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COMPARISON BETWEEN THEORETICAL AND EXPERIMENTAL INVESTIGATION OF COMPOSITE BEAMS WITH SPIRAL SHEAR CONNECTORS

by

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مقارنة بين الدراسة النظرية و العملية للكمرات المركبة مستخدماً دسر القص الحلازونية

الخلاصة:

يمتد البحث لدراسة سلوك الكمرات المركبة ليشمل المقارنة بين الدراسة النظرية والنتائج العملية لهذا النوع من الكمرات. ففي هذا البحث طريقة تحليلية لحساب مقاومة القص للدسر الحلزونية الشكل من أسياخ حديد التسليح لربط الكمرة الحديدية بالبلاطة الخرسانية المسلحة. كما تم إشتقاق معادلة جديدة لحساب مقاومة القص التصميمية للدسر الحلزونية في حدود اللدونة والمرونة. وتشمل المقارنة بين الدراسة العملية والنظرية للكمرات المركبة أحمال الإنهيار والعزوم القصوى والترخيم وكذلك حساب الإجهادات على القطاع العرضي لهذه الكمرات. كما يشمل البحث أيضاً تحليل ومناقشة معادلة المواصفات المصرية لحساب مقاومة القص للدسر المشكل من حديد التسليح المكسح بزوايا مختلفة داخل البلاطة الخرسانية مقارنةً بالتجارب العملية ومعادلة المواصفات الأوربية وإعطاء بعض التوصيات لتحسين هذه المعادلة. ولقد بينت المقارنة ككل مدى التوافق الجيد بين النتائج العملية والتحليلات النظرية لهذا النوع من الكمرات.

1. ABSTRACT

Investigation of the behavior of composite beams with different types of shear connectors (spirals and bent bars) extends to include the comparison between the theoretical study and the experimental results of such types of beams.

In this paper, a proposed technique for predicting the shear resistance of the spirals as shear connectors in the composite beams is presented. In this technique, the behavior of the connectors in the elastic and the plastic ranges is analyzed. Hence, a proposed formula to predict the design shear resistance of the spirals is suggested. The comparison between the suggested formula and the experimental results showed a good agreement between them.

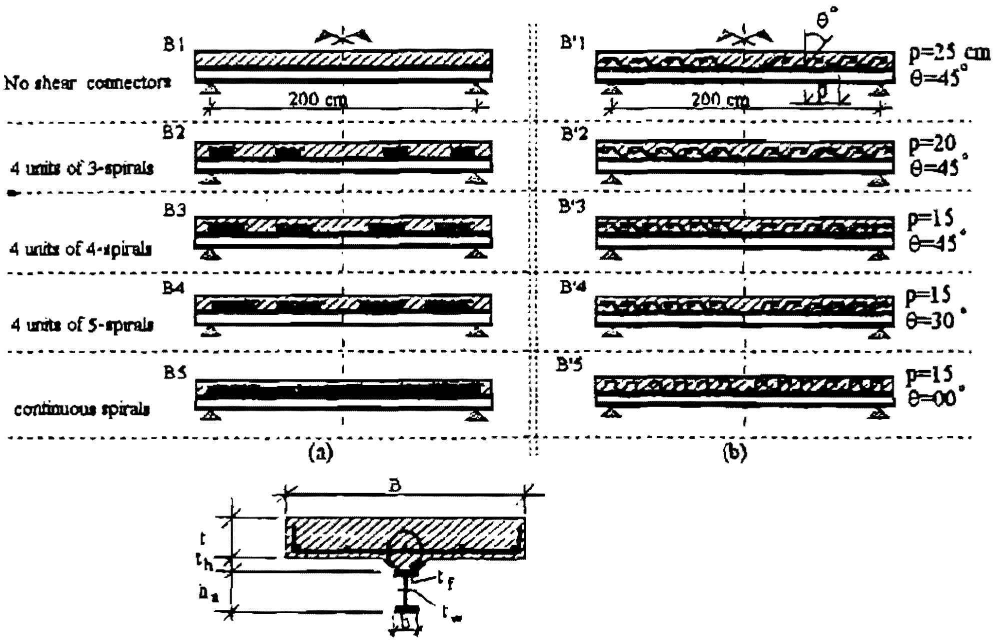
Moreover, a comparison between the theoretical and the experimental results for the tested composite beams with spirals and bent bars as shear connectors is carried out with respect to ultimate loads, ultimate moments and deflections.

The common type of shear connectors as bent bars is compared with spiral connectors as well. A discussion for the given formulae of the bent bar shear resistance in both Egyptian and European (EC4) codes together with the experimental results is conducted which may lead to the modification of the Egyptian formula.

The comparison between the present theoretical work and the experimental results has shown good agreement.

2. INTRODUCTION

Experimental investigation for the composite beams using round steel bar spirals as shear connectors has been carried out [1]. In these tests, shear connectors as spirals have been placed in four units along four composite beams. Four units of three spirals each, four units of four spirals each, four units of five spirals each and continuous spirals have been applied as shown in Fig. (1a). The experimental work for composite beams has been extended and included bent bar shear connectors having different pitches and slope angles [2]; as shown in Fig. (1b).



concrete	steel profile	reinforcement
f_{cu} = cubic strength = 2.5 kN/cm^2	f_a = yield stress = 29.93 kN/cm^2	f_y = yield stress = 30.0 kN/cm^2
f_{cb} = working stress = 0.65 kN/cm^2	f_u = tensile stress = 38.0 kN/cm^2	f_s = working stress = 15.0 kN/cm^2
B = 50.0 cm	b = 6.00 cm	5 Φ 10 in longitudinal direction
t = 10.0 cm	t_w = 0.60 cm	7 Φ 10 mm /m in cross direction
t_h = 5.00 cm	t_f = 1.00 cm	
A_c = 500 cm^2	h_a = 12.0 cm	
	$Z_{x,el}$ = 69 cm^3 , $Z_{x,pl}$ = 81 cm^3	
	A_a = 18 cm^2	

(c)

Fig. (1) Types of composite beams tested

In this study, a theoretical analysis for the experimental behavior in the elastic and ultimate ranges of the spiral shear connectors is presented. A proposed technique for predicting the shear resistance of spiral shear connector is based on the fact that the unit of spirals including the concrete inside the spiral space acts as a flexible block unit connected with the steel flange. In this technique, a formula is proposed for both maximum elastic and ultimate resistance of the shear connector appears to be in a good agreement with the experimental results. Comparison between ultimate resistance of spirals and bent bars connectors is also discussed.

Moerover, comparison between the theoretical analysis and the experimental results of tested beams (B1, B2, B3, B4 & B5) and (B'1, B'2, B'3, B'4 & B'5) with spirals and bent bar connectors is discussed. The comparison includes ultimate loads and deformations of both beams with spirals and bent bar shear connectors.

Recommendations for the modification of Egyptian formula for predicting the shear resistance of bent bars is drawn and discussed together with the experimental results and the European Code of Composite Structures [3, 2 & 4].

3. ANALYSIS OF SPIRAL SHEAR CONNECTOR

3.1 General

As is well known, the behavior of the shear connectors plays an important role on the behavior of composite beam. In other words, the interface between the reinforced concrete slab and the steel beam has a great influence on the strength of the composite beam due to the behavior of the shear connectors. Many researches and design rules are concerned with predicting the shear resistance of the shear connectors.

As a matter of fact, the behavior of a shear connector between the concrete slab and the steel beam may be described through three aspects. These aspects are the strength of the connector (full, partial or no strength), the stiffness of the interface (rigid, semi-rigid or non-rigid) and the ductility of the shear connector (ductile, semi-ductile or non-ductile); [5]. However, the rigidity of the shear connectors has a great influence on the distribution of the shear flow in a composite beam and hence it affects greatly the behavior of composite beams.

As shown in Fig. (2a), the distribution of the shearing forces of rigid non-ductile connectors along the beam interface has, more or less, the same triangular shape but with different values for both elastic and plastic stages. On the other hand, Figures (2b & 2c) show that the forces carried by the shear connectors along the beam are very nearly equal at the plastic stage [5, 6].

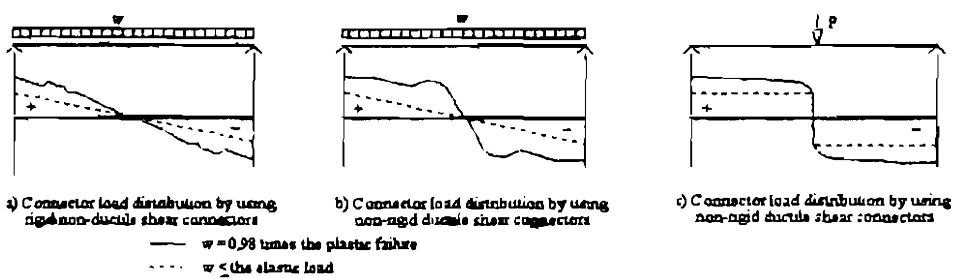


Fig. (2) Effect of rigidity of shear connectors on the shear flow

3.2 Analysis of Experimental Results of Spiral Shear Connectors

Push-out tests have been carried out for spiral shear connectors. Three units of spiral connectors are tested. These units (connectors) have 3-spirals, 4-spirals and 5-spirals as shown in Figures (3 & 4). The following analysis has been carried out for a particular spiral's circle of 8 cm diameter, 10 cm pitch and formed from a steel bar of 10 mm diameter as shown in Fig. (3).

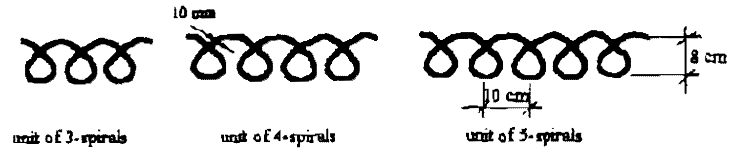
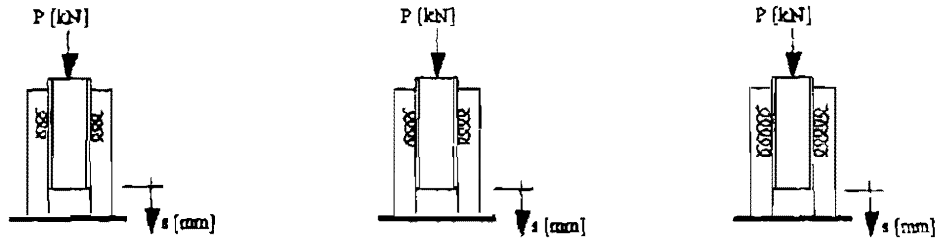


Fig. (3) Different units of shear connector



a) Push-out test for 3-spirals   b) Push-out test for 4-spirals   c) Push-out test for 5-spirals

Fig (4) Push-out tests on spiral shear connectors

Table (1) shows the maximum elastic and ultimate loads as well as the corresponding slips of the push-out tests for the different units of spiral connectors. The mean values of the loads and the slips are also tabulated.

In Fig. (5), the experimental results of the push-out tests for units of 3-spirals, 4-spirals and 5-spirals are plotted as Load-Slip relationship per spiral in the form of scattered points. Curve (a) of Fig. (5) represents the mean value of these results of the Load-Slip relationship. The mean ultimate value of the load per spiral is found to be 46.78 kN at a slip of 8.45 mm.

Table (1) Load per one spiral for the different units

Type of connector ⟹	unit of 3-spirals	unit of 4-spirals	unit of 5-spirals	**mean value/spiral** D_m and s_m
Elastic load D_{el} [kN]	36.67	41.25	35	**37.64**
Elastic slip s_{el} [mm]	3.18	4.31	3.56	**3.68**
ultimate load D_u [kN]	44.02	47.57	48.74	**46.78**
ultimate slip s_u [mm]	8.47	7.13	9.74	**8.45**

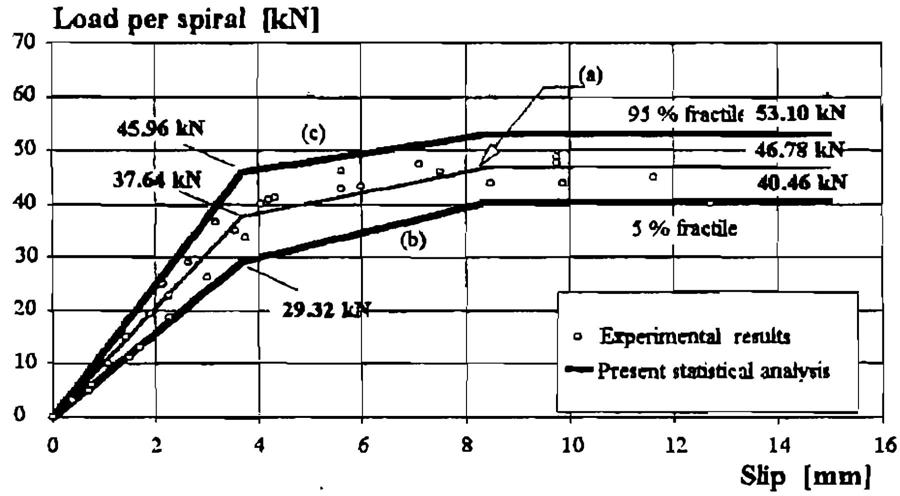


Fig. (5) Upper, lower and middle limits of load slip relationship of one spiral

According to EC4 [4], the previos value of slip at maximum load is enough to classify the spiral connectors of the above dimensions as ductile non-rigid connectors.

The above values of curves (b) and (c) in Fig. (5) are statistically calculated using 75% level of confidence. The fractile factors K_s for estimating 5% fractiles may be taken 3.15 from a background report to EC3 [7].

Thus,

$$D_{95\%,5\%} = D_m \pm K_s\ \sigma_n \qquad (1)$$

where, σ_n is the standard deviation.

Curves (b & c) are for 5% and 95% fractile, respectively. These curves may be safely used as lower and upper limits for the design loads of shear connectors.

3.3 Proposed Technique for the Shear Resistance of Spiral Connectors

According to Egyptian Code of Practice [3] and the European Code EC4 [4], the shear resistance of such a type of non-rigid connector is partly due to the bearing pressure between the connector and the concrete slab and partly due to shearing of the connector base. Also, depending on the results of the experimental programs [1, 2] which show a high shearing resistance of the spirals compared with the bent bar connectors, the spiral connector may be modelled as a short flexible block of reinforced concrete; Fig. (6). In this model, the concrete inside the spiral space and the spiral itself act together as one unit to resist both the bearing pressure and the shear at the connector base.

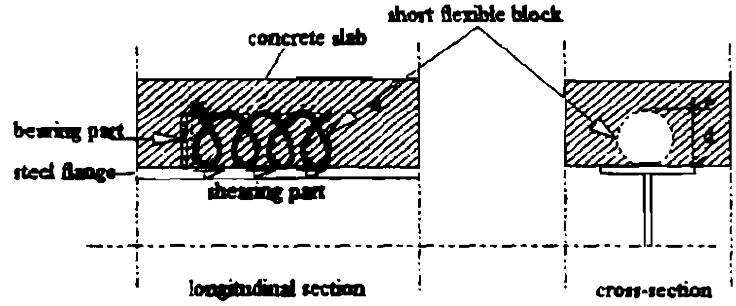


Fig. (6) Suggested model of spirals as shear connector

From Fig. (6), the ultimate design resistance of the spiral shear connector ($D_{d,u}$) is the summation of the bearing part ($D_{b,u}$) and the shearing one ($D_{s,u}$) as follows:

$$D_{d,u} = D_{b,u} + D_{s,u} \tag{2}$$

where,

$$D_{b,u} = \frac{\pi d^2}{4} \cdot \frac{0.67 f_{cu}}{\gamma_c}, \tag{2a}$$

$$D_{s,u} = N \cdot \frac{2\pi\phi^2}{4} \cdot \frac{f_y}{\gamma_s \cdot \sqrt{3}}, \tag{2b}$$

d is the diameter of spiral's circle, f_{cu} is the concrete cubic strength, γ_c is the partial safety factor of the concrete, N is the number of spirals per unit, ϕ is the diameter of spiral's bar, f_y is the yield stress of spiral's bar and γ_s is the partial safety factor of the spiral's material. γ_c and γ_s may be taken [4] 1.5 and 1.15, respectively, according to EC4 .

Previous equation (2) may be developed to obtain the design resistance in the elastic stage $D_{d,el}$ by using the working stresses of the materials used. Equation (3) which is identical in the form to the equation of non-rigid closed ring bar connector in the Egyptian Code of Practice [3], but with different model for the shear connector, is suggested for the calculation of the design elastic resistance of the spiral connector $D_{d,el}$.

Then,

$$D_{d,el} = D_{b,el} + D_{s,el} \tag{3}$$

where,

$$D_{b,el} = \frac{\pi d^2}{4} \cdot f_{cb}, \tag{3a}$$

$$D_{s,el} = N \cdot \frac{2\pi\phi^2}{4} \cdot \frac{f_s}{\sqrt{3}}, \tag{3b}$$

f_{cb} is the concrete working stress and f_s is the working stress of the spiral's bar material.

3.4 Comparison Between Experimental Results and Suggested Equations

By using equations 1, 2 and 3, the experimental and theoretical values of the ultimate and maximum elastic resistance for the different types of spiral connectors are evaluated. These are summarized in Tables (2, 3), respectively. The results are also plotted in figures (6, 7) for more clarification.

Table (2) Comparison between theoretical and experimental ultimate resistance of spirals

$D_{d,u}$ No. of spirals	Experimental limits, Eq.(1) or Fig.(5)			Theoretical limits	
	5% fractile	mean value	95 % fractile	Eq. (2) Design values	Eq. (2) with $\gamma_c=\gamma_a = 1.0$
3-spirals	121.4	140.3	159.3	127.1	165.8
4-spirals	161.8	187.1	212.4	150.8	193.0
5-spirals	202.3	233.9	265.5	174.4	220.2

Table (3) Comparison between theoretical and experimental max. elastic resistance of spirals

$D_{d,el}$ No. of spirals	Experimental limits, Eq.(1) or Fig.(5)			Theoretical limits	
	5% fractile	mean value	95 % fractile	Eq. (3) Design values	Eq. (3) with $\gamma_c=\gamma_a = 1.0$
3-spirals	088.0	112.9	137.9	73.5	114.3
4-spirals	117.3	150.6	183.8	87.1	141.5
5-spirals	146.6	188.2	229.8	100.7	168.7

Figures (6, 7) show the relationship between the number of spirals and the ultimate shear resistance and the maximum elastic shear resistance of spirals, respectively. It can be distinctly seen form the curves in Fig.(6) of the ultimate design resistance, that the scattered values of equation (2) lie under the lower limit of the experimental results. Similar comparison has been conducted by using the working stresses of the materials used to predict the maximum elastic capacity of the spiral connectors, Fig. (7). If the partial factors of safety γ_a and γ_c are omitted from equations (2) and (3), the resistance of spiral values will be within the experimental limits, Figures (6, 7).

In this sense, equation (2) may be used when the design is carried out by the ultimate limit state method while equation (3) is suitable for working stress method. However, equations (2) and (3) would be safely applied to predict the ultimate design and the maximum elastic shear resistance of spirals, respectively.

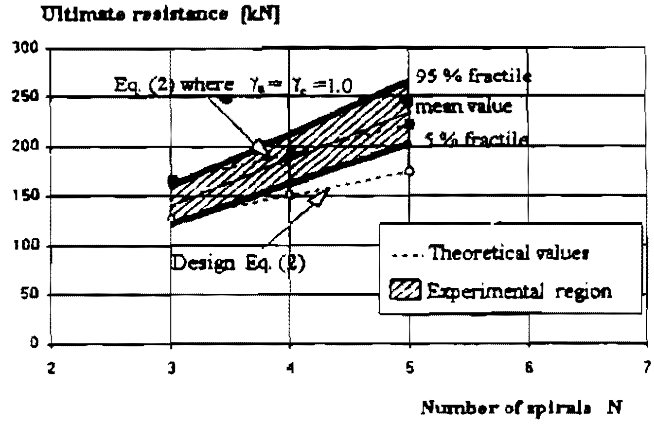


Fig. (6) Theoretical and experimental ultimate shear resistance

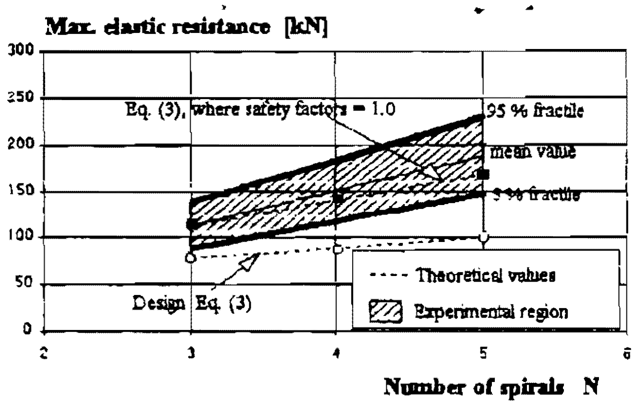


Fig. (7) Theoretical and Experimental max. elastic shear resistance

4. COMPOSITE BEAMS WITH SPIRAL AS SHEAR CONNECTORS

A typical composite beam comprises a concrete solid or hollow slab and a rolled or built -up steel profile. The composite action between the steel profile and the concrete slab is performed by the use of shear connectors.

The proposed technique for the shear resistance of spirals is applied to obtain the load-deflection relationship for the tested composite beams shown in Fig.(1). The behavior of a composite beam depends on the properties of the cross section, materials used and the behavior of the connectors. The behavior of the spiral connector may be deduced from push-out tests. Out of the push-out tests, the secant stiffness modulii K_i , [5], are given in Tables (4, 5 & 6).

$$K_i = P_i / s_i \qquad (4)$$

where, P_i is the shear resistance of the connector at slip s_i (from the push-out test).

Table (4) Connector stiffness modulii for 3-spirals unit

s_i [cm]	0.0565	0.2118	0.3176	0.4059	0.6	0.8471	1.5176
K_i [kN/cm]	354.0	354.0	347.0	296.0	217.0	156.0	66.0

Table (5) Connector stiffness modulii for 4-spirals unit

s_i [cm]	0.0523	0.4313	0.5625	0.7125	0.975	1.625
K_i [kN/cm]	383.0	383.0	329.0	267.0	195.0	111.0

Table (6) Connector stiffness modulii for 5-spirals unit

s_i [cm]	0.0305	0.3	0.42	0.5625	0.75	0.975	1.7625
K_i [kN/cm]	492.0	492.0	488.0	382.0	307.0	225.0	100.0

The stress distribution in the plastic stage on the composite cross-section is shown in Fig. (8). From this figure, two extreme cases of shear interaction can be distinguished.

The first case is the full shear connection between the steel beam and the concrete slab. In such a case, the connectors carry the least value of the plastic normal force of the concrete section or the steel section. This leads to give full plastic capacity of the composite cross-section.

The second extreme case of connection is where no connectors are provided. In this case, both the concrete slab and the steel beam deflect individually with a maximum value of relative slip between the steel beam and the concrete slab at the ends. The maximum carrying capacity of the cross-section in this case is the capacity of the steel beam only neglecting the effect of the concrete slab.

The partial shear connection is a case that falls between the last two extreme cases. In such a case, the relative slip between the steel beam and the concrete slab is controlled according to the desired strength of the cross-section and the construction requirements [5, 6 & 8].

The degree of shear connection for the composite beams is determined [5, 6], as follows:

$$\text{degree of shear connection} = \eta = \frac{D_{u,N,spirals}}{\text{Least of } F_{a,n} \text{ or } F_{c,n}} \% \qquad (5)$$

where, $D_{u,N,spirals}$ is the longitudinal shear between the considered cross-section with a sagging bending moment and a simple end.

According to the previous proposed technique (Eq. 2), the ultimate shearing resistance of the different types of spiral shear connector (3-spirals unit, 4-spirals unit, 5-spirals unit & continuous spiral) is determined. Composite beams, with 10 cm of spiral's circle shear connectors, are classified according to their degree of shear connection by using the previous proposed technique; (see Table (7)).

Table (7) Classification of the beam shear connection

Beam	No. of spirals per connector	D_{du} per connector Eq. (2) [kN]	No. of connectors on half span	η % Eq. (5)	Type of shear connection
B1	free	00.00	0	00 %	no
B2	3-spirals	213.18	2	79.1 %	partial
B3	4-spirals	240.38	2	89.2 %	partial
B4	5-spirals	267.59	2	99.3 %	partial
B5	continuous	294.80	continuous	109.4 %	full

Refering to Table 7, the plastic capacities of the composite beam cross-section with and without shear connectors have been calculated as follows:

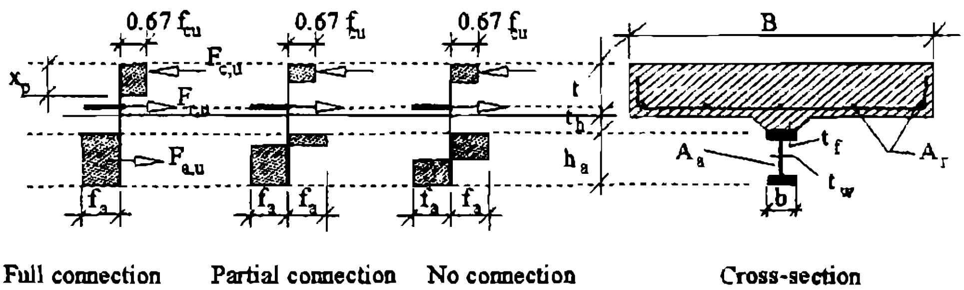


Fig. (8) Stress distribution of composite cross-section at plastic stage

Case of no- shear connectors:

The plastic moment $M_{th,u,0\%}$ of beam B1 is calculated as follows:

$B \,.\, x_p \,.\, 0.67 \,.\, f_{cu} \quad = A_r \,.\, f_{yr}$

$x_p \quad = 1.406$ cm

Plastic moment of the slab $M_{c,u}$ $= B \,.\, x_p \,.\, 0.67 \,.\, f_{cu} \,.\, (t \text{ -cover} - 0.5 \,.\, x_p)$

$= 918.11$ kNcm

Plastic moment of the steel profile $M_{a,u}$ $= Z_p \,.\, f_a$

$= 2424.33$ kNcm

Plastic moment of the composite section $M_{th,u,0\%}$ $= M_{c,u} + M_{a,u} = 3342.44$ kNcm

If the ultimate tensile stress (38 kN/cm^2) is used instead of the yield stress (29.93 kN/cm^2), $M_{th,u,0\%}$ will be 3996.11 kNcm.

Case of full- shear connection:

Beam B5 has full shear connection as well be described later in Table (8) and the plastic moment $M_{th,u,100\%}$ is obtained as follows:

The total plastic force in the concrete slab $F_{c,u}$ $= A_c \,.\, 0.67 \,.\, f_{cu}$ $= 837.50$ kN

The total plastic force in the steel section $F_{s,u}$ $= A_s \,.\, f_s$ $= 538.74$ kN

The total plastic force in the reinforcement $F_{r,u}$ $= A_r \,.\, f_{yr}$ $= 117.75$ kN

$F_{c,u} > F_{s,u}$, then the plastic neutral axis lies within the concrete slab and the plasticity of the cross section is governed by the yielding of the steel profile.

$$B \,.\, x_p \,.\, 0.67 \,.\, f_{cu} = A_r \,.\, f_{yr} + A_s \,.\, f_s$$

$$x_p = 7.839 \text{ cm}$$

The actual total plastic force in the concrete $F_{c,u}$ $= B \,.\, x_p \,.\, 0.67 \,.\, f_{cu}$ $= 656.51$ kN

$$M_{th,u,100\%} = F_{c,u} \,.\, 0.5 \,.\, x_p + F_{r,u} \,.\, (t\text{-cover-}x_p) + F_{s,u} \,.\, (t+t_b+0.5 \,.\, h_s - x_p)$$

$$= 9741.22 \text{ kNcm}$$

Case of partial- shear connection:

The method of calculation of the theoretical plastic moment of B2 depends on the partial-interaction theory of composite beams [5]. A linear formula (6) is given in [4, 5] which determines the partial plastic moment $M_{th,u,\eta\%}$ of a composite cross-section with a certain degree of shear connection η.

$$M_{th,u,\eta\%} = M_{th,u,0\%} + \eta \,.\, (M_{th,u,100\%} - M_{th,u,0\%}) \qquad (6)$$

$$= 8403.88 \text{ kNcm.}$$

It can be easily observed from figures. (9, 10) that the theoretical curve (a), using partial-interaction theory [5], gives lower values than the experimental results, while Fig. (11) gives close theoretical and experimental results in which the shear connection is of the full type. However, all theoretical values are on the safe side. The calculation of the deflection by using the method of elastic rigid shear connection, curve (b), gives smaller amounts of deflection than the experimental results.

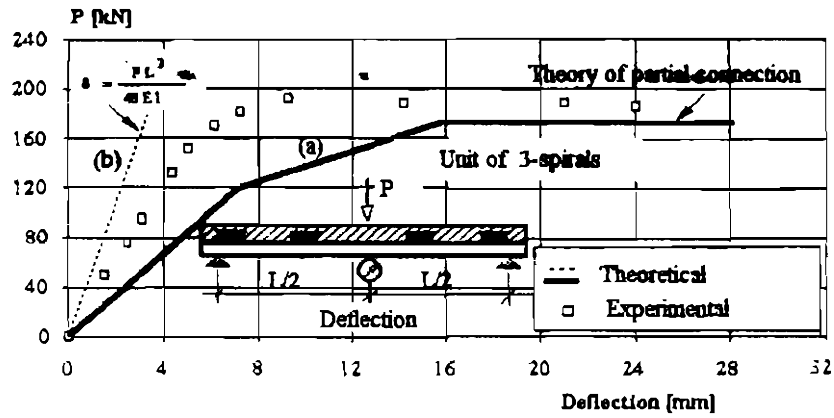


Fig. (9) Load-deflection relationship of composite beam (B2)

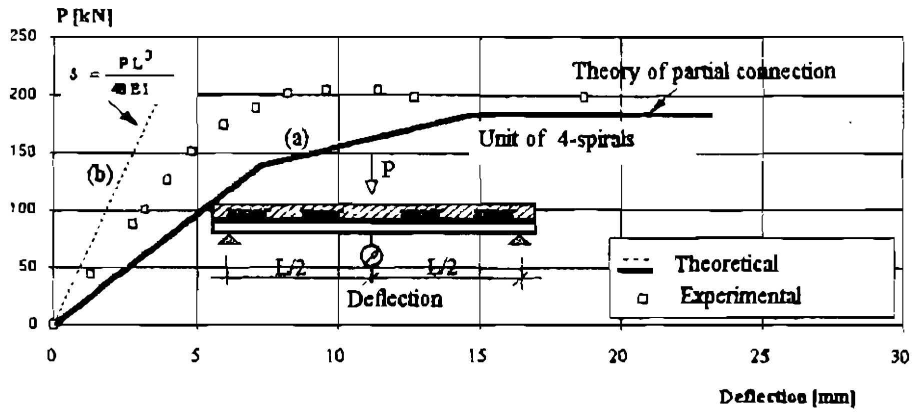


Fig. (10) Load-deflection relationship of composite beam (B3)

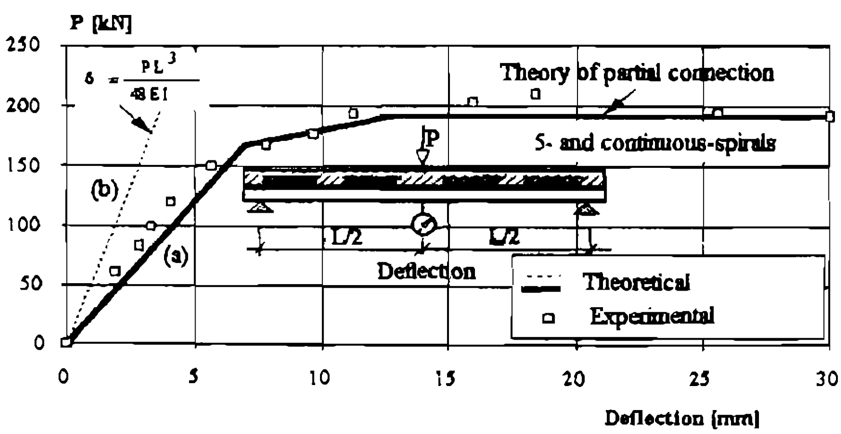


Fig. (11) Load-deflection relationship of composite beams (B4 & B5)

Table (8) Ultimate capacity of composite beams with spiral connectors

Beam	*η %	No. of spirals per unit	Experimental		Theoretical		$\frac{M_{exp,u}}{M_{th,u}}$
			$P_{exp,u}$ [kN]	$M_{exp,u}$ * [kNcm]	$P_{th,u}$ [kN]	$M_{th,u}^{*}$ [kNcm]	%
B1	00.0 %	free	078.6	03930.0	066.85	3342.50	117.6 %
			078.6	03930.0	079.92	3996.11	98.3 %*
B2	79.1 %	3-spirals	191.8	09590.0	168.08	8403.88	114.1 %
B3	89.2 %	4-spirals	204.4	10220.0	181.00	9050.15	112.9 %
B4	99.3 %	5-spirals	210.7	10535.0	193.93	9696.43	108.6 %
B5	109.4 %	continuous	223.3	11165.0	194.82	9741.22	114.6 %

* In case of no shear connection, the ultimate tensile strength of steel is used.

Table (8) shows a comparison between the theoretical results (depending on the previous technique given in 3.2) and the experimental ultimate moments and loads. The theoretical values are reasonably compatible with the test results. However, all the calculated values of the section carrying capacity are on the safe side.

5. COMPOSITE BEAMS WITH BENT BAR SHEAR CONNECTORS

Composite beams with bent bars as connectors are widely used in construction. The bent bars are already specified in many codes of practice as well as in the Egyptian Code [3]. It is noticeable that the formula of calculating the shear resistance of bent bars is stated in EC4 with a different form than that given in the Egyptian Code [3, 4]. The discussion of both codes in, such a point, would be useful for the development of the Egyptian Code formula for the shear connectors. Moreover, the experiments of the push-out tests and on the composite beams with bent bars will support this discussion and will be compared with alternative cases with spiral connectors.

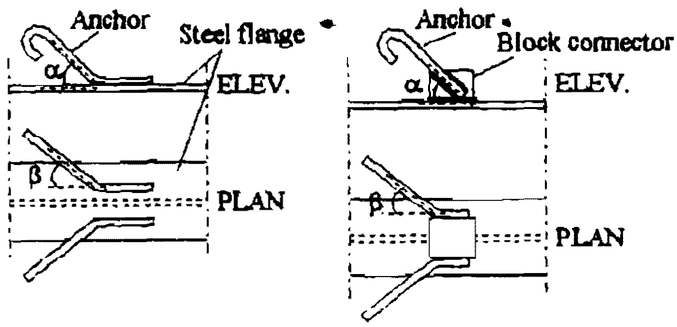
The shear resistance of bent bars may be calculated from Egyptian code [3] as follows:

Fig. (12) Example of bent bar [3]

$$D = \mu \cdot f_s \cdot A_r \cdot \cos\alpha \qquad (7)$$

where, μ is a coefficient equal to 0.5 for hooked bars and equal to 0.7 for closed rings, f_s is the allowable tensile stress in steel, A_r is the cross sectional area of the steel connecting bar and α is the angle of inclination between the steel bar and the top flange of the beam.

The same value of the shear resistance D may be calculated from EC4 [4] as follows:



Use equation (7) Use equation (8)

Fig. (13) Example of bent bar [4]

$$D = \frac{A_r\, f_{yd}}{\sqrt{1 + \sin^2\alpha}} \cos\beta \tag{8}$$

$$D\ \text{combined} = D\ \text{block} + \mu \, . \, D \tag{9}$$

where,

μ is a coefficient equal to 0.5 for hooked bars and equal to 0.7 for closed rings, α is the angle between the anchor bar and the plane of the flange of the beam, β is the angle in the horizontal plane between the anchor bar and the longitudinal axis of the beam as shown above, f_{yd} is the design strength of the material of the bar.

By using the maximum stress of the steel bar material without the factor of safety to compare the calculated values with the experimental ones, the ultimate tensile strength of the steel bars would be taken 40 kN/cm².

Table 9 Ultimate resistance of bent bars

Slop angle α°	Theoretical resistance D Eq. (7) Egyptian code [kN]	Eq. (8) European code [kN]	D_{Exp} [kN]	D_{Exp}/D_{th} Egyptian code Eq. (7)	D_{Exp}/D_{th} European code Eq. (8)	D_{Exp}/D_{th} Recommended Eq. (7) $\mu = 1.0$
45°	0.5 . 22.21	25.65	26.65	240 %	104 %	120 %
45°	0.5 . 22.21			240 %		120 % **
60°	0.5 . 15.71	23.75	26.25	334 %	111 %	167 %
30°	0.5 . 27.21			193 %		96.5 % **
90°	0.0	22.21	34.60	∞ %	156 %	∞ %
00°	0.5 . 31.42			220 %		110 % **

**α° is the suggested angle between the bent bar and the perpendicular plane of the flange of the beam for the Egyptian formula only.

Table (9) shows the ratios between the experimental tests [2] and both of Egyptian and European codes results. It can be observed that the factor μ is used only, according to EC4, where the weld of the bent bar (anchor) is not direct on the steel flange.

There are three comments on the Egyptian code of practice concerning bent bars. The first one is concerned with the angle on the horizontal plane between the anchor bar and the steel flange β. Although the effect of the angle β is important, yet it is not taken into consideration. The second comment is about the factor μ which gives very conservative values of the shear resistance compared with the experimental and European code results [4]. The last comment refers to the angle of inclination (α) between the bent bar and the top flange of the beam. With respect to the last comment, formula (7) gives zero shear resistance for the vertical bar (α=90°) which is not realistic.

From table (9), the coefficient μ may be increased by a value between 80 % to 120 % and the angle α becomes with the vertical direction (90° - α) in the Egyptian formula, it can be noticed that a reliable ultimate shear resistance of bent bars is obtained which agrees with the experimental results.

Results of the tested composite beams (B'1, B'2, B'3, B'4 & B'5) are analyzed and compared with the theoretical values. Table (10) shows this comparison taking into account the degree of shear connection which has a great influence on the theoretical ultimate values.

From Table (10), it can be noticed that the degree of shear connection is very low and the experimental ultimate moments are higher than the corresponding theoretical ones. Regarding the relatively large differences between the theoretical and the experimental values of the ultimate moments a considerable redistribution of stresses could obviously occur in the tested beams with the smallest degree of shear connection. This justification has been also mentioned in [8] for small tested models with low degree of shear connection.

Table (10) Ultimate capacity of composite beams with bent bars connectors

Beam	η %	Type of connection	Experimental		Theoretical		$M_{exp,u}$ / $M_{th,u}$
			$P_{exp,u}$ [kN]	$M_{exp,u}$ [kNcm]	$P_{th,u}$ [kN]	$M_{th,u}$ [kNcm]	%
B'1	20 %	θ = 45° p = 25 cm	157.2	07860.0	092.4	4620.0	170 %
B'2	25 %	θ = 45° p = 20 cm	163.5	08175.0	98.8	4940.0	165 %
B'3	30 %	θ = 45° p = 15 cm	167.0	08350.0	105.2	5260.0	159 %
B'4	29 %	θ = 30° p = 15 cm	210.7	10535.0	104.0	5200.0	203 %
B'5	39 %	θ = 00° p = 15 cm	170.0	08500.0	116.8	5840.0	146 %

6. CONCLUSIONS

From the results and discussions presented in this work, the following conclusions can be drawn:

1. The paper presents a method of analysis of spiral shear connectors where the push-out results present by upper and lower limits of ultimate and maximum elastic resistance of the spiral shear connectors. In addition, a proposed technique introduces to an explicit formula from which the ultimate and the maximum elastic shear resistance of any type of spiral shear connector can be obtained with reasonable accuracy.

2. A comparison between the theoretical and experimental results for composite beams with spiral shear connectors, using the present formula of spiral shear connectors, shows a good agreement.

3. A comparison between the Egyptian formula [3] and European formula [4], for predicting the shear resistance of bent bar shear connectors, highlights the importance of the factor μ in Egyptian formula. If this factor equals (1.0), the results of this formula would be compatible with push-out results . Also, the angle α in formula (7) may be taken $(90 - \alpha)$ as recommended in this paper.

4. The spiral shear connectors are much applicable and give higher shear resistance than bent bar connectors in composite beam construction.

5. Reduced scale testing, for composite beams with partial shear connection, suffers from scale effects and does not present of full scale performance.

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