The Effect of Confinement Shapes on Over-Reinforced HSC Beams

Ross Jeffry, and Muhammad N. S. Hadi

Abstract—High strength concrete (HSC) provides high strength but lower ductility than normal strength concrete. This low ductility limits the benefit of using HSC in building safe structures. On the other hand, when designing reinforced concrete beams, designers have to limit the amount of tensile reinforcement to prevent the brittle failure of concrete. Therefore the full potential of the use of steel reinforcement can not be achieved. This paper presents the idea of confining concrete in the compression zone so that the HSC will be in a state of triaxial compression, which leads to improvements in strength and ductility. Five beams made of HSC were cast and tested. The cross section of the beams was 200×300 mm, with a length of 4 m and a clear span of 3.6 m subjected to four-point loading, with emphasis placed on the midspan deflection. The first beam served as a reference beam. The remaining beams had different tensile reinforcement and the confinement shapes were changed to gauge their effectiveness in improving the strength and ductility of the beams. The compressive strength of the concrete was 85 MPa and the tensile strength of the steel was 500 MPa and for the stirrups and helixes was 250 MPa. Results of testing the five beams proved that placing helixes with different diameters as a variable parameter in the compression zone of reinforced concrete beams improve their strength and ductility.

Keywords—Confinement, ductility, high strength concrete, reinforced concrete beam.

I. INTRODUCTION

THE development of high strength concrete has been taken place in the last thirty years or so. Due to industrial demand the development of high strength concrete have improved rapidly because the industrial demand of new features in concrete members with serious advantages such as increased capacity and stiffness [1]. The benefit of increased compressive strenth is to lower volumes and produce smaller designs in terms of design prospective, thus allowing its immediate application into design. The concept of helical reinforcement of beams came after the demand of industry due to the improvement of stiffness factor; this improvement was associated with increasing of brittleness phenomenon in the compression zone, having said that, it is significant to minimize this problem. For the last few years there was a remarkable increase in the compressive strength of structural concrete. In Australia concrete has been used up to 100 MPa in some cases while in some countries they used concrete with compressive strength up to 130 MPa. Due to industrial demand the development of high strength concrete have improved rapidly because the industrial demand of new features in concrete members with serious advantages such as increased capacity and stiffness, the development of high strength concrete has been taken place in the last thirty years or so. The brittle nature of high strength concrete is a major obstacle in its widespread use, as any benefits in terms of reduced member size are negated by the need for increased factor of safety to prevent brittle failure.

II. CONFINEMENT MECHANISM

The confining reinforcement increases ductility and compressive strength of concrete under compression by resisting lateral expansion due to Poisson's effect upon loading. The behaviour of confined concrete depends on the effectiveness of the confinement, which in turn is affected by several important variables such as helical pitch, helix yield strength and helix bar diameter. There is no confining effect after loading, until a particular lateral stress due to Poisson's effect is reached and then the confinement commences. Confinement does not increase strength or ductility initially, but when the axial stress is about 60% of the maximum cylinder strength, the concrete is effectively confined [2]. Fig. 1 shows the differences between confined and unconfined reinforced beam.
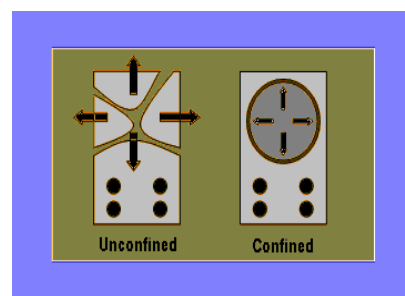

Fig. 1 Effect of helical reinforcement in beams compressive region

III. REVIEW OF LITERATURE

The brittleness of high strength concrete is significant when used in concrete structures, in other words using of high strength concrete without preventing the brittle failure is risky

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and unacceptable. Ductility is an important factor as it ensures large deformation to occur under overload conditions and high ductility enables a concrete segment or a joint to sustain plastic deformations without reduction in stress. Large deflections in structures provide a good warning of failure in the form of tensile cracks prior to complete failure of the beam. Most of the studies concerning confinement of the compression zone in beams is based on the results of research on columns, because the idea of a confined compression zone in beams has only been developed recently. Having said that, the literature and data available about columns confinement is more than for beam confinement. It was first observed by Ritchart et al. [3] that confined concrete from the surrounding steel sections showed great increase in the maximum compressive strength, stiffness, and extended strain at which the peak stress was reached. When the concrete experiences deformation, there is no substantial reduction of the load bearing capacity and it fails gradually in a ductile way. Mrtinez et al. [4] investigated the difference in behavior between spirally confined NSC and HSC column. They tested 94 small diameter columns, which were divided into four groups, The first group specimen had 102 mm diameter by 203 mm high, The second group specimen had 102 mm diameter by 406 mm high, The third group specimen had 127 mm diameter by 610 mm high and the fourth group specimen had 152 mm diameter by 610 mm high. The concrete compressive strength used varied between 21 to 69 MPa and no longitudinal reinforcement was included. The first 78 columns had no protective concrete cover over the spiral steel while the rest (16 columns) had concrete cover over the spiral steel. They measured the strains and the total axial deformation in the lateral steel. Based on these experimental results, Martinez et al. [4] proposed an equation for predicting the confined strength of HSC and NSC.

$$f_{cc} = 0.85 f'c + 4.0 f2\left(1 - \frac{s}{dc}\right) \qquad (1)$$

Where:

f'$_c$ = Compressive strength of concrete (MPa)

f$_2$ = lateral pressure (MPa)

s = pitch of helical reinforcement (mm)

d$_c$ = outside diameter of confined concrete (mm)

In addition they concluded from their experimental investigation, in case of using helical steel with yield stress exceeding 414 MPa it will probably result in unconservative design if the steel used based on assumption at yield point at the computed failure load of the column. In regard of modulus of elasticity they concluded that, there is no difference between the confined and unconfined concrete of these spirally columns. Finally, they concluded that, if the helical pitch was equal to the confinement then the effect of confinement is negligible. Kwan [4] tested 20 reinforcement concrete beams and they claim that to avoid brittle failure and

ensure minimum ductility, it is proposed to set a maximum limit to the tension steel to balanced steel ratio. The values of the proposed maximum limit, which gradually decreased as the concrete strength increased to account for the lower ductility of higher strength concrete, since the balanced steel ratio increases with the concrete strength, the maximum allowable tension ratio still increases with the concrete strength equal to 80 MPa. Thus, the use of HSC in place of normal strength concrete does allow the bending strength of the beam to be increased while maintaining similar ductility. However, the net increase in bending strength due to use of high strength concrete is relatively small compared to the increase in concrete strength. Eventually, from what [5] concluded that, the ductility for reinforced concrete beam using HSC with compressive strength greater than 80 MPa needs to be significantly improved. Hadi and Shmidt [6] tested seven beams with a cross section of 200×300mm^2 by 4060 mm long with a clear span of 3700 mm. The concrete cover was 20mm. The main objective of this study was to investigate the beams ductility when helical reinforcement in the compression region was applied. From their study, Hadi and Shmidt [2] concluded that the beam without helix was very brittle in its failure, while the beam with helix continued to deflect for a longer time. The conclusion they came out with was, if the correct pitch is utilized for effective confinement, helical reinforcement will provide an economical solution for enhancing the strength of flexural members [6].

Whitehead and Ibell [7] tested seven rectangular steel reinforcement beams. Each helically reinforced specimen contained a single helix with a 20 mm pitch. The helices were formed from either 3 mm or 4.8 mm diameter mild steel wire. To show the full benefit of the presence of a circular helix, control specimen (no helix) and specimens containing a similar volume by weight of rectangular links were tested for direct comparison purposes. They came out with conclusion of, placing a steel helix (of 3 mm or 4.8 mm diameter wire) in the compre-ssion zone of a heavily over reinforced (with steel reinf-orcement bars) concrete beam, considerable ductility has been achieved, even using a longitudinal steel percentage of about 7%.This finding is considered exciting in an attempt to achieve shallower concrete structures that are heavily over-reinforced, but which are nonetheless ductile.

Elbasha and Hadi [8] investigated five beams of 4000 mm length and a cross section of 200 mm in width and 300 mm in depth and a clear span of 3600 mm subjected to four point loading, with emphasis placed on the midspan deflection. All variables such as concrete compressive strength and longitudinal reinforcement ratio, helix diameter have been kept same, and the only parameter changed was the helical pitch. The helix pitch was 25, 50, 75 100 and 160 mm. The output of this experimental program indicate that the helix had negligible effect when the helical pitch was 160 mm (helix diameter) in other words the behaviour of the beam with helical pitch of 160 mm which is equal to the core diameter of the beam, was shown to be very brittle in its failure, providing

no plateau region in its load deflection or moment curvature curves. While the behaveour of the other beams with helical pitch 25, 50, 75 and 100 mm was shown to be ductile and the level of ductility based on helical pitch. In addition, the concrete cover spalling-off load increased linearly as the helical pitch increased, which means the spalling-off load is directly proportional to the helical pitch and the ultimate load decreased as the helical pitch increased. From what [3] achieved there is a need of investigating the effect of helix diameter as a variable parameter and to investigate the effect of this parameter on neutral axis depth at the post-peak stage on flexural strength and ductility.

IV. EXPERIMENTAL PROGRAM

Five beams have been designed, constructed, and tested according to AS3600 [9], in order to examine the effect of different types of confinement at the compression zone area of each beam. Table I shows details of each beam, all beams have the same dimensions 4000 mm (length), 300 mm (height) and 200 mm (width), with a concrete compressive strength of 85 MPa (at the time when the beams were tested) used in this experimental program.

For the steel reinforcement, yield strength of500 MPa h-as been chosen and 250 MPa for shear reinforcement and helixes. The beams were classified as a reference beam Fig. 2, SHS Fig. 3 (single helix with stirrups along the beam), SH Fig. 4 (single helix without stirrups at the midspan) and DH Fig. 4 (Double Helix without stirrups at midspan) and finally DHS Fig. 3 (Double helix and stirrups along the beam), these beams have different forms of reinforcing confinement in the compression zone, with various amounts of tensile steel in the tension zone. All beams were designed to be an over-reinforced beam according to Australian Standard AS3600 [9].

In addition, the reference beam would be considered as a benchmark beam to compare it with the other beams. Both the SHS and SH beams had a single helix in the compression zone with 160 mm diameter and the difference between these two beams is Beam SHS has stirrups, while beam SH has no stirrups at mid span. Both DH and DHS were confined with double helices, the helix diameter was 80 mm and the only difference between these two beams is Beam DH has no stirrups at mid span while Beam DHS has continuous stirrups along the beam span. The helix pitch for all beams is constant which was 50 mm, the reference beam was kept without helix. Finally, different types of strain gauges have been used in this experimental program, in order to determine the internal strain within the beam while applying the load. Each strain gauge has a resistance that increased or decreased as the strain gauge extends or shortens. Around 8-12 strain gauges have been located in each beam and at different locations to measure the actual strain in these locations. All strain gauges were placed in mid span of each beam so as to capture the behavior of the beams in the section where the maximum deflection and stresses would occur.
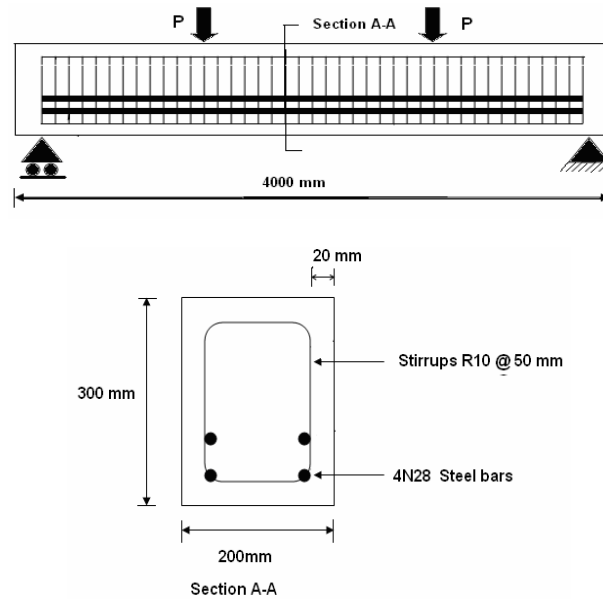

Fig. 2 Reference beam details

V. RESULTS ANALYSIS

A. Load Deflection Behaviour

Fig. 5 shows the load midspan deflection of all tested beam specimens. It can be noticed that, the beam's behaviour can be classified in two stages, the first stage is the elastic range up to the yield load and the second stage was the post yield. A linear deflection curve associated with the deflection of between 35 to 45 mm was shown by all beams during the elastic stage. Beam DH and Beam DHS have a steeper elastic region which indicates a higher flexural stiffness. Both beams reached 420 kN, which is approximately 10% more than that of the other beams. These two beams failed at similar ultimate load strength, and they have similar behaviour in the fact they have multiple failures. The next stage of Beam DHS after reaching an ultimate load of 420 kN was the curve increases almost back to the ultimate load and plateaued for a small period of time, at 132 mm deflection a second failure occurred and the load dropped by 75 kN, then a small minor failure occurred between 175 mm and 245 mm deflection before the testing concluded. The curve for Beam DH was similar as Beam DHS but with less ductility as can be seen in Fig. 5. There is a significant secondary failure by 150 kN and that occurred after a small plateau after the initial failure occurred. Then the load carrying capacity kept decreasing ending this performance approaching zero. The Reference Beam and Beam SHS failed at similar values, and as beam SH was not compacted properly, it can be assumed that the ultimate load of Beam SH was close to the ultimate load of Beam SHS and the Reference Beam. Beam SHS at the second stage of loading (post-yield) performed in almost horizontal plateau stretching until the end of beam loading. The beam reached a load capacity of 320 kN. This plateau indicates that this beam is behaving in a very ductile manner. Beam SH showed some ductile behaviour but not as much as Beam SHS and Beam DHS. After the beam reached yield load there was a slight loss

in load capacity, then an increase occurred until reached a peak load of 260 kN, this was followed by a decrease until a plateau at 95 kN at the end of beam loading.

TABLE I
DETAILS OF STEEL REINFORCEMENT AND HELIX DIMENSIONS OF THE TESTED BEAM

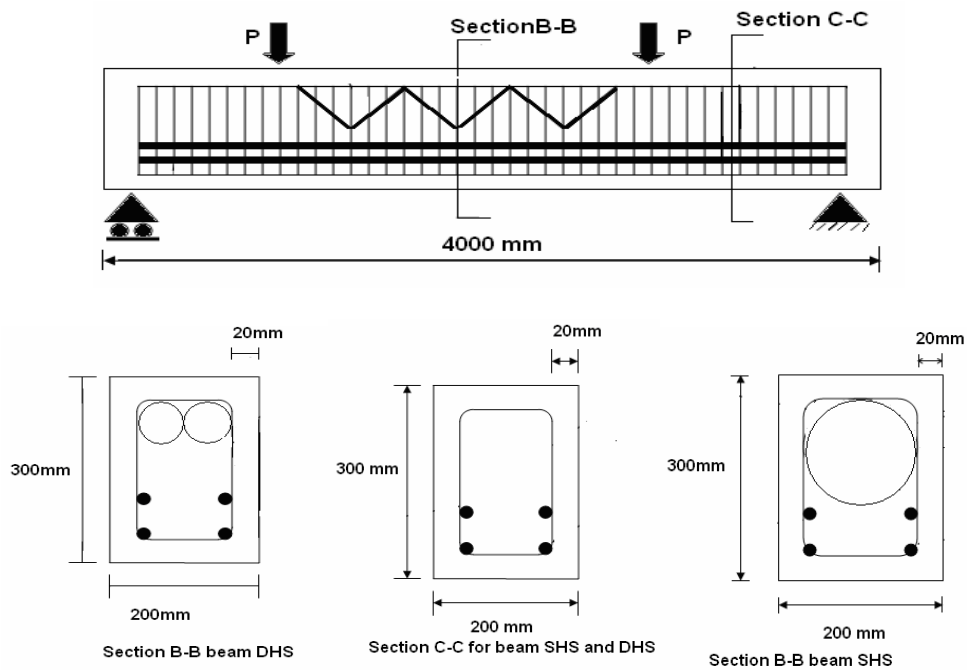| Beam | Longitudinal Reinforcement | | Stirrups | | Helical Reinforcement |
	Number of Bars	Diameter (mm)	Pitch at Ends (mm)	Pitch at Middle (mm)	Helix Diameter (mm)
Ref.	4N 28	10	50	50	None
SHS	2N24+2N28	10	50	50	160mm
SH	2N32+2N24	10	50	None	160mm
DH	2N32+2N24	10	50	None	2×80mm
DHS	2N32+2N32	10	50	50	2×80mm

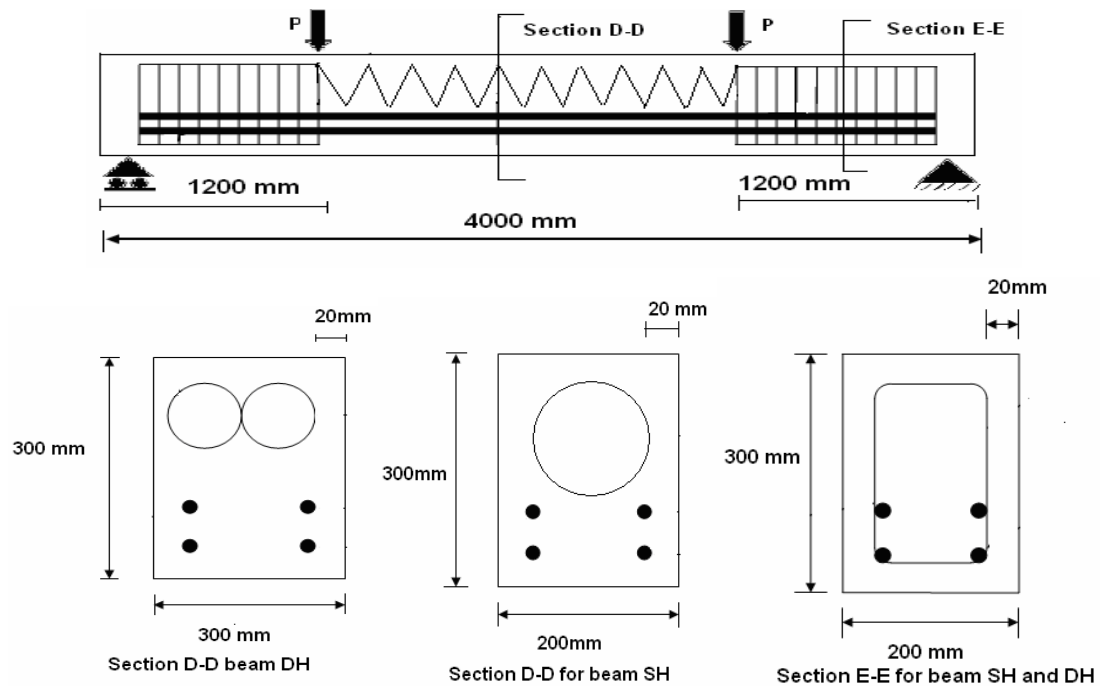Fig. 3 Beam SHS and DHS (Refer to Table I for more details)

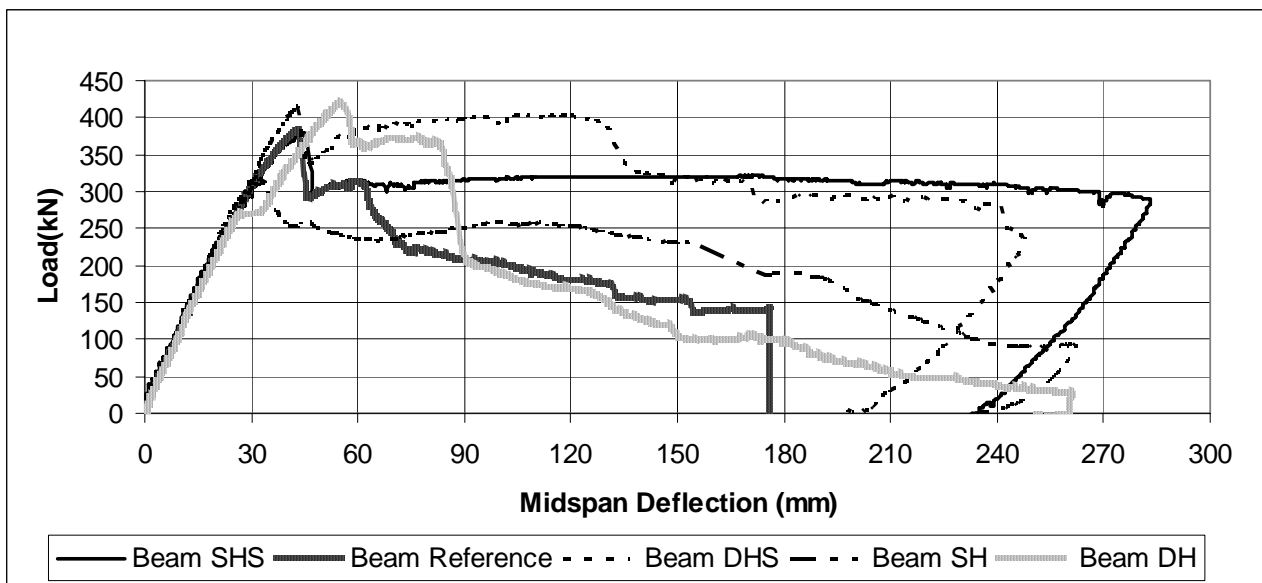Fig. 4 Beam SH and DH (Refer to Table I for more details)

Fig. 5 Load vs Deflection at midspan for all five beams

B. Ductility

The measurement of ductility is varied and there are several ways to measure ductility such as area under curve and ductility index. Having said that Table II and Table III show two different ways of ductility measurement Table II measures the ductility index based on the deflection at yield and ultimate load at 80 % of its value, it shows that each beam achieved a level of ductility. Beams SHS and DHS were the most ductile beams followed by Beam DH and SH then the Reference behaved in a less ductile way compared with the other beams.

The second method to measure the ductility was the area under the curve, this method is based on the ratio of the area under the curve for the plastic stage to the total area of the curve which includes the area under the curve of elastic stage. The values in Table III indicate Beams SHS and SH are the most ductile beams followed by Beams DHS, DH and then the Reference Beam. This high ratio of ductility is due to the beams with single helix confined more amount of concrete within the compression area, while the beams with double helix confined less amount of concrete at the compression zone. On the other hand, the area ductility ratio of the Reference beam was less than the other beams due to a very short increase in load capacity after the initial failure.

From Table IV it can be noticed that beams with double helices have less reduction of neutral axis depth compared with beams containing a single helix, in other words, Beams DHS and DH kept the confined area in the compression zone until the final failure occurred, this explains the high value of strength obtained by these two beams. The reductions of the neutral axis of beam SHS and SH was greater than DHS and DH especially the neutral axis of SHS reduced by 7% compared with beam DHS.
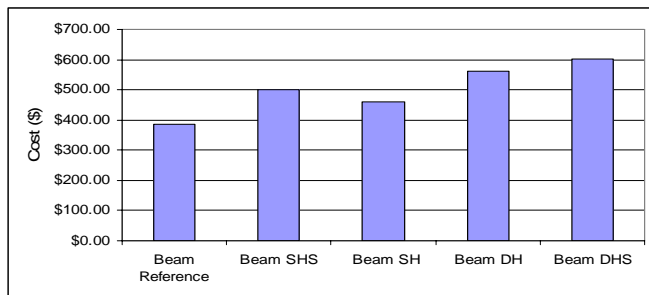

Fig. 6 Beam Cost Comparison

After calculating the construction cost of each beam, Fig. 6 which shows the cost of all beams, it can be noticed that the highest total cost were Beams DH and DHS, and that was because of the two 80mm diameter helixes. While the other two Beams SHS and SH were less than Beams DH and DHS by 20% and 30%, respectively. The reference beam is the cheapest cost, which was approximately $396.

Fig. 7 shows the ductility and strength of each beam against the beam cost in order to find which beam is the most cost-effective. The strength plotted in kN, ductility as area under

curve with units of kN-mm, and the cost is plotted in dollar units. It can be noticed that beam SHS was the most ductile beam for its cost, followed by Beam DHS. In terms of strength, the yield load in Beam DHS and DH was greater than beam SHS and SH this increase is due to the fact that using double helix in Beam DHS and DH increase the overall cost of these beams, on the other hand, the strength per a dollar cost for these Beams DHS and DH were cost effective beams, and this can be noticed in Fig. 8.
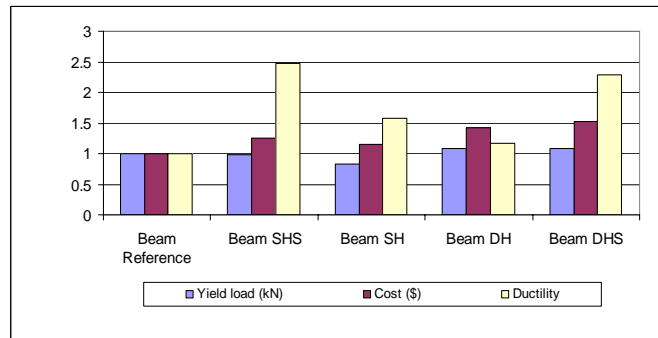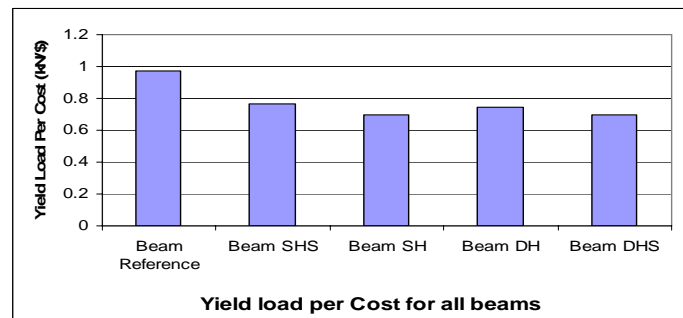

Fig. 7 Beam cost VS Ductility and strength

Fig. 8 Yield load per cost of each beam

VI. CONCLUSION

The aim of this experimental program was to improve the ductility of high strength concrete by using helices in the compression area of the beam, in other words, by confining the compression area with the use of these helices. In terms of ductility, the load midspan deflection behaviour of the beams in the elastic region are all very similar with the double helix beams reaching a higher strength than the rest of the beams as the twin helices allowed an earlier confinement of the concrete in the compression zone.

The Beam SHS was the most ductile beam followed by Beam DHS and that due to the fact that the helical reinforcement with stirrups acted as a confining mechanism for the concrete in the compression area, traixially stressing the concrete and improving the strength and strain capacity of the concrete. This failure occurred due to the fact that the neutral axis was not deep enough to enable adequate concrete to be in compression, and hence be in confined compression, to take full advantage of the confining reinforcement available. The failure enabled the confining reinforcement to

act more significantly due to the movement of the neutral axis. The cover spalling off reduced the effective depth and cross sectional area of the concrete, forcing an immediate drop in the depth of the neutral axis. This drop in the neutral axis engages a larger percentage of the concrete in the helical reinforcement to be in compression and as a result in a confined state.

The higher ultimate load achieved by Beams DHS and DH indicates that there was a larger area of concrete in confinement at the earlier stages of loading compared to the single helix and the reference beam, which allows for a greater ultimate strength being reached, they reached approximately 10% more than the other beams.

From the load deflection curves of all beams, the beams behaved in a ductile manner, and ductility has presented due to the fact that the gradual decrease in capacity after initial failure does not continue, instead turns around and increases. Beam SH achieved a reasonable (due to improperly compaction during the construction stage) ductility after the initial failure occurred, in addition a minor strength capacity gained by this beam before decreasing again.

The strain profiles and neutral axis depths are less well predicted, with tensile steel strains generally being significantly underestimated. It is possible that the ultimate strain of 0.003 assumed for normal-strength concrete is too low a value for high-strength concrete, from the fact that the top fibre strain in all five beam was well above 0.004 at yield. The unconfined concrete that spalled off in a brittle manner at yield failed at an assumed ultimate strain of 0.003. However the confined concrete's ultimate strain was well above this assumed value. The helices and the stirrups confined the concrete significantly hence the large experimental values obtained. In general, comparing Beams SHS, SH, DH, DHS and the Reference Beam it can be seen that the inclusion of a helix is an effective type of confinement reinforcing. The inclusion of helices in the compression zone of the four beams dramatically changed their behaviour. The helices effectively confined the compressive region of the beams allowing greater loads to be held after the initial spalling of the unconfined concrete. The neutral axis depths were well predicted in this experimental program. After spalling of the concrete had occurred to the outermost fibre, there would be a significant neutral axis shift due to the cross section of the beam reducing in size. This change of neutral axis location was taken into consideration in the theoretical calculations and predicted results were relatively close to the experimental results.

The impending demand of high strength materials to be used in the construction of beam members currently can not be fully utilised, as both materials suffer from limited ductility. This deficiency in ductility reduces the ability to take full advantage of the increase in strength of both materials. Using steel helices to encase the concrete in the compression region of the beams increases their performances dramatically as revealed in this experimental program. Beam SHS was the most ductile beam for its cost, followed by Beam DHS. In terms of strength, the yield loads of Beams DHS and DH were greater than Beams SHS and SH this is due to the fact that Beams DHS and DH included double helices. Having said that, the strength per dollar cost for Beams DHS and DH being the cheapest made these two beams as most cost effective beams. Finally, it can be concluded that this helices in the compression zone of beams increase their strength and ductility.

REFERENCES

[1] Nawy, E. (2001). Fundamentals of High-Performance Concrete. Canada: John Wiley & Sons.
[2] Sargin.M (1971). Stress-strain relationships for concrete, and the analysis of structural concrete sections, Waterloo, Solid Mechanics Div., Univ. of Water loo.
[3] Richart, F., Brandtzaeg, A., and Brown, R. (1929). the Failure of Plain and Spirally Reinforced Concrete in Compression. Illinois University, Engineering Exp-erimental Stat ion, Bulletin, No. 190, Vol.26, No.31, pp 1- 74.
[4] Martinez, S., Nilson, A.H., and Slate, F.O. (1984) Spirally Reinforced High Strength Concrete Columns, Journal of the American Concrete Institute, No. 81, Vol. 5, pp 431-442.
[5] Kwan, A. (2004). Effects of Confinement on Flexural Strength and Ductility Design of HS Concrete Beams. Structural Engineer, 38-44.
[6] Hadi, M.N.S. and Schmidt, L.C. (2002). Use of Helixes in Reinforced Concrete Beams, ACI Structural Journal, Vol. 99, No2, pp 191-198.
[7] Whitehead. P. A. and Ibell T. J. (2004). Deformability and Ductility in Over- reinforced Concrete Structures. Magazine of Concrete research, Vol. 56, No. 3, pp 167-177.
[8] Hadi, M.N.S. and Elbasha, N. (2005). Effect of Tensile Reinforcement Ratio and Compressive Strengths on the Behaviour of Over-Reinforced Helically Confined HSC Beams, Construction and Building Materials, Vol. 21, pp 269-276.
[9] Australian Standard for Concrete Structures. (2001). AS3600 .North Sydney, Australia: Standard Association of Australia.
[10] Warner, R., Rangan, B., Hall, A., and Faulkes, K. (1998). Concrete Structures, Longman.
[11] Warner, R., Rangan, B., Hall, A., and Faulkes, K (1998). Concrete Structures, Longman.
[12] Onesteel Reinforcing Design Guide (2004) 500+ Rebar, Onesteel Australia.
[13] Onesteel Reinforcing Product Catalogue (2004) 500+ Rebar, Onesteel Australia.

TABLE II
DUCTILITY DEFLECTION INDEX

Beam	Py kN	0.8 Py kN	Δy (mm)@ 0.8Py	Δu (mm)@ 0.8 Py	μd $(\Delta u/\Delta y)$
Reference	385	308	30	47	1.56
SHS	380	304	30	183	6.1
SH	320	256	24	40	1.67
DH	420	336	33	84	2.54
DHS	420	336	30	107	4.25

TABLE III
AREA DUCTILITY RATIO

Beam	Yield Load Py (kN)	Area under Elastic Curve (kN-mm) (A1)	Area under Plastic Curve (kN-mm) (A1)	Ductility $\mu d= (A2/A1+A2)$
Reference	385	7700	28581	0.787
SHS	380	7980	70826	0.898
SH	320	5583	45001	0.889
DH	420	9450	33484	0.779
DHS	420	9450	65179	0.873

TABLE IV
THE EFFECT OF HELIX ON NEUTRAL AXIS DEPTH

Beam	Py (kN)	Neutral axis depth for UnCracked section (mm)	Neutral axis depth for UnCracked section (mm)	%Reduction of Neutral axis Depth
Reference	385	166	130	0.21
SHS	380	164	124	0.24
SH	320	165	128	0.22
DH	420	165	129	0.21
DHS	420	168	138	0.17