Flexure Behavior of Hybrid Continuous Deep Beam Strengthened by Carbon Fiber Reinforced Polymer

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Abstract

This study present an experimental investigation for overall flexure behavior of reinforced concrete continuous deep beams (RCCDB) made of hybrid concrete, normal strength concrete (NSC) and high strength concrete (HSC) at different location and percentage.

The experimental work includes testing of sixteen specimens of RCCDB under two points loads. The effects of HSC layer thickness and CFRP on strength of RCCDB had been studied.

The experimental results showed that the strengthening of RCCDB by HSC layer from top is better than from bottom, where the increment in the ultimate flexural strength increased by (14,21,27)% for top strengthening and (12,15,13)% for bottom strengthening for (25,50,75)% thickness of total depth of beam respectively. The optimal strengthening of RCCDB by HSC layer at top was of 25%. The results also proved that the strengthening of hybrid RCCDB by (10,15)cm CFRP strip at the bottom for flexure gave increment in the ultimate strength by (32, 29)% respectively, and the strengthening by CFRP strip for flexure at the bottom is better than at top for hybrid RCCDB. The shear strengthening of hybrid RCCDB increases the ultimate strength by 23.4% and 13.8% if the strengthening has O and U shape respectively.

Keywords: Continuous Deep Beam, hybrid, strengthening by CFRP

الخلاصة :

يتضمن هذا البحث دراسة تصرف الانثناء العام للأعتاب الخرسانية الهجينة العميقة المستمرة المتكونه من خرسانه طبيعة المقاومة وخرسانة عالية المقاومة لنسب ومواقع مختلفة من المقاطع . تضمن الجزء العملي من البحث فحص ستة عشر عتب مستمر هجين تحت تأثير نقطتي تحميل لدراسة عمق وموقع الخرسانة عالية المقاومة وكذلك تأثير التقوية بواسطة الالياف الكربونية.

اظهرت النتائج المختبرية ان تقوية الاعتاب الخرسانية المستمرة العميقة الهجينة المتكونة من خرسانة عالية المقاومة في الطبقات العليا افضل من تقويته في الطبقات السفلى ، حيث كانت الزيادة في تحمله الاقصى للانثناء (27,21,14)% للأعتاب التي تم تقويتها من الاعلى و (12,15,13)% للأعتاب التي تم تقويتها من الاسفل لنسب تقويه هي (75,50,25)% من العمق الكلي للعتب على التوالي.ان أفضل تقويه للأعتاب الخرسانية الهجينة المستمرة هي بوضع 25% خرسانة عالية المقاومة في الطبقة العليا.

اما بالنسبة للاعتاب المقواة بالألياف الكاربونية (المقواة بشرائح بعرض 15و10 سم) من الاسفل فقد اعطت زيادة في مقاومة الانثاء القصوى بنسبة (29%و32%) على التوالي وأكدت النتائج إن وضع الاياف الكاربونيه في الاسفل افضل من وضعها في الاعلى من اجل الحصول على مقاومه اعلى للانثناء لهذا النوع من الأعتاب. ان تقويه القص للاعتاب الخرسانية الهجينة العميقة المستمرة يؤدي الى زيادة المقاومة بنسبة 23.4% و 13.8% اذا ما كانت التقوية على شكل حلقي وحرف U على التوالي .

الكلمات المفتاحية :- الاعتاب الخرسانية المستمرة العميقة ، والمقواة باللالياف الكاربونية .

1. Introduction

Reinforced concrete continuous deep beam is a subject of considerable interest in structural engineering practice. A deep beam is a beam having a depth much greater than span length. RCCDB is fairly commonly used as load distribution elements such as transfer girders, pile caps, tanks, folded plates, and foundation walls, often receiving many small loads and transferring them to a small number of reaction points (**Kong *et.al.*, 1978**).

RCCDB develop a distinct ‘tied arch’ or ‘truss’ behavior that are not found in shallow continuous beams. The net result of this is that conventional reinforcement detailing rules, based on shallow beams or simply span deep beams, are not necessarily appropriate for continuous deep beams. The coexistence of high shear and high moment within the interior shear span in continuous deep beams has a considerable effect on the development of cracks, leading to a significant reduction in the effective strength of the concrete strut, which is the main load transfer part in deep beams (**Kong, 2003**).

There is no universally accepted definition of deep beam. In general, European deep beams are approximately twice as deep as North American deep beams. For example, **CEB-FIP (1970 and 1982)**(European Committee for Concrete) suggests that simply supported beams of span/depth ratio *L/D* (where *L* is the beam span in m, the smaller of the centre to centre span, or 1.15 times the clear span; *D* is overall beam depth in m) less than 2 and continuous beams of *L/D* ratio less than 2.5 be designed as deep beams. **(ACI-318code)** suggests that beams with clear span to overall depth ratios are not greater than 4 (and loaded at the top or compression face) be treated as deep beams. The ACI deep beam definition is based on shear behavior while CEB definition is based on flexural behavior. It is important to recognize the different definitions when reviewing design recommendations. In reality the deep beam problem is a coupled problem (**Kong and Singh, 2003**).

Deep RC with smaller (a/d) ratio exhibits higher load carrying capacity, less deformation , and lower ductility than of higher (a/d) ratio. Increasing concrete compressive strength leads to a more brittle behavior with increased the load carrying capacity and stiffness at different levels **(Beshara, 2014)**.

2. Experimental Work

2.1 Description of Specimens

The experimental program includes twelve hybrid and four homogenous RCCDB that had cross section dimensions of (300mm) overall depth and (100mm) width. All specimens were tested under two points loads at top surface and at the center of span. Figure (1) shows the geometrical details and the steel reinforcement of specimens

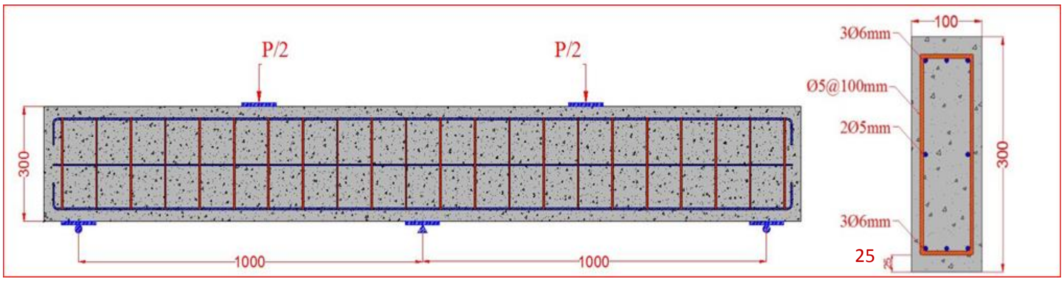

Figure (1): Geometrical details and steel reinforcement of all specimens.

The clear concrete cover of the reinforcement was (25mm). The specimens were reinforced with three (Ø6mm) deformed bars and they provided as main reinforcement at each top and the bottom and two (Ø5mm) deformed bars were provided as horizontal stirrups at mid depth and provided closed stirrups of (Ø5mm) spaced at (100mm along the specimen).

2.2 Specimens strengthed by HSC.

This group consists eight specimens strengthened by HSC layer as shown in Figure(2) to Figure(9).

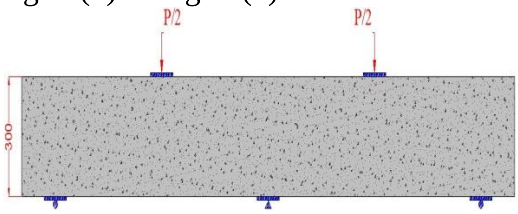

Figure (2): 1DB1 full NSC.

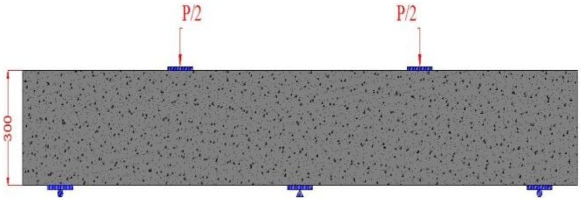

Figure (3):1DB2 full HSC.

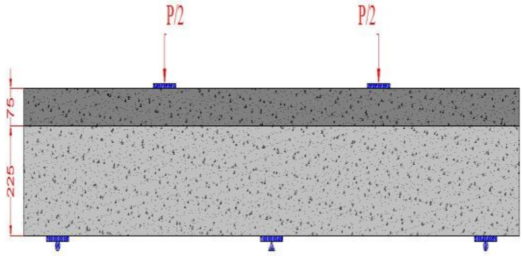

Figure (4):1DB3 25%HSC from top.

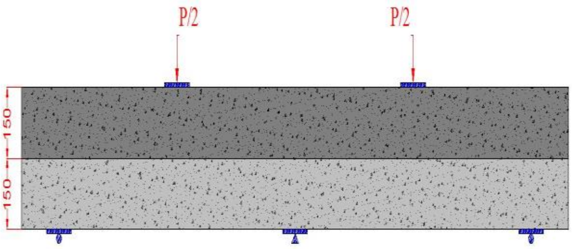

Figure (5):1DB4 50%HSC from top.

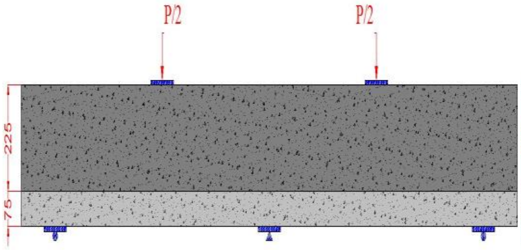

Figure (6): 1DB5 75%HSC from top.

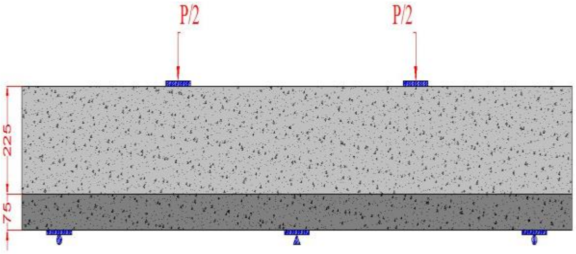

Figure (7): 1DB6 25%HSC from bottom.

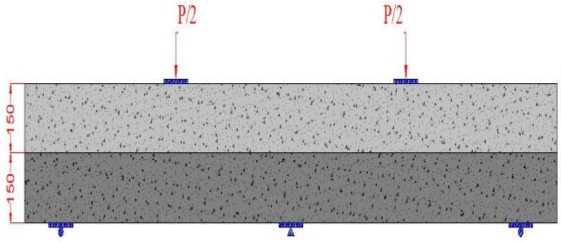

Figure (8): 1DB7%HSC from bottom.

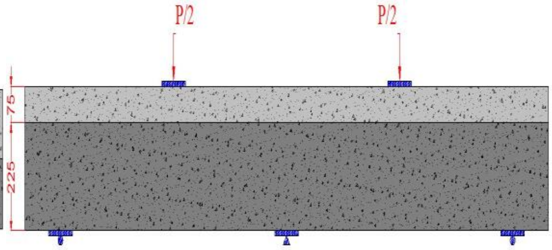

Figure (9): 1DB8 75%HSC from bottom.

2.3 Specimens strengthed by HSC and CFRP

These specimens are consist eight specimens strengthed with CFRP as well as HSC as shown in Figure (10) to Figure (16). The thickness of CFRP sheets used in this study was (0.17mm).The percentage of CFRP strip of width 10cm equals to 0.056% from beam cross section while the percentage of CFRP of width 15cm equals to 0.0085% from beam cross section.The details of concrete and reinforcement as mentioned previously. The load of first crack, ultimate load and type of failure of these specimens are listed in Table (1).

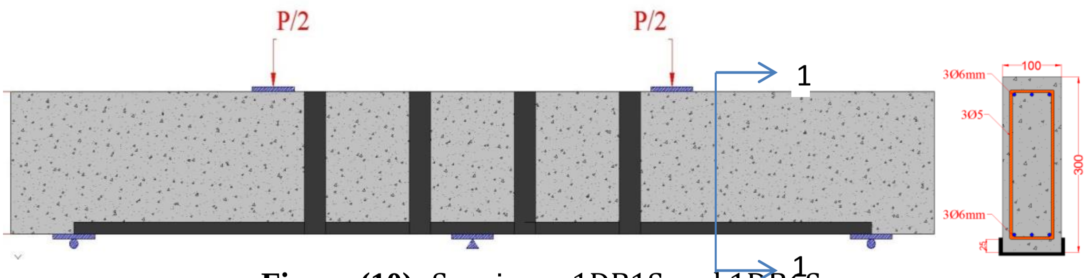

Figure (10): Specimen 1DB1S and 1DB6S.

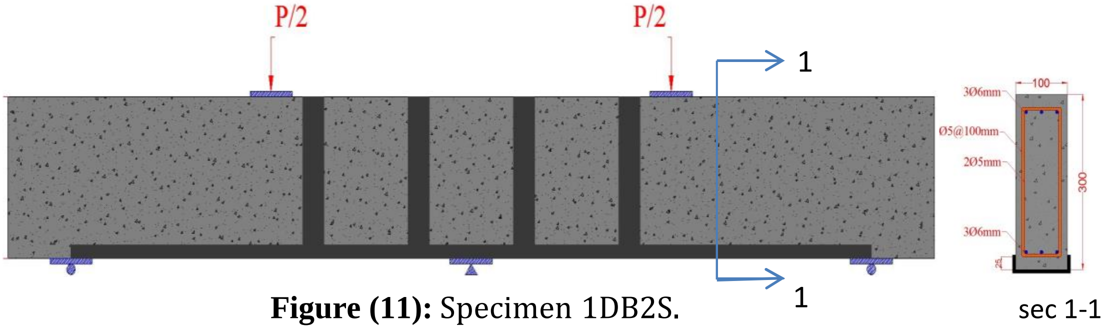

Figure (11): Specimen 1DB2S.

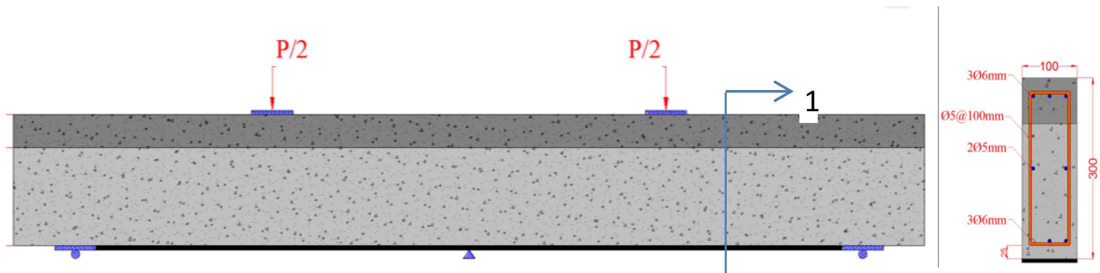

Figure (12): Specimen 1DB3S.

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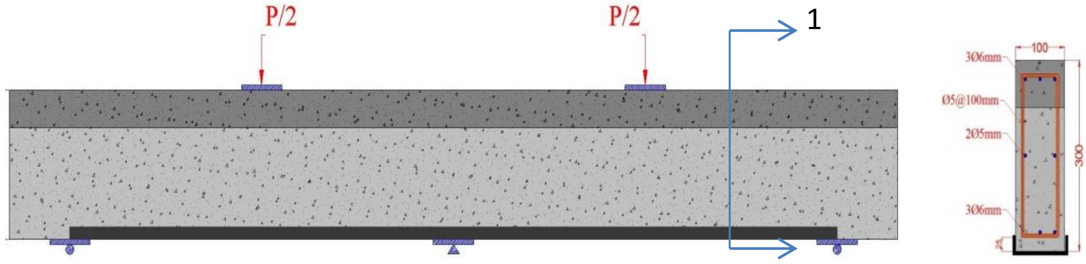

Figure (13): Specimen 1DB4S.

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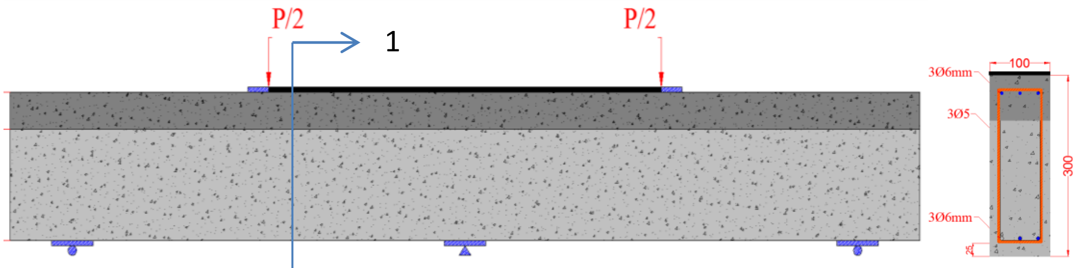

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Figure (14): Specimen 1DB5S and 1DB6S.

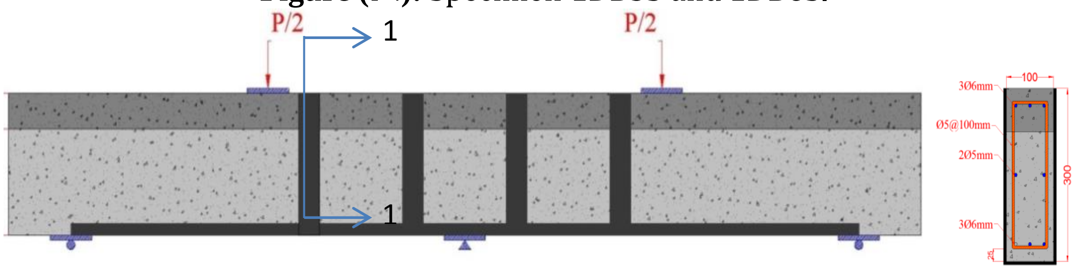

Figure (15): Specimen 1DB7S.

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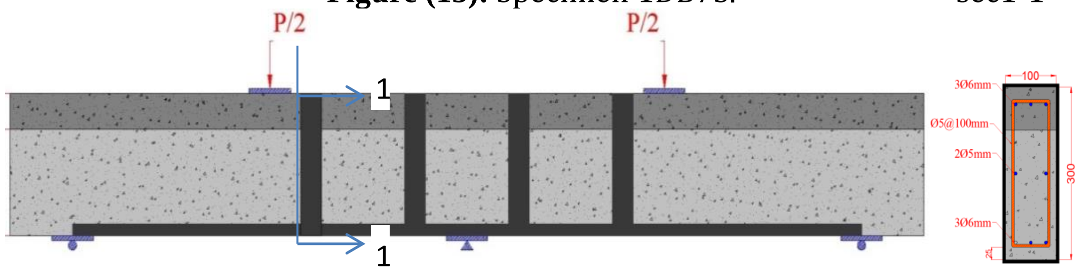

Figure (16): Specimen 1DB8S.

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Table (1): Specimens codes and descriptions.

Beam code	description
1DB1	Full normal strength concrete
1DB2	Full high strength concrete
1DB3	25% HSC from top
1DB4	50% HSC from top
1DB5	75% HSC from top
1DB6	25% HSC from bottom
1DB7	50% HSC from bottom
1DB8	75% HSC from bottom
1DB1S	Full NSC with 15cm CFRP strip at bottom and two strips of 5cm near the center of U shape
1DB2S	Full HSC with 15cm CFRP strip at bottom and two strips of 5cm near the center of U shape
1DB3S	25% HSC from top with 10cm CFRP strip at bottom for flexure
1DB4S	25% HSC from top with 15cm CFRP strip at bottom for flexure
1DB5S	25% HSC from top with 10cm CFRP strip at top for flexure
1DB6S	25% HSC from top with 10cm CFRP strip at top for flexure
1DB7S	25% HSC from top with 15cm CFRP strip at bottom and two strips of 5cm near the center of U shape
1DB8S	25% HSC from top with 15cm CFRP strip at bottom and two strips of 5cm near the center of O shape

2.4 Material Properties of Tested Specimens

2.4.1 Concrete

The materials used in producing concrete were:

1. Ordinary Portland cement was used throughout this investigation for casting all the specimens.
2. Natural sand of maximum size of 4.75 mm.
3. Gravel of maximum size of 14.5mm.
4. High range water reducing admixture is used in order to reduce the water used in mix design.
5. Tap water has been used for mixing concrete and curing all the beams.

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

2.4.2 Reinforcing Steel

Two sizes of reinforcing steel deformed bars were used in this study; deformed bars of size Ø6 mm were used as main reinforcing steel and Ø5mm were used as closed stirrups.

Figure (17): Cage of steel reinforcement.

Tensile test of steel reinforcement was carried out on at least three specimens, prepared for each type of the used reinforcing steel bars to determine their tensile properties. According to ASTM A370 the tensile test was performed in laboratory of

Material Engineering College, University of Babylon. The main properties are summarized in **Table (2).**

Table (2): Details of steel reinforcing bars.

Nominal diameter(mm)	Measured diameter(*) (mm)	Yield stress(*) (MPa)	Ultimate strength(*) (MPa)
5	4.7	520	630
6	5.6	480	550

(*)Each value is an average of three specimens.

2.4.3 Carbon Fiber Reinforced Polymer (CFRP)

Sika® CarboDur® sheets are pultruded carbon fiber reinforced polymer (CFRP) sheet selected for strengthening concrete. The mechanical properties of CFRP plate are written in Table (3) according to manufacturing specifications of Sika Company.

Table (3): Technical properties of CFRP sheet.

Properties	**Sika® CarboDur® XS514/80**
Tensile strength MPa	4'900 N/mm^2
E-modulus MPa	230’000 N/mm^2
Density	1.80 g/cm^3
Fiber Orientation Degree	0°
Strain at break (min. value)	2.1%
Thickness mm	0.17mm

2.4.4 Epoxy Adhesive Properties

Sikadur®-330 is a thixotropic, structural two part adhesive, based on a combination of epoxy resins and special filler. Its main properties as supplied by the manufacturer are shown in Table (4). Figure (18) shows the CFRP and epoxy used in strengthening of specimens.

Table (4): Technical properties of adhesive material.

Properties	**Sikadur®-330**
E-modulus (MPa)	Flexural : 3800 N/mm^2 Tensile: 4500 N/mm^2
Compressive Strength (MPa) (N/mm^2)	50-95
Tensile strength (MPa)	30 N/mm^2 (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 (18): CFRP and epoxy used in strengthening of specimens.

2.5 Mechanical Properties of Hardened Concrete

The compressive strength ($f$cu) and splitting tensile strength (ft) of concrete for each specimen are presented in Table (5). Each value represents the average of testing three cube specimens (150mm × 150mm) for ($f$cu) and three cylindrical specimens (100mm × 200mm) for (f_t). The modulus of rupture test was carried out on concrete prisms (100 x 100 x 300) mm one for each specimen.

Table (5): Properties of Hardened Concrete.

Specimen No.	Compressive strength of concrete (f_{cu}) (MPa)		Compressive strength of concrete(f_c') (MPa)(*)		Splitting tensile strength (f_t) (MPa)		Modulus of rupture (f_r) (MPa)		Modulus of Elasticity (Ec) (*1) (MPa)	
Concrete strength	Normal strength	High strength	Normal strength	High strength	Normal strength	High strength	Normal strength	High strength	Normal strength	High strength
1DB1	36.5	------	29.2	-----	3.212	-----	4.162	-----	25397	------
1DB2	-----	77.5	-----	62	-----	6.2	-----	6.95	------	37007
1DB3	34	74.1	27.2	59.3	2.992	5.93	3.942	6.68	24512	3619.
1DB4	34	74.1	27.2	59.3	2.992	5.93	3.942	6.68	24512	36193
1DB5	34	74.1	27.2	59.3	2.992	5.93	3.942	6.68	24512	36193
1DB6	38	77.1	30.4	61.7	3.344	6.17	4.294	6.92	25914	369189
1DB7	38	77.1	30.4	61.7	3.344	6.17	4.294	6.92	25914	36918
1DB8	38	77.1	30.4	61.7	3.344	6.17	4.294	6.92	25914	36918
1DB1S	37.7	------	30.2	----	3.322	-----	4.272	-----	25828.63	-------
1DB2S	-----	75.5	-----	60.4	------	6.04	-----	6.79	------	36527.2
1DB3S	40.1	78.1	32.1	62.5	3.531	6.25	4.481	7	26628.7	37156.7
1DB4S	40.1	78.1	32.1	62.5	3.531	6.25	4.481	7	26628.7	37156.7
1DB5S	40.1	78.1	32.1	62.5	3.531	6.25	4.481	7	26628.7	37156.7
1DB6S	37.6	78.1	30.1	62.5	3.311	6.25	4.261	7	25785.8	37156.7
1DB7S	37.6	78.1	30.1	62.5	3.311	6.25	4.261	7	25785.8	37156.7
1DB8S	37.6	78.1	30.1	62.5	3.311	6.25	4.261	7	25785.8	37156.7

(*) $fc' = 0.8fcu$ (ACI code)

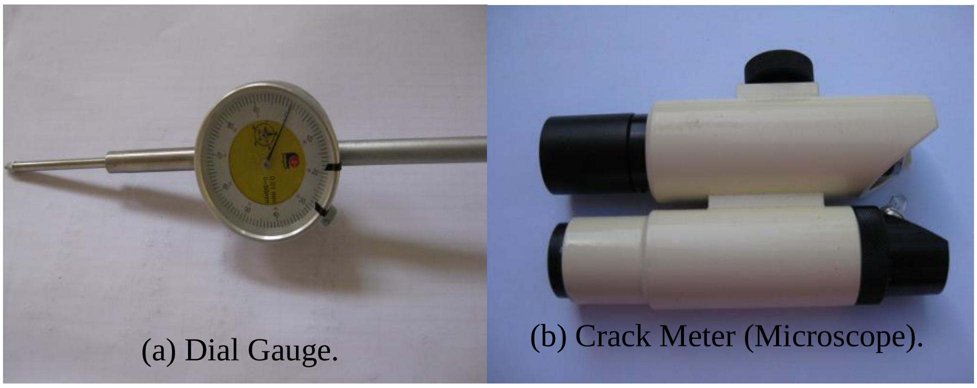
(*1) $\mathrm{Ec} = 4700\sqrt{fc'}$ (ACI code)

2.6 Test Procedure

Tests were carried out using hydraulic testing machine in Civil Egineering Department of Babylon University as shown in Figure (19).

Figure (19): Hydraulic testing machine.

The main characteristics of the structural behavior of the beam specimens were detected at every stage of loading during test. A dial gage of 0.01 mm accuracy was used at the midspan in order to record the deflection, Figure (20) shows the instruments that used in the testing. The beams were placed on the supports of the testing machine, and then the first readings of the gages were recorded. After that, the beams were loaded with a constant rate of loading. Readings of deflection were recorded at each interval of load as well as recording the first crack load, tracing the cracking patterns and the failure load.



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

Figure (20): Instruments used in testing deep beam specimens.

3. Experimental results

All the specimens tested under two point loads. First crack include ultimate load, mode of failure and the change in the ultimate loads with respect to the control beam 1DB1 for all the tested beams are tabulated in Table(6).

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

Beam code	First positive flexural crack load (kN)	First negative flexural crack load (kN)	First shear crack load (kN)	Type of Failure	Ultimate load (kN)	Increasing in ultimate load* (%)
1DB1	80	100	140	Flexure	240	0%
1DB2	132	115	240	Flexure	320	33%
1DB3	84	100	170	Flexure	275	14%
1DB4	112	90	----	Flexure	291	21%
1DB5	127	80	192	Flexure	305	27%
1DB6	114	75	172	Flexure	269	12%
1DB7	105	116	184	Flexure	278	15%
1DB8	92	120	188	Flexure	271	13%
1DB1S	120	90	145	Shear	335	39.5%
1DB2S	160	140	200	Flexure	405	68.7%
1DB3S	100	90	172	Flexure	319	32%
1DB4S	121	95	160	Shear	311	29%
1DB5S	90	148	130	Shear	256	6%
1DB6S	83	128	137	Shear	258	6%
1DB7S	115	92	165	Shear	354	47%
1DB8S	120	85	190	Shear	384	60%

*: $\frac{ultimate\ load\ of\ beam - ultimate\ load\ of\ 1DB1}{ultimate\ load\ of\ 1DB1}$

The figures from (21) to (23) show the load deflection of all specimens

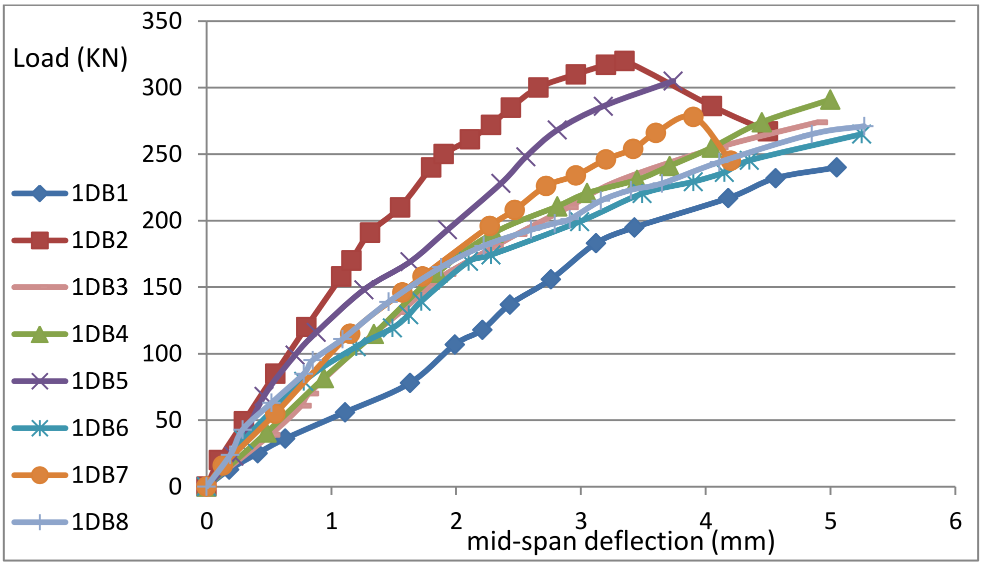


Figure (21): Load-deflection curves of the specimens unstrengthened with CFRP.

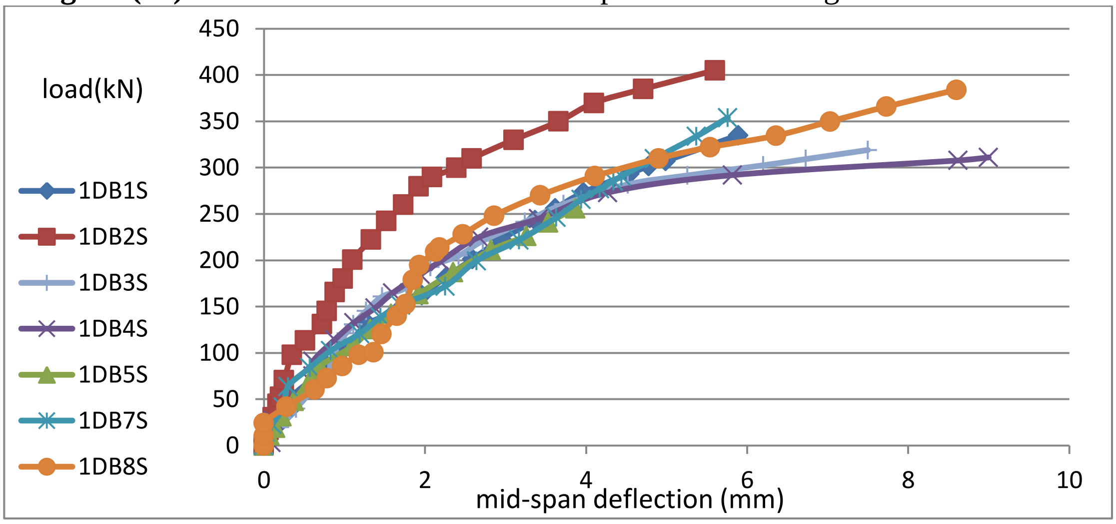
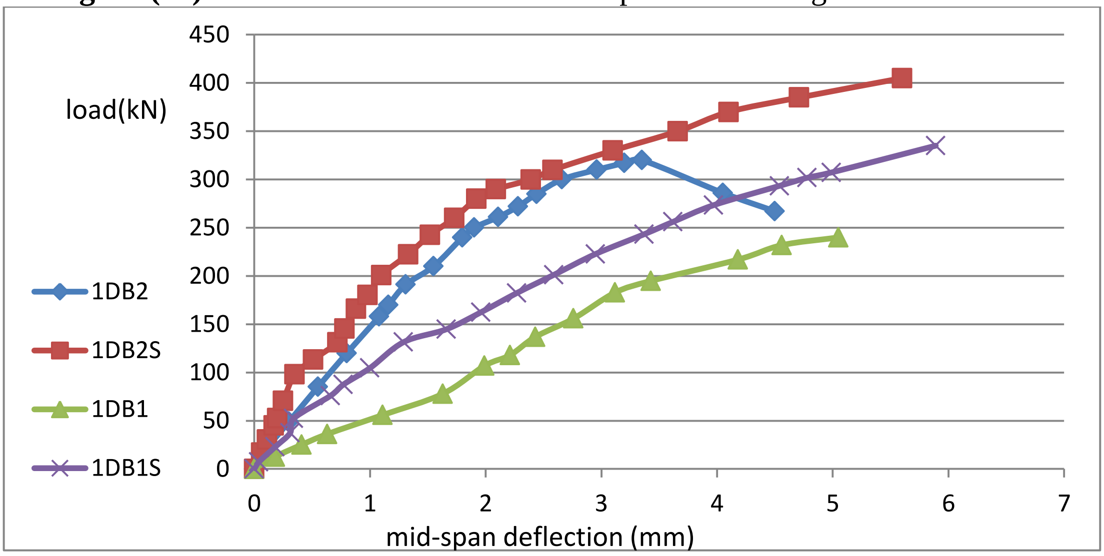


Figure (22): Load-deflection curves of the specimens strengthend with CFRP.



Figure(23): Load-deflection of full NSC and HSC specimens strengthened and unstrengthened with CFRP.

The figures from (24) to (31) show the failure mode of specimens unstrengthed with CFRP. While the figure from (32) to (39) show the failure mode of strengthened beams with CFRP.

Figure (24): Specimen(1DB1) after testing.

Figure (25): Specimen (1DB2) after testing.

Figure (26): Specimen (1DB3) after testing.

Figure (27): Specimen (1DB4) after testing.

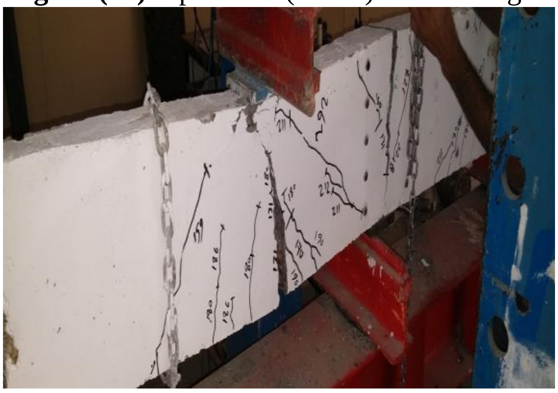


Figure (28): Specimen (1DB5) after testing.

Figure (29): Specimen (1DB6) after testing.

Figure (30): Specimen (1DB7) after testing.

Figure (31): Specimen (1DB8) after testing.

Figure (32): Specimen (1DB1S) after testing.

Figure (33): Specimen (1DB2S) after testing.

Figure (34): Specimen (1DB3S) after testing.

Figure (35): Specimen (1DB4S) after testing.

Figure (36): Specimen (1DB5S) after testing**.**

Figure (37): Specimen (1DB6S) after testing.

Figure (38): Specimen (1DB7S) after testing**.**

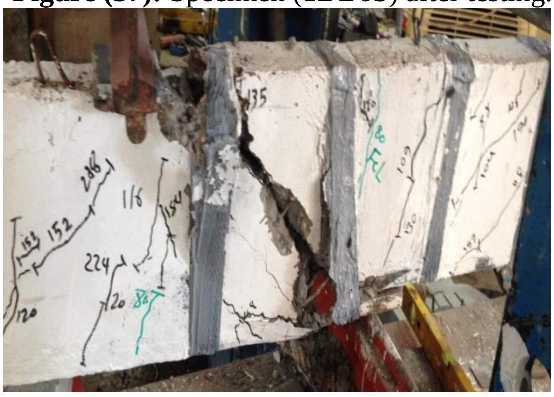


Figure (39): Specimen (1DB8S) after testing**.**

4. Conclusions

1- The results indicate that the ultimate load enhanced with the increase in the thickness of HSC layer, where the thickness of HSC layer increased from 0% to 100% the increment was 33%.

2- The load of first crack in positive and negative moments regions increased by 65% and 15% respectively when fc' increased from (29.2 MPa-1DB1) to (62 MPa-1DB2).

3- The location of first crack was at mid span for specimen 1DB1 while was at middle support of specimen 1DB2, where the HSC increases the responsibility of middle strut to carrying load.

4- The results proved that the increase in the HSC layer from 25% to 75% at top leads to the increase in the load of first crack in positive moment region and decrement in the load of first crack in negative moment region, while the increment in the HSC layer from 25% to 75% at bottom leads to the increment in the load of first crack in negative moment region and decrement in the load of first crack in positive moment region. This phenomena insures that the strengthening of concrete increases the responsibility of layer to carrying the loads.

5- The full HSC specimen gave an improvement in first shear crack load by 71% when compared with full NSC, while the hybrid concrete specimens did not give much variation in first shear crack load.

6- The result proved that the optimal location of strengthening RCCDB by HSC layer is at the top and the optimal percentage was equal to 25%.

7- The results of specimen 1DB1S gave an increase in strength by 39.5% from the same specimen unstrengthed with CFRP, while the specimen 1DB2S gave an increase in strength by 26% from the same specimen unstrengthed with CFRP, these comparisons show that the effect of strengthening of full NSC is more efficient than strengthening of full HSC.

8- Also the results of specimens 1DB3, 1DB3S, and 1DB5S proved that the flexural strengthening of Hybird RCCDB by CFRP at the bottom is batter than at the top and the increment in the ultimate strength due to flexural CFRP(0.056%) was 16%.

9-The results of 1DB7S and 1DB8S proved that the shear strengthening of hybrid RCCDB increase the ultimate strength by 23.4% and 13.8% if the strengthening has O and U shape respectively.

10-The strengthening with CFRP in positive moment region caused in delay the appearance of first crack in the positive moment region but this leads to earlier appearance of first crack in negative moment region when compared with the specimens unstrengthed with CFRP and vice versa. The results approved that the strengthening of hybrid RCCDB (25% HSC from top) must be strengthed at bottom by CFRP whenever required.

5. References

American Concrete Institute, 2008, "Building Code Requirements for Reinforced Concrete", Detroit, ACI-318-08.

ASTM A370, 2007,"Standard Test Methods and Definition for Mechanical Testing of Steel Product".

Beshara F.B.A., I.G.Shaaban, and T.S.Mustafa,"Behaviour and Analysis of Reinforced Concrete Continuous Deep Beams", 2014, (The 12th Arab Structural Engineering Conference).

Comite Europeen du Beton, 1970, "International Recommendations for the Design and Construction of Concrete Structures", CEB-FIP", pp. 460.

Comite Europeen du Beton, 1982, "International Recommendations for the Design and Construction of Concrete Structures", CEB-FIP.

Iraq Specification No. 45/ 1984, " Aggregate from Natural Sources for Concrete and Construction", وزارة التخطيط, الجهاز المركزي للتقييس و السيطرة النوعية .

Kong F.K., and Singh A., 1972" Diagonal Cracking and Ultimate Loads of the Lightweight Concrete Deep Beams", ACI Journal, August, pp. 513-521.
Kong F.K., Robins P.J., and Coke D.F., 1978" Web Reinforcement Effects on Deep Beams", ACI Structural Journal, Vol. 67, No. 12, Dec., pp.1010-1017.
Kong F.K.,"Reinforced Concrete Deep Beams", University of Newcastle-upon-Tyne**,** United kindom, 2003, pp. 299**.**
Sika Wrap-301 C product Edition 2014-06-1 www.sika.my.
Sikadur-330 Proudect Data sheet Edition 13/6/2006 Identification no 02 04 01 04 001 www.sika.com.bh.