Journal of Engineering Sciences, Assiut University, Vol. 35, No. 3 pp. 617-633, May 2007

EFFECT OF CONFINING METHOD ON THE DUCTILITY OF OVER-REINFORCED CONCRETE BEAMS

M. M. Ahmed, O. A. Farghal,
Ass. Prof. Dr of Civil Eng. Faculty of Eng. Assiut University, Assiut, Egypt.

A. K. Nagah and
Lect. Dr of Civil Eng. Faculty of Eng. Al-Azhar University, Qena, Egypt.

A. A. Haridy
Demonstrator in Civil Eng. Faculty of Eng. Al-Azhar University, Qena, Egypt.

(Received March 29, 2007 Accepted April 16, 2007)

Most design codes limit the amount of tensile reinforcement in beams to avoid the brittle failure. However, sometimes a high percentage of steel reinforcement is used in order to minimize structural depth and still provide adequate stiffness. The objective of this work is to investigate and evaluate the methods of improving the behavior of over-reinforced beams. The effect of different types of techniques on the enhancement of strength and ductility of such beams was presented. An experimental and theoretical study of the behavior of fourteen over-reinforced, either internally confined or externally plated C. beams with 240 cm length and a cross-section of 15 x 23cm were carried out. Variables such as helix pitch, helix diameter, concrete compressive strength, longitudinal reinforcement ratio and confining with steel plate were considered. The results were discussed, analyzed and compared with those obtained theoretically. The results indicated the contribution of proposed techniques to the structural ductility for improving behaviour of such over-reinforced concrete beams. Finally some valuable conclusions and recommendations were given.

***KEYWORDS**: over-reinforced, confining, helically reinforced, ductility, plated beams.*

INTRODUCTION

The brittleness of reinforced concrete members increases with the use of high percentages of longitudinal reinforcement. Over-reinforced sections fail suddenly by crushing of the compression concrete when their ultimate compressive strain has been exceeded, while the strain in the longitudinal reinforcement has not reached yield. The limited extent of the deflection and cracking found in over-reinforced beams gives insufficient warning of impending failure. At present, in order to avoid brittle compression failures, codes of practice sensibly prohibit the use of over-reinforced sections [1].

Previous researches had shown that the ductility and flexural response of over-reinforced and prestressed concrete beams can be enhanced by the use of full-depth rectangular steel wire helical reinforcement [2,3]. However, circular helical reinforcement, located entirely above the longitudinal reinforcement and enveloping the entire compression zone, provides greater confinement [4,5].

Whitehead and Ibell [6] concluded that, by placing a steel helix of 3 or 4.8mm wire diameter in the compression zone of heavily over-reinforced concrete beam considerable ductility has been achieved, even when using a longitudinal steel percentage of about 7%. Providing a longitudinal compression reinforcement or using randomly oriented steel fibers or by installing rectangular stirrups in the compression zone which restrains the lateral expansion enhance the strength and ductility of over-reinforced beams. [5, 6, 7, 8, 9, 10, 11, 12]

However most of the previous works have dealt mainly with the internal confinement in the compression zone of these members, especially those of high strength concrete. Few available researches have studied the effect of internal confinement with long and short stirrups together or the external confinement in case of the existing members such as glueded and bolted steel plates to the top surface of the beams.

This paper presents an experimental study to investigate the contribution of some proposed techniques for improving behavior of such over-reinforced concrete beams. The test program includes two main parts: The first one is that related to some precautions provided internally during casting the beams by means of providing either steel helix or short rectangular stirrups. The second one deals with that provided to the external surface of the beams by gluing and bolting steel plates to the top surface of the beams.

EXPERIMENTAL PROGRAM

Details of the tested beams

Fourteen rectangular beams were tested in this work. All the tested beams have the same concrete dimensions. Beams of series A, B and C were reinforced with six bottom bars of 16 mm in diameter (A_s), two top bars of 10 mm in diameter and steel helix of 13 cm helical diameter with different values of wire diameter and pitch placed in concrete compression zone. Beams of series D and E provided with short rectangular stirrups instead of steel helix. Beams of series G provided with upper steel plates. Tables (1) as well as Fig (1) show the details of the main parameters considered in this investigation.

Materials

All beams were made using normal concrete having small range of variable concrete compressive strength, see table (1). The used concrete was made from Ordinary Portland Cement, local sand which has specific weight, bulk density and fineness modulus of 2.5, 1700 kg/m^3 and 2.43 respectively and Gravel of 20 mm maximum nominal size. The water cement ratio w/c was 0.55 for all batches. The high tensile steel with about 4370 kg/cm^2 proof stress, was used as main reinforcement, while the steel used as stirrups was mild steel of about 3222 kg/cm^2 yield strength, and for helical steel f_y was 3890, 6360 and 5090 kg/cm^2 for Φ6, Φ4 and Φ3 respectively.

Preparation of test specimens and Test procedure

The concrete was batched in the laboratory using a pan mixer. Control specimens including cubes of 15 cm side length were cast from each batch. The concrete was placed by hand in steel forms and compacted using a 2.5 cm diameter electric vibrator. The tested beams and the corresponding control specimens were tested in the same day after 28 days from casting. All the beams were simply supported and the load was applied to the beams through two points as shown in Fig (1). The loading was applied in increments of 0.5 ton. The crack pattern, cracking load, mid-span deflection, failure load and strains in longitudinal steel at mid-span were measured and recorded.

Table (1): Details of tested beams

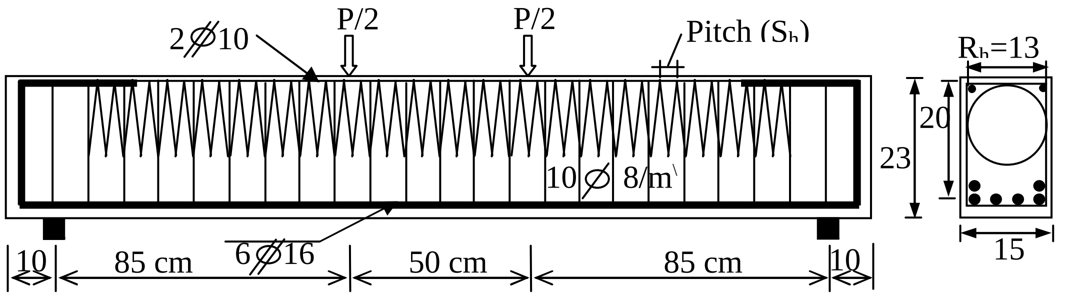
Series	Beam No.	f_{cu} Kg/cm^2	ϕ mm	S (cm)	ρ (%)	ρ_{max} (%)	Area of plates A_p (cm^2)	Type of technique
A	Bsp1	330	6	4	4.23	1.42	-	Steel helix
	Bsp3	265	6	4	4.23	1.14	-	
	Bsp2	225	6	4	4.23	0.97	-	
B	Bsp4	295	4	4	4.23	1.27	-	
	Bsp5	265	3	4	4.23	1.14	-	
C	Bsp6	215	3	3	4.23	0.93	-	
	Bsp7	215	3	2	4.23	0.93	-	
D	Bst1	265	-	10	2.82	1.14	-	Short stirrups
	Bst2	220	-	10	4.23	0.95	-	
	Bst3	265	-	10	5.64	1.14	-	
E	Bst4	265	-	-	4.23	1.14	-	-
	Bst5	265	-	5	4.23	1.14	-	Short stirrups
G	Bst6	265	-	-	2.82	1.14	12x0.6	steel plates
	Bst7	265	-	10	2.82	1.14	12x0.3	

S : pitch of the spiral or spacing between short stirrups.
ϕ : diameter of helical reinforcement.

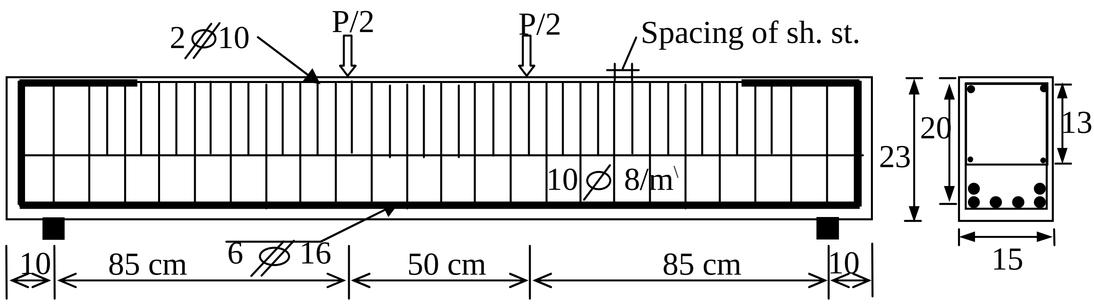
TEST RESULTS AND DISCUSSION

Ductility

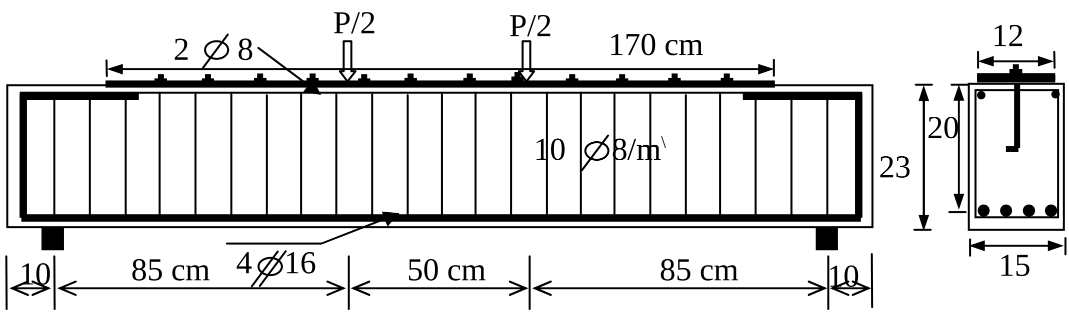
The displacement ductility index [μ_D] can be measured as the ratio between maximum deflection [Δ_{max}] (corresponding to 90% of the maximum recorded load capacity) and the deflection corresponding to cracking load [Δ_{cr}]. The ductility index ratio [R] is the ductility index of the different tested beams compared with the corresponding reference beam Bst4, table (2). It emerged that, both confined and plated beams showed an acceptable increased ductility in comparison with that of the corresponding reference beam Bst4, particularly in case of beams having higher concrete compressive strength and that having biggest wire diameter of spiral (Bsp1, Bsp4 and Bsp3).



a) Reinforcement details for beams of series A, B and C



b) Reinforcement details for beams of series D and E



c) Reinforcement details for beams of series G

Fig (1): Details of the tested beams

For helically confined beams of series B, as the wire diameter of the spiral increased the ductility index increased, see Fig (2). The ductility index for beams Bsp4 and Bsp5 was 108.2% and 141.8% of beam Bsp3 respectively. The increase in ductility index for beam Bsp3 having 6mm wire diameter of spiral was relatively smaller compared with the increase in Bsp4 with 4mm in wire diameter of spiral due to the smaller tensile strength of the confining steel Ø6mm. Fig (3) shows that, as the spiral pitch decreases within the relatively small range (from 4 to 2cm) the ductility index slightly increases. The ductility index for beams Bsp6 and Bsp7 was 104.7% and 107% of that of beam Bsp5 respectively. This result indicates again the fact that the increase in volume of helical reinforcement [V_{sp}] within the studied range can increase the ductility index. On the other side, as the concrete compressive strength increases the ductility index increases, as shown in Fig (4), where [μ_D] for beams Bsp1 and Bsp3 was 157% and 128%, of beam Bsp2 respectively

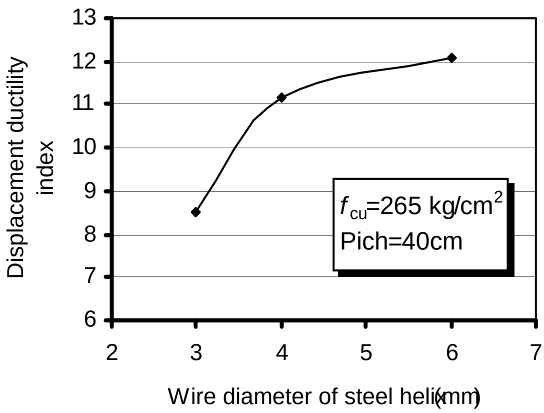


Fig. (2): Effect of values of spiral diameter on ductility.

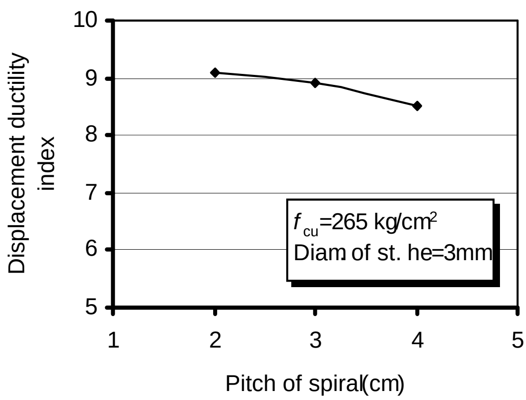


Fig. (3): Effect of values of spiral pitch on ductility.

For beams confined with short stirrups, Fig (5) a reduction in ductility occurs with increasing the ratio of (A_s/A_c%). The ductility index for beams Bst2 and Bst3 was 96% and 93% of that of beam Bst1 respectively. This indicates that the effect of the same confining reinforcement decreases as the percentage of longitudinal reinforcement increases. Furthermore, the ductility index for beams Bst2 and Bst5 having 10cm and 5cm spacing between short stirrups was 134% and 163%, respectively of that of reference beam Bst4. This result confirmed the benefit of providing the over reinforced concrete beams with short stirrups in compression zone as they increase the ductility index with decreasing the spacing between the short stirrups.

From table (2), it can be concluded also that concerning ductility the effectiveness of confining with steel helix is better than that of confining with short rectangular stirrups. This can be attributed to the fact that, helices apply a uniform radial stress to the concrete along the concrete member, while short stirrups tend to confine the concrete mainly at the corners. Moreover, it is obvious that, for beam Bst7 provided with steel plates, the ductility index was 116% of that of reference beams Bst1. The technique of providing steel plates on the top compression side seems to have a significant effect on the ductility index, provided that, the plates must be good fastened to the upper side. Furthermore, the ductility index for beam Bst6 in which the short stirrups were replaced by additional another steel plate was 105% of that of beam Bst7 having short stirrups. From the previous result, it is emerged that using additional steel plate instead of short rectangular stirrups gave a slightly better ductility index.

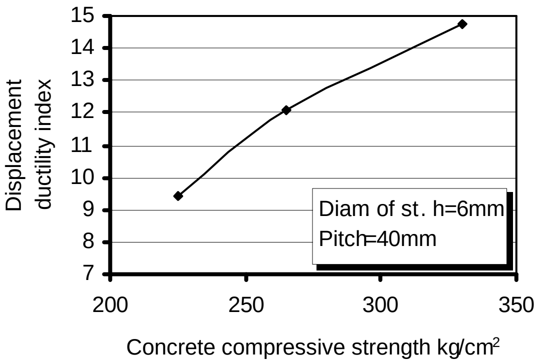


Fig. (4): Effect of (f_{cu}) on ductility.

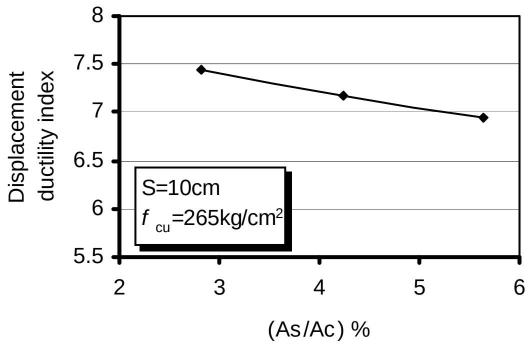


Fig. (5): Effect of (ρ%) ratio on ductility

Load Deflection Diagrams.

The results shows that confining the concrete in compression zone with steel helix or short stirrups or glued steel plates on to top face of over reinforced beams improve their ductility. The flat Plato of the curves for these beams showed a considerable increase. The maximum deflection decreases slightly as the concrete compressive strength increases. The maximum deflection for beams Bsp3 and Bsp1 was 90.3% and 120% of that of beam Bsp2 respectively, see Fig (6). On the other hand, the ultimate mid-span deflection decreases as the volume of helical reinforcement decreases, as shown in Fig (8). The deflection for beam Bsp6 at loads greater than $0.8P_u$ is bigger than the corresponding value of the reference beam Bst4. This can be attributed to the fact that the effect of confining is more activated at higher load levels.

The maximum deflection for beams Bst7 provided with steel plates was 195% of that of reference beams Bst1 without steel plates. Furthermore the maximum deflection for beam Bst6 having steel plate with cross section A_p=720 mm^2 and without short rectangular stirrups was more than that of beam Bst7 having A_p=360 mm^2 and with short rectangular stirrups, see table (2). This result confirmed again that replacing the short rectangular stirrups with steel plates gives a better ductility.

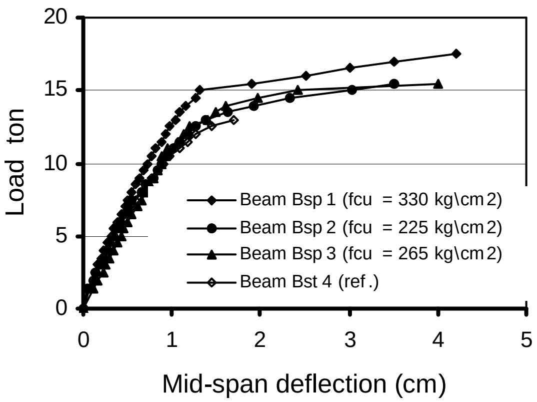


Fig. (6): Deflection curves for beams with different values of f_{cu}.

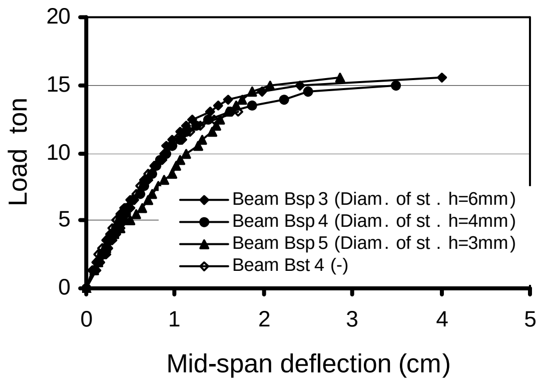


Fig. (7): Deflection curves for beams with different wire diameter.

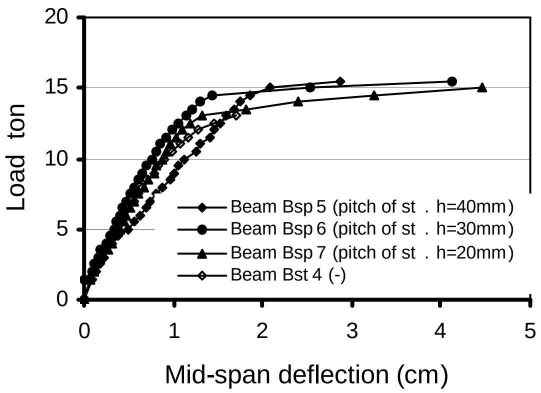


Fig. (8): Deflection curves for beams with different values of spiral pitch.

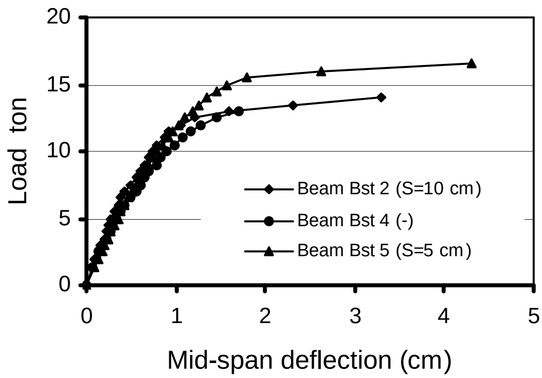


Fig. (9): Deflection curves for beams with different values of (ρ%).

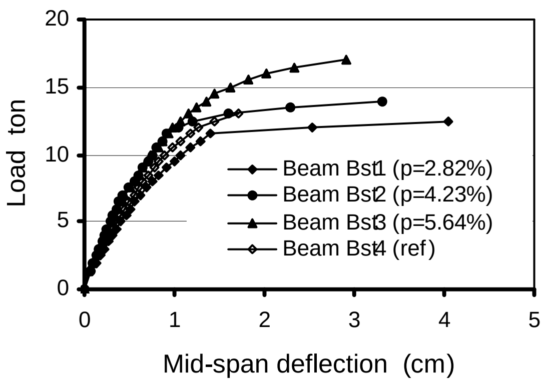


Fig. (10): Deflection curves for beams with different short stirrups space.

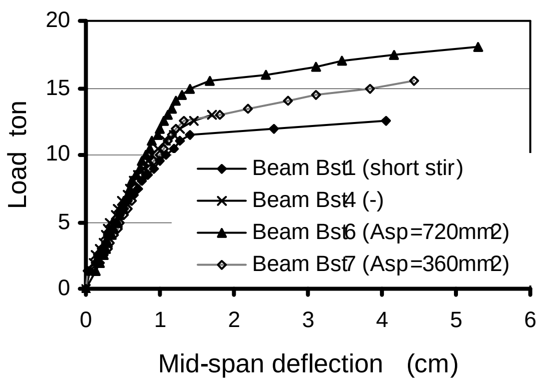


Fig. (11): Deflection curves for beams with different values of (A_p).

Crack Patterns and Modes of Failure

The initiation and propagation of cracks for the different tested beams Fig (12), was obtained visually with a magnifying glass. The cracks were first initiated at the bottom fibers in the constant moment zone for all confined and plated beams. As the load increased, new cracks were created along the beam and the formed cracks propagated towards the points of load application. The rate of cracks propagation was smaller than that of the reference beam Bst4. Prior to failure a horizontal crack was initiated near the upper side of the beam at the steel level and the concrete cover began to spell off. At failure the height of the cracked portion of the tested beams was some what more than that of the reference beam Bst4, particularly in case of beam Bst6 provided with steel plates on to the top compression fiber. The mode of failure for the different tested beams is also included in Table (2). The reference beam Bst4 (without any confining reinforcement) failed in a brittle flexural compression by sudden concrete crushing in the compression zone and the spalling off concrete cover occurred just prior to crushing of concrete. For the confined beams, the modes of failure were also flexural compression, however they changed from a brittle to a relatively ductile one in a gradual manner through crushing in the compression zone and buckling in upper steel was observed. The spalling off concrete cover began earlier than in the corresponding reference beam Bst4. The obtained results showed a similar trend as was observed in a previous work by Whitehead and Ibell [6]. The plated beams Bst6 and Bst7 failed in flexural compression in a ductile way by gradual crushing of the compression zone and local buckling of the fixed steel plate. The horizontal upper cracks were observed at about 95% of its ultimate load, and then the local buckling of steel plat occurred at the instant of failure.

Cracking and Ultimate Loads

The cracking load was not influenced considerably with the presence of steel helix or steel plate. This can be attributed to the fact that the first crack was a flexural one and had occurred at relatively low load before confining effect took place.

From table (2), it can be seen that there was a gain in load capacity for all beams especially in case of beams Bsp1, Bst3, Bst5 and Bst6. The increase in concrete compressive strength or longitudinal reinforcement ratio or glued steel plate showed a considerable increase on the ultimate load as shown in Figs (13) and (15). The ultimate load of beams in series D having f_{cu} = 330, 225 and 265 kg\cm^2 was 1.33, 1.17 and 1.18 times that of the corresponding reference beam Bst4 respectively. The obtained results confirm the previous results reported by Whitehead and Ibell [6]. However the spiral pitch, diameter of spiral wire, and spacing of short stirrups, within the studied range have not affected considerably P_u, see Fig (14). Beams Bst2 and Bst5 having spacing of short rectangular stirrups equal to 10 and 5 cm, the ultimate loads, were 1.11 and 1.27 times that of the reference beam Bst4 respectively.

Fig (12):Pattern of cracks and modes of failure for tested beams.

Table (2): Results of tested beams

Beam No.	P_{cr} ton	P_u ton	P_u/P_{cr}	P_{sp}/P_u	Ductility				Mode of failure
					Δcr (cm)	Δ_{max} (cm)	μ_D	R %	
Bsp1	2.5	17.4	6.96	0.86	0.14	2.1	14.8	276	Flex. Comp. (ductile)
Bsp3	2.5	15.5	6.2	0.9	0.13	1.58	12.0	225	Flex. Comp. (ductile)
Bsp2	3	15.4	5.13	0.93	0.18	1.75	9.4	176	Flex. Comp. (ductile)
Bsp4	2.5	14.7	5.88	0.88	0.15	1.74	11.1	208	Flex. Comp. (ductile)
Bsp5	3	15.1	5.05	0.92	0.2	1.7	8.5	159	Flex. Comp. (ductile)
Bsp6	2.5	15.3	6.14	0.91	0.12	1.14	8.9	166	Flex. Comp. (ductile)
Bsp7	2.5	15.2	6.1	0.92	0.21	1.99	9.1	170	Flex. Comp. (ductile)
Bst1	2.5	12.4	4.96	0.84	0.17	1.28	7.45	139	Flex. Comp. (ductile)
Bst2	3	14.6	4.86	0.92	0.17	1.27	7.17	134	Flex. Comp. (ductile)
Bst3	4	16.9	4.2	0.88	0.23	1.6	6.95	130	Flex. Comp. (ductile)
Bst4	3.5	13.1	3.74	0.95	0.23	1.23	5.34	100	Flex. Comp. (brittle)
Bst5	2.5	16.6	6.64	0.9	0.17	1.55	8.75	163	Flex. Comp. (ductile)
Bst6	3	18	6	0.95	0.27	2.46	9.11	170	Flex. Comp. (ductile)
Bst7	3	15.3	5.1	0.95	0.29	2.5	8.65	116	Flex. Comp. (ductile)

P_{cr} : the Cracking load, P_{sp} = spalling off concrete cover load.
P_u : the ultimate load, ρ = the Longitudinal reinforcement ratio.
ρ_{max} : the maximum longitudinal reinforcement ratio.

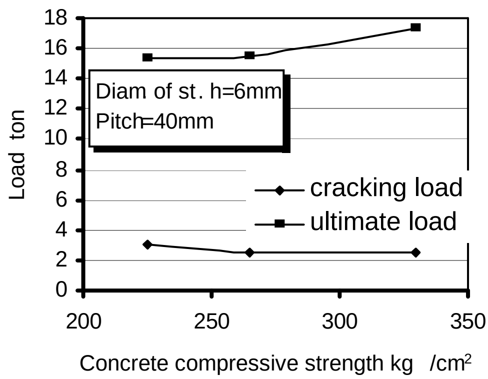


Fig. (13): Effect of f_{cu} on cracking and ultimate load.

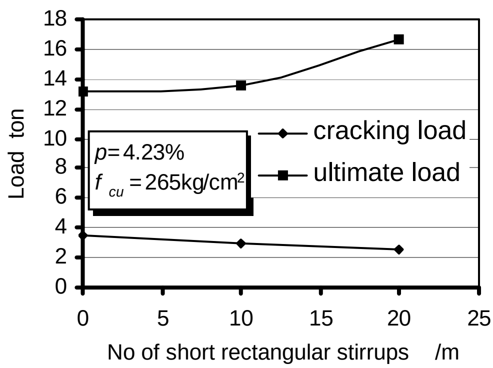


Fig. (14): Effect of No. of short stirrups on cracking and ultimate load.

The ratio between the ultimate and the cracking loads (P_u/P_{cr}) for all the confined and plated beams was bigger than that of the corresponding reference beam Bst4, which reflect also the improvement of their ductility, table (2).

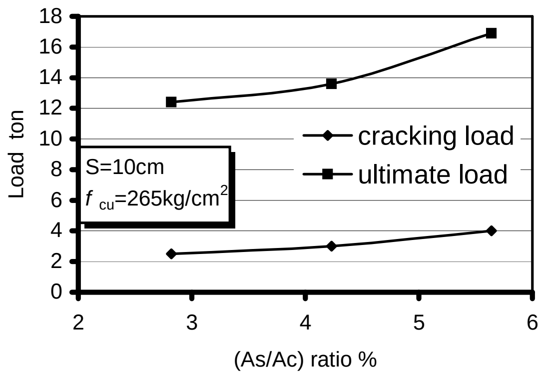


Fig. (15): Effect of (ρ%).on cracking and ultimate load.

PREDICTION OF THE BEHAVIOUR OF CONFINED BEAMS

The tested beams with helically confined concrete were theoretically analyzed. The ultimate load and the induced deflection at mid-span for different helically confined over reinforced beams were estimated at different stages of loading; non cracked, cracked and ultimate stages. The following equations in both non linear and cracked stage with respect to Figs (16), (17), (18) and (19) are used.

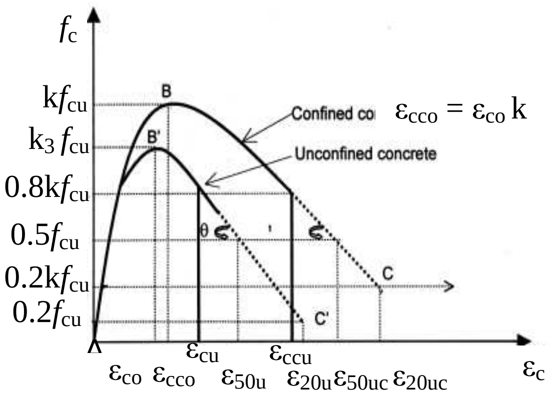


Fig (16): stress-strain model for concrete confined by circular spirals. [13]

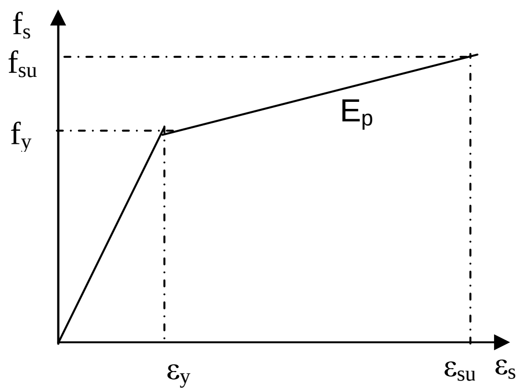


Fig (17): Stress strain curve for steel

$$f_{cc} = K \; f_{cu} \; [(2\varepsilon_c / \varepsilon_{cco}) - (\varepsilon_c / \varepsilon_{cco})^2],$$

where $K = 1 + 2.05(\pi D_h^2 / (R_h - D_h) S_h)(f_{yw} / f_{cu})$

$f_{yh} = f_y + (\varepsilon_s - \varepsilon_y)\, E_p$

$$\varepsilon_{ccu} = K(0.2 / \Psi_c + \varepsilon_{co})$$

$$\psi_c = \tan\theta_c \,/\, f_{cu} = (K - 0.5)/(\varepsilon_{50u} + 0.75\rho_h \sqrt{R_h / S_h} - \varepsilon_{co} K)$$

where E_p is modulus of plasticity of steel, f_{yh} is value of the upper yield strength, ε_{cco} is strain of concrete at compressive cylinder strength of the unconfined concrete, f_{cu} is the compressive strength of unconfined concrete, f_{cc} , ε_c are stress and strain in concrete, ε_y is the yield or proof strain for steel, K is confinement coefficient.

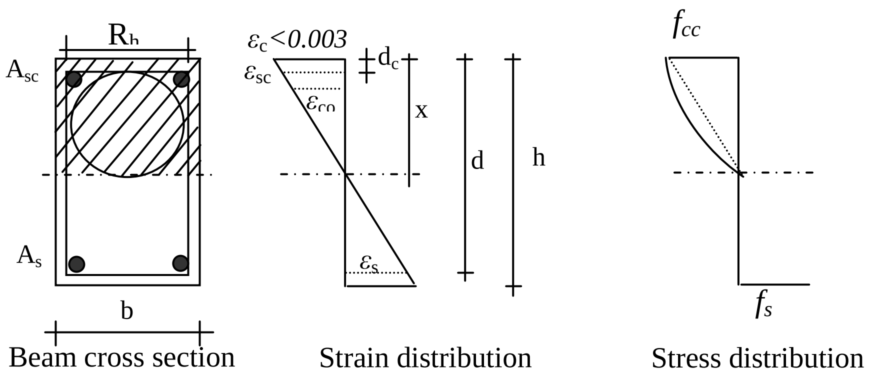


Fig. (18): Section analysis for cracked stage

The Neutral axis position for the cracked nonlinear stage may be given as:

$$0.67bf_{cc}x^2 + (A_{sc}E_s\varepsilon_{cc} + A_sE_s\varepsilon_{cc})x - A_sE_s\varepsilon_{cc}d - A_{sc}E_s\varepsilon_{cc}d_c = 0 \qquad (4.6)$$

The flexural strength (M) is therefore:

$$M = A_sf_s(d - 0.375\,x) + A_{sc}f_{sc}\,(0.375x - d_c) \qquad (4.7)$$

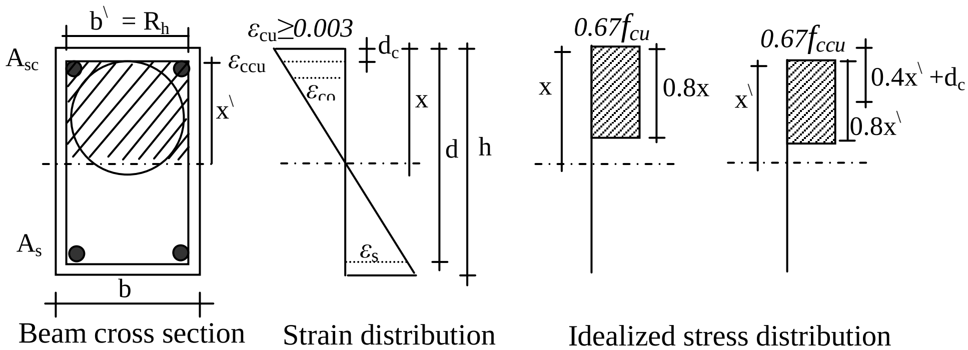


Fig. (19): Section analysis for ultimate stag

The equilibrium of internal forces at ultimate gives the neutral axis position:

$$0.536\,f_{cu}\,x\,b + 0.536\,x^{\backslash}\,b^{\backslash}\,(f_{ccu} - f_{cu}) + A_{sc}f_{sc} - A_sf_s = 0$$

The ultimate flexural strength (M_u) is therefore:

$$M_u = A_sf_s(d - 0.4x) + A_{sc}\,f^{\backslash}_y\,(0.4x - d_c) - 0.536\,x^{\backslash}\,b^{\backslash}\,(f_{ccu} - f_{cu})\,(0.4x^{\backslash} + d_c - 0.4x)$$

A comparison between the load deflection curve obtained from experimental work and that predicted theoretically, are presented in figures (20 to 26), for beams Bsp1 to Bsp7. The figures indicated that, the deflection curves are typically similar and there is almost full agreement between the theoretical estimated up to about 50% of the ultimate load. After this limit, there is a deviation between the experimental and the theoretical curve. This deviation increased with increasing the load up to failure. The theoretical values of maximum deflection at ultimate are smaller than the experimental values except for beams Bsp1, Bsp2 and Bsp3. The experimental measured ultimate load for the tested beams was always slightly bigger than the corresponding predicted values, see Fig (20). The maximum deviation between theoretical and experimental values reaches about 18% for beam Bsp1.

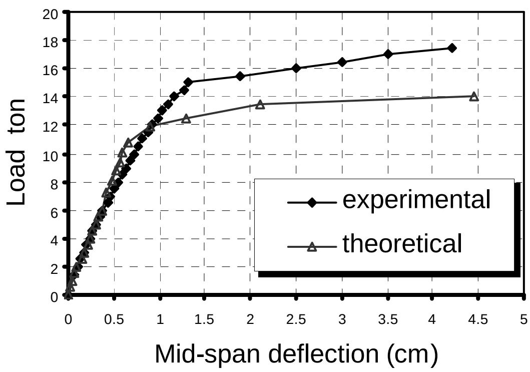


Fig. (20): comparison of experimental and theoretical deflection of beam Bsp1

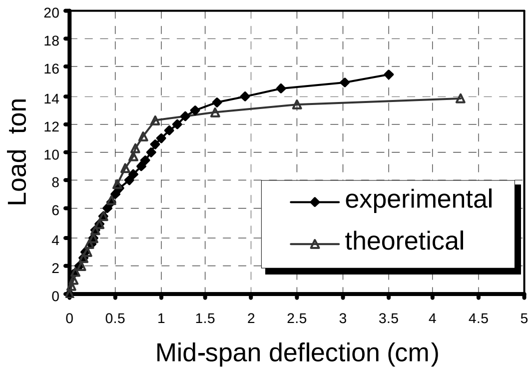


Fig. (21): comparison of experimental and theoretical deflection of beam Bsp2

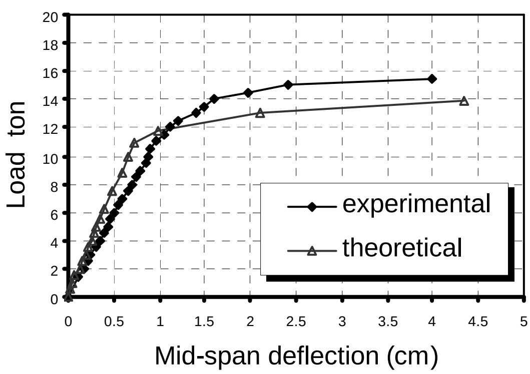


Fig. (22): comparison of experimental and theoretical deflection of beam Bsp3

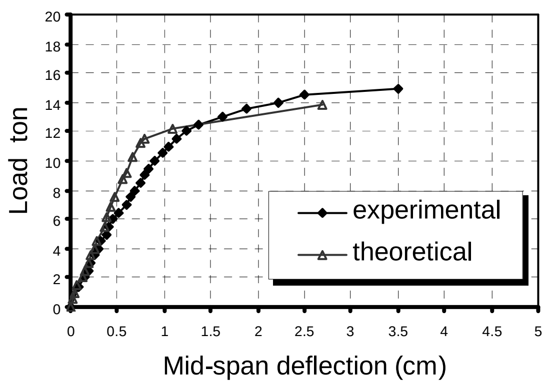


Fig. (23): comparison of experimental and theoretical deflection of beam Bsp4

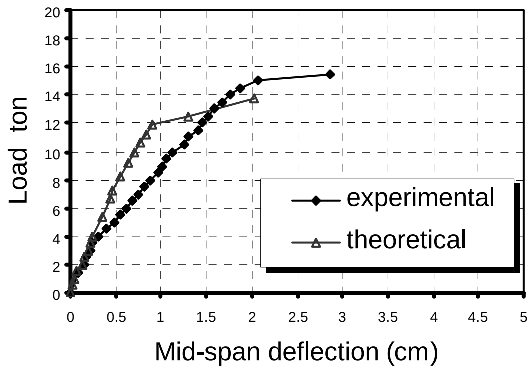


Fig. (24): comparison of experimental and theoretical deflection of beam Bsp5

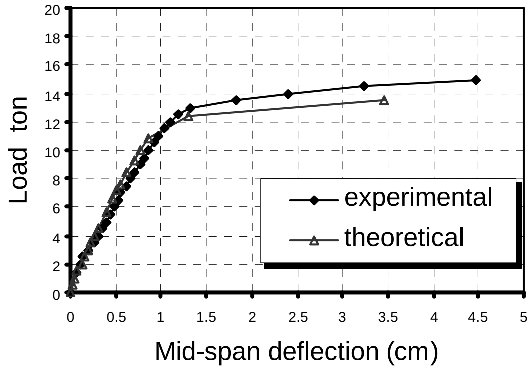


Fig. (25): comparison of experimental and theoretical deflection of beam Bsp6

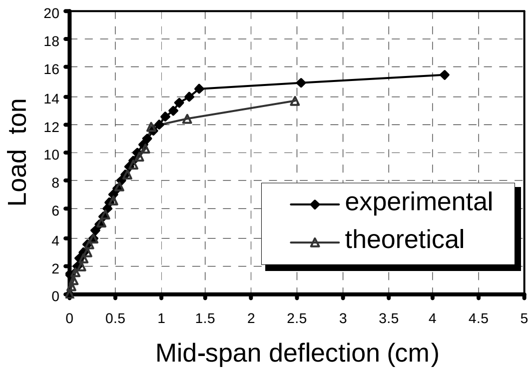


Fig. (26): comparison of experimental and theoretical deflection of beam Bsp7

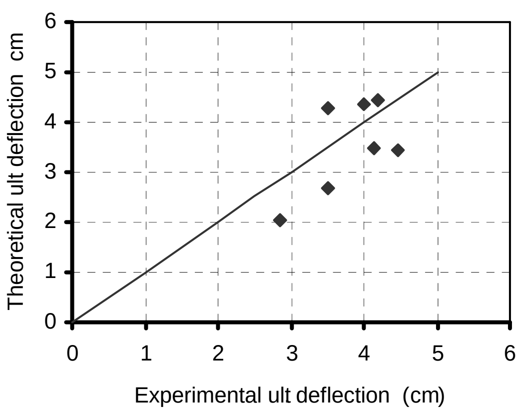


Fig. (27): Experimental versus theoretical ultimate deflection

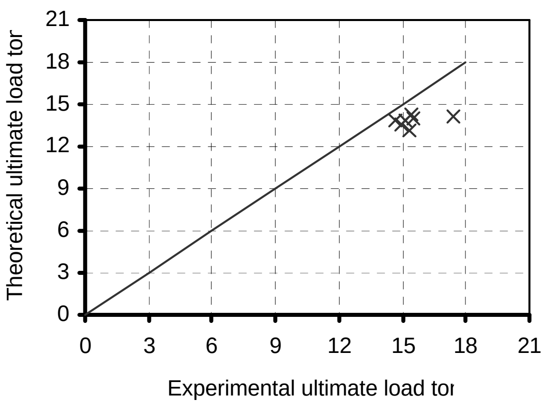


Fig. (28): Experimental versus theoretical ultimate load

Table (3): Comparison between theoretical and experimental results

Beam No	Ultimate load (ton)		Exp/ Theo	Ultimate deflection (cm)		Exp/ Theo
	Exp	Theo		Exp	Theo	
Bsp1	17.4	14.13	1.2	4.2	4.45	0.94
Bsp3	15.5	13.94	1.1	4	4.35	0.92
Bsp2	15.4	13.82	1.1	3.5	4.29	0.81
Bsp4	14.7	13.89	1.05	3.5	2.7	1.29
Bsp5	15.1	13.81	1.09	2.86	2.03	1.4
Bsp6	15.3	13.56	1.13	4.13	3.47	1.19
Bsp7	15.2	13.58	1.12	4.46	3.45	1.28

CONCLUSIONS

Based on the results obtained in this work, the following conclusions can be drawn:

- Considerable increase in ductility has been achieved by providing the over-reinforced concrete beams with steel helix in the compression zone, even when using high longitudinal steel percentage of about 5.64%. This increase in ductility increases as the volume of steel helix increases.
- The effect of the same helices decreases as the percentage of the main reinforcement increases. In addition to that, the characteristics strength of the confining steel influenced the behavior of the confined beam.
- With the same confining reinforcement as the concrete compressive strength increases the ultimate deflection at failure increases.
- The structural ductility and the load capacity of over-reinforced concrete beams can be increased by confining the concrete in the compression zone either internally with steel helix or short rectangular stirrups or externally by glueded and bolted steel plate.
- The external confinement by attaching steel plates on to the compression zone of these beams can be as effective as the internal one if it is properly provided.
- The flat portion of mid-span deflection curves for over-reinforced beams were significantly increased by the corresponding internal and/or external confining techniques.
- Both the internal and external confinement of the compression zone for over-reinforced beams can change the modes of failure from a brittle flexural compression to a ductile flexural compression in a gradual crushing in the compression zone.
- Using helical confinement in the compression zone of rectangular beams is more effective than short rectangular stirrups. This can be attributed to the fact that, helices apply a uniform radial stress to the concrete along the concrete member, while short rectangular stirrups tend to confine the concrete mainly at the corners.

- The behavior of such over-reinforced beams, the ultimate load and the corresponding deflection can be satisfactorily predicted using the well known formulas for the analysis of R. C. members besides the given equations which considered the effect of confinement. However, the used equations overestimate the ultimate deflection while underestimate the ultimate load.

REFERENCES

1. Ziara M. M, D. Haldane and S. Hood "Proposed changes to flexural design in BS 8110 to allow over-reinforced sections to fail in a ductile manner" Magazine of Concrete Research, 2000, 52, No. 6, Dec., 443-454.
2. Nawy, E. G, Denesi, R. F. and Grosko, J. J, "Rectangular Spiral Binders Effect on Plastic Hinge Rotation Capacity in Reinforced Concrete Beams", Journal of American Concrete Institute, Vol 65, No 12, December, 1968, pp. 1001-1010.
3. Muguruma H., Watanabe F., Tanak a H., Katsuda S. and Iwashimizu T. "Improving the flexural ductility of prestressed concrete beams by using the lateral hoop reinforcement of high yield strength". Transactions of the Japan Concrete Institute, 1980, 2, 391–398.
4. Base G. D. "Helical reinforcement in the compression zone of concrete beams". Concrete and Construction Engineering, 1962, 57, No. 12, 456–461.
5. Hadi M. N. S. and Schmidt L. C. "Use of helixes in reinforced concrete beams". American Concrete Institute Structural Journal, 2002, 99, No. 2, 191–198.
6. P. A. Whitehead and T. J. Ibell "Deformability and ductility in over-reinforced concrete structures" Magazine of Concrete Research, 2004, 56, No. 3, April, 167–177.
7. Base GD, Red JB. "Effectiveness of helical binding in the compression zone of concrete beams". J Am. Con. Inst Proc 1965;62(July):763–781.
8. Bing Chen and Juanyu Liu "Properties of lightweight expanded polystyrene concrete reinforced with steel fiber" Cement and Concrete Research, 2004, 34, 1259–1263.
9. Cusson D, Paultre P. "High-strength concrete columns confined by rectangular ties". ACI Struct J 1994;120(3):783–804.
10. Chan, W. L., "The Ultimate Strength and Deformation of Plastic Hinges in reinforced concrete Frameworks" Magazine of Concrete Research, 1955, 7, No. 21, Nov, 121–132.
11. Galeota D, Giammatteo M. M, Marino R. "Strength and ductility of confined high strength concrete". In: Proceedings of 10th world conference, Toronto, on earthquake engineering, Madrid, vol. (5); 1992. p. 2609–13.
12. Hatanaka S, Tanigawa Y. "Lateral pressure requirements for compressive concrete". In: Proceedings of 10th world conference on earthquake engineering, Madrid; 1992. p. 2603–8.
13. Alacali. S. N and Doran. B "The Effects of Confinement and Strain Hardening on the Performance and Design of Short Circular Columns" Yıldız Technical Univ, Main Campus, 80750 Yildiz / Istanbul / Turkey.

14. Mohamed. M. A. “Static and repeated response of strengthened R. C. beams made of significantly low strength concrete”. Inter Build 99 , Cairo, Egypt, 1999, 26-30 June, pp, 673-685.

تحسين سلوك الكمرات الخرسانية عالية التسليح في الانحناء

معظم أكواد تصميم وتنفيذ المنشآت الخرسانية توصي بالابتعاد قدر الإمكان عن تصميم القطاعات الخرسانية عالية التسليح حيث أنها قليلة الممطولية وقد تتسبب في الانهيار المفاجئ. ولكن قد نضطر لتنفيذ مثل هذا النوع من القطاعات لأغراض معمارية كتقليل عمق الكمرة مثلاً، أوقد يحدث عن طريق الصدفة كأخطاء في التنفيذ مثلاً. لذلك فإن هدف هذا البحث هو دراسة تأثير بعض التقنيات المقترحة لتحسين سلوك هذه الكمرات في الانحناء من ناحية الجساءة والممطولية وكيفية توقع سلوكها.

في هذا البحث تم عمل دراسة نظرية وعملية على سلوك أربعة عشرة كمرة خرسانية عالية التسليح مزودة إما بحصر داخلي (حلزوني أو كانات قصيرة) أو بحصر خارجي (ألواح مستمرة علوية ملصوقة بمنطقة الضغط ومدعمة بالجوايط)، وكانت هذه الكمرات ذات أبعاد ثابتة حيث بلغ طولها 240سم وعرضها 15سم وارتفاعها 23سم. تم أخذ مجموعة من المتغيرات في الاعتبار ممثلة في قطر الحديد الحلزوني وخطوات انتشاره ومقاومة الخرسانة ونسبة حديد التسليخ والمسافات بين الكانات القصيرة وأيضاً وجود الألواح الخارجية وذلك على كل من قدرة تحمل وتشكل وطراز الانهيار لهذه الكمرات المختبرة. وأخيراً تم حساب قيم التشكلات عند كل حمل وأقصى ترخيم وأقصى حمل نظرياً ومقارنتها مع النتائج المعملية. النتائج أظهرت التأثير المفيد لهذه التقنيات على ممطولية هذه الكمرات وتحسين سلوكها وكذلك إمكانية التنبؤ نظرياً بمقدار هذا التحسن في الممطولية والحمل الأقصى.