm and depth of 235mm to obtain the span- depth ratio equal to (12).The overall span length was 3000mm with limited width of the concrete slab of 400 mm to simplify laying the joist between the screw shafts of the testing flexural machine,. The joist members were arranged in a warren truss configuration. Steel double angles were used to construct top and bottom chords have the same cross sectional. Double angles, also used for the web members, except the web of joist 5 were constructed using rounded steel bar with 25mm diameter. Figs 2, 3 and plate 2 showed above details. Headed studs of 10mm diameter as a shear connectors were used. 30mm stud height for all joist, except joist 7 has 50mm stud height. Fig. 4 shows the stud distribution.

Plate 2: Corrugated sheets and shear connectors

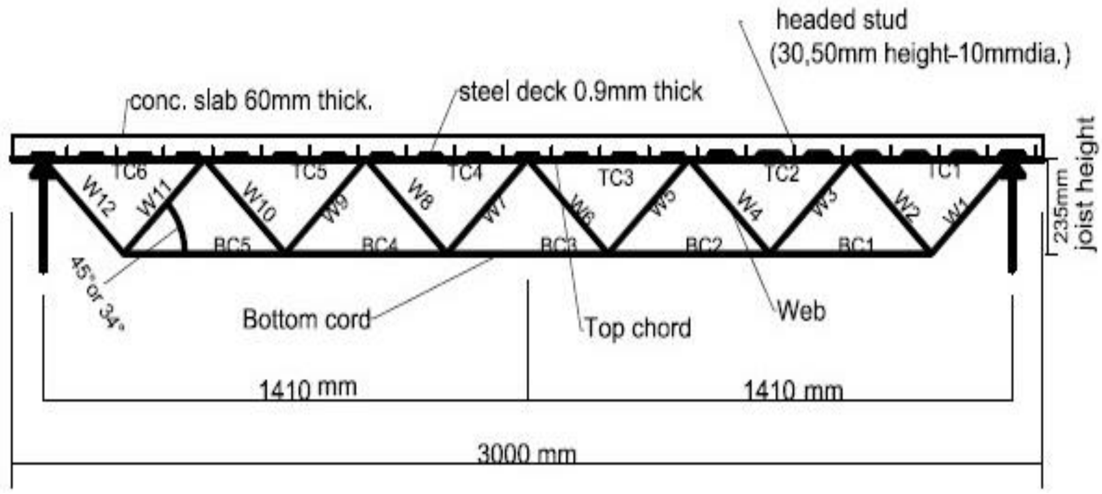

Figure 2: Joist configuration details

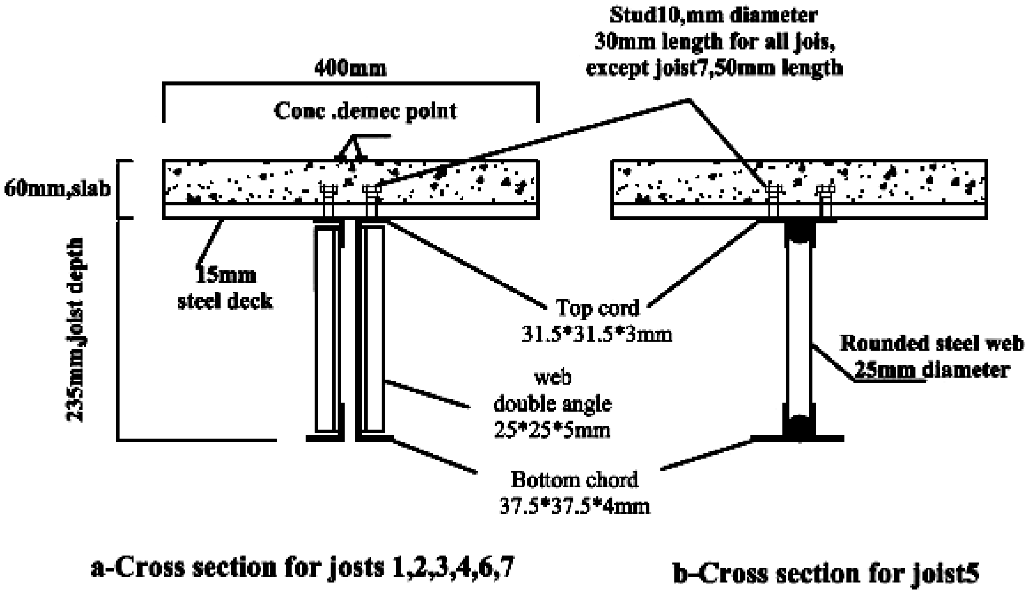

Figure 3: Cross sections for Joists

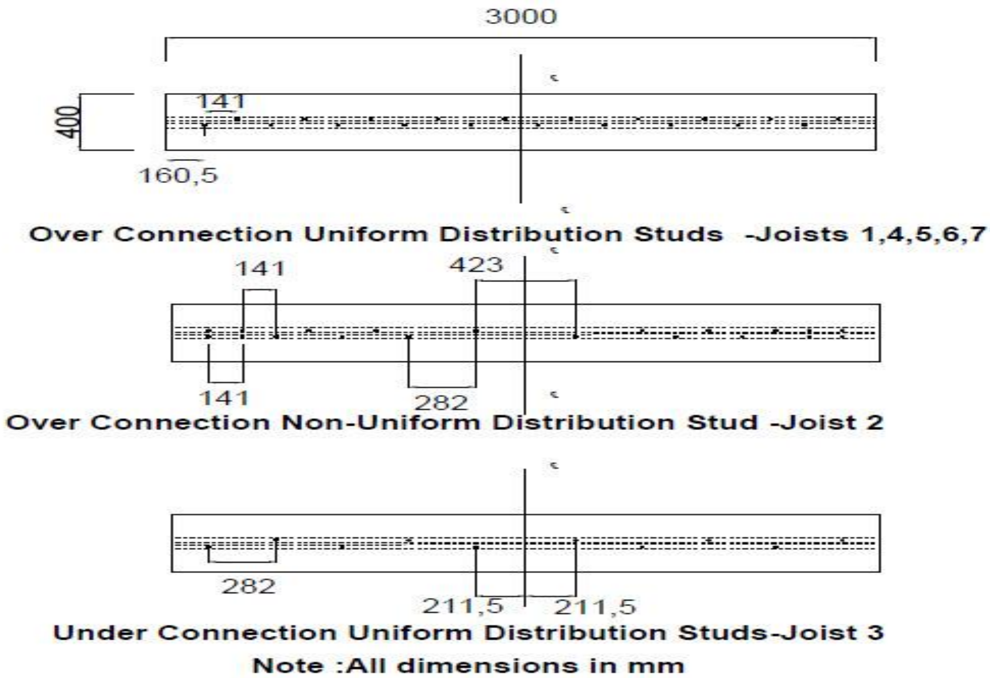

Figure 4: Distribution of shear connectors (Studs)

Similar instrumentation patterns were used for each joist test. A mechanical method was used for strain measurement at the mid span top slab; one row of demec point was fixed at the top face of slab shown in Fig.3, in order to record slab strains .Its reading scale accuracy was 0.002mm. Top and bottom cords were labeled (TC1toTC6) and (BC1 to BC5) respectively. The symbols (R) and (L) represented right and left leg of cord. For webs the symbols were (W1toW11). Joist 4 instrumentations have little difference. Metal Strain gauges were installed with labeling (G1to G8and glued by adhesive P-2). (R) and (L) indicated the gauge installed at right or left cord leg respectively. Two LVDTs (Linear variable differential transformers) instruments of 100mm capacity were located at the two ends of each joist to read the relative movements (slips) at the interfaces between the top cord and the slab. LVDTF9 and LVDTB10 indicated to the front and back locations, respectively. Mid-span and quarter-span deflections were measured using dial gauges of 0.01 mm accuracy. Electrical data logger denoted TDS 530 of ten reading channels was used to control the strains of the cords and the end relative slips for each joist. Fig. 5 and plate 3 and 4 gave the instrumentations details.

Plate 3: Metal strain gauge and its P-2glue

Plate 4: Data Logger TDS 530

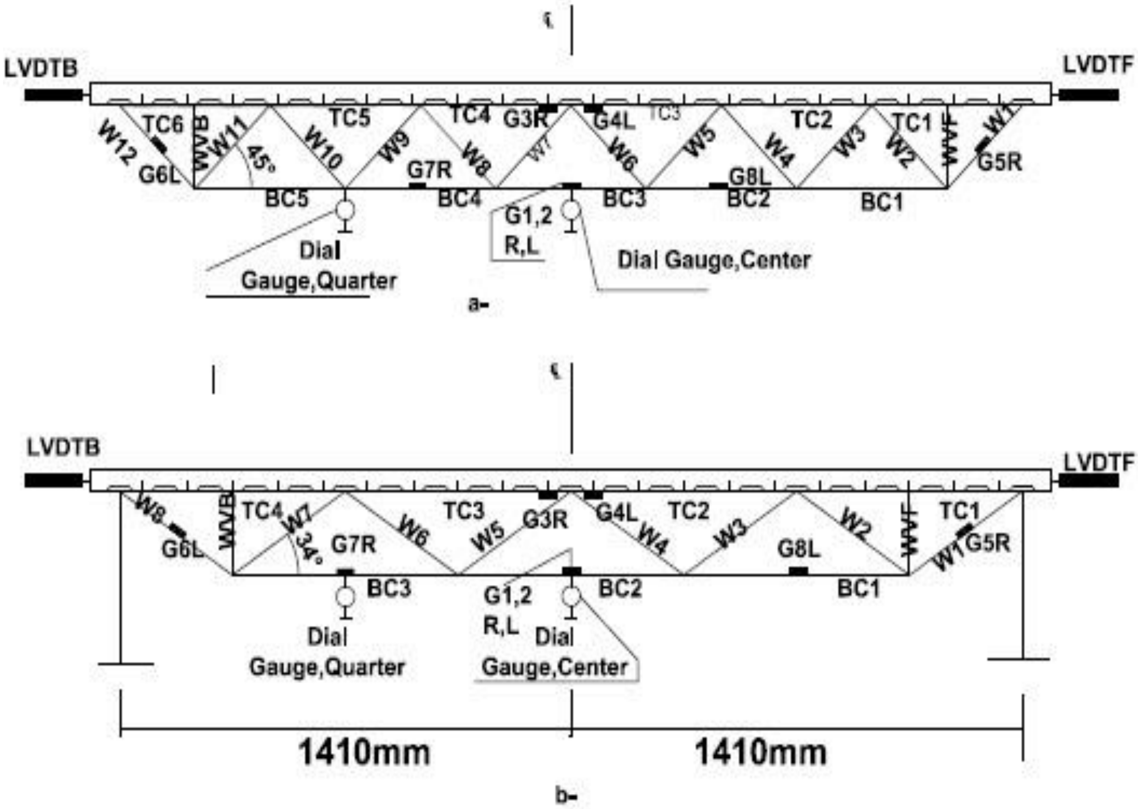

Figure 5: Members designation and strain gauges locations **a-** joists (1,2,3,5,6,7), **b-** joist4

2.4 Joist Loading Procedure

A flexural machine with 3000kN hydraulic jack was used to apply equal load to two points over a series of spreader beams (which their weights taken in dead load computations), used to distribute the applied load to the specimens by eight load points as shown in Fig.6 and Plate 5.

3 Results and Discussion

3.1 Push- out Test Results

The experimental results of push-out test are compared with the (AISC-2005)[6] and euro-code (1994)[8] equations, these results revealed acceptable values. Long headed stud exhibited more strength than others due to the ductility gained by increasing in its projection embedded in slab above the top of the steel deck. The results are given in table 2.

Euro code Equation (1994) [8]

$$q = 0.8 * f_u * (\pi d^2 / 4) \leq q = 0.29 d^2 \sqrt{(f_{ck} E_{cm})} \quad ..1$$

Where, the applicable stud shear reduction factor for steel deck from Lawson study (1993) [9] is:

$$r_p = 0.75/\sqrt{N} * b_a/D_p * [h/(h+D_p)] \leq 1.0 \quad ....2$$

AISC Equation (2005)[6]

$$Q_n = 0.5 A_{sc} \sqrt{(E_c f'_c)} \leq A_{sc} F_u \quad ....3$$

The strength reduction factor for metal deck oriented perpendicular to the joist span from the LRFD specification is:

$$RF = 0.85/\sqrt{N_r} * w_r / h_r [(H_r / h_r) - 1] \leq 1.0 \quad ....4$$

If ***RF*** more than 1.0 the ratio of actual flange thickness to the recommended minimum flange thickness must be checked, which is considered as a reduction factor for thin steel flange, so **Minimum t_f =*0.4(stud dia.) Goble(1968)*** [10]. In this study, 3mm thick top chord angles and 10 mm diameter studs result in the provided flange thickness (in terms of stud diameter). ***Actual t_f***

=0.3(stud dia). Reduction factor for thin base metal ***(R_t)=0.3/0.4=0.75***, this reduction was considered for the calculation of the stud shear capacity**.**

3.2 Ultimate load carrying capacity for joists

The experimental load capacities of the joists ranged from (160 -225) kN as shown in table 3.The joist of long studs bears maximum load was about 225kN.

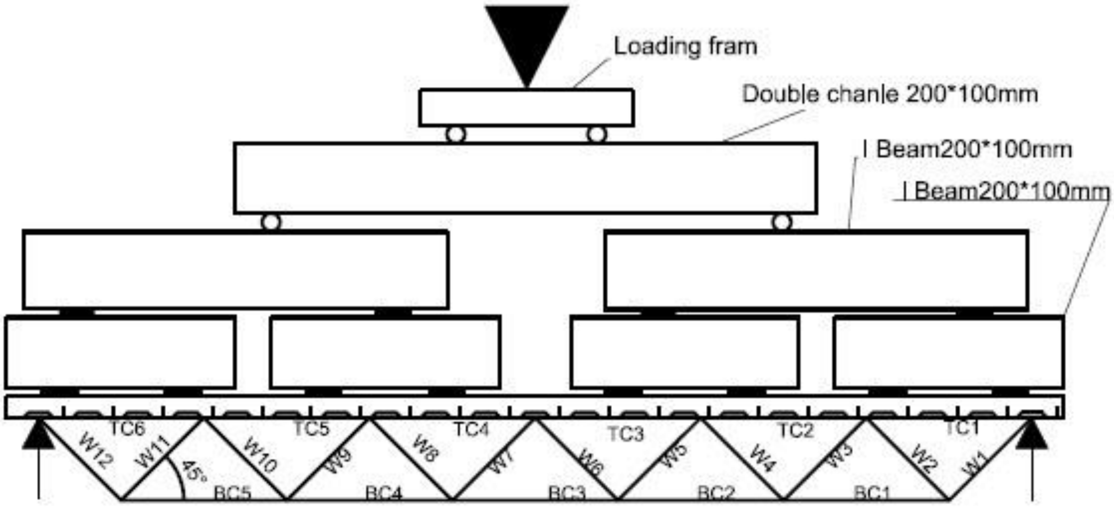

Plate 5 Flexural test Machine

Table 2 Push out test results

Speci-men	Ulti-mae load (kN)	Ulti-mae slip (mm)	Shearing Resist-ance per stud (kN)	Slip at 50% of stud shear force (mm)	Shearing Rigidity (Ks) (kN/mm)	Shearing Rigidity (Ks) (kN/mm) Euro-Code Equation	Sheari-ng Rigidity (Ks) (kN/mm) AISC Equation	Mode of Failure
Slab NWC, Short Stud	120	1.48	30	0.21	71.43	23.83	24.4	Concrete Crushing
Slab NWC , Long Stud	140	2.54	35	0.94	19	23.83	24.4	Concrete Crushing
SlabL WC, Short Stud	106	0.71	26.5	0.26	51	0.85*23.83 = 20.26	0.85*24.4 =20.74	Concrete Crushing

Table 3: Ultimate load capacity

Test Designation	Characteristic	Load per Joist at failure (kN)
CSJ-1(B1)	Reference ,uniform stud distribution	210
CSJ-2(B2)	Non uniform stud distribution	198
CSJ-3(B3)	Under connection-shear stud	190
CSJ-4(B4)	Web inclination - 34°	190
CSJ-5(B5)	Rounded web shape	189
CSJ-6(B6)	LWC slab	160
CSJ-7(B7)	Long shear stud	225

3.3 CSJs Experimental Response

The linear behavior shown in the curves that will be discussed below indicates the full composite action for joists whereas the non-linear behavior indicates the non composite action due to the deterioration of shear connection between the slab and the studs.

3.3.1 CSJ-1 Experimental Response

This joist was considered as a reference for general structural behavior for other joists. At applied load of 151.35kN *the bottom chord* yielded at load stage of 185.62kN.The system deflection and strains of top and bottom chords behaved linearly elastically up to a load approximately 150kN in addition for slab strain as in Figs (7, 8, 9, and 12). The cords were under tension action. Failure occurred at a load of 210kN.*W1*and *W11* suffered from tension and compression strains respectively during the test as in Fig 10. At 150kN load level there was *noticeable slips*. The presence of slip indicates *no full interaction* between the slab and the steel top chord, as in Fig. 11**.** It was found the 1st and 2nd headed stud's shank exhibited distorted shape due to the horizontal shear flow, as in Plate 6. Full separation between the steel deck and slab was observed uniformly along the span within the loading of 210kN, indicating full shear connection deterioration. The uniform stud's distribution had enabled to control the uniform separation along span, as in Plate 7.

Plate 6 Fracture pattern of CSJ-1

Plate 7 Distorted headed stud of CSJ-1

3.3.2 CSJ-2 Experimental Response

Bottom chord reached yield limit of 185.62kN when applied load was 137.7kN. The system deflection and strains of top and bottom chords behaved linearly elastically up to a load approximately 100kN also for slab strain as in Figs. (7, 8, 9, and 12). The cords were under tension action in this stage. Full failure occurred at 198kN.The strains in the chosen webs *(W1, W11)* indicated tension and compression response respectively as designed and shown in Fig.10. An existing end slips until 100kN load level at both joist ends due to the shear studs amounts near the supports as in Fig.11. Deck separation began near the supports at the150kN and directly ahead at mid span in the range of 170-195kN loading stage. The non uniform distribution of studs along the span caused non uniform deck separation. Plates 8 and 9 shows the deck separation and stud distortion receptively. This joist exhibited well ductile manner as in fig.7.

Plate 8 Steel deck separation pattern and top cord buckling shape in CSJ-2

Plate 9: Distorted headed stud of CSJ-2 at failure

3.3.3 CSJ-3 Experimental Response

Yielding strength of 185.62kN occurred at applying load of 134.6kN. Bottom chord reached yield limit of 185.62kN when applied load was 137.7kN. The system deflection and strains of top and bottom chords behaved linearly elastically up to a load approximately 160kN but for slab strain was about 80 kN as in Figs. (7, 8, 9, and 12). The cords were under tension action in this stage. Beyond that, the specimen acted none linearly due to non- composite action, caused by the separation of the shear connection studs beginning after the load stage of 120kN as shown in plates 10 and 11.Failure load was 190kN.*The webs W1and W11*, undergoing tension and compression strains, respectively as in Fig 10.There is tension relative slips at the joist ends because of the low interaction between the contacted faces, due to less amount of shear studs (under connection) as in Fig.11. The under connection, uniform distribution of stud along the joist span exhibited well stiff behavior that relieved in load-deflection curve given in Fig.7

Plate 10: Fracture pattern of CSJ-3

Plate 11: Distorted headed stud of CSJ-3

3-.3.4 CSJ-4 Experimental Response

Bottom chord was yielded when applied load had reached 121kN. Deflection has approximately linear behavior up to120kN, also for top and bottom chord as in Figs (7, 8, and 9), but concrete in slab crushed at 80 kN load (strain= 0,003) as in Fig.12. Failure took place at 190kN loading stage. *W1* suffered from tensile strain, while *W11* in compressive strain as in Fig.10. *Relative slip* at ends of the joist between was existing, pointing no full interaction. *Steel deck separation* started at 90 kN loading stage and propagated towards mid span through the load increments as shown in Plate 12. Plate 13 shows the stud's distortion.

Plate 12: CSJ-4 after loading up to failure

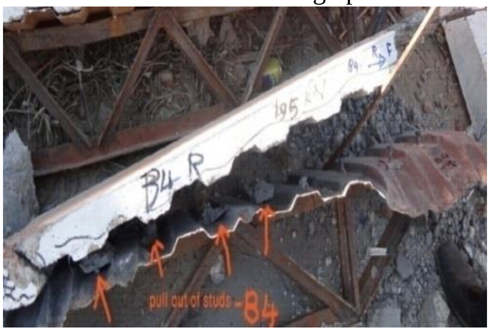

Plate 13: Studs distortion of CSJ-4

3.3.5 CSJ-5 Experimental Response

Bottom chord yielded at 147.5kN applied load level. For deflection behavior, an approximately linear trend up to a load level of 120 kN the same behavior for the top and bottom chords as in figs.7, 8 and 9. *Right side of bottom chord* exhibited more strain than left side at failure stage, due to lateral movement, because of single rounded web configuration, so the horizontal bridging must be taken in the consideration during the construction. Fig. 10 shows no yielding happened in the *web members* coinciding with their over deigned. *Small relative horizontal slips* exist until 40kN as in Fig. 11. *Strains in slab* behaved in compression linearly till a loading level of 120 kN, shown Fig.12**.** *The separation of steel deck* was clearly seen when the loading level reached 187 kN indicating non- composite action as shown in Plate 14. Plate 15 shows *five distorted studs.* The system failed at 189 kN load stage.

Plate 14: Steel deck separation pattern for CSJ-5

Plate 15 Studs distortion at failurefor CSJ-5

3.3.6 CSJ-6 Experimental Response

At the 130.03kN load level the bottom chord yielded by 185.62kN force.*This joist revealed linear behavior* until 130kN load level according the deflection curve presented in Fig.7.Figs 8and 9 show the same behavior in the top and bottom chords. As designed, the W1 and W11 resisted tensile and compression strains respectively as in Fig 10. *Relative slips* occurring showed partial connection in the composite system that is shown in Fig 11. *Slab deformation* was linear compression behavior continued till 50kN loading stage that shown in Fig. 12, the slab withstood compressive action to the crushing load level of 160kN.That was due to the exits of deck ribs which acted as reinforcement at the bottom fiber of the slab, besides the exits of studs, restrained the concrete in between. *The deck separation* was at 9okN load level while *the studs* not distorted and their strength may be more than that of light concrete. The joist failed at 160 kN load level.plate 16 shows the joist after failure, while plate 17 depicted no distortion in studs.

Plate 16: Deck separations, slab crushing, CSJ-6

Plate 17: Never distorted headed stud, CSJ-6

3.3.7 CSJ-7 Experimental Response

Bottom chord yielded with strength of 185.62 kN when applied load reached 133.5 kN. Linear deflection behavior reached the 150 kN load level, that was the same for top and bottom chords indicating composite action as shown in Figs.(7, 8 and 9). *W1 and W11* exhibited tensile and compression strains, as expected as in Fig.10. Figure 11 shows *the slip progressing* indicating no full interaction at the interface region. *Strains at top face of the slab* varied linearly until the 70kN load level as in Fig.12. Due to long studs, *the uplift and separation* did not occur up to 220kN loading stage as in plate.18. Plate 18 shows the joist at failure. There was no significant headed *stud's distortion*, however, the first and second headed stud near the support suffered from the small distortion and little curvature at its mid height due to high horizontal axial block concrete stress as shown in Plate 18. Failure load was 225kN.

Plate 18: CSJ-7specimen after failure

Plate 19: Studs distortion in CSJ-7specimen

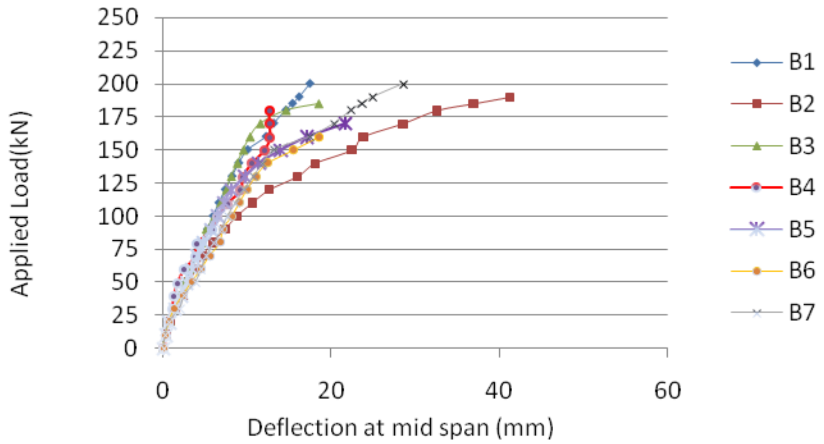

Figure 7: Joists load- deflection behaviors

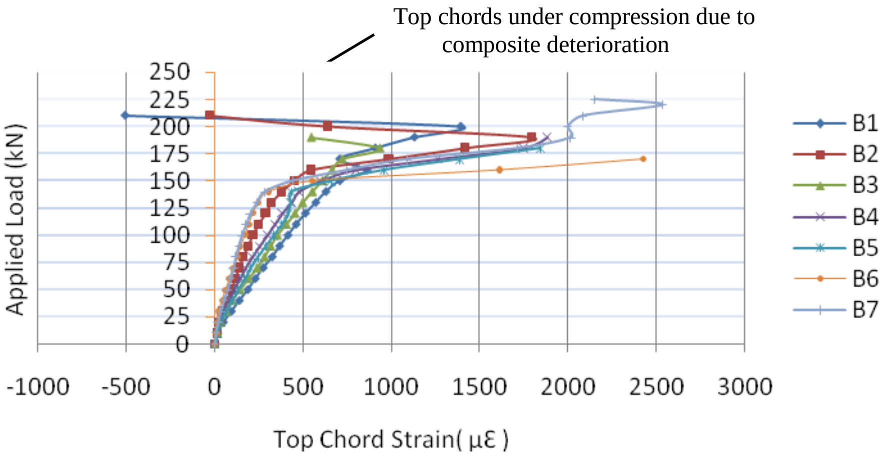

Figure 8: Joists load-top chord strain

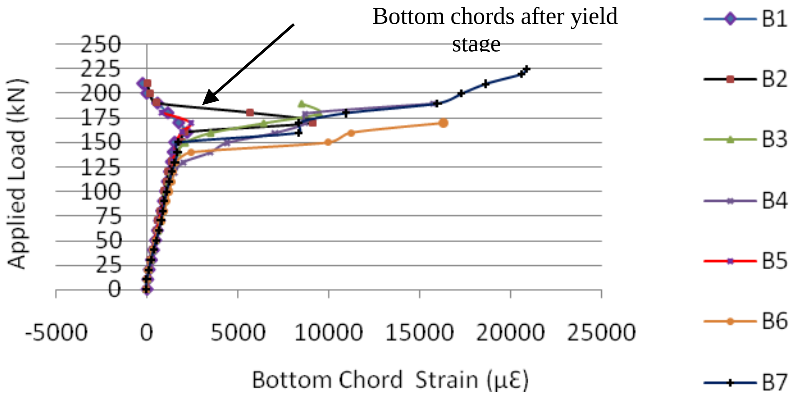

Figure 9: Joists load- bottom chord strain

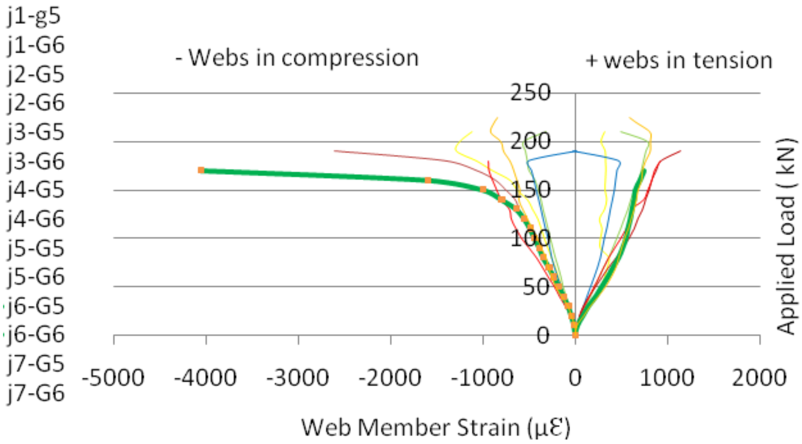

Figure 10: Joists load-web strain

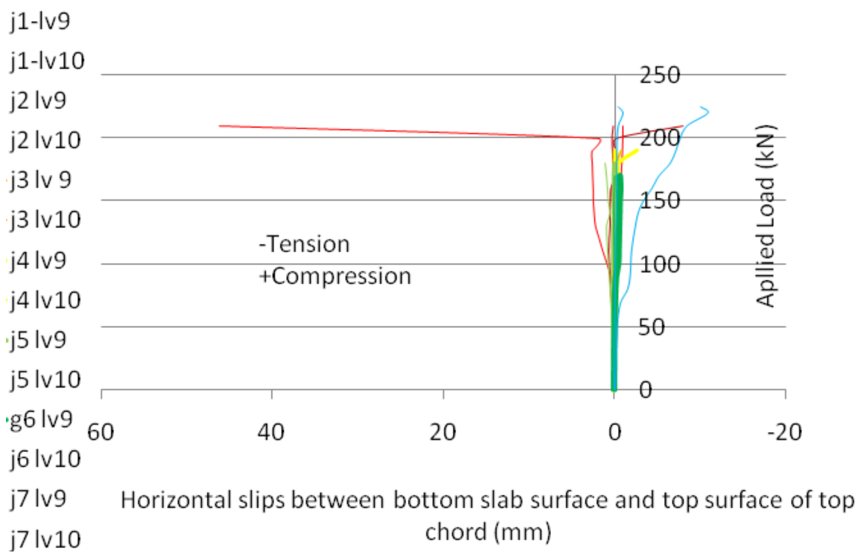

Figure 11: Joists load-slab, top chord horizontal slip

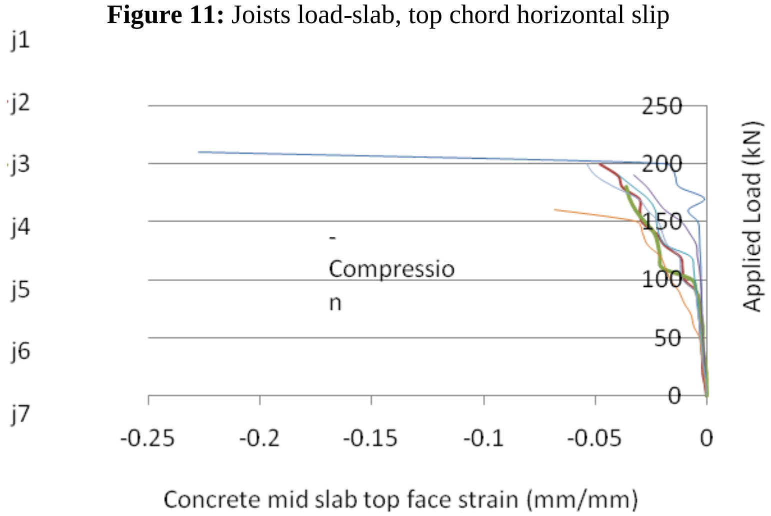

Figure 12: Joists load-mid top face slab strain

4. Conclusions

1- It was found the long shear stud that embedded in the normal weigh concrete exhibited more ductility (slip = 0.94mm), less rigidity (19 kN/mm) with higher shear strength (35kNper stud), While the short stud exhibited less ductility (slip= 0.21mm), more rigidity (71.43kN/mm) with shear strength (30kN per stud) which embedded in the same media. On other hand, the shot studs that fixed in light weight concrete slab exhibited ductility(slip=0.26mm) and shear strength(26.5kN/stud) close to that embedded in normal weight concrete but with rigidity(51kN/mm) less amount than that for stud in normal concrete.

2- Joist of light weight concrete slab resisted lower load carrying capacity of 160kN, while the joist which built with long shear connector resisted 225 kN.The remain five joists had the closed value of load capacity ranged from 189-210 kN.

3- Comparing with the capacity of each joist with that for the reference joist (joist1) the relative load capacity as follow:

94% for Joist 2, 90.5% for Joist 3, 90.5% for Joist 4, 90%for Joist 5, 76.2% for Joist 6 and Joist 7 has relative capacity 107.143%.

4-From the load- deflection relationships of the seven joists(COWSJs), the joist of the uniform over-connecting stud distribution as CSJ-1, and the joist of the under-connection condition as CSJ-3 have behaved in a high stiffness flexural manner .The Joist of the non-uniform distribution of shear connectors as CSJ-2 has exhibited more ductile performance. The Joist of web-member inclination less than 45 degrees as CSJ-4 , and the joist of rounded web members as CSJ-5 have suffered transverse bottom chord displacement , thus , the lateral bracing for the bottom chord is important to avoid such undesired movement. All tested joists have performed quite efficiently as composite flexural members within elastic range behavior.

References

[1] Oehlers, D.J. and Bradford, M. A., "Elementary Behavior of Composite Steel and Concrete Structural Members", Butterworth Heinemann, 1999, London, 259 p.9.

[2] Jamal, A.F., "Development of Three- Layers Composite – Concrete Steel Beam Element with Application", Ph.D. Dissertation, University of Technology, June, 2009.

[3] David Samuelson, "Composite Steel Joists", Engineering Journal / Third Quarter, 2002, 111 p.

[4] Supplement No 1," Standard specifications for Composite steel Joists and code of standard practice", Steel Joist institute (SJI), Vol. 32, Dec., 2010.

[5] Douglas F. Lauer," Ultimate strength analysis of composite and fully composite open - web steel joists", M.Sc. Thesis, Virginia Polytechnic Institute and state University, October, 1994.

[6] AISC, "Steel Construction Manual", American Institute of Steel Construction, Chicago, Thirteenth Ed., IL, 2005.

[7] BS 5400: Part 5, "Design of composite bridges", British standards institution, London 1979.

[8] ENV 1994-2," Euro Code - 4; " Design of Composite Steel and Concrete Structure, Part 2; General Rules for Bridges ", CEN; 1997.

[9] Mark Lawson and Peter Wickens,"Steel Designers' Manual – Chapter Composite Beams", 6th Edition, 2003.

[10] Colaco, J. P. "A Stub-Girder System for High-Rise Buildings." AISC Engineering Journal, 1972, pp. 89-95.

البحث العملي لتصرف العتبات الفولاذية المفتوحة الوترات

علي فرحان حديد
دكتور مدرس
قسم هندسة تقنيات البناء والانشاءات
كلية الاسراء الجامعه

ليث خالد الحديثي
استاذ مساعد
قسم الهندسة المدنية
جامعة النهرين

رياض جواد عزيز
استاذ مساعد
قسسم الهندسة المعمارية
كلية الاسراء الجامعة

الخلاصة

الروافد الفولاذية المركبة ذات الوترات المفتوحة شائع استعمالها للعديد من السنين لاسناد الارضيات والسقوف وهي اقتصادية وخفيفة الوزن وتغطي فضاءات كبيرة ويمكن تمديد كيبلات الكهرباء وقنوات التكييف والانابيب من خلال فتحات الوترات وبالتالي نحصل على اقل ما يمكن من الا رتفاع الصافي بين ارضيات الطوابق . لغرض دراسة التحمل لهذه الروافد تحت مختلف المتغيرات تم تشكيل سبعة روافد مركبة من الحديد بطول 2820 ملم مركز الى مركز الاسناد وعمق كلي 235 ملم ربطت عليها بلاطة من الكونكريت بواسطة روابط قوى القص لدراسة التحمل وفق المتغيرات التالية (أسلوب توزيع روابط القص studs ،درجة الربط بين البلاطة والوتر الفولاذي العلوي ، زاوية ميل اضلاع الجذع ، شكل الجذع ، كثافة الخرسانية للبلاطة،طول رابط القص).أضهرت النتائج العملية ان العتبات ابدت اقل تحمل قدره 160 كيلونيوتن للرافدة ذات البلاطة خفيفة الوزن واعلى تحمل قدره225 كيلونيوتن للرافده ذات الرابط الطويل (50 ملم) عند الفشل.هذه النتائج قورنت تحليليا وفق نموذج عزمي .