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Abstract

This study is performed to investigate experimentally the behaviour of steel deck composite slabs with different end anchorages. End anchorage as a type of shear connection for composite slabs plays an important role to prevent relative slip between concrete and steel deck. The presented composite slab specimens are made of high strength concrete and loaded at a specific shear span. Objectives of this study is to evaluate experimentally load carrying capacity, end slip, mode of failure, shear bond capacity, and the end anchorage contribution to the whole composite slab behaviour. Research also presents a comparison between the experimental results and the theoretical results derived according to m-k and partial shear connection methods included in these standards (BS 5950-4:1994, CSSBI S3-2003, and EC4 EN 1994-1-1:2004).

Keywords : Composite slabs, End anchorage, Longitudinal shear capacity, Moment capacity, Steel deck, High strength concrete, End slip

1.Introduction

The use of steel deck in the construction of floors began in the 1920's. The deck commonly was the main structural component for the floors of steel framed buildings. The addition of concrete cover provides structural strength; serve the purposes of fire protection, acts as a mean to level the top surface of the floor, and distribute the loads. The accepted practice by manufactures is to perform multiple full-scale slab tests of the steel deck to determine its performance. Composite slab has a similar definitions by EC4 ENV 1994-1-1:2004 [1], BS-5950 Part 4 (1994) [2], and CSSBI S3-2003 [3] and its generally defined as "a slab system comprising normal weight or lightweight structural concrete placed permanently over steel deck in which the steel deck performs dual roles of acting as a form for the concrete during construction and a positive reinforcement for the slab during service". Composite action is defined by BS-5950 Part 4 (1994) [2] as "the structural interaction which occurs when the composite slab interact to form a single structural element". Fig.1 shows composite floor with steel deck system and the location of end anchorage.

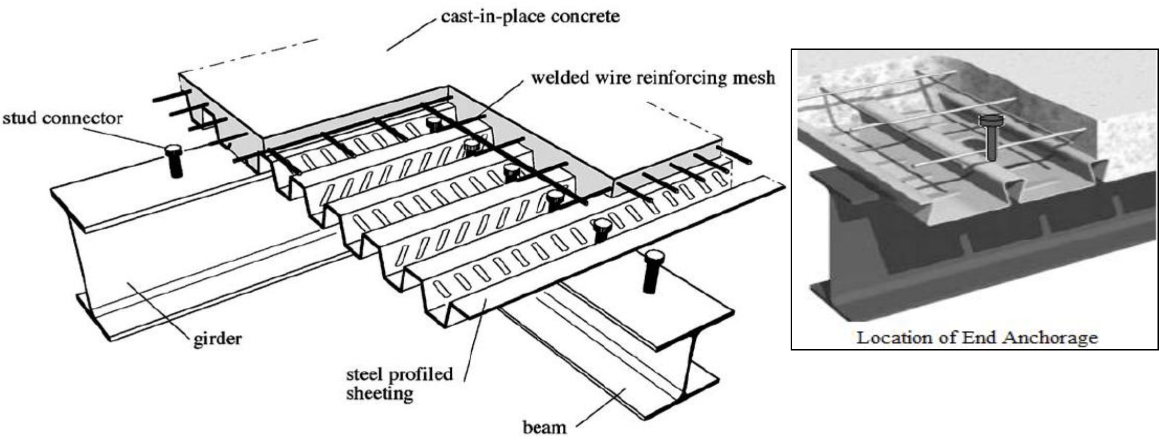

Fig.1:Composite Floor with Steel Deck

The types of shear connection for steel deck composite slabs are:

(1) Chemical bond (frictional bond); resulting from the chemical adherence of cement paste to the steel sheeting (natural bond), once this bond is broken, slip is initiated and the chemical bond strength reduces to zero.

(2) Mechanical bond (Physical interlocking at contact surface);the interlocking is developed by clamping action caused by bending of the steel deck and friction at contact surface due to steel surface roughness (indentation or embossment).

(3) End anchorage; prevent relative slip between concrete and steel deck as shot fire pins, welding studs…etc.

The composite slab under bending can exhibit three major modes of failure according to BS-5950 part 4(1994) [2], EC4 ENV 1994-1-1:1992[1], and (Johnson, 1994) [6] as shown in Fig.2.

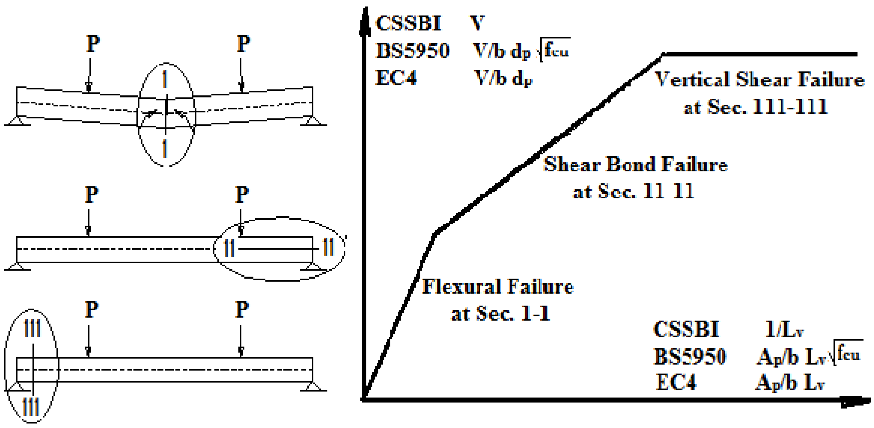

Fig. 2: Modes of Failure

First mode of failure is ***flexural failure*** and is based on the full shear connection at the interface between steel deck and concrete. This mode usually occurs in long thin slabs and analysis is as the case of ordinary reinforced concrete procedures, not dominant design criterion because steel and concrete interaction is usually incomplete.

Shear bond failure is the second mode of failure and is characterized by the formation of diagonal tension crack in the concrete at/or near the load points followed by a loss of bond between steel deck and concrete (observable end – slip) within the concrete shear span as shown in Fig.3

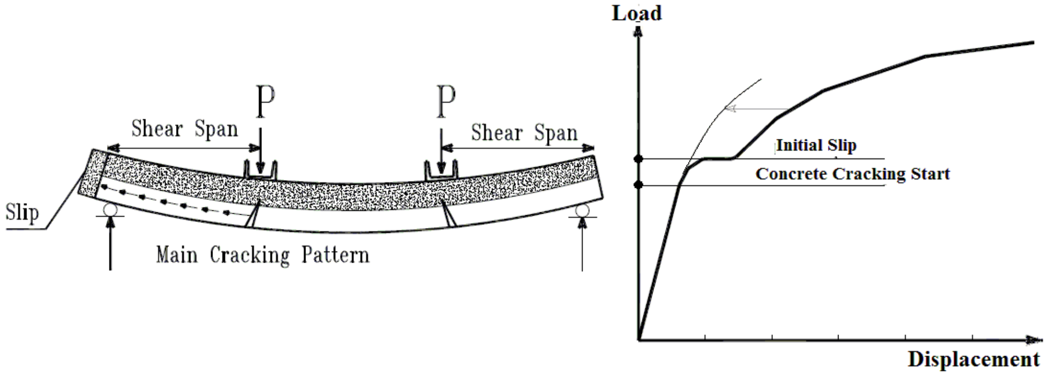

Fig. 3:Shear Bond Failure

The third mode of failure is ***Vertical shear failure*** which will be critical only in special cases e.g. in deep slabs of short span with loads of relatively large magnitude and this mode is typically ignored in the design calculations.

In 1987, Jolly and Zubair[9] experimentally evaluated evaluate the effect of different indentations on the shear bond strength for the web of deck profile. Luttrell, L.D. (1987) [10] carried out a research involved testing 25 slabs (both single span and two-span continuous) of varying width in which embossed steel deck acted as the only reinforcing. K. Roik, H.Bode(1992) [11] describes the design of composite slab with ductile shear behaviour takes account the incomplete interaction and partial shear connection. Van der sanden, Stark j.W.B, H.H.Snijder, H.W.Bennenk (1996) [12] carried out 22 tests to investigate the behaviour of headed studs in ribbed slabs and evaluated. The parameters which were varied are: the steel sheeting (with and without sheeting, with and without embossments, thin and thick sheet), the geometry of the rib, the place of the stud within the rib. Thorsten Faust(1997) [13] presents test data on the longitudinal design shear strength of a composite floor slab with lightweight aggregate concrete, and re-entrant steel sheet with indentations on the top flange. H.Bode, F .Minas (1998) [14] deals with the strength and behaviour of composite slab with three typical, but different profiled steel sheet geometries used with and without end anchorage means (headed studs and bent rib anchors). L.H. Lee (2001) [15] studied the behaviour of cold-formed steel deck and concrete composite slab under hogging moment using 10 specimens of different thickness and reinforcement ratio. S.A.L. de Andrade (2004) [16] presented an analytical investigation backed by experimental results of the structural behaviour of composite slabs with steel decks. G. Marciukaitis (2005) [17] presented a method for calculating deflections of slabs. The deflection of composite slabs depends directly on the shear stiffness of the connection between profiled steel sheeting and concrete. V. Marimuthu (2006)[18] carried out an experimental study to investigate primarily the shear bond behaviour of the embossed composite deck slab under simulated imposed loads and to evaluate the m–k values with M20 grade concrete.

2 RESEARCH SIGNIFICANCE AND METHODOLOGY

Based on the available literatures, the prediction of the effect of different end anchorage on the whole behaviour of composite slabs with high strength concrete is undetermined and the contribution of end anchorage on the whole load carrying capacity, shear bond capacity is obscure. Because of the leakage in the methods of analysis of end anchorages in composite slabs, the investigation of the effect of end anchorage type upon composite slab behaviour is carried out and discussed.

The performed laboratory test program based on the full-scale tests of composite slabs utilizing trapezoidal deck profiles that are commonly available in Egypt. This experimental program includes; testing of specimens with various end anchorages, with or without internal reinforcement mesh. The study investigate the contribution of the end anchorage to the composite slab load carrying capacity, shear bond capacity, end slip, deflection, and toughness ratio for different end anchorages. The experimental results are discussed in the light of various specifications in order to study the composite slab behaviour focusing on the effect of the end anchorage of the slab on the interaction property and slab performance.

3 EXPERIMENTAL TESTING PROGRAM

The experimental program includes the testing of nine full scale specimens; first one is the benchmark sample without end anchorage and the other eight specimens were with different end anchorages means. All specimens were built in a simple span set up and experimentally tested at the heavy structures and reinforced concrete laboratory at the Faculty of Engineering, Tanta University, Egypt. The main idea was to create slabs specimens that resemble the composite slabs normally found in actual construction practice. The benchmark sample was designed to have shear bond failure which is characterized by a relative movement (end slip) between steel deck and concrete at the ends of the tested specimens. Fig.4 shows the basic dimensions of specimens; all specimens had 1800 mm length, 690 mm width, 100 mm depth, and subjected to loads on a certain shear span equal 450 mm.

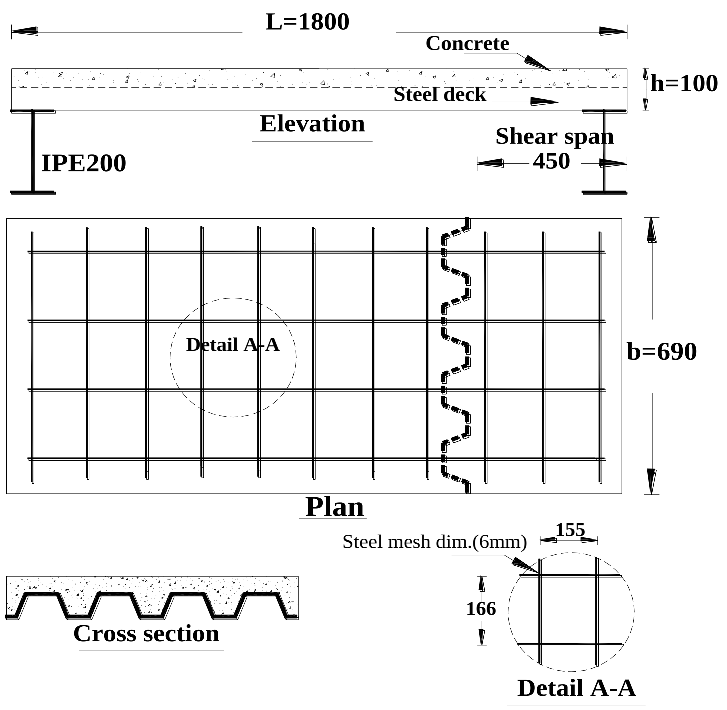

Fig. 4:Geometry and Basic Dimensions of Specimensin mm

Steel deck properties were as follows; the shape is trapezoidal with embossments on adjacent webs with 0.85 mm thickness as shown in Fig.5, with 305 N/mm^2 yield strength, ultimate tensile strength equal 377 N/mm^2 ,and mean shear stress per unite horizontal area $\tau_u$equal 305.95 kN/m^2 [5].

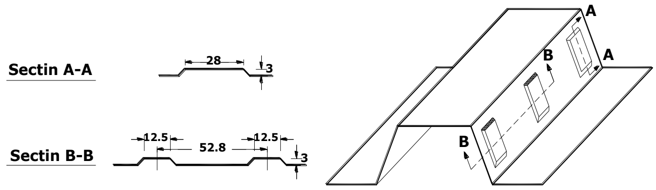

Fig. 5:Details of Steel Deck Embossmentsin mm

The design mix of high strength concrete for one cubic meteris as presented in Table 1.

Table 1: HIGH STRENGTHCONCRETEMIX DESIGN

Concrete grade	Cement (Kg)	Sand (Kg)	Crushed gravel 10mm (Kg)	Crushed gravel 19mm(kg)	Water (Liter)	Silica fume (Kg)	Admixture Type F (Liter)
C 65/70	550	680	460	612	138	38.5	11

The stud shear connectors properties were as follows; the yield stress equal 43.4 kN/mm^2, the tensile strength equal 73.05 kN/mm^2, and the Elongation equal 7.55 %.

Instrumentation and measurements includes; (1) Hydraulic jack used to apply vertical load to composite slab specimens with 300 kN capacity and 0.01 mm accuracy load cell. (2) Digital vernier calliper used to measuring thicknesses with precision 0.01mm. (3) Load indicator used to controls rate of loading for every load increment. (4) Mechanical dial gauges with 0.01mm accuracy used to measure slip between concrete and steel deck and deflection at composite slab mid span. Composite slab behaviour, vertical deflection at middle of slab, relative slip between concrete and steel sheeting, and composite slab mode at failure were investigated and recorded at each stage of loading. Schematic drawing of test set up illustrate load arrangements and instrumentations are as shown in Fig.6.

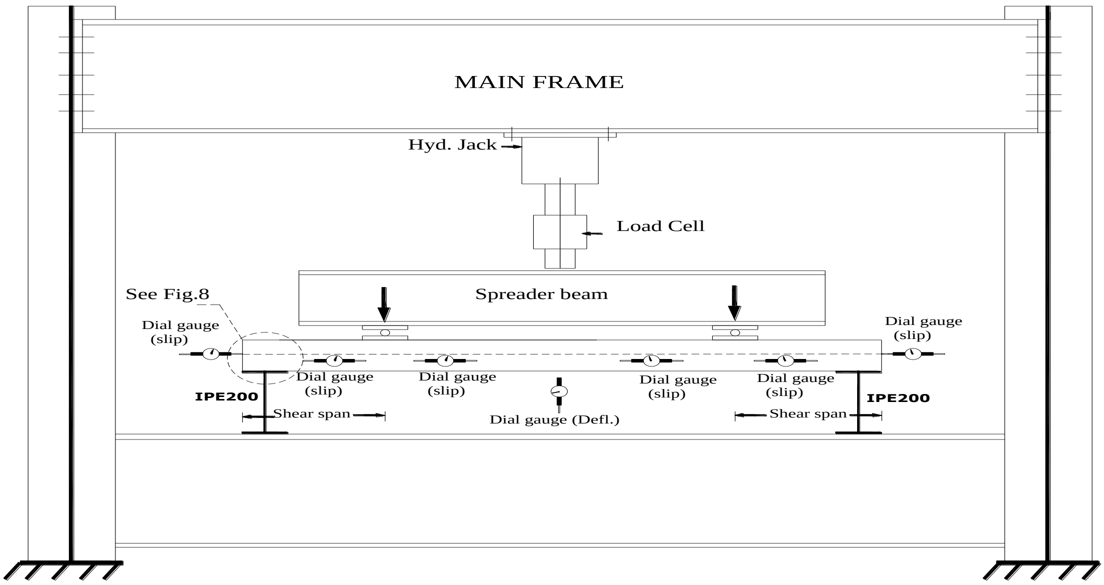

Fig. 6:Schematic Drawing of Test Set Up

Table 2 shows details of the specimen's characteristics.

Table 2: SPECIMEN'S DESCRIPTION

Specimen No.	End anchorage	Reinforcement Mesh	Shear span / Total span	Concrete Strength N/mm^2	Concrete Modules of elasticity N/mm^2
S1	Without end anchorage	5φ 6/m'	0.25	59	33797.04
S2	One vertical row of studs (19 mm diameter)	5φ 6/m'	0.25	60	34082.25
S3	End angle with horizontal studs(19 mm diameter)	5φ 6/m'	0.25	70	36813.041
S4	Last transverse bar of steel mesh line welded to steel deck	5φ 6/m'	0.25	67.5	36149.68
S5	End angle with horizontal studs + (- ve) steel mesh	5φ 6/m'	0.25	67	36015.552
S6	Reinforced Concrete end	5φ 6/m'	0.25	64	35200
S7	Transverse wire spot welded to steel deck	-	0.25	72	37075.05
S8	Transverse mesh (2 bars) spot welded to steel deck	-	0.25	67	36015.552
S9	Transverse mesh (3bars) spot welded to steel deck	-	0.25	72	37075.05

There are eight end anchorages shapes used in this study as shown in Fig.7 and its details as shown in Fig.8.

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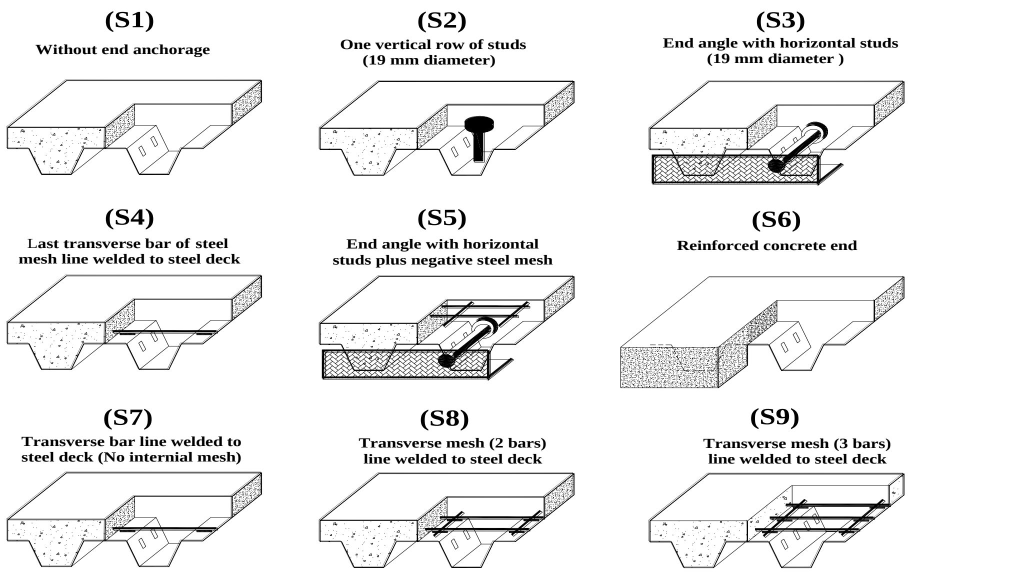


Fig. 7:End Anchorages Description

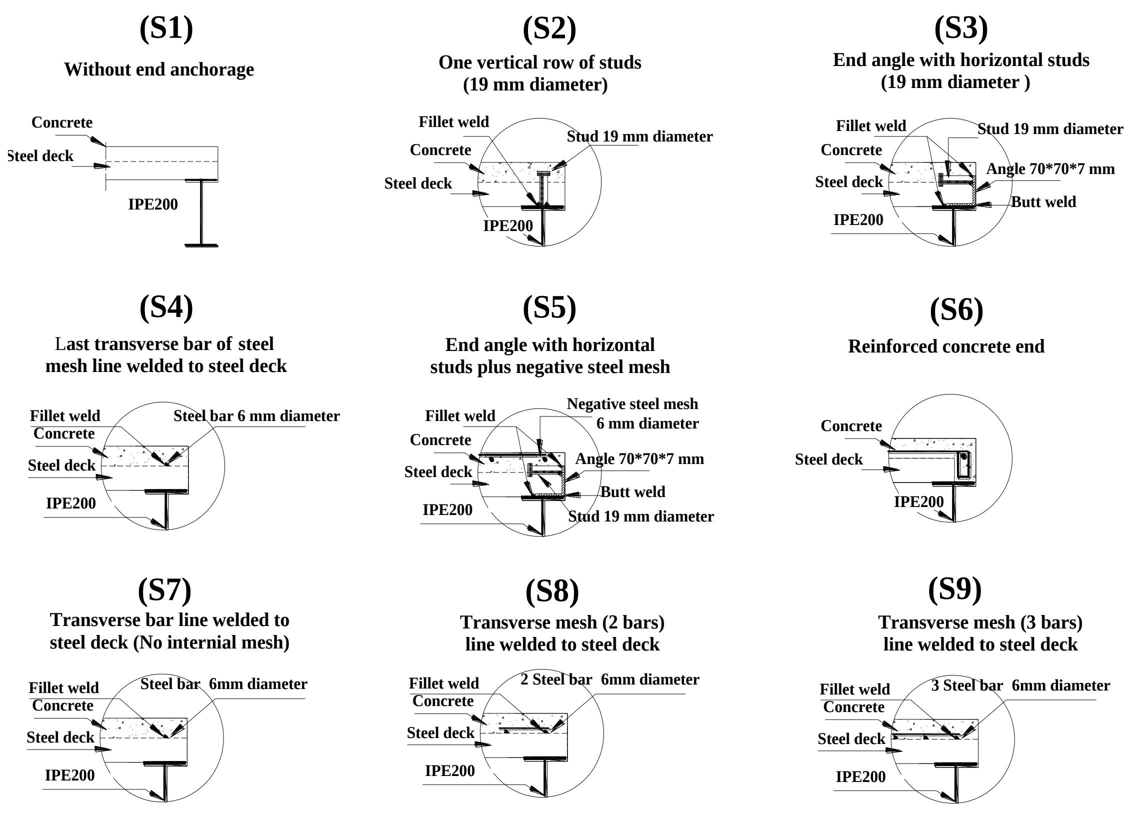


Fig. 8:End Anchorages Details

4.Test results and disscution.

Specimens from S2 to S6 are presented to study the effect of the end anchorages on composite slabs with internal reinforcement mesh, while specimens from S7 to S9 are presented to study the effect of end anchorages for specimens without internal reinforcement mesh. All these specimens are compared to S1 (Benchmark specimen without end anchorage). Composite slabs tested were loaded at the predetermined increments to get the best output data.

4.1 Behaviour of Composite Slabs with Internal Reinforcement Mesh

4.1.1 Crack & Failure Load, and Mode of Failure

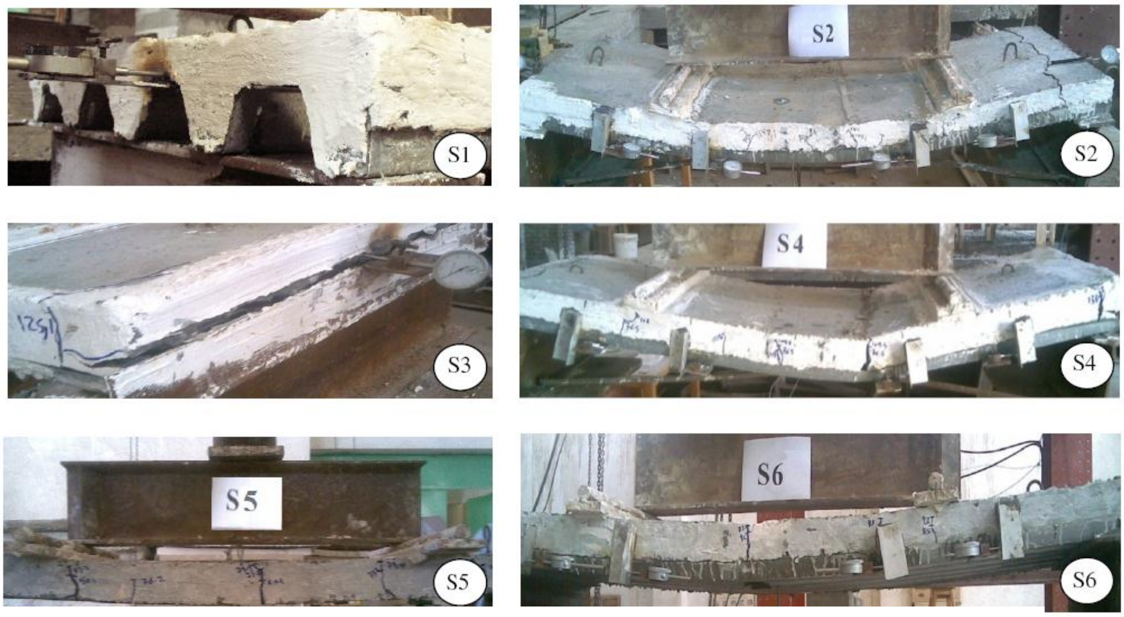
First crack load as a service load, maximum load carrying capacity, and mode of failure for specimens S1 to S6 are presented in Table 3.

Table 3: Experimental results of composite slabs with internal reinforcement mesh

Specimen No.	Mode of failure	First crack load(KN)	First crack Location	Maximum load(KN)	Maximum load Actions
S1	Shear bond	67.089	Left line load	68.889	- Middle span cracks - Sound of steel deck separation
S2	Shear bond	91.289	Right line load	153.589	- Sudden transverse crack appears parallel to the right end of the slab with about 2-3 mm width, See Fig.9
S3	Flexural	83.689	Left line load	128.589	- Sudden concrete breaking at right support - Horizontal separation in concrete slab above the plane contains horizontal shear studs at right end. See Fig.9
S4	Flexural	64.889	Left line load Slab middle	137.189	- Increasing of hair cracks at the right slab end till reaching 5mm width. - Local failure at the two ends - Negative mesh controlled cracks at specimen upper surface
S5	Shear bond	53.489	Right line load	86.789	- Sound of steel deck separation
S6	Shear bond	88.339	Left line load Right line load Slab middle	95.189	- Separation between left supporting beam and concrete end with 3mm height

Crack growth is monitored at every load increment and during the loading procdure the following actions should be highlighted;

- S1 heared a sound of steel deck separation and increasing in crack pattern in the middle of specimen when the maximum load obtained.
- S2 had started cracks of middle at the load of 99.789kN.
- S3, at a load of 83.689kN hearted a sound of concrete breaking under the right line load and a transverse crack with the whole slab width offset to the right end of about 12 cm was started. See Fig.9
- S4, at a load of 128.389kN observed inclined hair cracks at the end angles in both slab sides.
- S5, at a load of 76.589 kN the crack under left line load was began, the cracks continued at the middle of the specimen
- S6, had a totally separation between the steel deck and concrete with significant deformation in steel deck at the bottom of the left end.



S1; Left end slip at maximum load
S2; Sudden transverse crack at right end
S3; Horizontal separation above studs plan transverse crack at the right support
S4; Horizontal separation above studs plan at right end but without cracks at

Fig. 9: Cracks Pattern of Composite Slabs with Internal Reinforcement Mesh

The ratio of crack load relative to S1 (benchmark specimen) for specimens S2, S3, S4 ,S5,and S6 are 36.07 %, 24.47 %, -3.28 %,-20.27%, and 31.16 % respectively and the improvements of failure load are 122.95 %, 86.66 %, 99.14 %, 20.17%, and 38.17 % respectively relative to S1 (benchmark specimen).The crack and failure loads are as shown in Fig.10

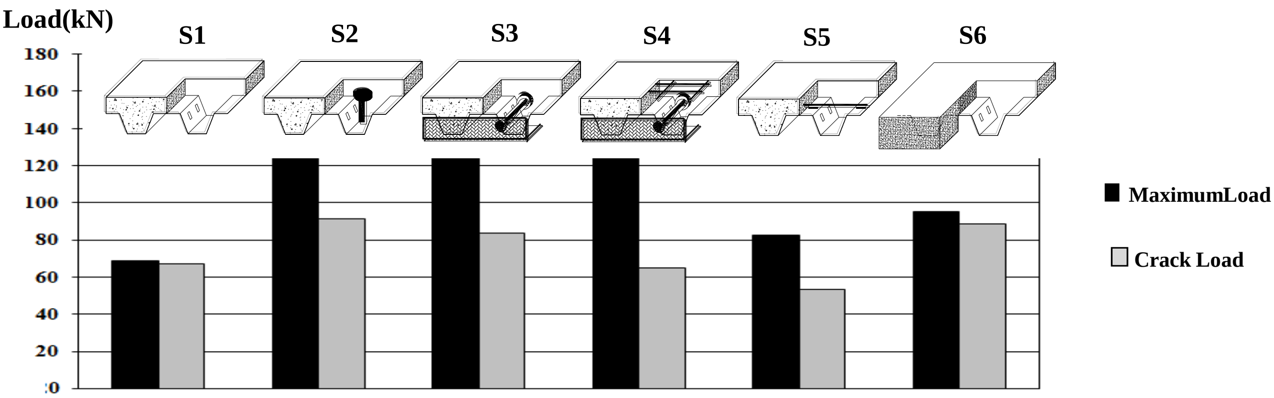


Fig. 10: Comparison between Maximum Load and Crack Load for S1 to S6

From the previous discussion, specimen with one vertical row of studs (diameter 19 mm) achieves the maximum load carrying capacity and maximum crack load. Sequentially, it may be considered as better choice of end anchorages than the other specimens.

4.1.2 Deflection and Ductility

Effect of different end anchorages on load- deflection curve and ductility is presented in this part for specimens from S1 to S6. Fig.11 shows comparison of load–mid span deflection relationships for specimens S1 to S6.

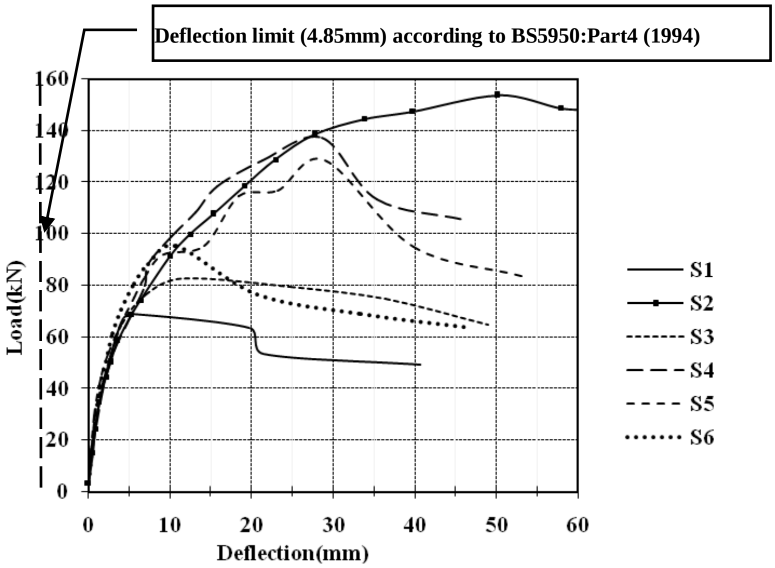


Fig.11: Load –Deflection Curve for Specimens S1 to S6

According BS5950:Part4 (1994) [2] deflection of composite slab should not normally exceed effective span/350 or 20 mm, whichever is the lesser. Therefore, the deflection allowed at service level (4.85mm). For specimen S1 to S6, the loads against this deflection were 68.08 kN, 66.21 kN, 68.29 kN, 66.07 kN, 70.75 kN, 75.8 kN respectively. Table 4 describes with details the value of deflection against first crack load, maximum load, and at first slip. It was clearly seen that the highest crack load is 91.789kN is for slab with end angle with horizontal studs.

Table 4: DEFLECTION VALUES FOR SPECIMENS S1 TO S6

Specimen No.	Deflection at first crack load		Deflection at max. load		Deflection at first slip	
	Load (kN)	Deflection(mm)	Load (kN)	Deflection (mm)	Load (kN)	Deflection (mm)
S1	67.089	4.26	68.889	5.33	68.889	5.33
S2	91.289	9.99	153.589	50.15	68.789	5.29
S3	91.789	9.10	128.589	28.94	-	-
S4	64.889	4.68	137.189	28.58	-	-
S5	53.489	2.82	82.789	11.77	65.29	4.07
S6	88.389	7.33	95.189	10.73	61.19	2.93

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From the distinctly study of the different relationships of Fig.11 and Table 4, it can be emphasis that the slab with one row vertical studs has a better behaviour and ductile curve than the others. Ductility measurements may be defined using toughness ratio of load deflection curve as follows:

$$\text{Toughness Ratio} = \frac{\text{Area under load deflection curve up to maxmuim load}}{\text{Area under load deflection curve up to end of linear zone}} \quad [8]$$

Specimen S2 with one vertical row of studs (diameter 19 mm) showed the best result and improved toughness ratio by 1255 %. The other specimens S3, S4, S5 and S6 had toughness ratio less than specimen S2 with one vertical row of studs (diameter 19 mm) but more than benchmark specimen by 675 %, 587.8 %, 550 %, and 301.4 % respectively.

Composite slab ductility classification is presented in EC4 1994-1-1:2004[1] ,the behaviour considered ductile if the failure load exceeds the load causing a recorded end slip of 0.1 mm by more than 10%. Therefore according this classification and data presented in Table 4, specimen S1 and S5 are considered Non ductile specimens and S2, S3, S4, S6 classified as ductile specimens.

4.1.3 End Slip

End slip for specimen S1, and S5 is approximately from one side and the other side without unrespectable slip because first crack began only under one line load and the steel deck is deformed under the same load. So, with continues loading process, the main crack widen and concrete part within shear span slipped relatively away from steel deck. Specimen S2 had a slip from two sides with different value of end slip. For specimen S3 and S4, it is observed that the slip was restrained at both ends due to welded end angle, so the value of slip equals zero. For specimen S6, when load reached 95.189 kN, a separation between left supporting beam and concrete end was appear with 3mm height as shown in Fig.12.The load – maximum end slip relationship is as shown in Fig.13

Fig.12: Slip at left side of specimen s6

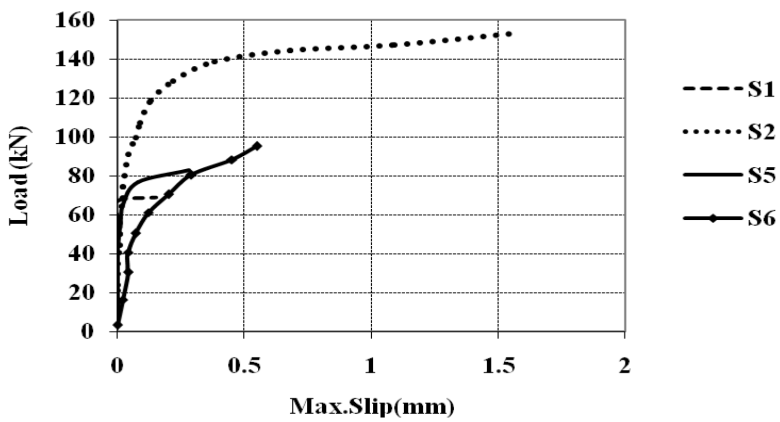

Fig.13: Load –max. slip curve for specimens s1 to s6 at maximum load

It is clear from the previous that relatively to specimen S1 and at maximum load, specimen S2 with one vertical row of studs (diameter 19 mm) had an increasing in end slip ratio by 953.3% due to existing of end anchorage. Specimen S3 with end angle attached by horizontal studs (diameter 19 mm), and Specimen S4 with end angle attached by horizontal studs (diameter19 mm) plus negative mesh as an end anchorages had no slip then the end slip is decreased by 100%. For specimen S5, with last transverse bar of steel mesh line welded to steel deck only increases end slip ratio by 86.8 %.Specimen S6 with reinforced concrete end, the end slip was increased by 266.6%.

It can be clearly noticed that the best behaviour for the above specimens is for specimen with one row vertical studs (diameter 19 mm) as an end anchorage. It has good toughness ratio, maximum load carrying capacity, and good load deflection curve than the other specimens although it had maximum end slip. Table 5 presented summary of test results for specimens S1 to S6.

TABLE 5: Summary of Results for Specimens S1 to S6

Specimen No		Experimental results					Ductility EC4 1994-1-1:2004	Significant Events
		P_{exp} (kN)	$P_{0.1\ mmS}$ (kN)	Δ (mm)	Slip $_L$ (mm)	Slip $_R$ (mm)		
S1		68.889	68.29	5.30	0.15	0.0	Non Ductile	
S2		153.589	95.539	50.15	0.99	1.58	Ductile	
S3		128.589	-	28.49	0.0	0.0	Ductile	Local end failure –No end slip
S4		137.189	-	28.58	0.0	0.0	Ductile	Local end failure –No end slip
S5		82.789	77.209	11.77	0.28	0.01	Non Ductile	
S6		95.189	56.696	10.73	0.55	0.07	Ductile	

4.2 Behaviour of Composite Slabs without Internal Reinforcement Mesh

4.2.1 Crack & Failure Load, and Mode of Failure

First crack load as a service load, maximum load carrying capacity, and mode of failure for specimens S1 to S6 are presented in Table 6.

Table 6: Experimental results of composite slabs with internal reinforcement mesh

Specimen No.	Mode of failure	First crack load(KN)	First crack Location	Maximum load(KN)	Maximum load Actions
S7	Shear bond	64.589	Right line load Slab middle	73.489	- Right side end slip
S8	Flexural	76.589	Slab middle	83.389	- Middle slab cracks increased -Inclined crack under the right line load
S9	Shear bond	54.789	Slab middle	75.589	- Welds of transverse mesh (3bars) broken at the left side

Crack growth is monitored at every load increment,see Fig14, and during the loading procdure the following actions should be highlighted;

- S7 heared a sound of steel deck separation at a load of 68.389 kN.
- S8, at the middle of the specimen observed middle slab cracks at a load of 76.889 kN.
- S9, at a load of 64.989 kN observed inclined crack under the left line load.

Fig.14:Cracks Pattern of Composite Slabs without Internal Reinforcement Mesh

First crack load as a service load for S1, S7, S8, S9 are 67.089 kN, 64.689 kN, 76.589 kN, and 54.789 kN respectively. The ratio of crack load relative to S1 (benchmark specimen) for specimens S7, S8, and S9 are -3.75%, +14.16%, and -18.33% respectively. The maximum load carrying capacities of slabs S1, S7, S8, and S9 are 68.889 kN, 73.849 kN, 83.389 kN, and 75.589 kN respectively. The improvement for load carrying capacity for S7 with last transverse bar of steel mesh line welded to steel deck is 7.19%. The other two specimens S8 [with transverse (2bar) mesh line welded to steel deck], and S9 [with transverse (3bar) mesh line welded to steel deck] had an improvements by 21.04% and 9.72% relative to the benchmark specimen S1. The crack and failure loads are as shown in Fig.15.

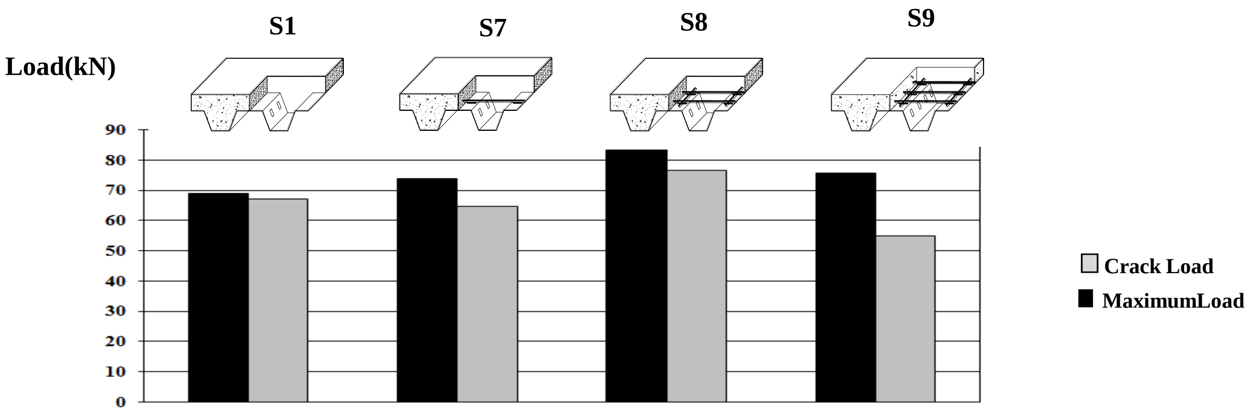

Fig.15:Comparison between Maximum Load and Crack Load for S1, S7, S8, S9

4.2.2 Deflection and Ductility

Effect of different end anchorages on load- deflection curve and ductility is presented in this part for specimens S1, S7, S8, and S9. Fig.16 shows comparison of load – mid span deflection relationships for specimens S1, S7, S8, and S9.

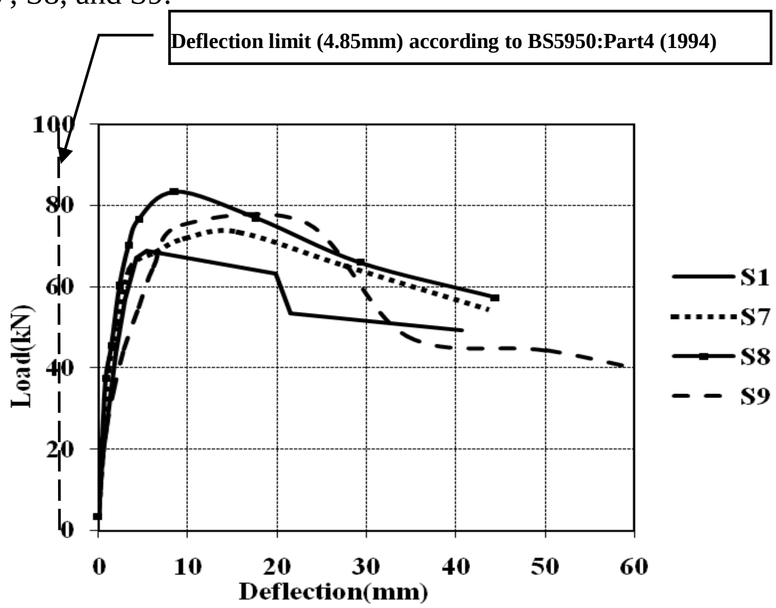

Fig.16: Load –Deflection Curve for Specimens S1, S7, S8, S9

According BS5950:Part4 (1994) [2] deflection of composite slab should not exceed (4.85mm). For specimen S1, S7, S8, and S9, the loads against this deflection were 68.08 kN, 66.45 kN, 77.13 kN, 56.95 kN respectively. Table 7 describes with details the value of deflection against first crack load, maximum load, and at first slip. It was clearly seen that the highest crack load is 76.589 kN is for slab with transverse mesh (2bars) line welded to steel deck.

Table 7: Deflection values for specimens s1, s7, s8, and s9

Specimen No.	Deflection at first crack load		Deflection at max. load		Deflection at first slip	
	Load (kN)	Deflection(mm)	Load (kN)	Deflection (mm)	Load (kN)	Deflection (mm)
S1	67.089	4.26	68.889	5.33	68.889	5.33
S7	64.589	3.48	73.489	15.71	68.389	6.35
S8	76.589	4.53	83.389	8.55	-	-
S9	54.789	4.49	75.589	23.34	75.189	9.31

Specimen S9 with transverse mesh (3 bars) spot welded to steel deck showed the best toughness ratio, then specimen S7 with last transverse bar line welded to steel deck. The lowest number for toughness ratio is for specimen S8 with transverse mesh (2 bars) spot welded to steel deck although it had no slip, and is the best in loading carrying capacity. The toughness ratio relative to S1 is with these maintained ratios -27.90%,-59.88%, and 10.31% respectively. According to EC4 1994-1-1:2004[1] for ductility classification, specimen S1 is considered Non ductile specimen and S7, S8, S9, S6 classified as ductile specimens.

4.2.3 End Slip

End slip for spesimen S7 is from two sides with different values. For spesimen S8, it is observed that there is no slip at both ends. End slip for spesimen S9 is from two sides with different values. The load – maximum slip relationship is as shown in Fig.17.

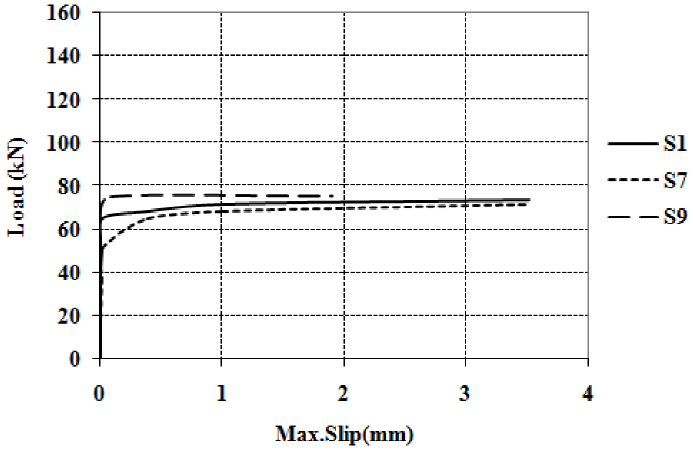

Fig.17: Load –Max. Slip Curve for Specimens S1, S7, S8, S9

It can be clearly noticed that the best behaviour for the above specimens is for specimen with transverse mesh (2 bars) spot welded to steel deck an end anchorage. It has good toughness ratio, no slip, maximum load carrying capacity, but the load deflection curve is not ductile than the other specimens. Table 8 presented summary of test results for specimens S1, S7, S8, and S9.

TABLE 8: Summary of Results for Specimens S1, S7, S8, S9

Specimen No.		Experimental results					Ductility EC4 1994-1-1:2004	Significant Events
		P_{exp} (kN)	$P_{0.1\ mm\ S}$ (kN)	Δ (mm)	Slip $_L$ (mm)	Slip $_R$ (mm)		
S1		68.889	68.29	5.30	0.15	0.0	Non Ductile	-
S7		73.489	65.1	15.71	0.45	3.52	Ductile	-
S8		83.389	-	8.55	0.0	0.0	Ductile	No Slip
S9		75.589	67.61	23.34	1.9	1.45	Ductile	-

5 Results Interpretation in Light of International Codes

The design codes used in this part to assessment the tested composite slabs are EC4 EN 1994-1-1:2004[1], BS5950-4:1994[2], and CSSBI S3-2003[3]. The calculations for moment capacity and shear-bond capacity are introduced in the light of the above maintained codes and then compared with the results obtained from the experimental testing program carried out in this research.

5.1 Moment Capacity

EC4 EN 1994-1-1:2004[1], BS5950-4:1994[2], and CSSBI S3-2003[3] design equations can only estimate moment capacity for specimens without end anchorages. BS5950-4:1994[2] present moment capacity based on plastic full connection moment which equal for S1 a value of 17073.944 kN.mm. CSSBI S3-2003[3] presented moment capacity as the value of over reinforced or under reinforced moment and after the estimation with this concept for the same specimen, it was found the under reinforced moment equal 17106.291 kN.mm. EC4 EN 1994-1-1:2004[1] estimate moment capacity by plastic full connection moment and by substitution in its equations, the plastic full connection moment equal 17727.943 kN.mm. The moment capacity for experimental tests is calculated based on the beam theory of Bernoulli and it is equal 30717.8 kN.mm for S2, 25716 kN.mm for S3, 27437.8kN.mm for S4, 16556.8 kN.mm for S5, 19037.4 kN.mm for S6, 14697.8 kN.mm for S7, 16677.8kN.mm for S8, and 15117.8kN.mm for S9 respectively.

Fig.18 shows comparison for moment capacity for specimens S1to S6 is including experimental moment capacity for these specimens and values of moment capacity according international codes.

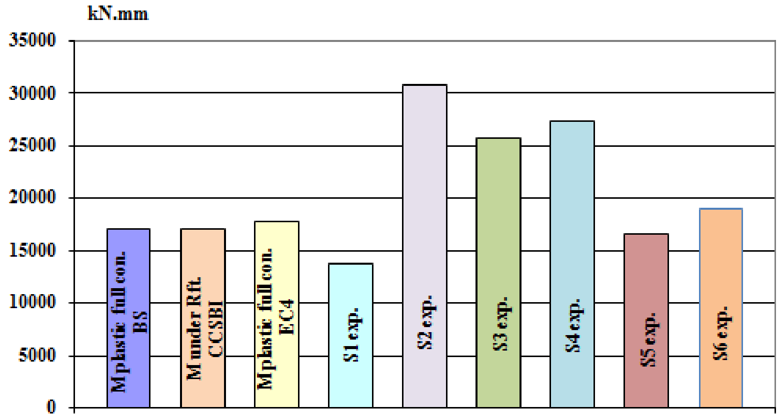

Fig.18: Moment Capacity Comparison for specimens S1to S6

Fig.19 shows comparison of moment capacity for specimens S1, S7, S8, and S9.

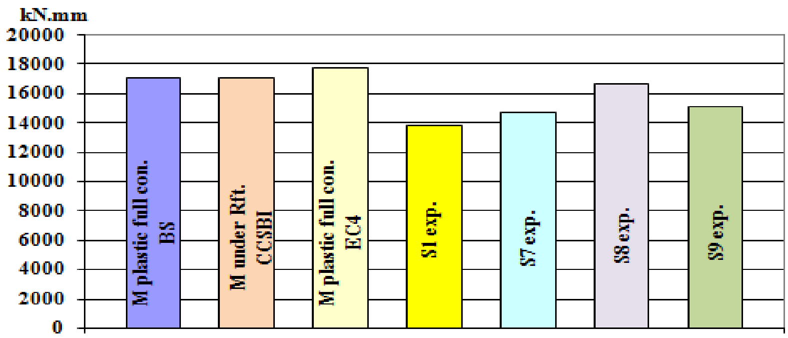

Fig.19: Moment Capacity Comparison for Specimens S1, S7, S8, S9

5.2 Shear Bond Capacity

All codes involved in this research can estimate the shear bond capacity based on m-k method which described in details in EC4 EN 1994-1-1:2004[1], BS5950-4:1994[2], and CSSBI S3-2003[3]. The only method which is applicable to estimate the contribution of end anchorage is partial shear connection method PSC which is clearly described in EC4 EN 1994-1-1:2004[1].

Shear bond capacity results of tested composite slabs are presented in Table 9.The equations used for estimating shear bond capacity are the followings for different codes:-

BS 5950-4:1994 equations;

(1) Composite slab without end anchorage;

$$V = \frac{bd_p}{1.25}\left(\frac{mA_p}{bL_v} + k\sqrt{f_{cu}}\right) \text{.........} \qquad (1)$$

(2) Composite slab with end anchorage

$$\text{.........} \qquad (2)\; V_a = NP_a\left(d_p - x/2\right)/L_v$$

When end anchorage is used in conjunction with the shear bond between the concrete and the profiled steel sheets, the combined resistance to longitudinal shear is limited as follows:

$$\text{......}\; (3)\; V_c = V + 0.5\ V_a \quad but\ V_c \le 1.5\ V$$

CSSBI S3-2003 equations;

- **Composite slab without end anchorage;**

$$V = b\, d_p\, [m/L_v + k] \text{.......} \qquad (4)$$

EC4 EN 1994-1-1:2004 equations;

(1) Composite slab without end anchorage (m-k) method

$$V = \frac{bd_p}{\gamma_v}\left(\frac{m\ A_p}{bL_v} + k\right) \text{..........}(5)$$

$$\tau_u = \frac{\eta N_{cf}}{b(L_v + L_o)} \text{..............}(6)$$

(2) Composite slab with and without end anchorage <u>(PSC) method</u>

TABLE 9: Summary of Shear Bond Capacity Results

Specimen No.	(m-k) method				PSCmethod EC4 EN 1994-1-1:2004		
	BS 5950-4:1994		CSSBI S3-2003 (N)	EC4 EN 1994 -1-1:2004 (N)	Shear bond capacity with end anchorage (N)	Mean shear stress $\tau_{u,Rd}$ N/mm^2	Contribution of end anchorage(N)
	Shear bond capacity with end anchorage (N)	Contribution of end anchorage (N)					
S1	27399.12	N.A	18451.01	19338.25	125978	0.405	-
S2	40799	13399.88	N.A	N.A	286400	0.405	160422
S3					286400	0.405	160422
S4					286400	0.405	160422
S5					217664	0.405	91686
S6	N.A				286400	0.405	160422
S7					189250	0.405	63497.5
S8					246286	0.405	120533.5
S9					197029	0.405	71276.5

The shear bond capacity calculations for specimens S1 and S2 based on m-k, and PSC methods are as shown in Fig.20.

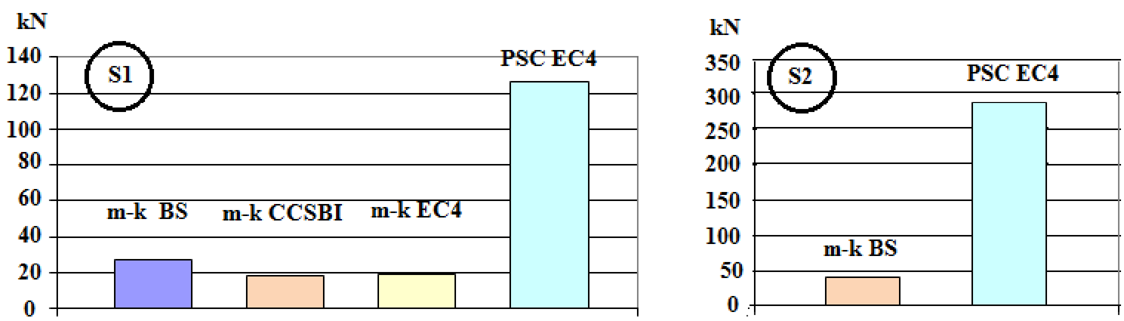

Fig.20: Shear Bond Capacity for Specimen S1, S2 according to Different Codes

It can be clearly notice that m-k method results approximately at the same range although the equations of design are different in the maintained codes of design. PSC method had value of shear bond capacity greater than values obtained from m-k method and it is represents the actual value of shear bond capacity. Fig.21 and Fig22 shows shear bond capacity and contribution of end anchorage using PSC method.

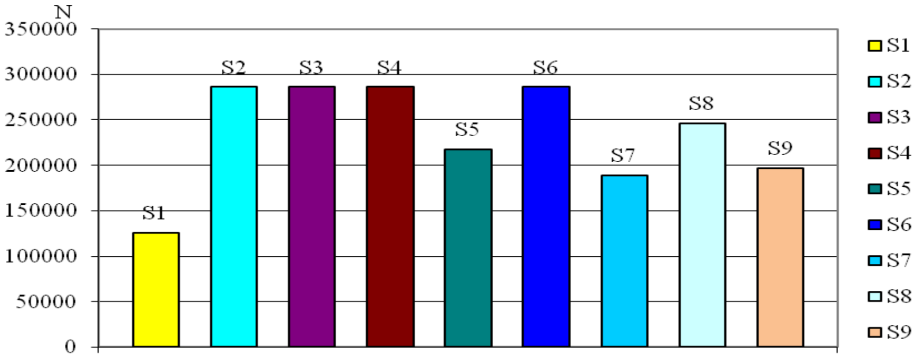

Fig.21: Shear Bond Capacity Results by PSC Method

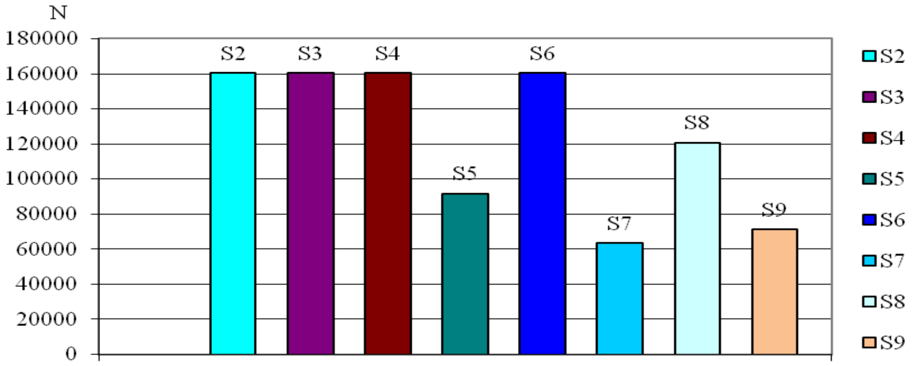

Fig.22: Contribution of End Anchorage Using PSC Method

6.Conclusions

-Using composite slab with end anchorage means increases the load carrying capacity, shear bond capacity, ductility and decreases end slip.

-End anchorage devices improved the load carrying capacity of composite slab with high strength concrete. One vertical row of studs showed the best results, then the end angle attached by horizontal studs with negative mesh then the end angle attached by horizontal studs, then the concrete end, then the last transverse bar of steel mesh line welded. Finally, benchmark specimen.

- End anchorage devices increases ductility of composite slabs except the case of (Last transverse bar of steel mesh line welded) due the brittle behaviour of high strength concrete.

- The specimens attached by end angles with horizontal studs with or without negative mesh showed the best end slip r.

- In case of composite slab without Rft mesh the load carrying capacity is lessr than the composite slab with Rft mesh.

- According BS 5950-4:1994[1], CSSBI S3-2003[9], and EC4 EN 1994-1-1:2004[2] shear bond capacity for composite slab can be estimated by full scale experimental testing results.

- (m-k) method according BS 5950-4:1994[1] can estimate shear bond capacity for composite slab without end anchorage and for composite slab with one row vertical studs only and can also estimate the contribution of the studded composite slab.

- (m-k) method according CSSBI S3-2003[9], and EC4 EN 1994-1-1:2004[2] can estimate shear bond capacity for composite slab without end anchorage only.

- Although differ between the relationships of getting (m-k) values according BS 5950-4:1994[1], CSSBI S3-2003[9], and EC4 EN 1994-1-1:2004[2], but all these results according these codes seems to be nearly similar values.

- PSC method according EC4 EN 1994-1-1:2004[2] can estimate shear bond capacity for composite slab without end anchorage and any shape of end anchorage, and we can also get the contribution of end anchorage to the hole composite slab shear bond capacity by full scale experimental tests.

- (m-k) method results are far away from the PSC method results, then the defects of (m-k) method is appearing. First, (m-k) method is not based on mechanical model so differ of materials, or loading from those used in tests needs approximate assumptions. Second, not suitable to evaluate composite slabs with ductile behaviour.

- End anchorages contribution according PSC are listed as :one row vertical studs had best results, then end angle with horizontal studs with or without (–ve) mesh, then concrete end, then last transverse bar of steel mesh spot welded. Finally, composite slabs without end anchorage.

Notations

A_p is the cross-sectional area of the steel deck
b is the width of the composite slab
d_p is the effective depth of slab to the centroid of the steel deck .
Δ is deflection (mm)
x is concrete depth in compression at mid span.
f_{cu} is the characteristic concrete cube strength
L_v is the shear span of the composite slab (for a uniformly loaded slab L_v is span/4).
L_o is the length of the overhang
k is an empirical parameter (N/mm)
m is an empirical parameter (N/mm^2)
is shear bond capacity per unit width. V
is shear bond capacity per unit width due to end anchorage. V_a
is total longitudinal shear capacity per unit width of slab. V_c
is partial safety factor for the ultimate limit state(1.25) γ_v
is mean shear stresses per unit area τ_u
is degree of shear connection η
N is number of shear connectors attached to steel deck end per unit length of beam.
P_a is end anchorage capacity per shear connector. $P_a = 0.4Q_k$
Q_k is characteristic resistance of the shear connector according BS 5950-3.1:1990.

REFERENCES

[1] EC4: European Standard ENV 1994-1-1:2004 "Design of composite steel and concrete structures".

[2] BS 5950-4:1994 Structural use of steelwork in building Part 4" Code of Practice for design of composite slabs with profiled steel sheeting".

[3] CSSBI S3-2003: Canadian sheet steel building institute" Criteria for the design composite

slabs"

[4] ACI 363R-92(State- of –the –Art Report on high strength concrete)

[5] Ahmed .M.Taha (2009), Dissertation for the degree of master in Civil Engineering: Behavior of composite slabs with different end anchorage.

[6] Johnson R.P.and and Anderson D. (1994), "Designers hand book for Eurocode 4:part1.1, composite steel and concrete structures". Stafford, London .to be published, 1994.

[7] Redzuan Abdullah (2003), Dissertation for the degree of doctor of philosophy in Civil Engineering: elemental test procedure and numerical analysis of composite slab.

[8] Mohamed A. Dabaon (2002), "Effective width of composite beam at a region of negative moment" PP: 85-109, Vol.37, No. 1, March31, 2002, scientific bulletin, Faculty of Engineering, AinShamis University, Egypt.

[9] Jolly, C. And Zubair, A. (1987). " The Efficiency of Shear-Bond Interlock Between Profiled Steel Sheeting and Concrete," Composite Steel Structures , Advances, Design, and Construction, Elsevier Science Publishing Co., Inc., pp. 127-136.

[10] Luttrell, L. D. (1987). " Flexural Strength of Composite Slabs", Composite Steel Structures ,Advances, Design and Construction, Elsevier Science Publishing Co., Inc., pp. 106-116.

[11] K.Roik, H.Bode (1993), "EC4: columns, slabs and some remarks on execution".

[12] Van der sanden, Stark j.W.B, H.H.Snijder, H.W.Bennenk (1996)"a new test for stud connector in ribbed slabs" Eindhoven University of technology.

[13] Thorsten Faust, (1997)"Lightweight Concrete in Composite Structures", Dipl.-Ing., InstitutfürMassivbau und Baustofftechnologie, Universität Leipzig. LACER No. 2, 1997.

[14] H.Bode, F.Minas (1997)," Composite slabs with and without end anchorage under static and dynamic loading ", Kaiserslautem, Germany. In: Proceedings of engineering conference composite construction, conventional and innovative.

[15] L.H. Lee, S.T. Quek, K.K. Ang (2001), “ Negative moment behavior of cold-formed steel deck and concrete composite slabs", Journal of Constructional Steel Research 57 (2001) 401–415.

[16] Sebastia A.L. de Andrade, Pedro C.G. da S. Vellasco, Jose Guilherme S. Da Silva, Tadeu H. Takey (2004), "Standardized composite slab systems for building constructions", Journal of Constructional Steel Research 494 60 (2004) 493–524.

[17] G. Marciukaitis, B. Jonaitis, J. Valivonis (2006)," Analysis of deflections of composite slabs with profiled sheeting up to the ultimate moment", Journal of Constructional Steel Research 62 (2006) 820–830.

[18] V. Marimuthu, S. Seetharaman, S. Arul Jayachandran, A. Cellophane, T.K. Bandyopadhyay, D. Dutta(2007) "Experimental studies on composite deck slabs to determine the shear-bond Characteristic (m–k) values of the embossed profiled sheet", Journal of Constructional Steel Research 63 (2007) 791–803.