# USE OF HORIZONTAL WEB BARS FOR UPGRADING REINFORCED CONCRETE BEAMS IN SHEAR

Jubin Motamed,
Formerly, Senior Research Fellow,
University of Greenwich and Research
Associate, Loughborough University
J.motamed@my.westminster.ac.uk

Mohammed Raoof,
Professor of Structural Engineering,
Loughborough University
Loughborough, United Kingdom.
M.Raoof@lboro.ac.uk

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ABSTRACT
The existing recommendations in Eurocode 2 and the British Codes of Practice BS8110 and BS5400 for the shear design of reinforced concrete beams are essentially based on research conducted on normal strength concrete (NSC) specimens with cube strengths of up to 50 MPa. Fairly recently, some limited tests on High Strength Concrete (HSC) beams, as reported elsewhere, appeared to suggest that, in certain cases, the shear strength of HSC members made with limestone aggregate may turn out to be less ( or equal) to the characteristic shear resistances of nominally identical NSC beams. The present paper reports the salient features of test data for five beams (four HSC specimens made with limestone plus one NSC beam made with river gravel as coarse aggregate) with the experimental results demonstrating this, perhaps, initially puzzling observation. In addition, test results for another set of six similar reinforced concrete beams (three HSC plus three NSC, which included 10 mm limestone and 20mm river gravel, respectively), will be used to demonstrate that, once upgraded with Horizontal Web Bars (HWB), the shear resistance of the HSC beams is highly dependent on the additional dowel action generated by such HWB positioned on the two sides of the beam, in the vicinity of the neutral axis. A new design rule is, therefore, reported for upgrading of HSC beams in shear, using such Near Surface Mounted (NSM) HWB. Finally, it is perhaps worth mentioning that for durable and effective upgrading, such Near Surface Mounted HWB is to be encased with epoxy resin filling the grooves made on either side of the beam within the cover of stirrups close to the neutral axis, with intumescent paint used to cover the repaired area of the beam for fire protection.

## INTRODUCTION

Since the 1980's, there has been an explosion of interest in developing economical, functional and safe methods for maintenance, repair and upgrading of structures as built. The financial implications of such works can often be considerable, and factors such as time and potential disruptions to the users of the structures in the course of remedial action can be of prime importance.

As regards reinforced concrete (RC) structures, it is now well-established that concrete can suffer from a wide variety of types of deterioration resulting in the main from chemical attack. Much effort has (over the last three decades or so) been made to investigate the causes of these chemical attacks and their effects on concrete and steel reinforcement on a material level and also on the means of reducing or eliminating such decay. Prevention is undoubtedly the best cure for the problem. In the meantime, however, many structures built in the last three or four decades will continue to deteriorate and a substantial number are in need of urgent repairs.

There is currently also a pressing need for upgrading (strengthening) of RC structures: this may arise for several reasons, including poor or outdated initial design, construction errors (e.g. missing reinforcement or incorrect placement/detailing of reinforcement), deterioration of structures due to both age and environmental factors and changing loading conditions (especially relevant to road bridges with increasing traffic loads and volumes over time). The repair industry is expanding rapidly, predominantly in the U.S.A. and Europe, but also now in the Middle East and other parts of the world.

The present paper introduces the use of horizontal web bars for upgrading normal as well as high strength reinforced concrete beams in shear, by utilising the potentially significant contribution of the dowel action, generated by employing additional near surface mounted horizontal web bars (HWB's) on either side of RC beams (in the vicinity of the Neutral axis), to the shear strength of RC beams.

It is, perhaps, worth mentioning that, over the last fifty years or so, a significant number of tests have been carried out to investigate the influence of dowel action on the shear behaviour of RC beams. As early as in the beginning of the last century, in his seminal work, Morsch (1902) discussed the potential contribution of the dowel action generated by the longitudinal tensile reinforcement to shear strength. It is also noteworthy that, since the 1970's, the dowel action has been suggested to be primarily dependent on the tensile (often related to the compressive) strength of concrete, and that increases in the dowel capacity depend on increases in both the tensile strength of concrete and the vertical crack displacement, with practically significant levels of dowel force developing when, in practice, one approaches the ultimate load, associated with which substantial opening(s) of the shear cracks is invariably found to take place (Baumann, 1968). Space limitations do not permit a detailed discussion of the previous literature on dowel action, here. Instead, the interested reader is referred to (Motamed, 2010) for a recent detailed and critical account of the previously available work in this area. For the present purposes, it perhaps suffices to provide a brief account of the most relevant works to the present discussion, as presented in the next section.

## BACKGROUND CODE EQUATIONS

When using the EC2 and/or BS8110 to design for shear, the characteristic shear resistance of RC beams with slender rectangular sections, are (in the present terminology: please refer to the NOTATION ) given as follows:

According to EC2,

$$V_{Rk,c} = 0.18(100 f_c . \rho_i)^{\frac{1}{3}} . \left(1+\sqrt{\frac{200}{d}}\right) .bd \tag{1}$$

with the BS8110's corresponding Equation, being

$$V_{Rk,c} = 0.27(100 f_c . \rho_i)^{\frac{1}{3}} . \left(\frac{d}{400}\right)^{\frac{1}{4}} .bd \tag{2}$$

where, both Equations (1) and (2) are only applicable to RC beams with no stirrups.

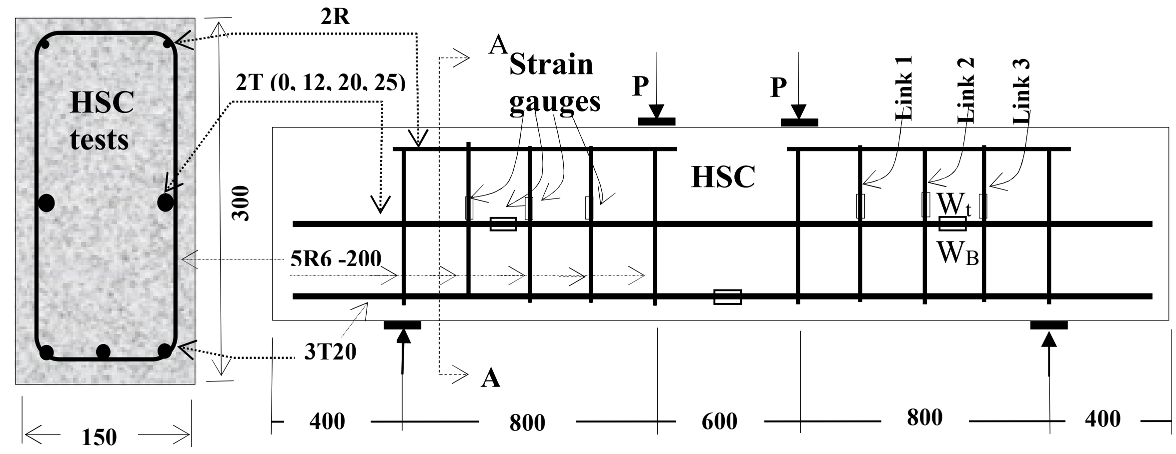
With shear stirrups present, both codes recommend the same additional term to be included in the above Equations (1) and (2)-i.e.

$$V_{Rk} = 0.18(100 f_c . \rho_i)^{\frac{1}{3}} . \left(1+\sqrt{\frac{200}{d}}\right) .bd + \rho_w f_{yv} bd \tag{3}$$

$$V_{Rk} = 0.27(100 f_c . \rho_i)^{\frac{1}{3}} . \left(\frac{d}{400}\right)^{\frac{1}{4}} .bd + \rho_w f_{yv} bd \tag{4}$$

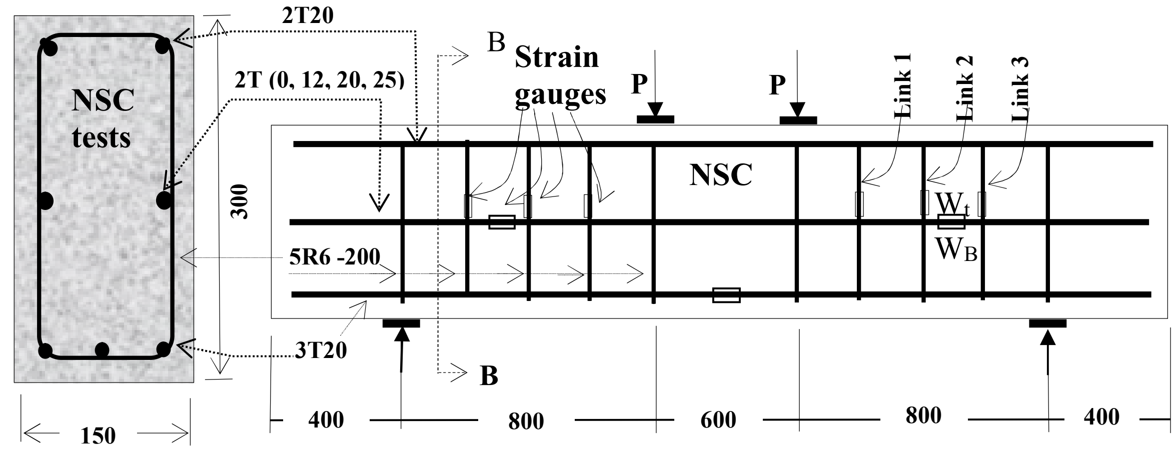
In Equations (1) to (4), the units are in N and mm. Coefficient 0.18 in Equation (1) is the one recommended in the general code, although it may be modified in certain National Annexes. In EC2, there is a limit of $f_c \leq 90$ N/mm$^2$.

In the present UK recommendations, BS 8110 restricts $f_{cu} \leq 40$N/mm$^2$ for Equations ( 2) and (4) and in the Concrete Society's recommendations of 1998, the concrete compressive strength, $f_{cu}$ , is restricted to less than 100N/mm$^2$ , with this figure amended (reduced) to 60N/mm$^2$ in 2004, motivated in part by the research conducted by Regan et al (2005).



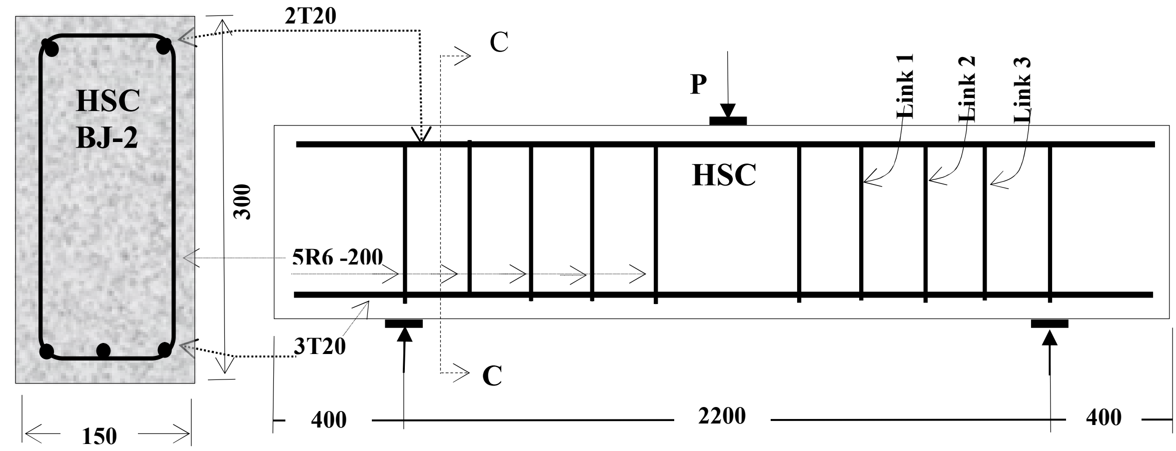

Section A-

Figure 1: Details of the HSC beams with or without HWB - a/d=3.02




Section B-

Figure 2: Details of the NSC beams with or without HWB- a/d=3.02.




Section C-

Figure 3: Details of the HSC beam BJ-2, which included stirrups-a/d=4.15.

The development of dowel action in RC beams, the contribution of which to shear resistance is already included in Equations (1) and (2), is a result of the longitudinal (main tensile) reinforcement taking (carrying) a portion of the applied shear force at a crack, with this initiated by the vertical movement(s) of the two opposite crack surfaces. Within the region of contact between concrete and the main tensile steel, and in the presence of stresses acting perpendicular to the direction of the longitudinal reinforcement, formation of a crack next (and in the same direction) to the steel bars is generally found to be associated with dowel failure. In such circumstances, the so-induced shear force in an individual bar increases with increasing relative movements at the face of the vertical crack, with this effect gaining more significance as one approaches the ultimate load with its associated invariably significant opening(s) of shear cracks in practice. It is noteworthy that, dowel action in reinforced concrete beams can occur at both flexural and also shear cracks, and that, in this context, the effectiveness (strength) of the concrete cover to resist such actions is, generally, believed to be decisive in terms of determining the level (significance) of the resulting dowel action. Dowel bars may also act against the core of concrete, with this resulting in a bending of the bar and it's associated local crushing of adjacent concrete. Indeed, this may even cause splitting of concrete in the plane of the bar if the cover thickness is sufficiently small.

Bearing the above in mind, then, in what follows, much experimental and theoretical attention will be devoted to the possibility of utilising the potentially significant dowel action contribution of HWB's, placed in the vicinity of the beam's Neutral axis, to increase (upgrade) the shear strength of RC beams. In this context, it is noteworthy that, since 1995, the first author and his associates have been actively involved with experimental, analytical and numerical work on the use of HWB's to increase the shear strength of RC beams (Motamed, 1997, 2010; Al-Hussaini & Motamed, 2002 and Motamed et al, 2012), and for the present purposes, the following sections report some of the salient features of our work in this area.

EXPERIMENTS

All the beams were designed to fail in shear, with ten specimens subjected to external loading with a/d=3.02 and one HSC specimen (BJ-2) having a/d=4.15: it is noteworthy that the beam BJ-2 was designed to BS8110 which imposes a limit of $f_{cu}$<40 MPa- hence, the use of 2T20 top steel in this particular test specimen. Otherwise, when designing all the other HSC beams in Table 1, use was made of the Concrete Society's recommendation, which recommended a limit of $f_{cu}$<100 MPa, in which case, the need to include any top steel is (unlike the BS8110's approach) eliminated. Moreover, the scale of the test specimens were chosen to be sufficiently large to correctly simulate the behaviour of full-scale structural elements. Figures 1 to 3 show details of the eleven test specimens, with the beams being 150×300mm in cross-section and having a clear span of 2.2m. All the beams had 3T20 as their main tensile steel (i.e. $\rho_l$= 2.37%) with their effective depth d=265mm. The shear links, placed within their shear spans, were R6 at 200mm centres. Both NSC and HSC beams were tested with and/or without horizontal web steel of 2T12, 2T20 and 2T25, Table 1.

The average measured (based on three tensile tests) yield strength for the stirrups $f_{yv}$=250 N/mm$^2$. The reinforcement used for the top, bottom and horizontal web steel was high yield, hot rolled deformed bars with a guaranteed yield value of $f_{yl}$= 460 N/mm$^2$. Details of concrete cube and cylinder splitting (tensile) strengths, $f_{cu}$ and $f_{sp}$ , respectively, are also given in Table 1. In the concrete mix design, Rapid Hardening Portland cement was used together with 20mm river gravel for the NSC and 10mm limestone for the HSC specimens, and the average values of $f_{cu}$ for the NSC and HSC beams were around 44 and 111 N/mm$^2$ , respectively. Moreover, apart from the beam BJ-2 which was subjected to a single external point load with a/d=4.15, all the other ten beams in Table 1 were subjected to symmetrical external two-point loading with a constant a/d=3.02. Finally, apart from the beam BJ-2 which, similar to the four NSC specimens in Table 1, had 2T20 bars as top (compression) steel, the other six HSC beams in Table 1 had no practically significant top steel, with the 2R6 bars placed in the top (compression) side of each of these beams merely used to hold the reinforcement cage together. Indeed, as clearly shown in

Figure 1, unlike all the other specimens with 2T20 which had their top steel running continuously along the full length of their spans, Figures 2 and 3, the R6 bars were all absent from the constant moment zone of all the HSC beams with a/d=3.02.

## Results and Discussion

As mentioned previously, Table 1 gives a summary of the test specimen details and their corresponding measured total ultimate loads, with a fairly detailed description of the experimental observations reported under the following five sub-section headings: (a) Shear failure loads; (b) Load-deflection behaviour; (c) Crack propagation; (d) Shear resistance of HSC beams compared to NSC and (e) Load-strain behaviour in reinforcement.

Table 1: Summary of the tests

| Beam No | Top Steel | Horizontal web bar (HWB) | Cube Strength ($f_{cu}$) N/mm$^2$ | Splitting strength ( $f_{sp}$) N/mm$^2$ | Total ultimate load, kN |
|---|---|---|---|---|---|
| NSC1 | 2T20 | 0 | 43.2 | 2.98 | 160 |
| NSC2 | 2T20 | 2T12 | 41.0 | 3.01 | 203 |
| NSC3 | 2T20 | 2T20 | 47.7 | 3.22 | 200 |
| NSC4 | 2T20 | 2T25 | 43.3 | 2.97 | 210 |
| HSC1-1 | 2R6 | 0 | 109.0 | 4.21 | 130 |
| HSC1-2 | 2R6 | 0 | 101.2 | - | 140 |
| HSC1-3 | 2R6 | 0 | 106.6 | - | 160 |
| HSC2 | 2R6 | 2T12 | 109.3 | 5.20 | 265 |
| HSC3 | 2R6 | 2T20 | 112.5 | 4.34 | 280 |
| HSC4 | 2R6 | 2T25 | 112.5 | 4.34 | 300 |
| BJ-2 | 2T20 | 0 | 118.1 | 4.3 | 142 |

Note: flexure (main tensile) reinforcement started to yield at failure only for the HSC beams with HWB

(a) *Shear failure loads.* The first HSC1 failure load of 130 kN ($f_{cu}$ = 109 N/mm$^2$) appeared low. The first result was compared with the second HSC1 failure load of 140 kN ($f_{cu}$ = 101.2 N/mm$^2$) and third failure load of 160 kN ($f_{cu}$=106.6 N/mm$^2$). The average ultimate load carried by these three similar HSC1 beams was 143.3 kN (the average $f_{cu}$=105.6 N/mm$^2$) as compared to the ultimate load of beam NSC1 which was 160 kN ($f_{cu}$=43.2 N/mm$^2$). The links were similar in the two and non contained any horizontal web steel. NSC1 did have 1.55% of compression reinforcement which was not present in HSC1. The inclination of the critical shear crack was much steeper in HSC1 at about 50°, compared with approximately 35° in NSC1.

The surprising reduction of shear resistance with increasing concrete strength found for the beams NSC1 and HSC1 was reversed when horizontal web steel was provided. With two 25mm web bars in both, the ultimate loads for HSC4 ($f_{cu}$=112.5 N/mm$^2$) and NSC4 ($f_{cu}$=43.3 N/mm$^2$) were 300 KN and 210 kN, respectively. The major increase of shear strength for the HSC beams occurred between HSC1 (without horizontal web bars) and HSC2 (2T12) with ultimate loads of 130 kN and 265 kN. When adding HWB's, HSC3 (2T20) carried 280 kN and HSC4 (2T25) took 300kN.With NSC the influence of horizontal bars was modest; NSC1 (no web bars)-160kN, NSC2 (2T12)-203kN, NSC3 (2T20)- 200kN and NSC4 (2T25)-210kN.

The results for the four high strength concrete beams with horizontal web steel demonstrated that (within the range of presently investigated parameters) no limit (upper ceiling) to improvement(s) in the shear resistance as a result of increasing the area of horizontal web reinforcement was reached. When the diameter of the horizontal web bars was increased from 20 to 25mm a further 7% improvement was recorded.

(b) *Load-deflection behaviour.* Mid-span deflections were measured by a single displacement dial gauge mounted from the laboratory floor. The deflection of beam HSC1 was fairly similar to that of NSC1. Both beams were without any horizontal web bars but NSC1 had 1.55% of compression reinforcement which would have been effective in reducing its deflection but the higher strength and elastic modulus of the concrete in HSC1 with no compression steel appear to have counter-weighed the effect(s) of compression steel in NSC1. The deflection of beam NSC1 was greater than that of NSC4 (2T25) at equal loads with NSC1's deflection near failure being 38% greater. The deflections of HSC2, HSC3 and HSC4 did not change by more than 15% as the area of horizontal web steel was increased in the HSC beams.

(c) *Crack propagation* . As a general observation, in the case of all the ten beams which were subjected to symmetrical two-point loading, at loads of around 40 to 60 kN, small flexural cracks were found to appear at the bottom surface (soffit) within the region of constant bending moment. As the load was increased, new flexural cracks were found to form within the shear spans spreading from the load application sections towards the supports with the flexural cracks located within the shear span(s) gradually becoming somewhat inclined. This was followed by a sudden occurrence of a wide shear crack within one of the shear spans, with this major crack eventually leading to ultimate failure. With the crack angle defined as the angle between a tangent to the crack at the centre of the depth of the beam and its longitudinal axis, the angle of the failure crack for the higher strength concrete beam(s) HSC1 was about 50° compared to the 35° for normal strength concrete beam NSC1.
Beams HSC2, HSC3 and HSC4 had respective major shear crack angles of about 43°, 45° and 42° , compared to the beams NSC2, NSC3 and NSC4 the major shear crack angles of which were 28°, 27° and 27°, respectively.
HSC1 and NSC2 exhibited dowel cracks at the level of the bottom steel. These cracks were formed at 120kN (92% $V_{test}$) and 140kN(64% $V_{test}$), respectively. NSC3 and HSC4 may possibly have had dowel cracks in mid-web formed at 190kN (86% $V_{test}$) and 230kN (77% $V_{test}$). HSC3 and NSC4, on the other hand, developed web dowel cracks at 210kN (75% $V_{test}$) and 200kN (95% $V_{test}$), respectively.

(d) *Shear resistance of HSC beams compared to NSC.* The set of test results presented in Table 2 suggest a potential practical problem with high strength limestone aggregate concrete. When considering these results, one needs to bear in mind that the amount of shear reinforcement used in the HSC beams was below the minima of both EC2 and the Concrete Society recommendations (which are $\rho_w f_{yv} \geq 0.08$ and $\rho_w f_{yv} \geq 0.039 f_{cu}^{2/3}$ , respectively). In Table 2, it is somewhat surprising that, the ratio of the measured ultimate shear strength, $V_{test}$, to the characteristic shear resistance, $V_{Rk}$, for the beam HSC1-1, as calculated by the BS8110's Equation (3) without a limit on $f_{cu}$ and ignoring the requirement on $\rho_w f_{yv}$, is as low as 0.69. Moreover, the results given in Table 2, indicate that the measured ultimate shear strengths of three of the four HSC beams (HSC1-1, HSC1-2 & BJ-2), $V_{test}$, are not only below that of the corresponding measured value for the reference normal strength concrete beam NSC1 (with river gravel as coarse aggregate and 1.55% compression steel) but that even the ratios of $V_{test}/V_{Rk,c}$ (with $V_{Rk,c}$ predictions based on BS8110's approach) for all these three specimens are somewhat less than 1, where $V_{Rk,c}$ ignores the contributions to the shear resistance from the stirrups. In Table 2, in the predictions based on BS8110, for beams without HWB, it is assumed that $f_c = 0.8 f_{cu}$**.** In practice, however, all the beams in Table 2 had stirrups with $\rho_w f_{yv} = 0.47$ N/mm$^2$ , with the additional contribution from the stirrups to the shear resistance of the specimens in Table 2, based on Equation (4), being: Vs=19.1 kN, where $V_{Rk} = V_{Rk,c} + Vs$.
Comparing the mean shear failure load, $V_{test}$ , of 71.7 kN for HSC1-1, HSC1-2 and HSC1-3 with that of NSC1 which had a shear failure load, $V_{test}$ , of 80 kN, the present HSC beams appear to have (on average) 11.6% less shear resistance compared to the equivalent NSC beam.
Past research demonstrates that the shear strengths of HSC members are often below the characteristic resistances calculated according to EC2 and BS8110 (Regan et al, 2005). In reinforced concrete members without shear reinforcement, shear resistance is mainly affected by the transfer of shear forces across cracks with a significant part of the applied shear carried across the flexural cracks. The force transfer across early 45°cracks develops a resistance greater than those anticipated for the 45° truss models when

shear steel is present. The magnitude of the shear transferred across a crack depends on the roughness of the crack surfaces and the width of the crack. Previously reported experimental investigations into the effect of the type of aggregate used in dense concrete on aggregate interlock (Taylor, 1970; Walraven, 1979; Motamed, 1997 and Regan et al, 2005) has indicated that the shear transfer strength of specimens made with limestone aggregate generally fail to increase with increasing concrete strength. The same trend seems to occur (to a lesser degree) in connection with HSC beams without shear reinforcement.

Table 2: Comparison of test results for the five beams without HWB with predictions based on Equations (2) and (4).

| Beam No. | $\rho_i$ % | a/d | $f_c$ (N/mm$^2$) | $V_{test}$ (kN) | $V_{Rk,c}$ (kN) | $V_{test}/V_{Rk}$ Eqn (4) | $V_{test}/V_{Rk,c}$ Eqn. (2) |
|---|---|---|---|---|---|---|---|
| **NSC1** | 1.58 | 3.02 | 34.6 | 80 | 51.6 | 1.08 | 1.44 |
| **HSC1-1** | 0.14 | 3.02 | 94 | 65 | 71.9 | 0.69 | 0.86 |
| **HSC1-2** | 0.14 | 3.02 | 86.2 | 70 | 69.9 | 0.76 | 0.95 |
| **HSC1-3** | 0.14 | 3.02 | 91.6 | 80 | 71.3 | 0.85 | 1.06 |
| **BJ-2** | 1.58 | 4.15 | 103.1 | 71 | 74.2 | 0.74 | 0.90 |

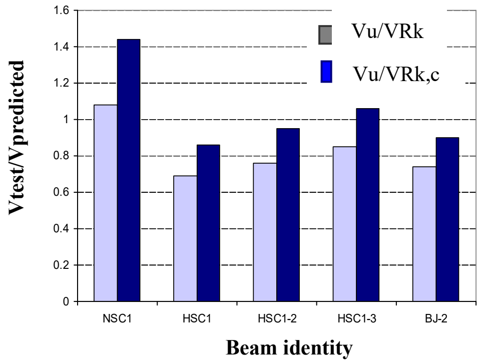



Figure 4: Comparison of the test results and the predictions based on Equations (2) and (4)-after BS8110.

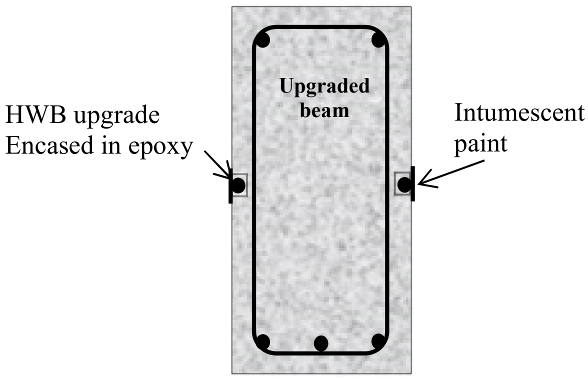



Figure 5: Near Surface Mounted Stainless HWB encased with epoxy resin filling the grooves on either side of the beam within the cover of stirrups

(e) *Load-strain behaviour.* As far as the axial measurements were concerned, Figures 1 and 2 give details of locations for the electrical resistance strain gauges placed on the main tensile bars, the two opposite sides (top and bottom) of the horizontal web bars, $W_t$ and $W_B$, and all the links in the NSC as well as the HSC specimens (whether , with or without HWB's ). In all cases, in the course of the installation of the gauges, once glued to the surface of the reinforcement, the strain gauges were all carefully covered by the application of a previously proven effective protective coating, prior to the casting of the concrete which then followed , in order to ensure that, during the application of the external loads, any potential shear deformations at the gauge/ protective coating/ concrete interface(s) did not have any adverse effect(s) on the gauge outputs (readings) , with the gauges providing reliable test results. It is particularly noteworthy that, as a typical example (Motamed, 2010), in the plots of Figure 6(a, b), the variations of the axial strain measurements with changes in the external loading, for all the links 1, 2 and 3 in all the 3 beams HSC1, HSC4 and NSC4, are found to follow very similar general trends, in the absence of any significant erratic (non-conforming) variations(patterns), providing very encouraging support for the reliability of the presently reported measurements.

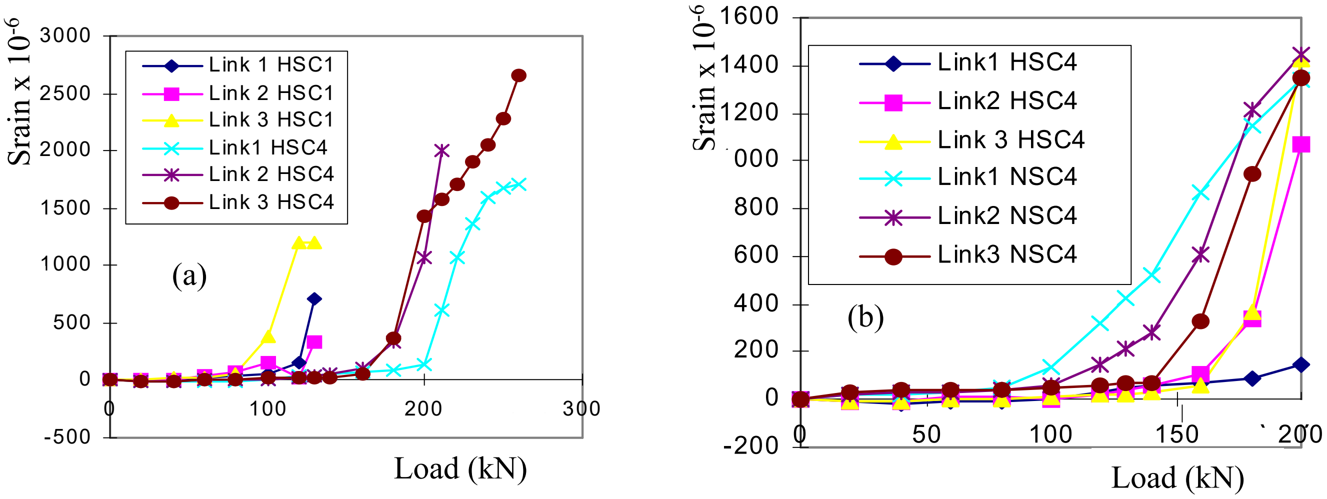



Figure 6: Axial strains in the links 1, 2, 3 of the beams: (a) HSC1, HSC4; and (b)HSC4, NSC4.

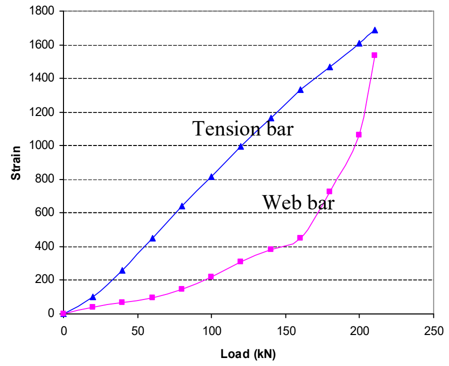
With the reliability of the axial strain measurements verified, in Figure 6b, a comparison is made between the axial strains in the links of the beams HSC4 and NSC4. Both beams had 2T25 horizontal web reinforcement, with (as mentioned previously) the locations of the individual strain gauges given in Figures 1 and 2; the NSC4 links 1, 2 and 3, have yielded at 200 kN, while in the HSC4 links 2 and 3, yielding of the steel has occurred at 200 kN with this followed by the yielding of the link 1 at about 230 kN- hence, suggesting that, at least in this respect (i.e. yielding of the links), the differences between the HSC and NSC specimens were (cf. their differences in failure loads) relatively small. Beam HSC4 continued to sustain load for another further increment of 100 kN after the links 2&3 had yielded, with the yielding of the link 1 followed by a further (additional) increment of 70 kN. The horizontal web bars (2T25) of HSC4, on the other hand, have yielded at 270 kN, Figure 7(d).

One possible explanation, for this observation, is that the horizontal web reinforcement in beam HSC4 was subjected to the stabilising effect(s) associated with an arching action, with this resulting in yielding of the links associated with which were increases in the main steel's axial force near the supports. This tie effect of the tension steel appears to have continued until the main tensile reinforcement has reached 90% of its yield strain at 300 kN at which the beam has failed, Figure 7(d), whereas in beam NSC4 its main (tensile) steel has reached 74% of its yield at failure load of 200 kN, Figure 7(a).

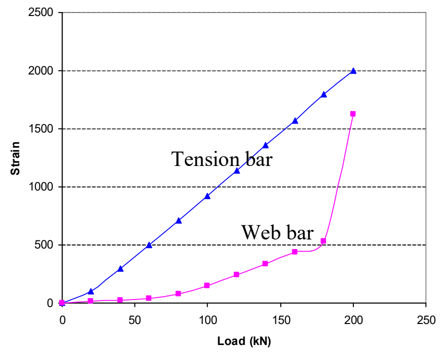
As shown in Figure 6a, in the beam HSC1, link 2 has yielded at about 100 kN, with the link 3 reaching 80% of its yield at 110 kN. In the course of the experiment, shear failure was found to occurr with a sudden formation of a crack located between the links 2 and 3, while link 1 had not yet reached 40% of its yield, and as shown elsewhere (Motamed, 2010), at failure, the axial strain at mid-span of the main (tensile) steel had only reached 40% of its yield.

Past research (Rogowsky and MacGregor, 1986; Hejazi, 1997) has shown, however, that the HWB has little, if any, effect on the shear strength of NSC beams. This is, perhaps, due to the comparatively low crushing strength of NSC which fails before reaching sufficient plasticity (with its associated significant deformations) to cause the main (tensile) bar(s) to yield. Similarly, as shown in Figures 7 (a, b & c), in NSC of a/d=3.05 with HWB and stirrups, the arching action did not develop sufficiently to cause the HWB or the main embedded bars to yield, whereas the plots in Figures 7 (d, e& f) suggest that similar HSC beams with HWB and a/d=3.05 had high enough concrete compressive strength(s) to cause the tension bar(s) to yield.

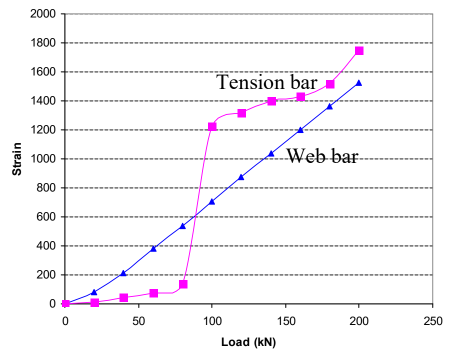
As regards the variations of axial strain(s) at top and bottom faces of HWB's, $W_t$ and $W_B$, respectively, with changes in the external load, for all the HSC and NSC specimens, full details are given in (Motamed, 2010). For the present purposes, in view of the space limitations, only the plots of Wt versus load (as well as the associated variations of axial strain in the main tensile bars versus load) for the six specimens (three HSC and three NSC) with HWB's are presented in Figures 7 (a-f). To start with the beam NSC4, the initial formation of inclined tension cracks was followed by a sudden formation of inclined web cracks at 160 kN which continued to develop further up to 200 kN. The measured axial strain on the bottom face of web bar, $W_B$ , was found to increase in-line with the corresponding readings associated with the top face of the web bar, $W_t$ , until an external load of about 160 kN, after which the bottom face direct strain, $W_B$ , was found to remain constant. In beam NSC3, inclined web cracks developed at 170 kN. It's associated axial strain, $W_B$ , was found to increase in-line with the corresponding $W_t$ readings, Figure 7(b), up to a loading of around 160kN, following which the axial strain $W_B$ was found to remain constant. In beam NSC2, the axial strains $W_B$ and $W_t$ , Figure 7(c), continued to increase in-line with each other up to an external loading of 130 kN, after which $W_B$ remained constant with further increases in the external load.



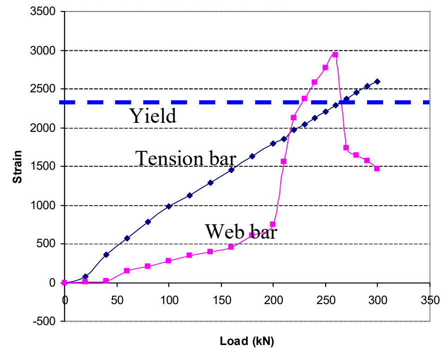

*a) Beam NSC4, strain $W_t$ on top of web bar (T25) and tension reinforcement (T20) not yielding.*



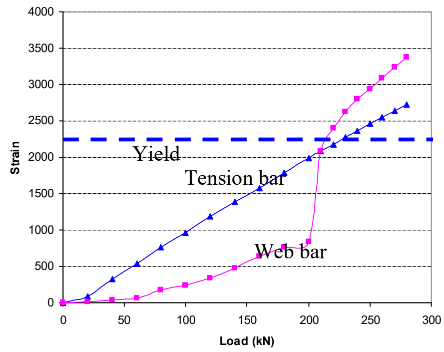

*b) Beam NSC3, strain $W_t$ on top of web bar (T20) and tension bar (T20) not yielding*



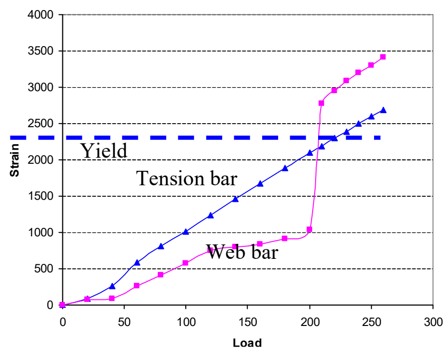

*c) Beam NSC2, strain $W_t$ on top of web bar (T12) and tension reinforcement T(20) not yielding.*




*d) Beam HSC4 with strain $W_t$ on top of web bar (T25) and tension reinforcement (T20) yielding.*



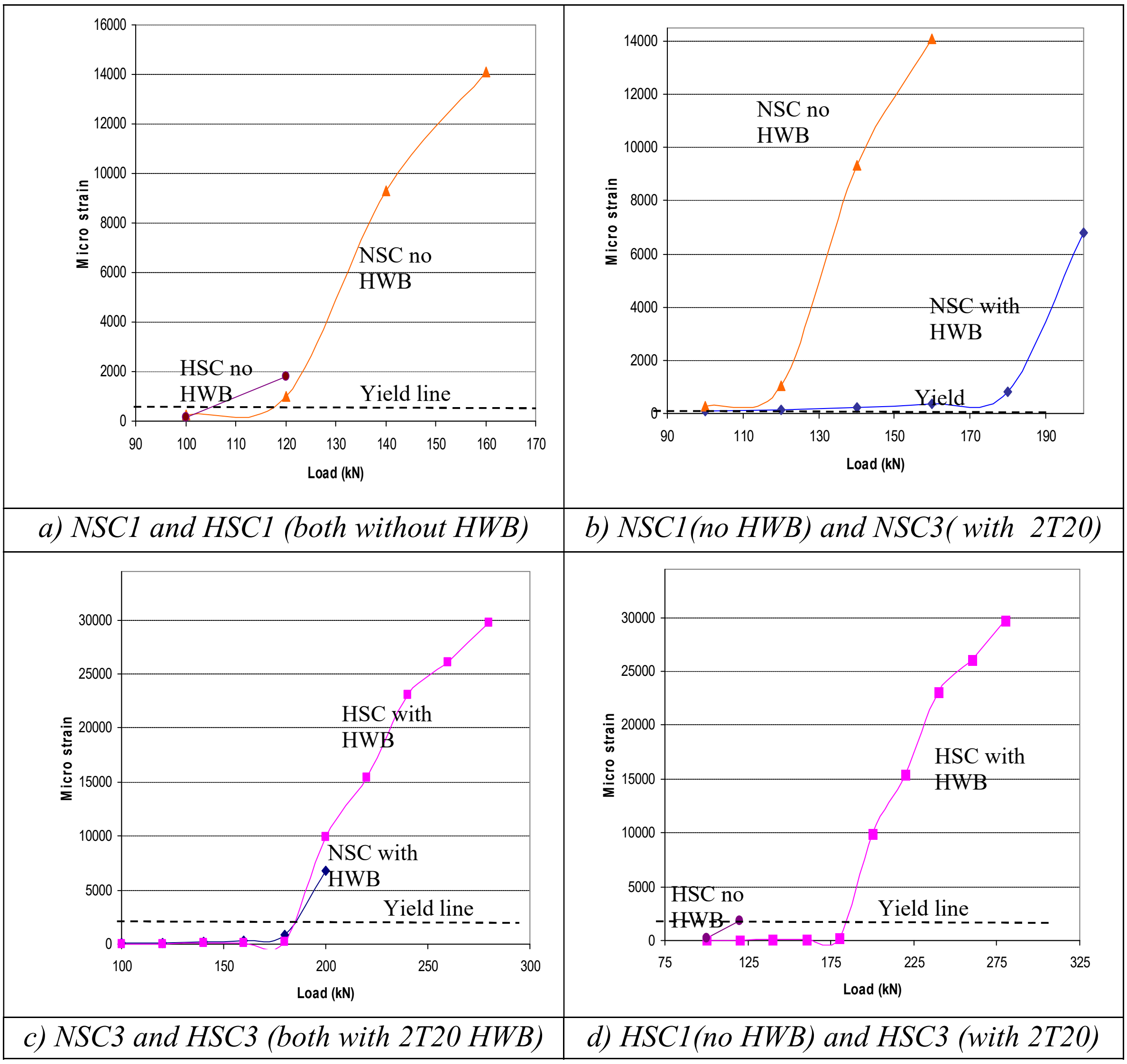

*e) Beam HSC3, strain $W_t$ on top of web bar (T20) and tension reinforcement (T20) yielding.*




*f) Beam HSC2, strain $W_t$ on top of web bar (T12) and tension reinforcement (T20) yielding.*

Figures 7: Measured axial strains in the main (tensile) reinforcements T20 and on the upper part of HWB

The effect of HWB's (with their significant associated dowel action(s)) on the patterns of variations in axial strains in the centre link 2 of the beams NSC1, NSC3, HSC1 and HSC3, are demonstrated in Figures 8, where the axial strains in link(s) 2 of specimens with and without HWB's are compared, noting that, the plots in Figures 8 are based on results for the specimens NSC3 and HSC3 with their associated limiting (maximum) size HWB's (T25) in the present experiments. According to the plots in Figure 8(c), in the case of the beam NSC3, the presence of HWB does not appear to make much difference on the magnitude of axial strains induced in the centre link 2 up until 120 kN. (cf. the specimen NSC1, which had  no HWB, also yielding at 120 kN). Experimental results for beams HSC1 and HSC3, Figure 8(d), demonstrate that, after 120 kN, as the axial strain in the centre link of HSC1 reached $1.8\times10^{-3}$ (corresponding to  138% of its yield value), the beam abruptly failed, whereas in the presence of HWB, the axial strain in the centre link 2 remained as little as $0.17\times10^{-3}$ (equal to 13 % of its yield value), at 180 kN loading. However, due to the formation of large shear cracks, the centre link 2 reached an axial strain of $9.9\times10^{-3}$ (760% of its yield) at 200 kN and that, at this stage, the HWB appear to provide additional shear resistance of 80kN-i.e. 40% increase in load carrying capacity.




*a) NSC1 and HSC1 (both without HWB)*

*b) NSC1(no HWB) and NSC3( with 2T20)*

*c) NSC3 and HSC3 (both with 2T20 HWB)*

*d) HSC1(no HWB) and HSC3 (with 2T20)*

Figures 8: Influence of HWB on the axial strains in  link 2 (located at the centre of the shear span)-for NSC and also HSC specimens.

## MODIFICATION OF BAUMANN'S DESIGN RULE

The shear resistance of rectangular reinforced concrete beams with vertical stirrups can be predicted by the BS8110 approach, using Equation (4), with limits of $\rho < 3\%$ and $f_{cu} < 40$ N/mm$^2$. One way of assessing the total shear resistance of a RC beam strengthened (upgraded) with a single layer of horizontal web steel is to add the predicted HWS's additional dowel action contribution to the original shear resistance of the RC beam, based on Equation (4), $V_{Rk}$ , using a modified (Motamed, 2010) version of Baumann's original predictive expression for dowel action contribution in the case of a beam with one layer of HWB, with the final modified version being:

$$V_{bu}=1.64.\ b_n\ d_b\ (n)^{1/4}.(f_{cu})^{1/3} \tag{5}$$

Moreover, it is important to note that implicit in Equation (5) is the assumption, based on the empirical works of others (e.g. Regan & Hamadi,1980) that, in practice, the movements of the cracks should be sufficiently large for the mobilisation of $\mathbf{V_{bu}}$: this is supported by the previously published measurements of vertical movements at the flexural cracks which have, in the course of experiments, eventually turned into shear cracks. Based on such studies, it appears that, sufficiently large movements for mobilisation of dowel resistance are primarily controlled by the tensile strength of concrete, with a movement of about 0.1 mm (Regan &Hamadi, 1980), generally being adequate for full mobilisation of such dowel action(s). Bearing the above in mind, then, with $\mathbf{V_{bu}}$ adequately mobilised, the presently suggested formula for the prediction of shear resistance of RC beams with stirrups, strengthened with horizontal web bars (HWB's), is:

$$V_{cu} = 0.27\ \ (100 f_{cu}.\rho_i)^{\frac{1}{3}}.\left(\frac{d}{400}\right)^{\frac{1}{4}}.bd\ +\ A_{sv}.d\frac{f_{yv}}{s}\ +\ b_n.d_b.\sqrt[4]{n}.\ f_{cu}^{\frac{1}{3}} \tag{6a}$$

or,

$$V_{cu}= V_{Rk}+ V_s +V_{bu} \tag{6b}$$

where, for our present test specimens: $A_{sv}$ = 56.6 mm$^2$, $f_{yv}$ = 250 N/mm$^2$, $d$= 270 mm & $s$ = 200 mm- hence, $V_s$= 19.1 kN .

Substituting for n=2 and $b_n$= b-2$d_b$ into Equation (5):

$$V_{bu}=1.95\ ( b-2d_b)\ d_b\ f^{1/3}$$

With the values of $V_{Rk}$ for all our test specimens given in Table 1, it is then simple to calculate the values of $V_{cu}$ for all our present test beams, Table 3.

For comparison purposes, in Table 3, other (alternative) results are also presented, with these based on the predictive expression for the shear resistance including the dowel action contribution from the HWB's, after Desai (1995):

$$V_{cu} = 0.27\ \ (100 f_c.\rho_i)^{\frac{1}{3}}.\left(\frac{d}{400}\right)^{\frac{1}{4}}.bd\ \ (1\ +\ 0.40\ \ \rho_b)+\ A_{sv}.d\frac{f_{yv}}{s}\ \le\ V_{\max} \tag{8}$$

with $V_{max}$ calculated from equation (8) with $\rho_b$=1.

## COMPARISON OF SHEAR DESIGN RULES FOR HWB

Table 3 presents the ratios of experimental to predicted values of shear resistance, $V_{test}/V_{cu}$ , based on both the present modified Baumman Equation (6a) and also the Desaie's Equation (8), with the correlations between theory and test data being encouraging in both cases, although (at least for the present (admittedly) rather limited test data) the presently proposed method appears to provide slightly better predictions. Moreover, in order to throw some light on the influence of stirrups on the shear strength of RC beams with HWB's, Table 4 presents results based on two nominally identical set of NSC beams with HWB, with one set of three beams being the presently tested beams NSC2, NSC3 and NSC4 (all three of which had stirrups), and the other set of three (with no stirrups), Figure 8, being those tested to failure by Hejazi (1997). It should be noted that, the NSC beams in Figure 2 have stirrups, compression reinforcement and 3 T20 tension reinforcement compared to those in Figure 8 which have no stirrups or compression steel and 3T16 tension reinforcement. Also noteworthy is the fact that, although in the tests the values of $f_{cu}$ for beams NSC3* and NSC3 (with HWB's of 2T-20) differed by 27%, the differences in $f_{cu}$ for the other two matching pairs of beams in Table 4 were negligibly small (0.03% to 0.08%) to be ignored for all practical purposes. A careful study of the (admittedly rather limited) results in Tables 1- 4 suggest that the modified Baumann design rule for shear prediction including the dowel action of the web bars (HWB's) remains conservative as the diameter of the web bars increase, with the effects of stirrups, in the case of HSC beams, on the shear resistance being less significant than that from the HWB's. When dealing with the NSC specimens, however, the stirrups and the associated HWB's appear to make largely similar contributions to the shear resistance. Finally, it is also worth mentioning that, in both cases of HSC and NSC, inclusion of HWB'S has been found to lead to relatively higher degrees of ductility at failure.

Table 3: Comparison of measured and predicted values of ultimate shear resistance based on the presently proposed and Desai's formulae for beams with HWB's.

| | | **NSC 2** | **NSC 3** | **NSC 4** | **HSC 2** | **HSC 3** | **HSC 4** |
|---|---|---|---|---|---|---|---|
| $\frac{V_{test}}{V_{cu}}$ | Modified Baumman | 1.02 | 0.93 | 0.97 | 1.06 | 1.06 | 1.11 |
| | Desai | 1.00 | 0.86 | 0.93 | 1.04 | 0.98 | 1.05 |

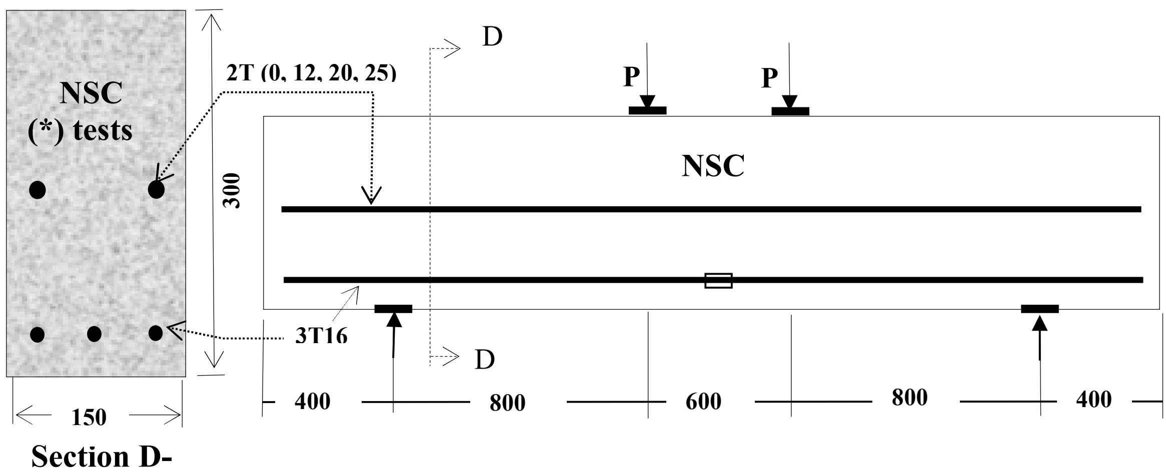



Figure 8: The Four NSC beams of Hejazi(1997), with and without HWB's, corresponding to the present NSC test specimens, but with no stirrups.

Table 4:Accuracy of the predictions, based on Equation (6a), for beams with HWB in the presence or absence of stirrups

| | Web bars | 2T12 | 2T20 | 2T25 | Average value |
|---|---|---|---|---|---|
| | $\rho_b$ % | 0.56 | 1.5 | 2.44 | |
| **NSC** (with stirrups) | $\frac{V_{test}}{V_{cu}}$ | 1.21 | 1.08 | 1.15 | 1.15 |
| **NSC*** (without stirrups) | | 1.06 | 1.02 | 0.95 | 1.01 |

## CONCLUSIONS

The structural behaviour of RC beams strengthened (upgraded) in shear, by using near surface mounted horizontal web bars, located in the vicinity of neutral axis, has been investigated and a simple method (amenable to hand calculations, using a pocket calculator) for predicting the shear resistance of such upgraded beams has been proposed. It has been shown that the presently proposed method provides more accurate predictions of the shear resistance of such upgraded RC beams compared with the current design equations in EC2 and the British code BS8110, with the proposed method being more accurate for high strength concrete beams with stirrups. Indeed, it has been demonstrated that the EC2 and BS8110 design rules for normal strength concrete beams, with stirrups but without horizontal web reinforcement are not valid if extrapolated to high strength concrete beams.

The present tests on normal strength concrete beams with stirrups support the previously reported findings by others that, in the case of normal strength concrete beams, there is a limit to the maximum contribution of HWB to shear resistance of beams with or without links.

Finally, it is perhaps worth mentioning that for durable and effective upgrading, the near surface mounted HWB is to be encased with epoxy resin filling the grooves made on either side of the beam within the cover of stirrups close to the neutral axis, with intumescent paint used to cover the repaired area of the beam for fire protection: in this context,it is also worth bearing in mind that, as previously suggested by others, the location of horizontal web bars in practice (which is in the vicinity of the neutral axis) would also prove advantageous.

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## NOTATION

| | | | |
|---|---|---|---|
| a | shear span from the centre of a point load to the centre of a support (mm) | $s$ | spacing of the shear links along the beam (mm) |
| $A_b$ | cross-section area of the horizontal web steel (mm$^2$) | $V_{bu}$ | contribution of the HWB's to Vcu (kN) |
| $A_{st}$ | total area of tension steel (mm$^2$) | $V_{Rk}$ | calculated characteristic shear strength (N) |
| $A_{sv}$ | cross-section area of a link (mm$^2$) | $V_{Rk,c}$ | calculated characteristic shear strength of a |
| b | breadth of the beam (mm) | $V_s$ | beam without shear reinforcement (N)<br>contribution of stirrups to the shear resistance of |
| $b_n$ | net breadth of the beam at the level of dowels reinforcement (mm) | $V_{test}$ | the beam (N)<br>measured ultimate shear strength (kN) |
| $b_w$ | web width of the beam (mm) | $V_{cu}$ | ultimate shear resistance including the HWB's contributions (kN) |
| $d$<br>$d_b$ | effective depth measured to the centre of main tensile reinforcement (mm)<br>diameter of each HWB (mm) | $\rho_b$ | area ratio of the horizontal web reinforcement ($A_b$/bd) |
| $f_c$<br>$f_{ct}$ | concrete cylinder compressive strength (N/mm$^2$)<br>concrete indirect tensile strength (N/mm$^2$) | $\rho_l$ | area ratio of the main (tensile) reinforcement ($A_{st}$/bd) |
| $f_{cu}$<br>$f_{yl}$ | concrete cube compressive strength (N/mm$^2$)<br>yield strength for the longitudinal reinforcement (N/mm$^2$) | $\rho'_l$ | area ratio of the compression reinforcement ($A'_s$/bd) |
| $f_{yv}$<br>n | yield strength of the shear stirrups (N/mm$^2$)<br>number of dowel bars | $\rho_w$ | area ratio of the web reinforcement (vertical stirrups) ($A_{sv}$/bd) |

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