EXPERIMENTAIL AND THEORETICAL INVESTIGATION FOR BEHAVIOR OF R.C BEAMS WITH TENSILE REINFORCEMENT LAP SPLICEC STRENGTHENED BY CFRP LAMINATES

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ABSTRACT

The main objective of the research reported in this paper was to present an experimental and analytical study for the effect of external strengthening by CFRP laminates on the behavior of beams containing tensile reinforcement lap splices such as ultimate load, ductility and mode of failure. To meet the objective, seventeen NSC beam specimens were tested with cross section of (200x300) mm and overall length 2100mm. Each beam was designed with bars spliced in a constant moment region at midspan. The variables used in the investigation were bar size, presence of transverse reinforcement in the splice region and strengthening scheme by CFRP laminates. From the experimental study the results show that the use of CFRP sheet to upgrade the R.C beams containing tensile reinforcement lap splices has significant effect on the behavior of these beams such as ultimate load, cracking load, deflection and mode of failure where the percent increase in ultimate load was about (11-47)% and with elimination the possibility of the brittle failure. Analytically, three dimensional finite element were used to analyze these members by nonlinear solution technique. ANSYS computer program (version 9.0, 2004) was performed throughout this study. Eight-node brick element has been used to model concrete, two node link element has been used to model steel reinforcement and four node shell element was used to model CFRP straps. The bond between steel reinforcement and concrete has been modeled by using two-node contact element (CONTAC52). The comparison between the experimental and analytical results referred to reasonable agreement and asserted the validity of the numerical analysis and methodology developed in this study with difference by about 10% for ultimate load and 16% for maximum deflection with good estimation for mode of failure.

KEYWORDS: Lap splices, CFRP sheet, stirrups

الخلاصة

ان الهدف الأساس من هذا البحث هو تقديم دراسة عملية وتحليلية عن تأثير التقوية الخارجية بأستخدام الياف الكاربون البولمرية على سلوك الأعتاب الخرسانية المسلحة والحاوية على وصلات حديد تسليح الشد المتراكبة مثل قابلية التحمل القصوى والمطيلية وكذلك نمط الفشل. من أجل القيام بالدراسة المشار اليها اعلاه تم صب سبعة عشر عتب خرساني مسلح بأبعاد مقطع (200*300) ملم وبفضاء كلي 2100ملم, تم وضع وصلات التراكب لجميع النماذج في منطقة العزم الثابت. ان المتغيرات االداخلة في هذه الدراسة هي قطر حديد التسليح, وجود حديد تسليح القص في منطقة الوصلة و نمط التقوية الخارجية. بينت النتائج العملية ان استخدام الياف الكاربون البولمرية لتقوية واعادة تأهيل الأعتاب الخرسانية الحاوية على وصلات التراكب لها تأثير أيجابي على سلوك هذه الأعتاب حيث انها اعطت زيادة في قابلية التحمل القصوى في مايقارب (11-47)% مع ألغاء نمط الفشل القصيف التي تعتبر النتيجة الأهم في مثل هذه الأعضاء الخرسانية. تحليلياً تم استخدام طريقة العناصر المحددة ثلاثية الأبعاد لتحليل هذه الأعتاب بأسلوب التحليل الاخطي بواسطة برنامج (ANSYS V 9.0) حيث تم تمثيل الكونكريت بأستخدام عنصر طابوقي ثماني العقد وحديد التسليح بأستخدام عنصر رابط ثنائي العقد و الياف الكاربون البولمرية تم تمثيلها بأستخدام عنصر قشري بأربع عقد اما بالنسبة للترابط بين حديد التسليح والكونكريت تم تمثيله بأستخدام عنصر سطحي داخلي. ان المقارنة بين النتائج العملية والنظرية أكدت صلاحية التحليل العددي بشكل واضح حيث كانت النتائج معقولة بنسبة اختلاف مايقارب 10% بالنسبة للحمل الأقصى و 16% بالنسبة للهطول الأقصى وبتخمين جيد بالنسبة لنمط الفشل النهائي.

INTRODUCTION

For almost 100 years, construction practices in the building of concrete structures have focused on the use of steel reinforcement to transfer tension and shear forces. Lap splicing has become the traditional method of connecting the steel reinforcing bars due to the discontinuity of rebars which are manufactured in standard length 6,9,12 meter. A lap can be defined as a two pieces of rebar overlap to form a continuous line which helps transfer loads properly throughout the structure [**Al-yassri 2011**]. In general the continuity of rebars can be provided in three methods of splicing, mechanical splices, welded splices and lap splices which is in two types, non-contact lap splices and contact lap splices where the last one is adopted here. Over the years, many structural engineers, architects and specifiers have been noticed that lap splicing has few advantages (no-cost splicing) and many disadvantages (poor under condition of reversed and impacting loading). ACI-code states that lap splices are not considered reliable under conditions of cyclic loading into the elastic range. The combination of shear and moment in the splice zone is less severe than moment alone. [**Ferguson and Briceno 1969**] and the mode of failure of beam containing tensile reinforcement lap splices is independent of the load history but it is a function of confining reinforcement, concrete cover, dimensions, compression reinforcement and bar size [**Ali 1990**]. The concrete strength has an important effect on the splice strength but there is a limit above which an increase in concrete strength begins to be detrimental to splice strength and this is probably due to shrinkage phenomenon, which generates tensile stresses in the surrounding concrete [**Tepfers 1973**]. Many researchers reported that the use of carbon fiber reinforced polymer (CFRP) has very significant in upgrading the weakness structure. The materials FRP holds many advantages over other materials in civil engineering. It has very high stiffness to weight ratio and high strength to weight ratio. The material exhibits excellent fatigue properties, non-magnetic properties, corrosion resistance, and is generally resistant to chemicals. Examining the strength and stiffness of beams with unidirectional CFRP plates was a primary focus of the past research. [**Meier et al. (1992)]** concluded beast results about using CFRP sheet for strengthening of minimally reinforced concrete beams by applying a unidirectional CFRP sheet to the tensile side where increase the the ultimate load about 100% compared to the control beam (unstrengthened) and the deflection of the strengthened beam was 50% less than the control beam. Experimental studies were conducted by [**Kim 2008**] by applying CFRP materials to the lap splice region in square and rectangular columns which exhibited a brittle splice failure as-built. After rehabilitating the columns using CFRP jackets and anchors, the failure mode changed from a brittle splice failure to yield of column reinforcement.

2. EXPERIMENTAL WORK

The experimental program included preparation and testing of seventeen beams. Tested beams were constructed with overall length (2100mm) and with rectangular cross section of (200x300)mm. Two Ø16mm or Ø12mm deformed bars were provided as longitudinal tension reinforcement. Rectangular closed stirrups (Ø10mm) were provided. All beams were tested under condition of two point loads at a distance of (600mm) from the support to produce a region of constant moment zone (800mm), where lapped splices of tension rebars are provided.

In order to identify the test specimens with different reinforcement ratios and strengthening schemes, the following designation system is used:

- (B) for control beams and beams without stirrups in the lap region, (BS) for beams with stirrups in the lap region, (BC) for beams with continuous bars (no lap splices), (R) for repaired beams and (P) for pilot beam.
- (12) for 12mm diameter of bars in tension reinforcement and (16) for 16mm diameter of bars in tension reinforcement.
- (F) for longitudinal strengthening by CFRP and (U) for lateral confinement by CFRP as U-jacket.

The beams (B-12, B-16) were kept without strengthening and were considered as control beams for comparison. To study the effect of external and internal strengthening in increasing capacity and ductility of beams containing lap splices, beams were designated in first segment of its symbol by (S) for internal confinement in the lap region by stirrups as shown in (**Figure 1)**. Beams were designated in second segment of its symbol by (F) for strengthened externally by two strips of CFRP sheet with (75mm) width at edges of bottom face as shown in (**Figures 2)** while the beams were designated in second segment of its symbol by (U) were wrapped partially (as U-jacket) by two strips of CFRP sheet with (75mm) width at ends of splicing region as shown in (**Figure 3)**. The repaired beams were loaded by 60% of ultimate load of original beams, with all techniques of strengthening above by CFRP were adopted to it. To prevent CFRP sheet end debonding or concrete cover separation, CFRP sheets strips were extended under supports **[Garden and Hollaway, 1998]**.

2.1 DESIGN OF TEASTED BEAM

The tested beams were designed so that flexural failure would occur before shear failure. To avoid shear failure design of shear strength at each section of a member is required. It is achieved by combined contributions of concrete and shear reinforcement. A critical part of the design was ensuring the beams failed due to flexure and not shear. The required length of lap for tension splices, established by the test, may be stated in terms of the development length (L_d). Two different classifications of lap splices are established corresponding to the minimum length of lap required: a Class A splice requires a lap of $1.0L_d$, and a Class B splice requires a lap of $1.3L_d$. In either case, a minimum length of (300mm) applies. Lap splices, in general, must be Class B splices, according to [**ACI Code-318M-08**], except that Class A splices are allowed when the area of reinforcement provided is at least twice larger than that required by analysis over the entire length of the splice and when one-half or less of the total reinforcement is spliced within the required lap length.

In this work the length of the lap splices of all specimens were designed so the yielding failure will occur in the tension steel reinforcement before the splice fails in bond ($L_s=1.3L_d$). Only beams type (BS) are including two vertical reinforcement closed stirrups of (10mm) bar diameter as confinement reinforcement at the lap region.

2.2 MATERIALS PROPERTIES

Normal weight concrete was used to cast the specimens. The 28-day concrete compressive strength was 25 MPa to mimic an older structural member that would be subjected to strengthening. The longitudinal steel reinforcement deformed bars have a 450 MPa, 580 MPa, 520 MPa yield stresses for bar diameters (16, 12, 10)mm respectively. A CFRP sheet has a tensile strength of 4.5 GPa, an modulus of elasticity of 238 GPa, the elongation at break of 1.8% and the thickness of 0.131 mm [**Sika, 2005**].

2.3 TEST SETUP

All tested beams are tested in two-points loading. Beam specimens were tested as simply supported beams over 2000mm span in 1500 kN capacity hydraulic machine as shown in (**Figure 4)**. Each beam specimen was supported and loaded by rollers. Forces were distributed through steel bearing plate 200mm in length to cover the entire beam width. To observe crack development, beam specimens were painted white with emulsion paint before testing. At the first the specimens loaded by 5 kN to seat the support and the load system, then reduce to zero. The load increment was 5 kN along the test. Dial gages of accuracy 0.01 mm installed at the mid span of the beam and under point of loading to measure the deflection.

3. EXPERIMENTAL RESULTS

The main objective of the current research work is to investigate the behavior of tensile reinforcement lap splices strengthened by CFRP laminates. (**Table 1)** shows a summary for test results. Test results were analyzed based on load-deflection response at the point of loading, cracking load, ultimate load and failure modes.

In beam (B-12) (control beam with spliced bars of 12 mm diameter and without any strengthening). The first crack occurred about 25 kN at the ends of lap region due to the discontinuity in rebar. As load increased the cracks propagated toward midspan and the ends of the lap region were stodgy by cracks afterwards the splitting bond failure occurred at a load of about 135 kN with a brittle configuration and noisily as shown in (**Figure 5a)**. In the beam (B-16) (control beam with spliced bars of 16mm diameter and without any strengthening), the first visible cracks appeared at ends of the lap region when the load reached to level about 30 kN. When the applied load reached 80 kN the cracks appeared in the midspan of the beam. With increasing applied load the cracks became wider and propagated rapidly indirection parallel to the splice region. When the applied load reached to level about 150 kN the splitting bond failure occurred in a brittle mode, very noisily and without any preconceived sign as shown in (**Figure 5b)**. The load deflection response of the point load for control beams are illustrated in (**Figure 6)**. From comparison between specimens (B-12) and (B-16) it's shown that the decrease in ultimate load and increase in ductility attribute to the decrease in diameter of bar. It can be noticed from the behavior of these beams there are three regions, elastic-uncracked, elastic-cracked and elasto-plastic, the first region terminate when the cracks occur. In beams (BC-12, BC-16) the tension reinforcement was continuous (no lap splices) the first cracks occurred at mid span of these beams at load of about 40 kN. The final mode of failure was flexural tensile failure (ductile failure) by yielding the rebar at the mid span sectionas shown in (**Figure 5c)**. For the beams (BS-12, BS-16) the same as control beams but they include 2-Ø10 closed stirrups in the lap region to provide internal confinement stress. The first cracks occurred at the ends of lap region and the cracks were large in number, more uniformly distribution with respect to the control beams that not include stirrups in the lap region. The final mode of failure was flexural tensile failure (ductile failure) as shown in (**Figures 5d, 5e)** due to the presence of the stirrups in the lap region that increasing the bond stress by allowing the more ribs to contribute to stress transfer between bars that form the splice and surrounding concrete. The load-deflection flexural stiffness of companion beams without and with transverse reinforcement in the splice region was identical below and above the flexural cracking load. The presence of transverse reinforcement did not affect the flexural cracking load of the beam specimens. In the beams (B-F12, B-F16) a two straps of CFRP sheet with 75 mm width were glued at the edges of the tension face along span of beam. In these beams the final mode of failure was rupture in CFRP followed by splitting bond failure (brittle failure) as shown in (**Figure 5f)** with increase in ultimate load about 43%. In the beams (B-U12, B-U16) two wrap of CFRP sheet with 75mm width as a U-jacket were glued at the ends of the lap splice in these beams to provide external confinement. The external confinement provided by CFRP wraps lead to ductile failure as shown in (**Figure 5g)** by eliminating splitting bond failure with increasing in the ultimate load about (11-20)%. Beams (BS-F12, BS-F16) strengthened externally by two straps of CFRP sheet of 75 mm width were glued at edges of bottom face along the span of beam and strengthened internally by using 2-Ø10mm stirrups. This scheme of strengthening gives good indication about using CFRP in improving the ductility by eliminating brittle failure and developing flexural failure out of lap region i.e. the strength of lap splice large of other sections nearly twice that of the other sections this attribute to good confinement provided by CFRP and stirrups and good continuity provided by longitudinal CFRP for the discontinuity in the ends of lap splices. The beams (BS-U12, BS-U16) were strengthened externally by CFRP sheet of 75mm width glued at the ends of the lap splice as a U-jacket to provide external confinement, and internally by 2-Ø10mm stirrups to provide internal confinement. This technique of strengthening gives good results about eliminating the brittle mode of failure with increasing in ultimate load about (11-20)%. It is decided to load (RBS-12 and RBS-16) approximately to 60% of the ultimate load of original beams (BS-12 and BS-16) respectively, then strength them by CFRP sheet in longitudinal direction and confined by wrapping. The results obtained from test for these beams gives good indication of efficiency of CFRP to rehabilitation the damaged beams by developing ductile failure with increasing in ultimate load about 17%. The load deflection curves for the tested beams are illustrated in (**Figure 6)**.

4. ANALYTICAL STUDY

In the Analytical study, three dimensional finite element analysis was used to investigate the performance of the R.C. beams with and without tensile reinforcement lap splices strengthened by longitudinal or lateral CFRP laminates. ANSYS computer program (**version 9.0, 2004**) was performed throughout this study. Eight-node brick element(Solid65) has been used to model concrete, two node link element (Link8) has been used to model steel reinforcement, four node shell element (Shell41) was used to model CFRP straps and eight node brick element (Solid45) was used to model loading plates. The bond between steel reinforcement and concrete has been modeled by using two-node contact element (CONTAC52). Geometry of these elements was illustrated in (**Figure 7)**. The full Newton-Raphson method was used for the nonlinear solution algorithm. The materials nonlinearity due to cracking, crushing of concrete, and yielding of reinforcement were taken into consideration during the analysis.

4.1 DESCRIPTION OF SPECIMEN IN FINITE ELEMENT

By taking advantage of the symmetry of the beams, one half of the full beam was used for modeling. This approach reduced computational time and computer disk space requirements significantly. The one half of the entire model is shown in (**Figure 8)**.

4.2 MESH REFINEMENT

An important step in finite element modeling is the selection of the mesh density [**Maekava et al 1983**].

Because a one half of the entire beam was used for the model, plane of symmetry was required at the internal face. At a plane of symmetry, the displacement in the direction perpendicular to the plane was held at zero. (**Figure 9)** shows boundary conditions for a typical finite element model.

5. FINITE ELEMENT RESULTS

The goal of the comparison of the FE model and the results of tested beams is to ensure that the elements, material properties, real constant and convergence criteria are adequate to model the response of R.C beams containing tensile reinforcement lap splices with or without strengthening. The results obtained from F.E analysis gave good agreement when compared with the experimental results which include, ultimate load, cracking load, maximum deflection and mode of failure as explained in (**Table 2)**.

Finite element analysis with partial bond gives better agreement (more than perfect bond) when compared with the experimental study as shown in (**Figure 10**). This can be attributed to the slip between reinforcing bar and surrounding concrete were neglected in case of perfect bond. The behavior of the two cases (perfect and partial bond) were approximately similar before cracking load but when the loading stages developed the slip occurred due to the crushing of the concrete in front of the bar ribs.

6. PARAMETRIC STUDY

From the reasonable agreement and accuracy of the finite element model including bond-slip phenomena with experimental results as explained above, effect of some selected parameters on beams containing tensile reinforcement lap splices are decided to study; this include:

- Wrapping shape of CFRP sheet.
- Diameter of reinforcing spliced bar.

6.1 EFFECT OF WRAPPING SHAPE.

To explain the effect of wrapping shape of CFRP sheet on the behavior of R.C. beams containing tensile lap splices such as ultimate load, cracking load, ductility and mode of failure; two of wrapping scheme are provided (full wrap and U-wrap). From **Figure (11)** it can be concluded there is no significant difference between beams confined externally by CFRP wraps (full and partial) because of the activity of confinement appear in portion abutting to the spliced bars and the U-wrap has sufficient length of development for bonding between CFRP and concrete therefore the confinement by CFRP sheet as a U-wrap have more practically and more economically.

6.2 Effect of Bar Diameter

When the diameter of spliced bars increases the splitting and bursting forces will be increased due to the larger stiffness of spliced bar and then lead to early splitting bond failure at low level of ultimate deflection as shown in (**Figure 12)**. Therefore the larger bar diameters demand adequate lateral confinement, concrete cover and sufficient anchored length to resist aggravated bursting forces and then prevent brittle failure.

7.CONCLUSIONS

1- The longitudinal strengthening by CFRP leads to increasing ultimate load by about (43-47)% but the mode of failure is still brittle whilst the external confinement by CFRP led to increasing ultimate load by about (11-20)% and the mode of failure was ductile.

2- Using longitudinal CFRP combined with external confinement of wrapping CFRP or internal confinement of stirrups gives the better behavior represented by increasing ultimate load by about (44-47)% and enhancing ductility in comparison with the case of using the longitudinal strengthening alone.

3- It can be observed from beams confined internally by stirrups combined with external wrapping of CFRP, there is no significant difference with the case of external confinement alone about (2)% in ultimate load. But this effect may be expected to appear clearly for splicing bars of larger diameter than 20mm.

4- In beams with no internal (stirrups) or external (CFRP) confinement in the tensile reinforcement lap splice region, the final mode of failure was a face-side splitting bond failure which was sudden, very brittle, and noisy with ultimate load less than that of a continuous bar by about 13%.

5- Beams with only external confinement of wrapping CFRP give well result of overall behavior through enhancing the mode of failure by eliminating the brittle failure as well as load-deflection response and cracking pattern, so this technique can be utilized as alternative technique for internal confinement by reinforcing stirrups.

6- The use of external longitudinal CFRP laminates with confinement wrapping of CFRP sheet, may be considered the best way for retrofitting the deteriorated or damaged beams containing tensile lap splices, with increasing ultimate load by about 17% and ultimate deflection 21% and mode of failure was still ductile.

7- The present finite element formulation with modeling of materials, cracking, crushing taking into consideration contact (interface element) between steel and concrete for tensile reinforcement lap splices, seems efficient and gives good accuracy through comparison with the experimental results where the maximum difference in the ultimate load was less than 10% and the maximum difference in the maximum deflection was less than 16%, as well as obtaining reasonable estimation for mode of failure (brittle or ductile).

8- The use of CFRP strips as lateral confinement in partial U-wrapping gives the same overall behavior as that obtained for full wrapping, where the first is more practical and economical in field of construction engineering.

9- Neglecting bond-slip phenomena in F.E. analysis of R.C. beams with tensile reinforcement lap splices leads to overestimation in; post-cracking, load-deflection response, ultimate load by about 47% and eliminating the possibility of splitting bond failure.

10- Increasing diameter of the spliced bars has unfavorable effect on the ductility and the mode of failure.

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Abbreviations

NSC: Normal Strength Concrete

CFRP: Carbon Fiber Reinforced Polymer

Table (1): Summary for test results

Beam Symbol	*Cracking load (kN)*	*Pcr/Pcr$_R$*	*Failure load (kN)*	*Pf/Pf$_R$*	*Under point load deflection (mm)*	*Δu/Δu$_R$*	*Mode of failure*
Pilot	*25*	*-*	*130*	*-*	*9.51*	*-*	*Splitting Bond failure*
®B-12	*25*	*-*	*135*	*-*	*9.47*	*-*	*Splitting Bond failure*
BC-12	*40*	*1.6*	*140*	*1.04*	*12*	*1.27*	*Flexural tensile failure*
BS-12	*25*	*1.0*	*150*	*1.11*	*11.3*	*1.19*	*Flexural tensile failure*
B-F12	*45*	*1.8*	*195*	*1.44*	*13.1*	*1.38*	*Rupture of CFRP followed by bond splitting failure*
B-S12	*35*	*1.4*	*150*	*1.11*	*12*	*1.27*	*Flexural tensile failure*
BS-F12	*50*	*2*	*195*	*1.44*	*13.3*	*1.4*	*Rupture of CFRP followed by flexural tensile failure*
BS-S12	*40*	*1.6*	*150*	*1.11*	*12.2*	*1.29*	*Flexural tensile failure*
®B-16	*30*	*-*	*150*	*-*	*7.5*	*-*	*Splitting Bond failure*
BC-16	*40*	*1.33*	*170*	*1.13*	*10.4*	*1.39*	*Flexural tensile failure*
BS-16	*35*	*1.17*	*170*	*1.13*	*9.5*	*1.27*	*Flexural tensile failure*
B-F16	*60*	*2.0*	*215*	*1.43*	*10.6*	*1.41*	*Rupture of CFRP followed by bond splitting failure*
B-S16	*50*	*1.67*	*180*	*1.2*	*10.8*	*1.44*	*Flexural tensile failure*
BS-F16	*70*	*2.33*	*220*	*1.47*	*14.1*	*1.88*	*Rupture of CFRP followed by flexural tensile failure*
BS-S16	*40*	*1.33*	*180*	*1.2*	*11*	*1.47*	*Flexural tensile failure*

® Reference beam

P_{cr} = Cracking load, Pcr_R = Cracking load of reference beam

P_f = Failure load, Pf_R = Failure load of reference beam

Δu = Under point load deflection, Δu_R = Under point load deflection of reference beam

Table (2): finite element results

Beam Symbol	Cracking load (kN)			Ultimate load (kN)			Max. deflection (mm)			Mode of failure
	Exp	Theo.	Exp./Theo.	Exp.	Theo.	Exp./Theo.	Exp.	Theo.	Exp./Theo.	
®B-12	25	30	0.83	135	140	0.96	11.0	13.0	0.85	Splitting bond failure
BC-12	40	45	0.89	140	140	1.0	12.4	13.1	0.95	Flexural tensile failure
BS-12	25	30	0.83	150	145	1.03	14	13.1	1.07	Flexural tensile failure
B-F12	45	40	1.13	195	185	1.05	16.3	15.1	1.08	Experimentally: Rupture of CFRP followed by splitting bond failure Analytically: Flexural tensile failure

Table (3): Continued

Beam Symbol	Cracking load (kN)			Ultimate load (kN)			Max. deflection (mm)			Mode of failure
	Exp	Theo.	Exp./Theo.	Exp.	Theo.	Exp./Theo.	Exp.	Theo.	Exp./Theo.	
B-U12	35	40	0.88	150	150	1.0	13.8	13	1.06	Flexural tensile failure
BS-F12	50	40	1.25	195	195	1.0	16.4	17.8	0.92	Experimentally: Rupture of CFRP followed by flexural tensile failure Analytically: Flexural tensile failure
BS-U12	40	35	1.14	150	145	1.03	12.8	14	0.91	Flexural tensile failure
®B-16	30	30	1.0	150	160	0.94	9.3	10.9	0.85	Splitting bond failure
BC-16	40	50	0.80	170	180	0.94	11.8	13.7	0.86	Flexural tensile failure
BS-16	35	30	1.17	170	170	1.0	12.4	13.9	0.89	Flexural tensile failure
B-F16	60	55	1.10	215	220	0.98	13.7	15.9	0.86	Experimentally: Rupture of CFRP followed by splitting bond failure Analytically: Flexural tensile failure
B-U16	50	40	1.25	180	180	1.0	12.3	13.5	0.91	Flexural tensile failure
BS-F16	70	55	1.27	220	220	1.0	15.2	17.0	0.89	Experimentally: Rupture of CFRP followed by flexural tensile failure

® Reference beam

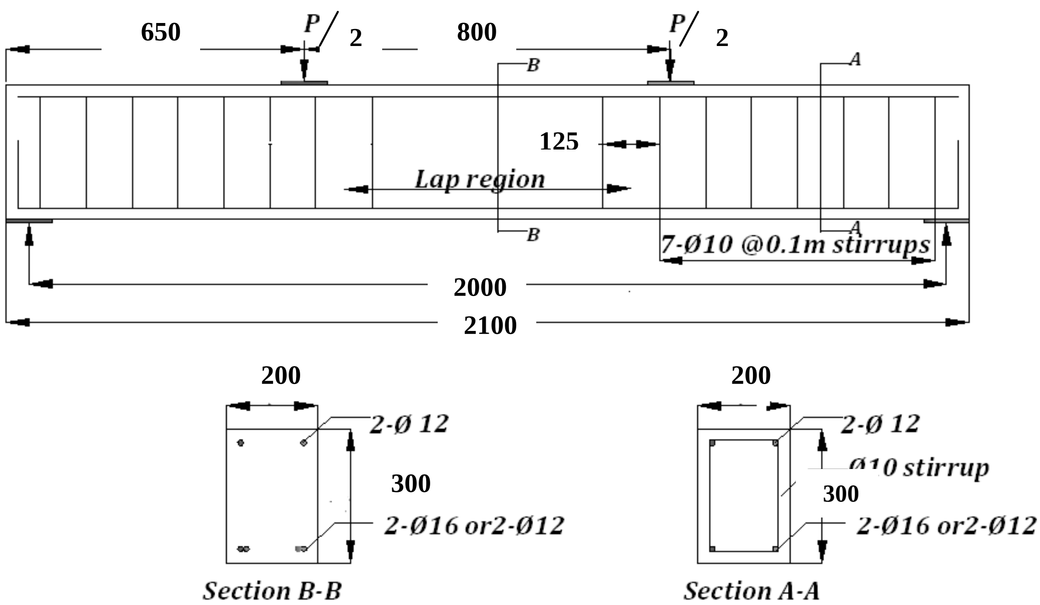


Fig.(1): Reinforcement details of specimens (BS)

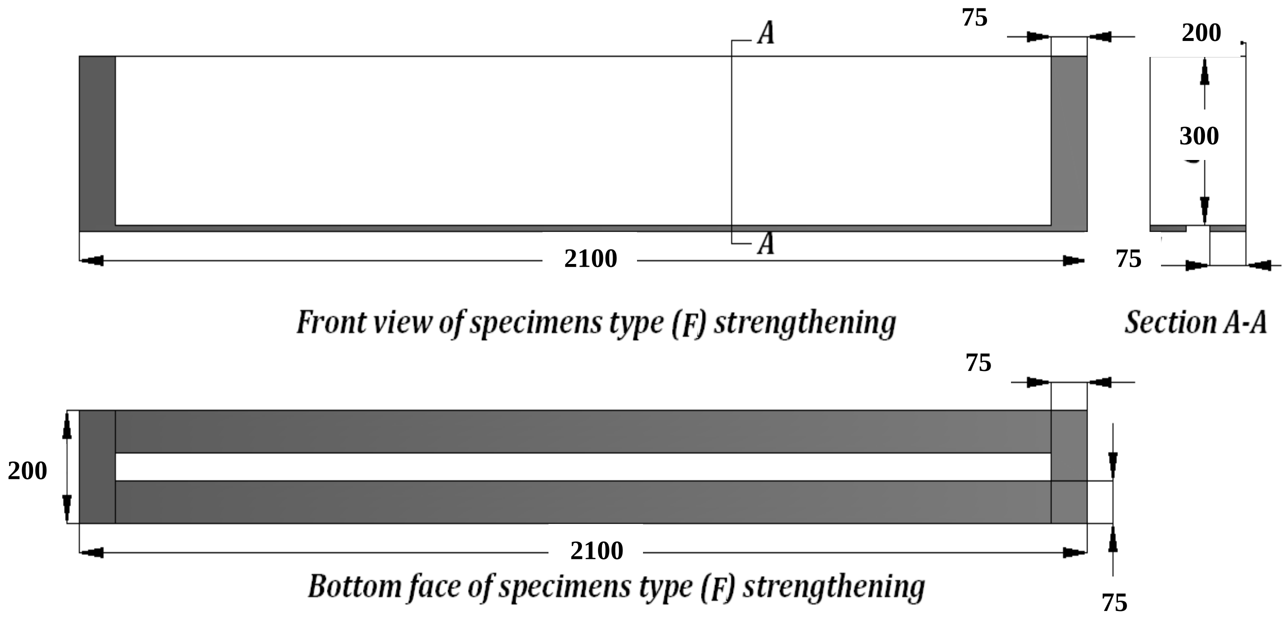


Fig.(2): Details of type (F) strengthening

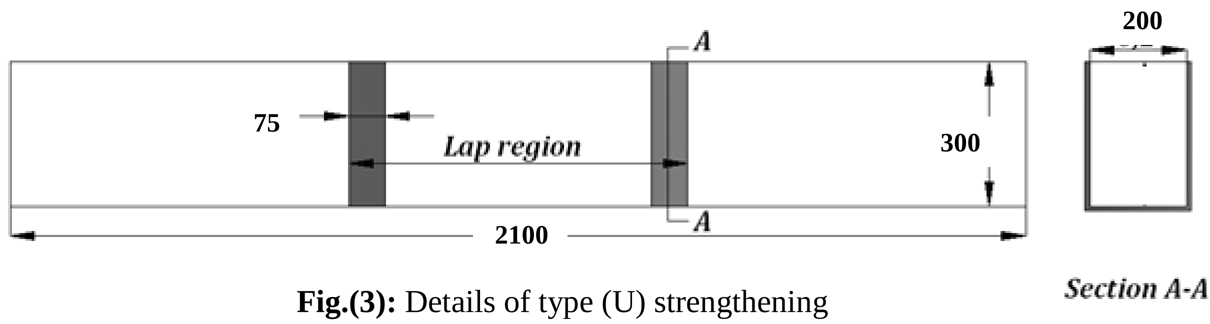
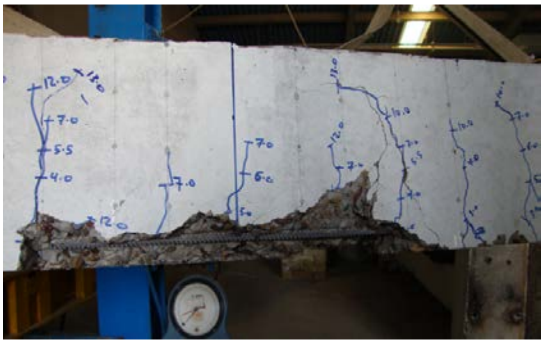


Fig.(3): Details of type (U) strengthening

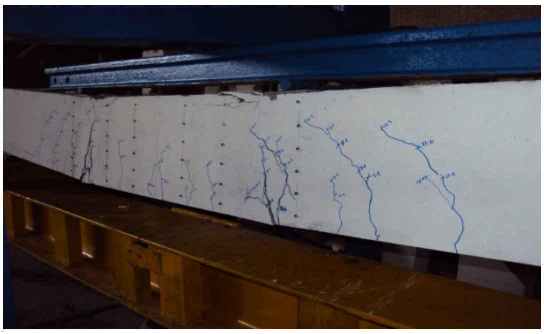
Fig.(4): Control beam in testing machine

(a) B-16



(b) B-12

(c) BC-16



(d) BS-12

(e) BS-16

(f) B-F12

(g) B-F16

Fig.(5): Modes of failure

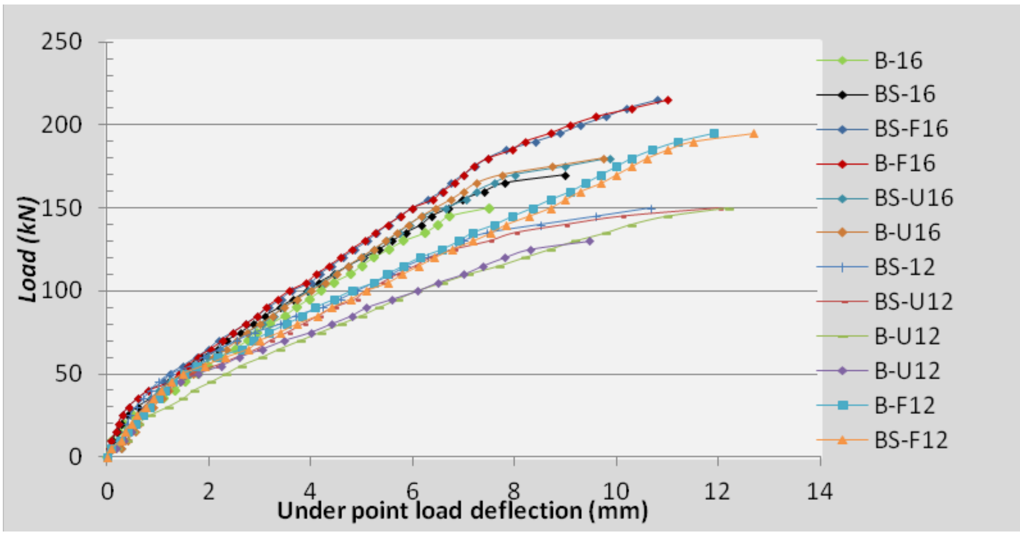
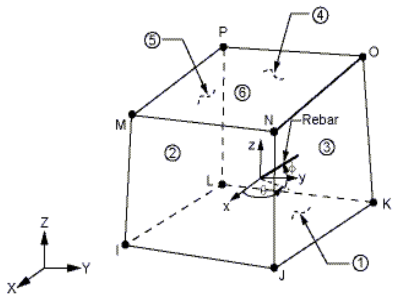
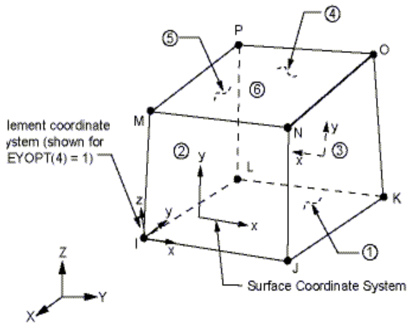


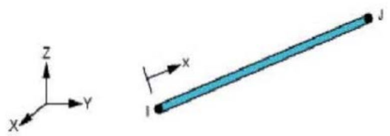
Fig. (6): Load deflection curves for tested beams



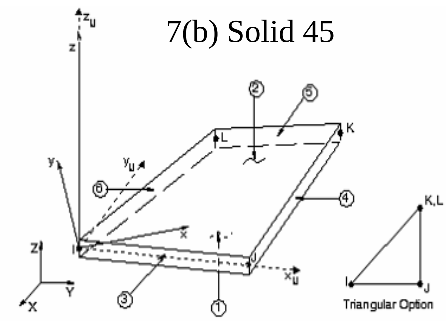
7(a) Solid65



7(b) Solid 45



7(c) Link8



7(d) Shell41

Fig.(7): Geometry of elements

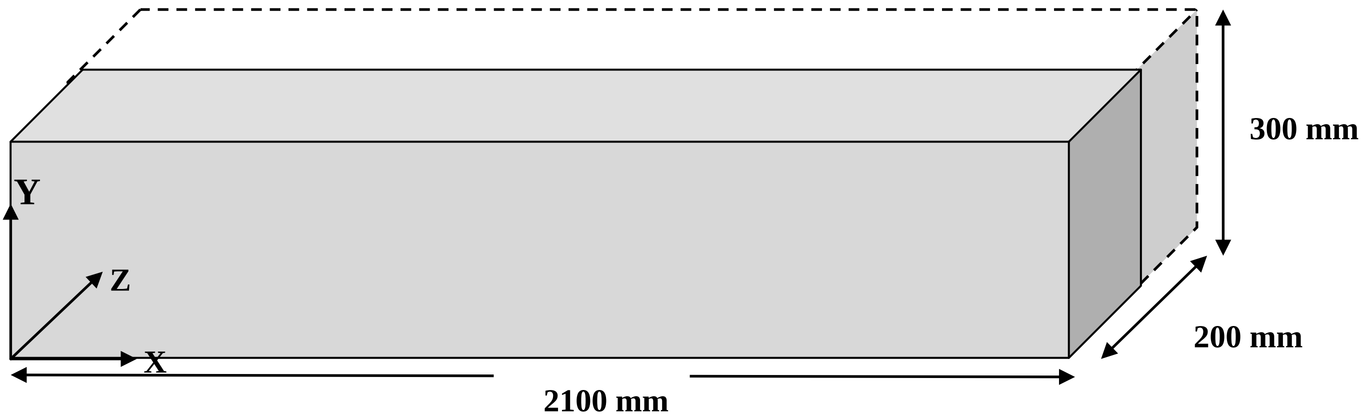


Fig.(8): Sketch for one half of the beam (not to scale)

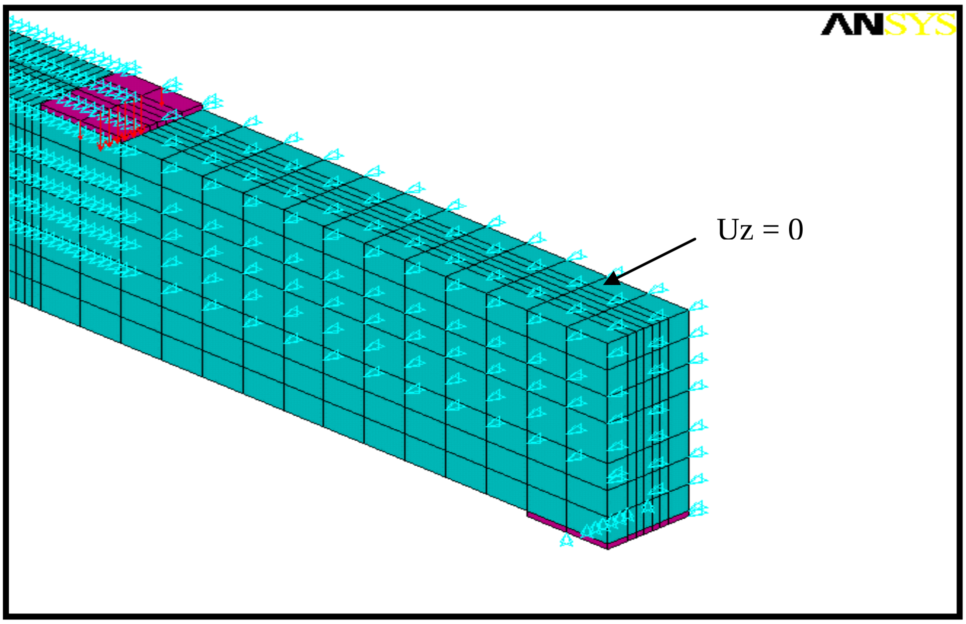
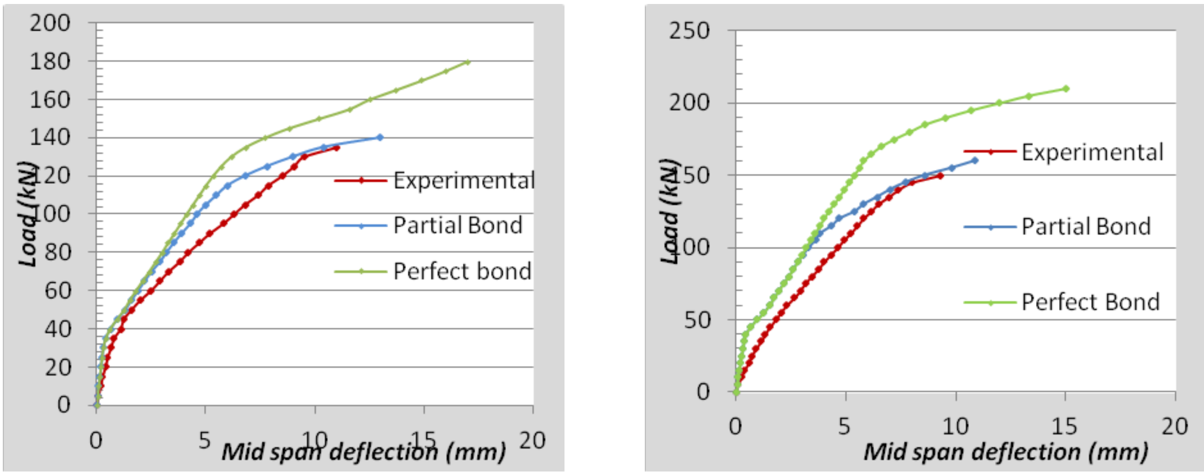
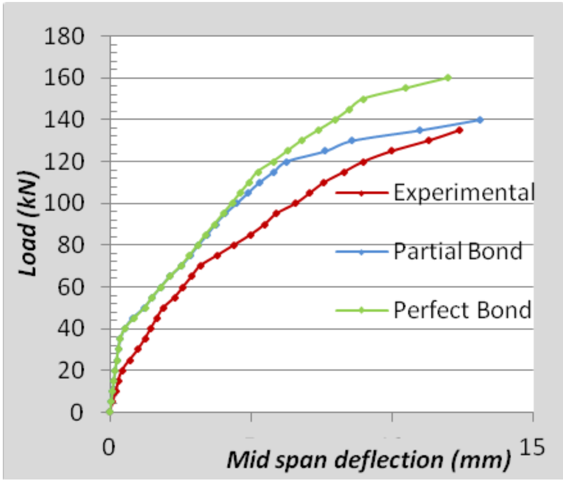


Fig.(9): Finite element modeling

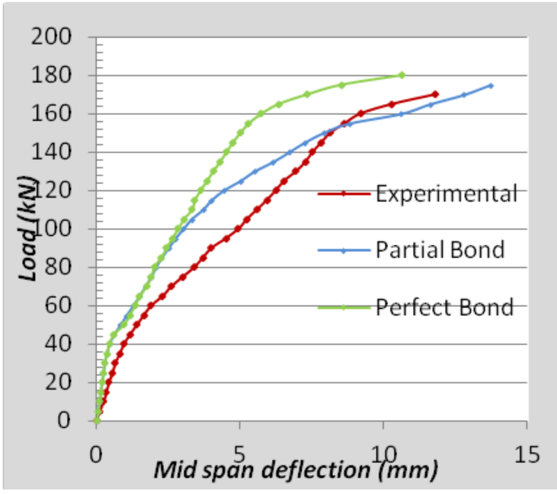


B-12 B-16

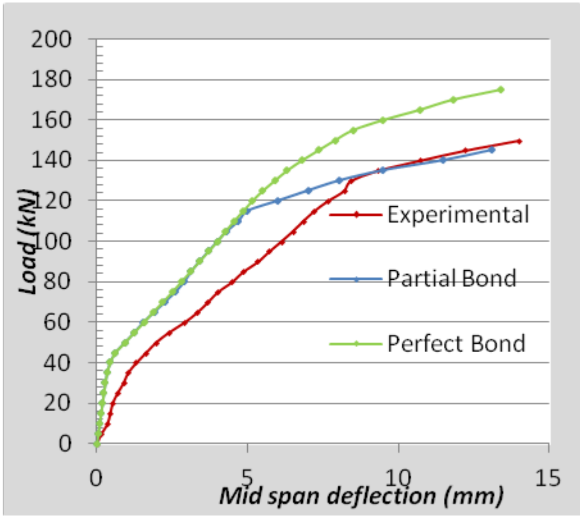
Fig.(10): experimental and theoretical load deflection curves for tested beams



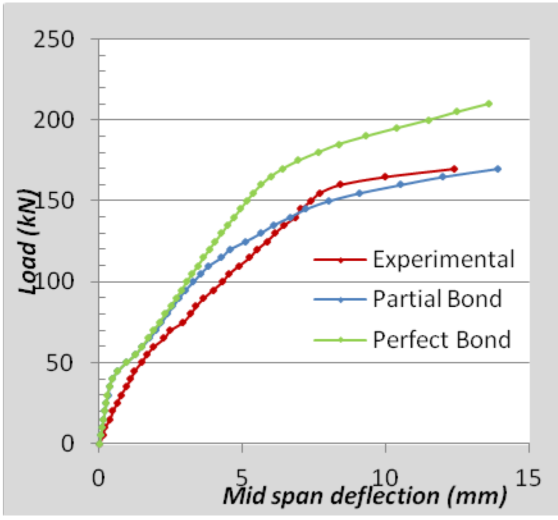
BC-12



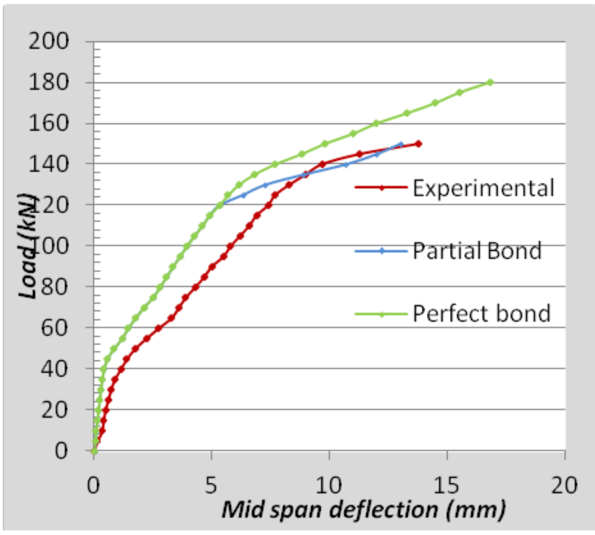
BC-16



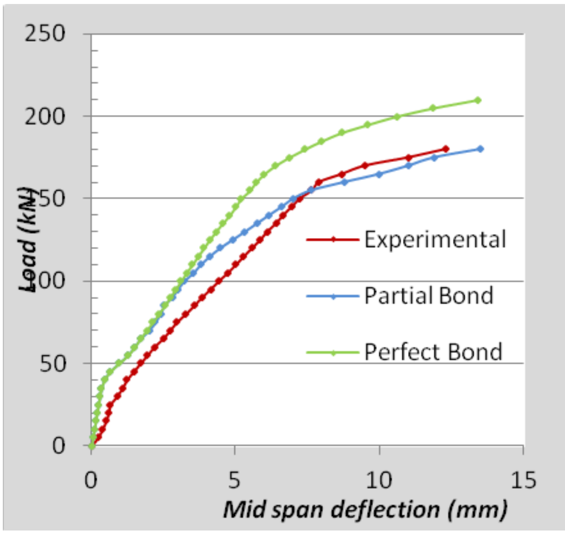
BS-12



BS-16

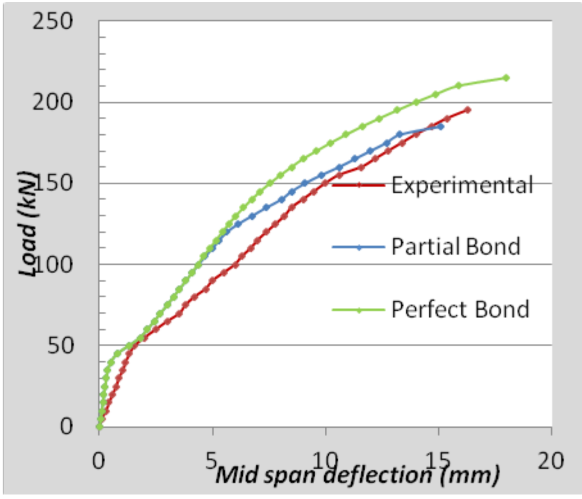


B-U12

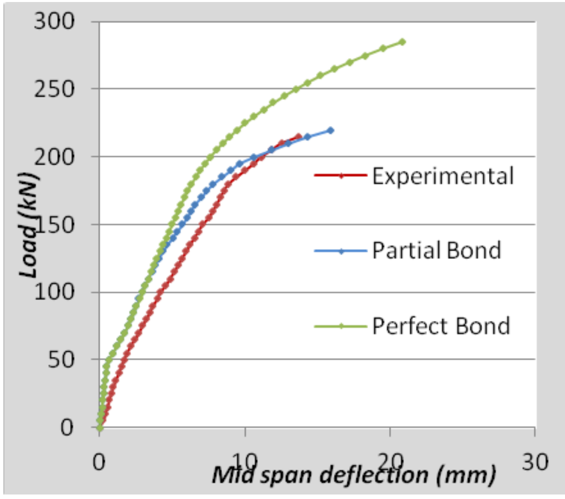


B-U16

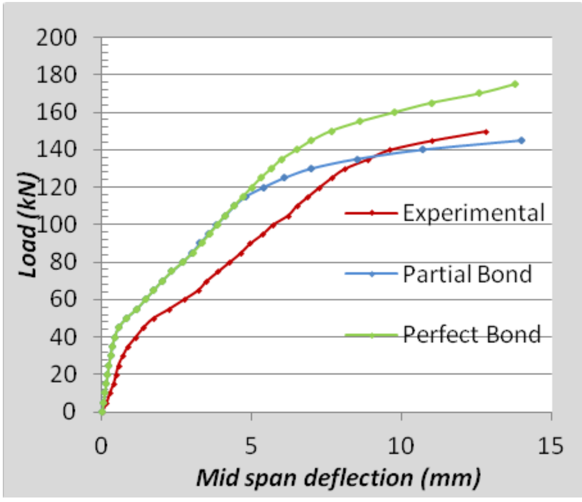
Fig. (11): Continued



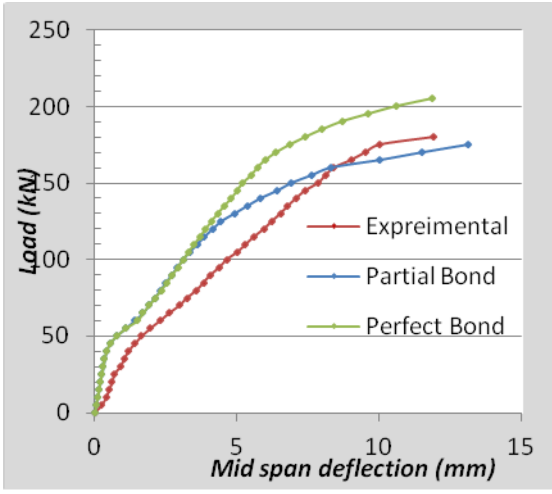
B-F12



B-F16



BS-U12



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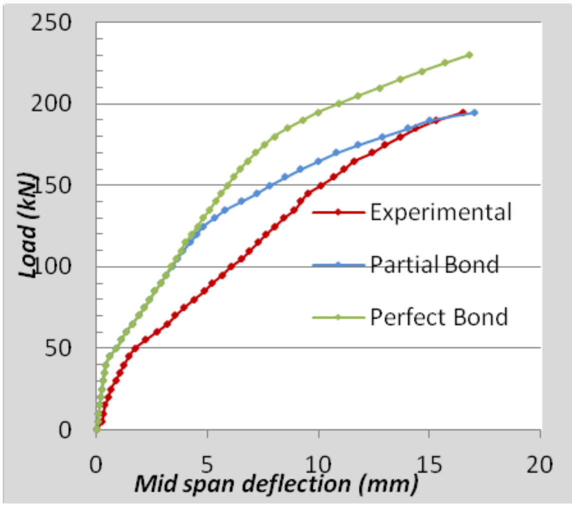
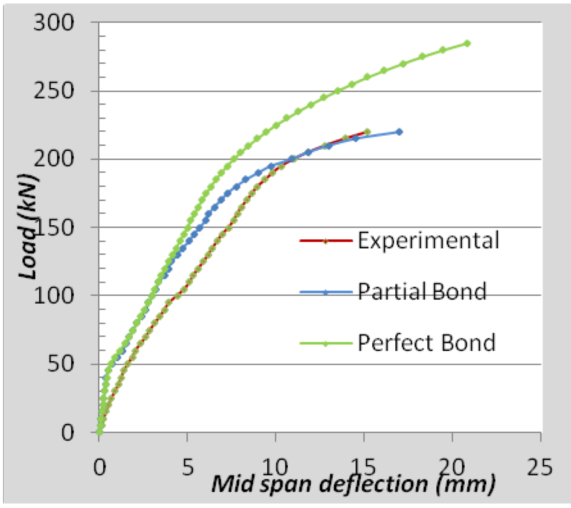


BS-F12



BS-F16

Fig. (12): Continued

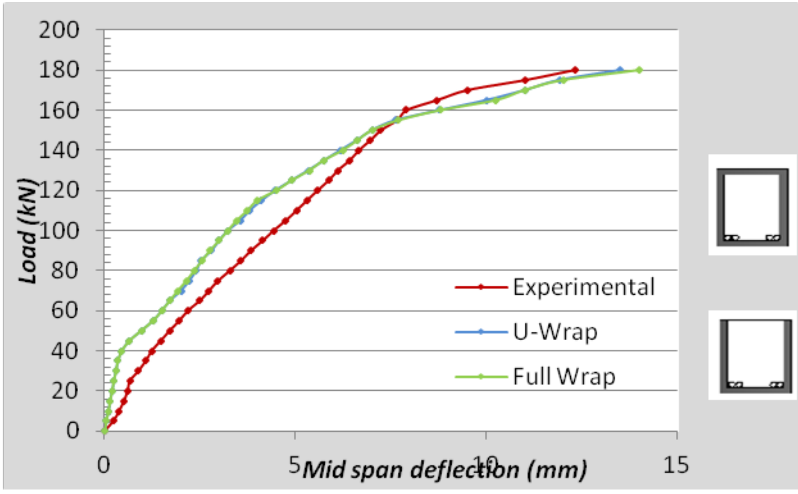


Fig. (13): Load deflection curves of beams with full and partial wrap

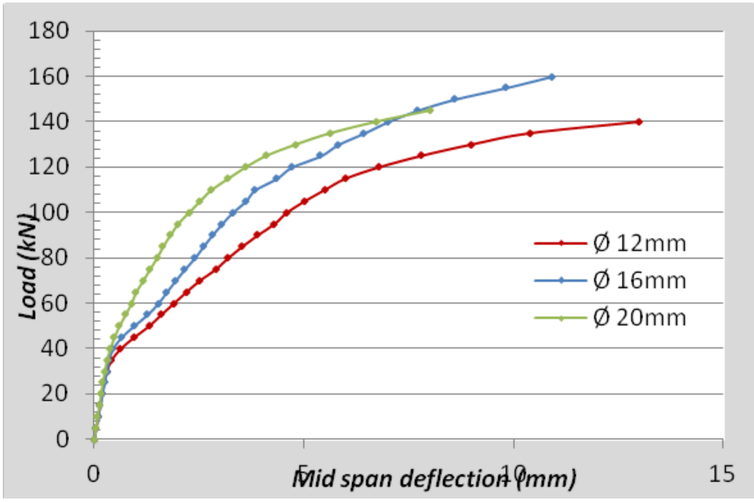


Fig. (14): Load deflection curves of beams with different diameters of bar