Shear behaviours of hybrid continuous deep beams strengthened with carbon fibre reinforced polymer

H M Al-Mutairee¹ and H A Al-Hamdani²

¹Department of Civil Engineering, University of Babylon, Iraq
Email: eng.hayder.mk@uobabylon.edu.iq, hayderalmutairee@gmail.com, Ass. Professor
²Department of Civil Engineering, University of Babylon, Iraq
Email: hussein.ali84@gmail.com, Ph.D. Student

Abstract. This paper investigates the shear strength characteristics of reinforced concrete continuous deep beams (RCCDB) consisting of homogenous versus hybrid concrete, and further looks at the effect of Carbon Fibre Reinforced Polymer (CFRP) on the shear strength of hybrid RCCDBs. The experimental work included testing ten specimens of RCCDB under two-point loads. The effects of high strength concrete (HSC) layer thickness and CFRP on the strength of RCCDB were thus studied. The experimental results showed that the strengthening of RCCDB by an HSC layer at the top is better than that from one at the bottom, as the increments in ultimate load strength were 17 and 34% for top strengthening and 8 and 26% for bottom strengthening for 25 and 50% thickness of total depth of beam, respectively. The optimal strengthening of RCCDB using an HSC layer at the top was at 50%. Shear strengthening using 5 cm width CFRP strips for hybrid RCCDBs increased the ultimate shear strength by 69% and 49% for strengthening with inclined and straight shapes, respectively, where such strengthening was applied near to the centre of the specimens.

Keywords: Continuous deep beam, Hybrid concrete, Carbon fibre, Shear behaviour.

1. Introduction

Deep beams represent an important structural element, being loaded as beams but having smaller shear-span to depth ratios. A deep beam, in general, has a depth much greater than normal, while the thickness in the perpendicular direction is much smaller than either its span or depth. ACI-code 318R-08 defines deep beams as those that have clear span to overall depth ratio less than four, ln/h ≤ 4, or a shear span to effective depth ratio less than two (a/d ≤ 2); the beam should also be loaded on one face and supported on the opposite face, such as compression struts that develop along the lines of loads and supports. Reinforced concrete deep beams are widely used in many structural engineering applications, such as transfer girders, pile caps, offshore structures (caissons), shear walls, wall footings, floor diaphragms, and complex foundation system, (ASCE committee 426). The shear strength of RCCDB increases with decreases in the a/d ratio, as found in simply supported deep beams. The position of maximum negative moment and the position of maximum shear force coincide in continuous beams, and the point of inflection is located next to the critical section of shear. The available empirical equations depend on the results of tests of simply supported beams, and thus cannot be applied to continuous beams without amendment. The case of continuous deep beams contains many parameters that differ from simple span beams, and until recently, few experimental works have been carried out to introduce empirical strength prediction equations for continuous deep beams (Kong, 2003). In deep beams, a significant proportion of the loads are transmitted to the supports directly by arch action, as shown in figure 1. The arch action phenomenon not only permits the transfer of the concentrated load directly to the support but also reduces the contribution of the other types of shear transfer. The major factors influencing this phenomenon are the ratio of shear span to effective depth (a/d) and the compressive strength of the strut. The compressive strength of the strut is closely dependent on the compressive strength of concrete and the tensile strength provided by the reinforcement at the top level.

Content from this work may be used under the terms of the Creative Commons Attribution 3.0 licence. Any further distribution of this work must maintain attribution to the author(s) and the title of the work, journal citation and DOI.
Published under licence by IOP Publishing Ltd
Maha (2013) showed results that for beams made with HSC (about 45 MPa) layers in a compression zone of thickness 25 to 50% of total beam depth: the ultimate shear strength was increased by about 11.2 to 19.5% for beams without web reinforcement and 16.75 to 22.25% for beams with minimum web reinforcement, respectively. The first cracking load was increased by about 32.8 to 48% and 43.4 to 57.9% for beams without and with web reinforcement, respectively. Salam’s (2014) experimental test results confirmed that the strengthening technique of a Near Surface Mounted CFRP system is applicable and can increase the shear capacity of strengthened RC beams. Salam concluded that the increments in the ultimate load capacity of the strengthened beams were between 13 and 33%. These strengthened beams also showed lower deflection at corresponding loads than the unstrengthened reinforced concrete beam.

2. **Experimental work**

2.1. **Description of specimens**

The experimental programme for the current work included eight hybrid and two homogenous RCCDB specimens with cross section dimensions of 300 mm overall depth and 100 mm width. All specimens were tested under two-point loads on the top surface at the centre of span. All details of the geometry and the reinforcement of the specimens is shown in figure 2. The clear concrete cover of the reinforcement was 25 mm, and the specimens were reinforced with two Ø12 mm deformed bars, which provided main reinforcement at the top and at the bottom, and three closed stirrups of Ø5 mm at the ends and at the middle support in order to affix the main reinforcement.

![Figure 2. Geometrical details and steel reinforcement of all specimens.](image)

2.2. **Specimens strengthened by HSC**

This group consists six specimens strengthened by an HSC layer, as shown in figures 3 to 8.

2.3. **Specimens of hybrid concrete strengthened by CFRP**
These four specimens were strengthened with CFRP as well as HSC, as shown in figures 9 to 12. The thickness of CFRP sheets used in this study was 0.17 mm, and the shape was a U shape (open). The load to first crack, ultimate load, and type of failure for all specimens are listed in table 1.

Figure 3. Specimen 3DB1 full NSC.

Figure 4. Specimen 3DB2 full HSC.

Figure 5. Specimen 3DB3 25% HSC from top.

Figure 6. Specimen 3DB4 50% HSC from top.

Figure 7. Specimen 3DB5 25% HSC from bottom.

Figure 8. Specimen 3DB6 50% HSC from bottom.

Figure 9. Specimen 3DB1S Hybrid RCCDB of 25% HSC from top reinforced with CFRP for shear near the interior support.
Figure 10. Specimen 3DB2S Hybrid RCCDB of 25% HSC from top reinforced with CFRP for shear near the exterior support.

Figure 11. Specimen 3DB3S Hybrid RCCDB of 25% HSC from top reinforced with CFRP for shear near the interior and exterior supports.

Figure 12. Specimen 3DB4S Hybrid RCCDB of 25% HSC from top reinforced with CFRP for shear inclined at 45° near the interior support.

Table 1. Specimens codes and descriptions.

| Beam code | Description |
|-----------|-------------|
| 3DB1      | Full normal strength concrete |
| 3DB2      | Full high strength concrete |
| 3DB3      | 25% HSC from top |
| 3DB4      | 50% HSC from top |
| 3DB5      | 25% HSC from bottom |
| 3DB6      | 50% HSC from bottom |
| 3DB1S     | 25% HSC from top with two shear strips of 5 cm width near the interior support |
| 3DB2S     | 25% HSC from top with two shear strips of 5 cm width near the exterior support |
2.4. The properties of Materials

2.4.1. Concrete
The following materials were used to cast all specimens:

- Portland cement (Iraqi manufactured) of the karasta type.
- Fine aggregate (sand) of maximum particle size of 4.75 mm.
- Semi-crushed gravel of a maximum particle size of 14.5 mm.
- Water reducing admixture to reduce the water-cement ratio in mix design.
- Tap water was used to mix the contents of the mixture as well as for curing the specimens.

All materials were selected according to ASTM and Iraq Specifications (Cement Iraqi Manufactured, Sand and Gravel Iraq specification NO 45/1984).

2.4.2. Reinforcement Bars
Two types of deformed bars were used: the first, of size Ø12 mm steel, provided longitudinal reinforcement, and the second, of size Ø5 mm, was used for lateral closed ties.

A test of tensile strength of reinforcement carried out for all specimens. The test was performed according to ASTM A370 in the laboratory of the Material Engineering College, University of Babylon. The obtained results are given in table 2 below.

```
| Trade diameter (mm) | Actual diameter (mm) * | Yield strength (MPa) * | Ultimate strength (MPa) * |
|---------------------|------------------------|------------------------|--------------------------|
| 5                   | 4.7                    | 520                    | 630                      |
| 12                  | 11.8                   | 550                    | 690                      |
```

* Each result represents the average of three specimens.

2.4.3. Carbon Fibre Reinforced Polymer (CFRP)
Sika® CarboDur® sheets are a type of pultruded carbon fibre reinforced polymer (CFRP) sheet selected for strengthening concrete. The mechanical properties of CFRP plates are given in table 3; these are taken from the manufacturing specifications of the Sika Company.

```
| Properties                         | Sika® CarboDur® XS514/80 |
|------------------------------------|--------------------------|
| Tensile strength MPa               | 4'900 N/mm²              |
| E-modulus MPa                      | 230'000 N/mm²            |
| Density                            | 1.80 g/cm³               |
| Fibre Orientation Degree           | 0°                       |
| Strain at break (min. value)       | 2.1%                     |
| Thickness mm                       | 0.17 mm                  |
```
Sikadur®-330 is a thixotropic material consisting of two parts, epoxy resins and special filler. Its main properties, as supplied by the manufacturer, are shown in table 4. Figure 13 shows the CFRP and epoxy used in the strengthening of specimens.

Table 4. Technical properties of adhesive material.

| Properties                      | Sikadur®-330          |
|---------------------------------|-----------------------|
| E-modulus (MPa)                 | **Flexural**: 3800 N/mm² |
|                                 | **Tensile**: 4500 N/mm² |
| Compressive Strength (MPa) (N/mm²) | 50-95                |
| Tensile strength (MPa)          | 30 N/mm² (7 days at +23 °C) |
| Shear Strength (MPa)            | 3-19                  |
| Bond Strength (MPa)             | concrete failure > 4  |
| Density                         | 1.3 kg/l + 0.1 kg/l (parts A+B mixed) (at +23 °C) |
| Mixing                          | Part A : part B = 4 : 1 by weight or volume |
| Layer Thickness                 | 30 mm max.            |
| Change of Volume                | 0.04%                 |

Figure 13. CFRP and epoxy used in strengthening of specimens.

2.5. Properties of Hardened Concrete

The compressive strength of cubes ($f_{cu}$) and splitting tensile strength ($f_t$) of concrete for all specimens are tabulated in table 5. The cube specimens had dimensions of 150 mm × 150 mm × 150 mm for $f_{cu}$, while the cylindrical specimens had dimensions of 100 mm × 200 mm for $f_t$. The modulus of rupture test was carried out on concrete prisms of 100 mm × 100 mm × 300 mm.

Table 5. Details of Properties of Concrete.

| Specimen No. | $f_{cu}$ (MPa) | $f_t$ (MPa) | Modulus of rupture ($f_t$) (MPa) |
|--------------|----------------|-------------|---------------------------------|
| Concrete strength | NSC | HSC | NSC | HSC | NSC | HSC |
| 3DB1         | 36.5 | ----- | 3.245 | ----- | 4.195 | ----- |
| 3DB2         | ----- | 78.8 | ----- | 6.31 | ----- | 7.06 |
| 3DB3         | 34.7 | 71.7 | 3.058 | 5.74 | 4.008 | 6.49 |
| 3DB4         | 34.7 | 71.7 | 3.058 | 5.74 | 4.008 | 6.49 |
| 3DB5         | 36.2 | 71.1 | 3.19 | 5.69 | 4.14 | 6.44 |
| 3DB6         | 36.2 | 71.1 | 3.19 | 5.69 | 4.14 | 6.44 |
| 3DB1S        | 35.7 | 72.5 | 3.146 | 5.8 | 4.096 | 6.55 |
| 3DB2S        | 35.7 | 72.5 | 3.146 | 5.8 | 4.096 | 6.55 |
| 3DB3S        | 37.6 | 72.5 | 3.311 | 5.5 | 4.261 | 6.55 |
| 3DB4S        | 37.62 | 72.5 | 3.311 | 5.8 | 4.261 | 6.55 |

* Mix design as illustrated in table A1.
\[ bE_{c}(\text{NSC}) = 4700 \sqrt{f_c} \quad (\text{ACI-318}). \]
\[ cE_{c}(\text{HSC}) = 3320 \sqrt{f_c} \quad (\text{ACI-363}). \]

2.6. **Testing Machine**

All specimens were tested using a hydraulic testing machine in the lab of the Civil Engineering Department of Babylon University, shown in figure 14.

![Hydraulic testing machine](image1)

Figure 14. Hydraulic testing machine.

The structural behavior of all specimens was detected and the important characteristics recorded at each stage of loading. The deflection was measured using a dial gage of accuracy 0.01 mm fixed at mid-span; figure 15 shows the instruments used in testing. After the specimen was placed on the supports of machine, the first readings of the gages were recorded, then the beam was loaded at a constant rate. The value of deflection was recorded at each stage of loading, and the first positive and negative flexural crack load, first shear crack, fixed cracking patterns, and the failure load were also recorded.

![Instruments used in testing](image2)

(a) Dial Gauge  (b) Crack Meter(Microscope)

Figure 15. Instruments used in testing.

3. **Experimental results**

Two-point loads were applied on all specimens. The load at first crack, ultimate load, type of failure, and the change in ultimate load with respect to the control beam 3DB1 for all tested beams are tabulated in table 6. As shown in this table, when the thickness of the high strength layer increases from 0 to 100% an increase in the failure load of more than 45% is obtained. The 25% HSC at the top gave an increase in ultimate strength of 7.7% when compared with the specimen with 25% HSC at the bottom; 50% HSC at the top also gave an increase in ultimate strength of 6.2% when compared with the specimen with 50% HSC at the bottom. The specimen with full HSC showed an increase in first shear crack load of 48.2% when compared with a specimen with full NSC. Shear resistance increases may arise from the increase in moment of inertia of the gross section due to the increase in ultimate compressive strength. The first flexure cracks in positive and negative moment regions of some specimens did not appear due to increments in the main steel reinforcement, which led to a change in the mechanism of failure. From table 6, it can also be observed that specimen 3DB1S gave an increase
of 75% in failure load when compared with specimen 3DB1 due to strengthening by stirrups of CFRP; closer locations fared better than far away ones, however, as the critical shear zone is near the middle support. Strengthening with stirrup CFRP near the outside supports thus had little effect on the ultimate load. The specimen (3DB3S) with mixed strengthening for shear both near to and far from the middle support did not improve in the strength compared with the specimen strengthened with CFRP near the middle support only. The specimen with inclined CFRP strips (3DB4S) fared better than the other shear strengthening options as it was approximately perpendicular in cracking and the development length was greater than that of vertical strengthening in the same location with the same width of strips; thus, the inclined CFRP gave the best strengthening for this group. The first shear crack load in specimen 3DB1S was higher than the first shear crack load of specimen 3DB2S due to the location of the CFRP: the first shear crack of specimen 3DB1S occurred in the region far from the centre while the first shear crack of specimen 3DB2S occurred in the region near the centre. The first shear crack load of specimen 3DB3S was also higher than the first shear crack load of specimen 3DB1S. Figures 24 to 27 show the failure of specimens in group 3, which were strengthened with CFRP. The increments in ultimate load when hybrid RCCDB (Beam 3DB3) was strengthened using two strips of CFRP (3DB1S and 3DB2S) were 49.5% and 18.1%, respectively. Thus, the strengthening by shear CFRP near the middle support is more efficient. Figures 16 and 17 show the load deflection of all specimens.

Table 6. First crack, ultimate load, and type of failure.

| Beam code | Load at the first positive flexural crack (KN) | Load at first negative flexural crack (KN) | Load at the first shear crack (KN) | Type of Failure | Mode of Failure | Ultimate load (KN) | Increment in ultimate load* (%) |
|-----------|---------------------------------------------|-------------------------------------------|----------------------------------|----------------|----------------|-------------------|-------------------------------|
| 3DB1      | 160                                         | 110                                       | 110                              | shear          | Inclined crack  | 179               | 0%                            |
| 3DB2      | 190                                         | ----                                      | 163                              | shear          | Inclined crack  | 260               | 45%                           |
| 3DB3      | 121                                         | ----                                      | 148                              | shear          | Inclined crack  | 210               | 17%                           |
| 3DB4      | 146                                         | 136                                       | 155                              | shear          | Inclined crack  | 240               | 34%                           |
| 3DB5      | 172                                         | ----                                      | 140                              | shear          | Inclined crack  | 195               | 8%                            |
| 3DB6      | ----                                        | 143                                       | 141                              | shear          | Inclined crack  | 226               | 26%                           |
| 3DB1S     | 152                                         | ----                                      | 204                              | shear          | Inclined crack and debonding | 314               | 75%                           |
| 3DB2S     | 145                                         | 173                                       | 168                              | shear          | Inclined crack  | 248               | 38%                           |
| 3DB3S     | 146                                         | ----                                      | 250                              | shear          | Inclined crack and debonding | 308               | 72%                           |
| 3DB4S     | 140                                         | 220                                       | 220                              | shear          | Inclined crack and rupture   | 355               | 98%                           |
From figure 16, the deflection at the same load (179 kN) decreased from 4.2 mm in NSC to 1.49 mm for the full HSC specimen. Also, the deflection of the full HSC specimen was less than that of the full NSC by 181.8% at load 179 kN. The load deflection curve of the hybrid RCCDB specimen with HSC layer at the top shows less deflection than the hybrid specimens with HSC layers at the bottom. From figure 17, there is a reduction in deflection at load 210 kN in specimen 3DB3 from the results of specimens 3DB1S, 3DB2S, 3DB3S, and 3DB4S by 86.4%, 17%, 74.2%, and 65.5% respectively. Figures 18 to 23 show the failure mode of specimens unstrengthened with CFRP, while figures 24 to 27 show the failure modes of strengthened beams with CFRP.
Figure 18. Specimen (3DB1) after testing.

Figure 19. Specimen (3DB2) after testing.

Figure 20. Specimen (3DB3) after testing.

Figure 21. Specimen (3DB4) after testing.
Figure 22. Specimen (3DB5) after testing.

Figure 23. Specimen (3DB6) after testing.

Figure 24. Specimen (3DB1S) after testing.

Figure 25. Specimen (3DB2S) after testing.
4. Conclusions

According to the experimental results, where the shear failure is controlled, the following conclusions can be drawn:

1- The ultimate load is enhanced with an increase in the thickness of the HSC layer; where the thickness of HSC layer increased from 0 to 100%, this increment was 45%.
2- The addition of 25% HSC at the top gave an increase in ultimate strength of 7.7% when compared with the specimen with 25% HSC at the bottom; similarly, the 50% HSC at the top gave an increase in ultimate strength of 6.2% when compared with the specimen with 50% HSC at the bottom. Thus, strengthening RCCDB by using an HSC layer at the top is better than adding one at bottom for shear control.
3- The specimen with full HSC showed an increase in first shear crack load of 48.2% when compared with the specimen with full NSC.
4- The load deflection curve of the hybrid RCCDB specimens with an HSC layer from the top showed less deflection than the hybrid specimens with HSC layers from the bottom.
5- The strengthening of hybrid RCCDB using shear CFRP vertical strips near the middle support is better than with strips further from the centre.
6- The strengthening of hybrid RCCDB using the proposed shear CFRP vertical strips near the middle support increased the ultimate load by 49.5%.
7- The strengthening of hybrid RCCDB with the proposed shear CFRP inclined strips represents the best option tested, with an increment in the ultimate load of 69%.
5. References

[1] ACI-ASCE Committee 426 1973 Shear Strength of Reinforced Concrete Members Proceedings ASCE ASCE Proc. 99 1091–1187

[2] Kong F 2002 Reinforced concrete deep beams

[3] Ammar P and Ali Y 2015 Experimental Investigation and Nonlinear Analysis of Hybrid Reinforced Concrete Deep Beams 8

[4] Khaleel H S 2014 Shear Strengthening of Reinforced HSC Continuous Beams with Near Surface Mounted CFRP

[5] Iraqi Specification NO.45 1984 Aggregate from natural sources for concrete and building construction vol 45

[6] ASTM-A370 1987 Standard Test Methods and Definitions for Mechanical Testing of Steel Products ASTM

[7] C S W-301 2014 product Data Sheet Edition www.sika.my.

[8] Sikadur-330 2002 Product Data Sheet Proteins

[9] ACI 363R-84 1984 State of the art report on high strength concrete (Detroit: American Concrete Institute)

6. Appendix A: Mix Design

It was found from the experiments on trial mixes that the mix proportion presented in Table 3.9 gave adequate strength and workability. The superplasticizer allowed enough mixing time and permitted the production of a uniform mix of concrete without any segregation.

Table A1. Properties of Concrete Mixes.

| Parameter                     | Normal strength concrete (NSC) | High strength concrete (HSC) |
|-------------------------------|--------------------------------|-----------------------------|
| Water/cement ratio (%)        | 0.45                           | 0.3                         |
| Water (kg/m³)                 | 204                            | 145                         |
| Cement(kg/m³)                 | 454                            | 485                         |
| Fine aggregate(kg/m³)         | 771                            | 800                         |
| Coarse aggregate (kg/m³)      | 907                            | 1120                        |
| Superplasticizer(l/m³)        | -                              | 4.85*                       |
| Density(kg/m³)                | 2336                           | 2550                        |

* 1 Litre/ 100 kg cement
