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Effect of Using Recycled Coarse Aggregate to the Bond Stress in Term of Beam Splice Specimens

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Abstract

In fact, demolition west disposal represents a serious problem in the civil engineering work since such materials are accumulated in large quantities. In this way, using these materials in new construction is considered a good sustainable and cost effective solution. The basic objective of this study is to investigate the behavior of lap splice when recycled coarse aggregate is used in structural members by experimental program. This program comprises casting 12 beam splice specimens. Two mix designs are proposed with nominal compressive strength of 20 and 30 MPa, more precisely, the degrees of coarse recycled aggregate partial replacement ratio that taken throughout this study are 0, 50 and 100% respectively using a crushed concrete casted with the same original mixes defined. Since a considerable lack of information was observed about the role of recycled coarse aggregate when the bond stress is taken into account, the beam splice specimens during this study were devoted to investigate lap splice bond strength in both singly and doubly beams to discover the desired behavior in tension and compression. The results showed that the degree of recycled coarse aggregate decreases the consequent bond stress in term of beam splice specimens for singly and doubly beams. The brittle failure behavior is evident in the entire beam specimens that conducted throughout this study.

Keywords: Recycled Coarse Aggregate; Concrete; Reinforcement; Bond Stress; Beam Splice Specimens.

1. Introduction

Usually, structures like bridges, roadways and buildings are still have a progressive increasing rate in the urban areas. When the old units of such structures reach the end of its service life and/or no longer satisfy their purposes, repairing or replacement processes are dictated which in turn increases the demand for a certain construction materials like concrete and asphalt aggregates. Concrete demolition aggregate or simply recycled aggregate concrete is a very common material that proved a significant role within this field as a cost effective and sustainable agent to substitute normal aggregate because such aggregate is generated in huge quantities every year as a waste material [1].

As a consequence, recycled aggregate concrete was used recently in different ways such as soil stabilization as well as being a recycled aggregate in concrete buildings construction.

More precisely, the main difference that can be recognized between the recycled and normal aggregate is the presence of the mortar reminders around its particles, however, such presence dictates more pores to be evident which means that many chemical and physical properties are dissimilar, due to that, the consequent characteristics and performance of the concrete can vary to a great concern. This variety is extended to the mechanical behavior and durability as well as low

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levels of density and specific gravity in addition to high water absorption. As a result, the specifications related to this field are still seeking for increasing confidence about the using of recycled materials in civil engineering applications [2-4].

On the other hand, when rebar's are used in concrete construction, the entire length may be not identical to the required value due the commercial length availability and / or the nature of the construction stages and this dictates that these rebar's should be fastened by suitable economic and fast ways. Overlapping is one of the most common methods to perform this task to ensure the required continuity. However, such continuity should be guaranteed to avoid undesirable slips and relevant excessive cracking.

In addition, when laps is used, the bars will continue to withstand stresses even after the failure, and the confinement will also enhanced because bars are more spaced, as a result, the risk of the joint brittle behavior in the post peak stage is reduced [4, 5].

Obviously, conducting a preliminary related tests is not enough to assess the performance of bond characteristics in RCA concrete applications since the results of such tests have very limited dependencies in the design considerations [6], in this way, implementing a parallel full scale lap splices specimens in addition to these tests is highly needed and justified.

Additionally, the mechanical properties of waste road concrete were studied in some detail [7]. On the other hand, such properties were also investigated in the presence of the partial replacement of fly ash and Granulated blast furnace slag [8]. The recycled coarse aggregate that produced from the construction repair and demolition was also included in some recent contributions [9-11] while some of these contributions were extended to produce high strength levels of recycled coarse aggregate concrete [13]. Additionally, the resource preservation and environmental concerns of this type of concrete ware also discussed in some details [14]. Accordingly, some other susceptible issues like the influence of age and successive recycling were taken into account [15]. Moreover, some of the scientific research programs were aimed in the paste to enhance some of the shortcomings like the surface low quality and performance degradation in this type of concrete [16-19].

However, some of the observed experimental programs throughout the past experience literature were devoted to investigate the bond behavior between full recycled aggregate concrete and steel reinforcement [20].

Furthermore, it can be clearly recognized throughout the entire literature that low information are now available about bond behavior and lap splice in tension and compression when recycled coarse aggregate is used, as a result, this stimulates scientific research within this field to understand the current issue more and more.

The basic aim of this study is to investigate the behavior of lap splice in recycled coarse aggregate in both tension and compression and characterize a descriptive view about the presence of such type of aggregate and its role as an alternative choice to natural aggregate regarding the lap splice bond stress.

2. Research Methodology

2.1. Used Materials

2.1.1. Cement

Type I of ordinary Portland cement (Tasluoja commercial brand) is used in the experimental program of this study. Tables 1 and 2 list the physical properties and chemical composition respectively while Table 3 lists the main compounds of such cement.

Table 1. Physical properties of cement used

Physical Properties	Test Results*	Limits of Iraqi Specifications No.5/198
Specific Surface Area (Blaine Method),cm^2/g	298.5	2
Setting Time (Vicat Apparatus)		
Initial Setting, (min)	166	Not less than 45
final setting, (min)	255	Not greater than 10 hr.
Compressive strength, MPa at 3 days	18.76	≥ 15.00
Compressive strength, MPa at 7 days	26.81	≥ 21.00
Soundness (autoclave Method), %	0.35	≤ 0.8

Table 2. Chemical composition and main compounds of cement

Compound Composition	Chemical Composition	Content%	Limits of Iraqi Specifications No.5/1984
Lime	CaO	62.7	---
Silica	SiO_2	18.45	---
Alumina	Al_2O_3	5.35	---
Iron oxide	Fe_2O_3	3.64	---
Magnesia	MgO	3.2	<5.00
Sulfate	SO_3	2.12	<2.80
Loss on ignition	L.O.I.	2.96	<4.00
Insoluble residue	I.R	0.95	<1.5
Lime saturation factor	L.S.F	0.8	(0.66-1.02)%

Table 3. Main compounds (bougue's equations)

Compounds Name	Symbol	Content %
Tricalcium Silicate	C_3S	67.76
Dicalcium Silicate	C_2S	1.85
Tricalcium Aluminate	C_3A	8.02
Tetracalcium Aluminoferrite	C_4AF	11.06

2.1.2. Fine Aggregate

Figure 1 shows the natural sand that used in the present study which brought from Al-Sudour suburb near Baqubah within Diyala governorate, Iraq. Table 4 lists the physical properties whereas the grain size distribution of that sand is also illustrated in Figure 2.

Figure 1. Fine aggregate before mixing

Table 4. Physical properties of fine aggregate

Physical properties	Test result	Limits of Iraqi Specifications No.45/1984
Specific gravity	2.60	-
Sulfate content	0.11%	0.5% (max)
Absorption	0.75%	-
Clay content	2.3	5% (max)

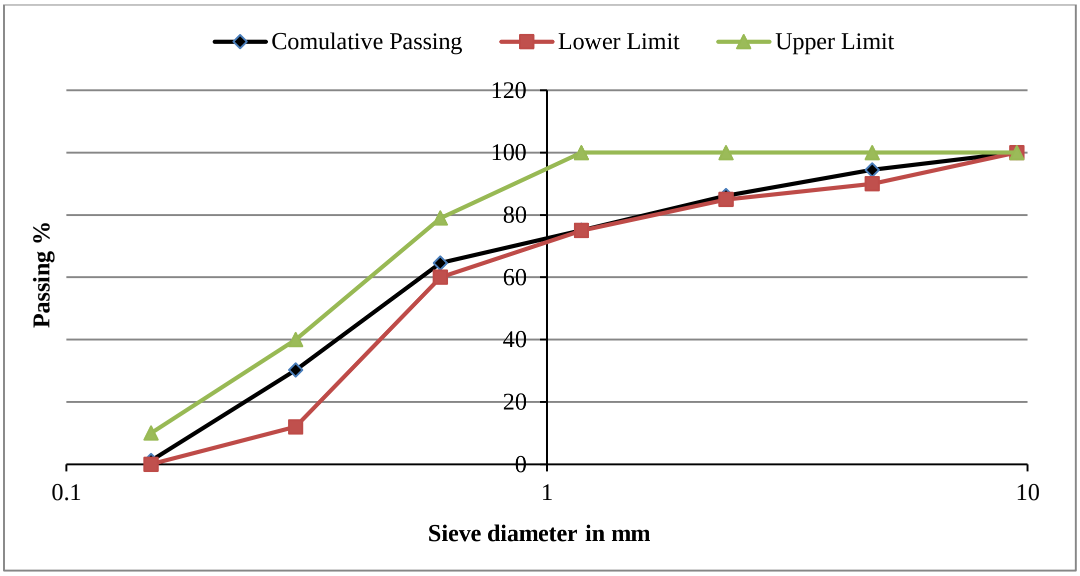

Figure 2. Grain size distribution curve of the fine aggregate

2.1.3. Course Aggregate

The natural coarse aggregate that wholly used in this study is crushed aggregate 19 mm maximum size, throughout this study, the coarse aggregate was cleaned, washed and air dried before mixing. The physical properties and the grain size distribution are listed in Tables 5 and 6 respectively.

Table 5. Physical properties of coarse aggregate

Physical properties	Test result	Limits of Iraqi Specifications No.45/1984
Specific gravity	2.60	-
Sulfate content	0.11%	0.5% (max)
Absorption	0.75%	-
Clay content	2.3	5% (max)

Table 6. Grain size distribution of coarse aggregate used

Sieve size (mm)	Test result	Limits of Iraqi Specifications No.45/1984
25	100	100
19	100	90-100
12.5	-	-
9.5	50	20-55
4.75	0	0-10
2.36	0	0-5

2.1.4. Recycled Course Aggregate

The physical properties of the RCA used entirely in this study are presented in Table 9.

Table 7. Physical properties of recycled coarse aggregate

Physical properties	20 MPa	30 MPa
Specific gravity	2.45	2.47
Dry specific gravity	0.11%	2.4
Absorption	3 %	2.4
Loss density kg/m^3	1300	1360
Compact density kg/m^3	1530	1580

2.1.5. Steel Reinforcement

16 mm in diameter deformed steel reinforcement bars are used as the main flexural reinforcement while bars of 10 mm in diameter are also used as shear reinforcement, in addition, bars of 12 mm in diameter were used to hold shear reinforcement. Three samples of each bar size are subjected to the tensile test according to ASTM C370-05a in the test machine shown in Figure 3 while Table 8 lists the general properties of such bars.

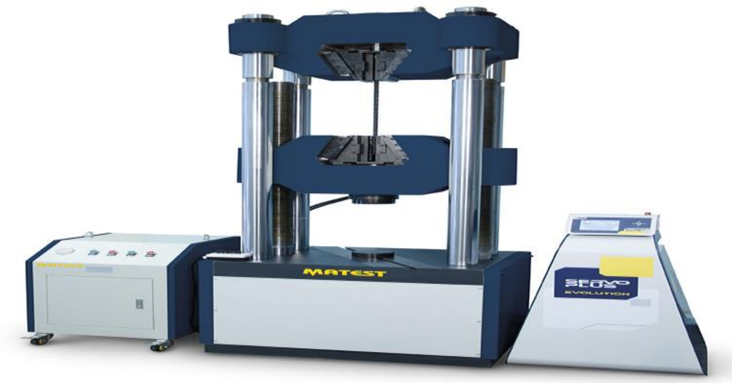

Figure 3. Machine used for testing steel bars in the present study

Table 8. Properties of the Reinforcing Steel Bars

Nominal Diameter (mm)	Nominal Area (mm^2)	*fy* (MPa)	*fu* (MPa)	*Es* (GPa)
16	201	420	630	200
12	113	430	645	200
10	79	430	450	200

2.2. Trial Mixes

Actually, series of trials were made to obtain two certain concrete mixes to achieve cylinder compressive strength 20 and 30 MPa respectively, however, the mix proportions that were established to get 20 and 30 MPa by weight [cement: sand: coarse aggregate] are (1:2.58:3.22) and (1:1.86:2.63) respectively.

In addition, another series of trials were also made to get the same limits of compressive strength for each degree of RCA replacement proposed using the produced RCA.

2.3. Bond Strength in Term of Lap Splice Specimens

Beam splice specimens were used in this study to inspect the behavior of splice bond strength. Twelve simply supported reinforced concrete beams were casted to do this role. In addition, such beams are inherently designed and reinforced to fail in flexural with tension failure (to develop actual tensile lap splice) and compression failure (to develop actual compression lap splice) according to ACI 318-14.

2.3.1. Lap Splice length Design

The lap splice length during this study is designed also according to ACI 318-14 as follows:

- **Lap Splice in Tension**

Tension Lap splice length is designed according to the following equation:

$$L_{db} = \left(\frac{fy}{1.1\lambda\sqrt{f\acute{c}}} \frac{\Psi t \, \Psi e \, \Psi s}{\left(\frac{ktr + cb}{d_b}\right)} \right) d_b \quad (1)$$

L_{db} = Development length;

f_y = Specified yield strength of reinforcement;

f'_c = Specified compressive strength of concrete;

Ψ_t = Reinforcement location modification factor;

Ψ_e = Reinforcement coating modification factor;

Ψ_s = Reinforcement size modification factor;

c_b = Smallest of distance from center of a bar to nearest concrete surface or one-half the center-to-center bar spacing;

K_{tr} = Transverse reinforcement index;

d_b = Nominal diameter of the reinforcing bar.

$$L_d = L_{db} * As(requirred)/As(provided) > 300mm \tag{2}$$

Lap splice finally were calculated equal to L_d if it is classified as class A or should be $1.3L_d$ when it is classified as class B.

In general, most of the lap splices are categorized as class B and class A exists when the provided steel is at least twice the required.

- **Lap Splice in Compression**

The Lap splice in compression is calculated as follows:

$$L_{db} = \left(\frac{d_b fy}{4\sqrt{f\acute{c}}}\right) \geq 0.04\ d_b\ f_y \tag{3}$$

In addition, the ACI code equations for lap splice for compression are:

For bars with $f_y \leq 420$ MPa $\quad 0.07 f_y\ d_b$ (4)

For bars $f_y > 420$MPa $(0.13 f_y - 24)\ d_b$ Not less than 300 mm (5)

Figure 4 shows total reinforcement before casting while Figures 5 and 6 shows schematic view of such splices in tension and compression.

Figure 4. Reinforcement of singly and doubly beams

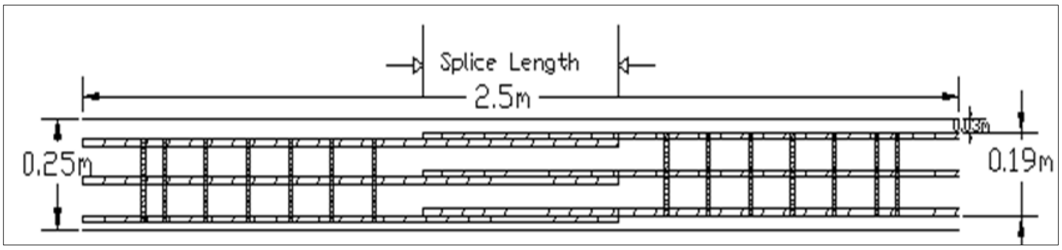

Figure 5. Lap splice scheme for singly beam

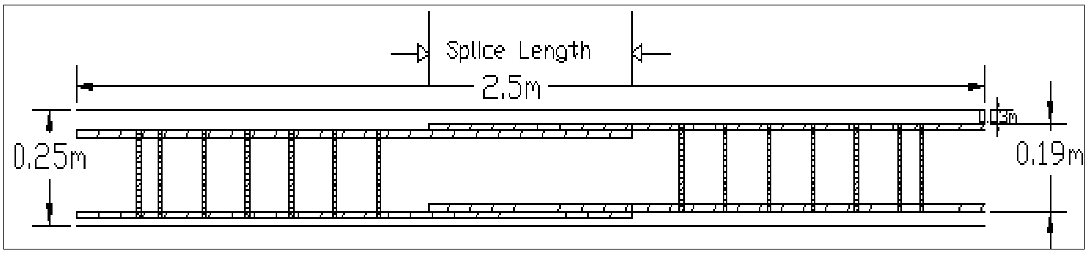

Figure 6. Lap splice scheme for doubly beams

2.3.2. Beam Splice Specimens Map

The proposed beams are divided into four groups with respect to the same proposed control mix designs. Additionally, these specimens are subdivided according to singly and doubly reinforced specimens to represent the lap splices in tension and compression respectively. Table 9 lists the specimens' dimensions while Figure 7 shows the beams dimensions of these groups.

Table 9. Map of the beam splice specimens groups

Group	f'_c (MPa)	Specimen designation	Replacement (%)	Development length (mm)	Lap Splice (mm)	Location of lap splice
		SBC20R0	0	432.75	560	bottom
A	20	SBC20R50	50	432.75	560	bottom
		SBC20R100	100	432.75	560	bottom
		DBC20R0	0	375.65	470	top
B	20	DBC20R50	50	375.65	470	top
		DBC20R100	100	375.65	470	top
		SBC30R0	0	353	460	bottom
C	30	SBC30R50	50	353	460	bottom
		SBC30R100	100	353	460	bottom
		DBC30R0	0	306.7	470	top
D	30	DBC30R50	50	306.7	470	top
		DBC30R100	100	306.7	470	top

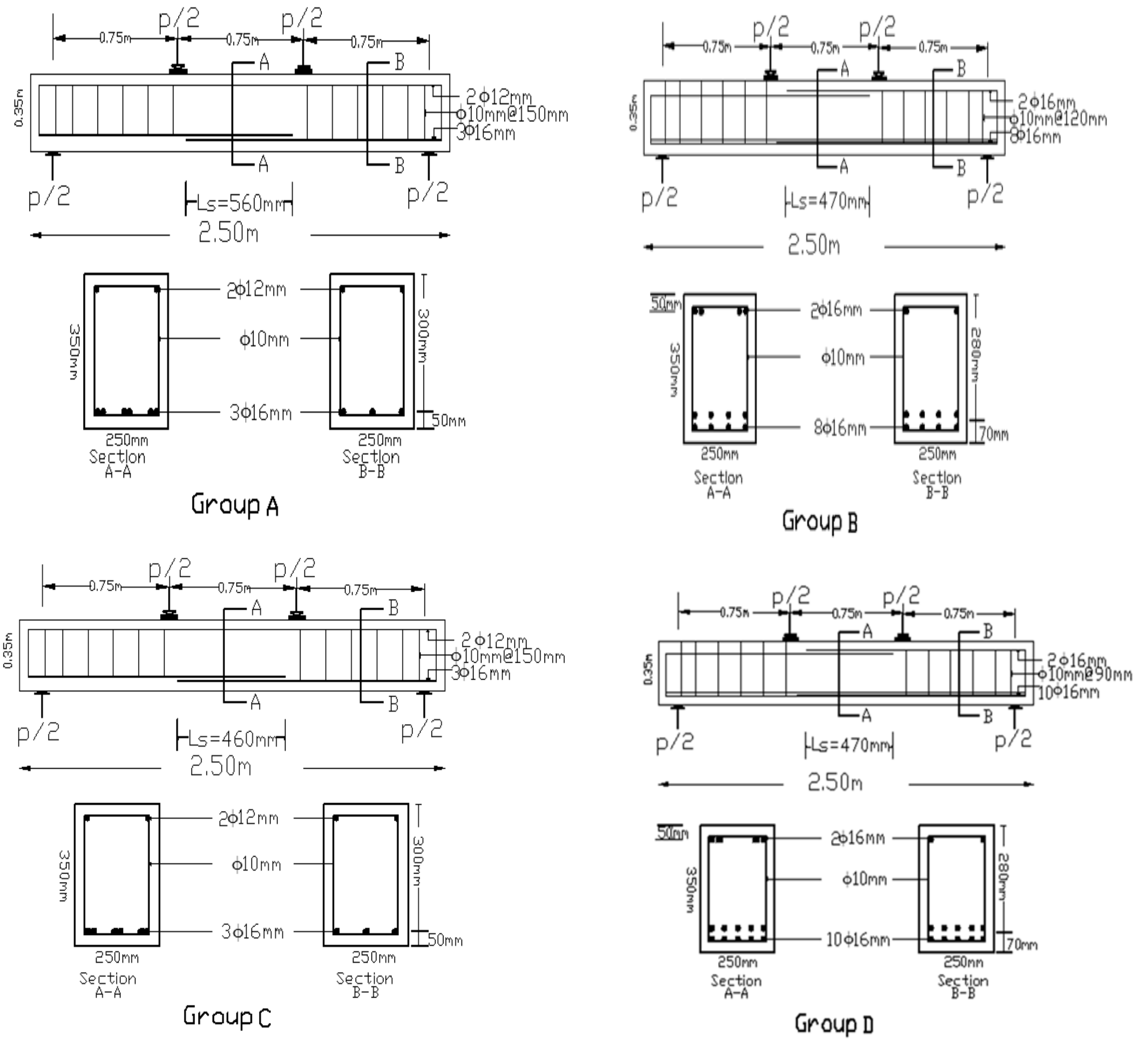

Figure 7. Dimensions details of beam specimens for the four groups

2.3.3. Strain Measurement of Steel Bar, and Concrete Surface

Two certain types of uniaxial certain gauges were used to calculate strain developed in steel bars and concrete surface, these types are illustrated in Figures 8 to 12.

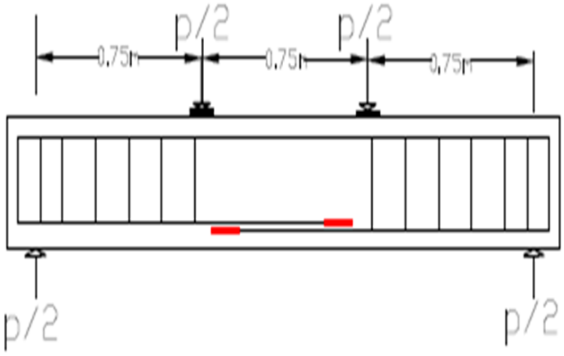

Figure 8. Scheme of strain gauges for singly beam

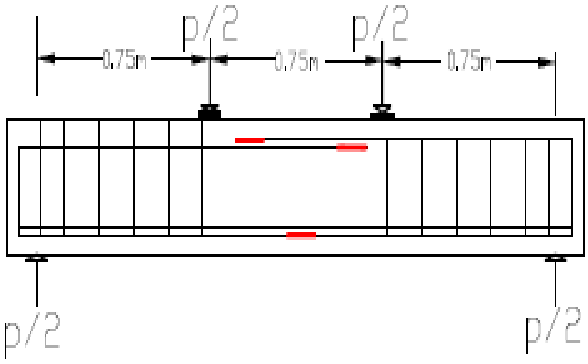

Figure 9. Scheme of train gauges for doubly beam

Figure 10. Strain gauges for singly beam

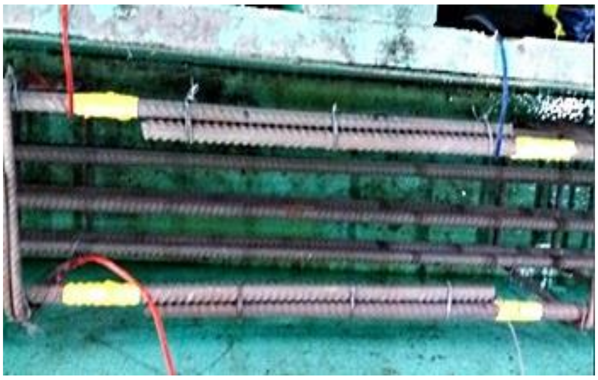

Figure 11. Strain gauges for doubly beam

Figure 12. Strain gauges on concrete surface

2.3.4. Beam Specimens Bond Stress Calculation

Figure 13 shows a steel reinforced concrete beam subjected to a known level of load in addition to an idealized segment within this beam to illustrate its free body diagram.

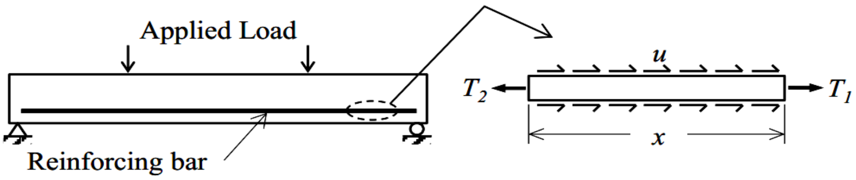

Figure 13. Bond stress calculation

Taking equilibrium of this segment within that beam:

$$T_1 - T_2 = A_s f_y = \pi u\, d_b\, L_d \tag{6}$$

This yields that the bond stress is calculated as follows:

$$u = \frac{f_s d_b}{4\, L_d} \tag{7}$$

Where:

U: Bond stress;

d_b: Bar diameter;

f_s: Steel stress;

L_d: Development length.

The equation used to compare with bond stress specimens is ACI committee 408R-03 as flows:

$$\frac{T_C}{{f_c}'^{1/4}} \left[1.43L_{d\ (c_{min}+0.5d_b)+57.4A_b}\right] 0.1\frac{c_{max}}{c_{min}} + 0.9 \tag{8}$$

Where:

T_C: Bond strength;

C_{min}: Smaller of minimum concrete cover;

C_{max}: Maximum of concrete cover;

A_b: Area of steel.

2.4. Test Measurements and Instrumentation

The methods followed as well as the instruments details used to do the aim required of beam splice specimens are illustrated as follows:

2.4.1. Deflection Measurements

The deflection of beam specimens was calculated by using dial gauges located at the mid span of each beam as shown in Figure 14. This dial gauges is 0.01 mm in accuracy.

Figure 14. Dial gauges position

2.4.2. Crack Width

The first flexural crack width was observed throughout this study for all load stages by using micro-crack meter device with a 0.02 mm accuracy as shown in Figure 15.

Figure 15. Micro crack meter device

2.4.3. TDS-530 Data Logger

(TML/ TDS-530) data logger was used to record the average strain that observed by the strain gages in concrete surface and steel reinforcement. This data logger is shown in Figure 16.

Figure 16. Strain gauges indicator used in the present research work

2.5. Beam Splice Specimens Testing

Finally, the 600 kN machine that illustrated in Figure 17.was used, before testing, the specimens were allocated in the specified position to get adequate loading positions, dial gages were established at mid span and strain gages were also wired to the required positions in data logger. In addition, bearing plates of 100×390×20 mm were used under the load positions and supports.

The load that conducted to perform the tests is monotonic in nature and applied in successive manner until failure, moreover, the required measurements (progress of cracks, crack width, strain readings, and deflection at the mid span) were recorded in the time laps between each step of load application and the testing process was considered to be accomplished when load begins to drop off.

Figure 17. Hydraulic machine used to test all the specimens

3. Results and Discussion

3.1. First Crack

Figure 18 shows the first crack loads for the three four groups.

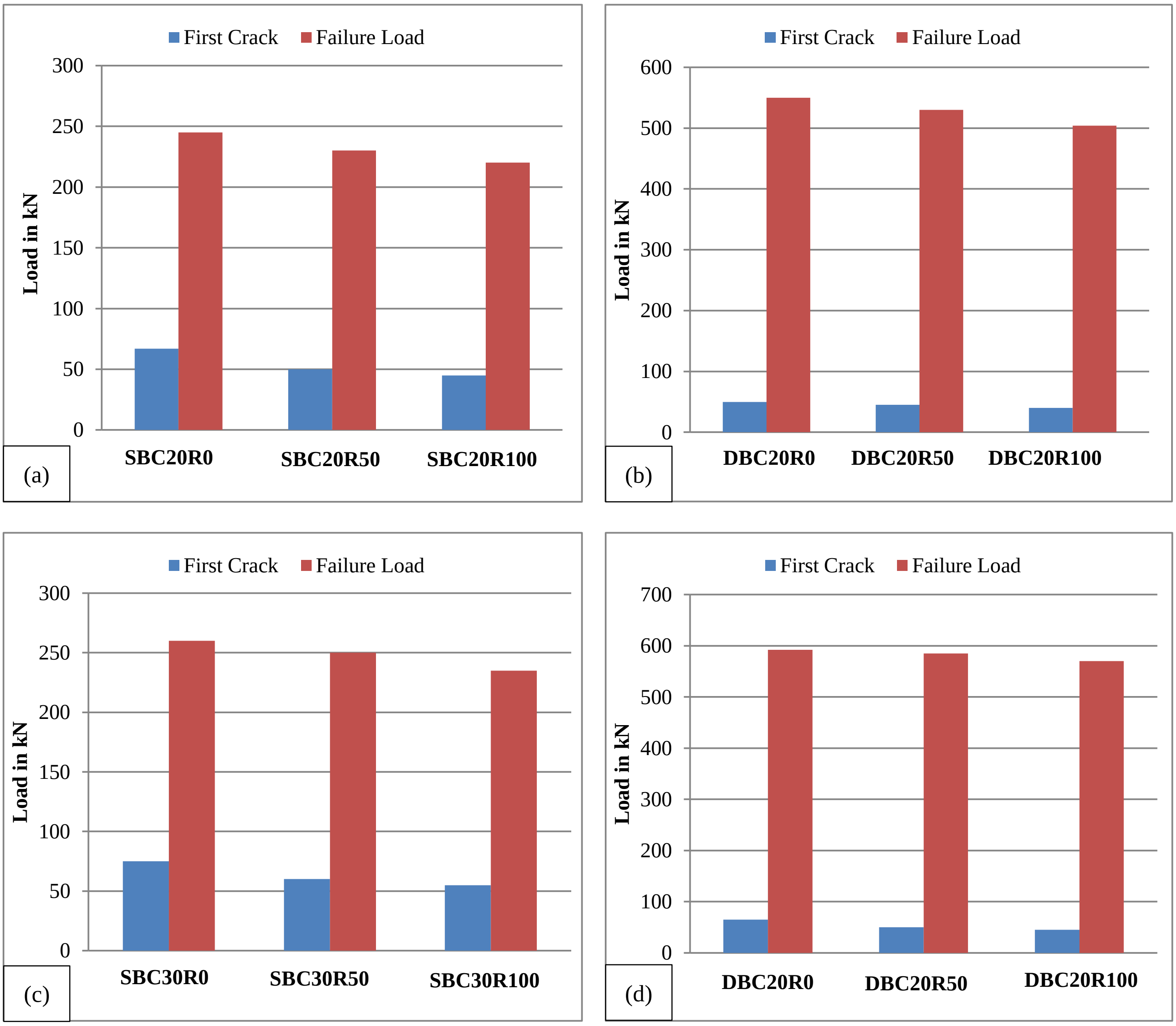

Figure 18. First crack loads: (a) Group A. (b) Group B. (c) Group C. (d) Group D

Actually, it can be recognized in the above figure that the progress degree of RCA replacement ratio makes the first crack early to appear due to the bond weakness which is inherently expected between the new paste and the old cement paste reminders. However, there is a general trend in the doubly designed beams to illustrate low levels of first crack load limit due to the inherent design, moreover, specimens of type 30 MPa mix exhibit that such limit are of high levels if compared with the corresponding type of 20 MPa mix, however, it is believed that this can be ascribed to the mechanical strength excellency of the 30 MPa mix.

Moreover, the first crack occurs below the two concentrated point load and then propagated toward the mid span till the sudden failure occurs.

3.2. Crack Width

The propagation of crack width is illustrated in Figure 19.

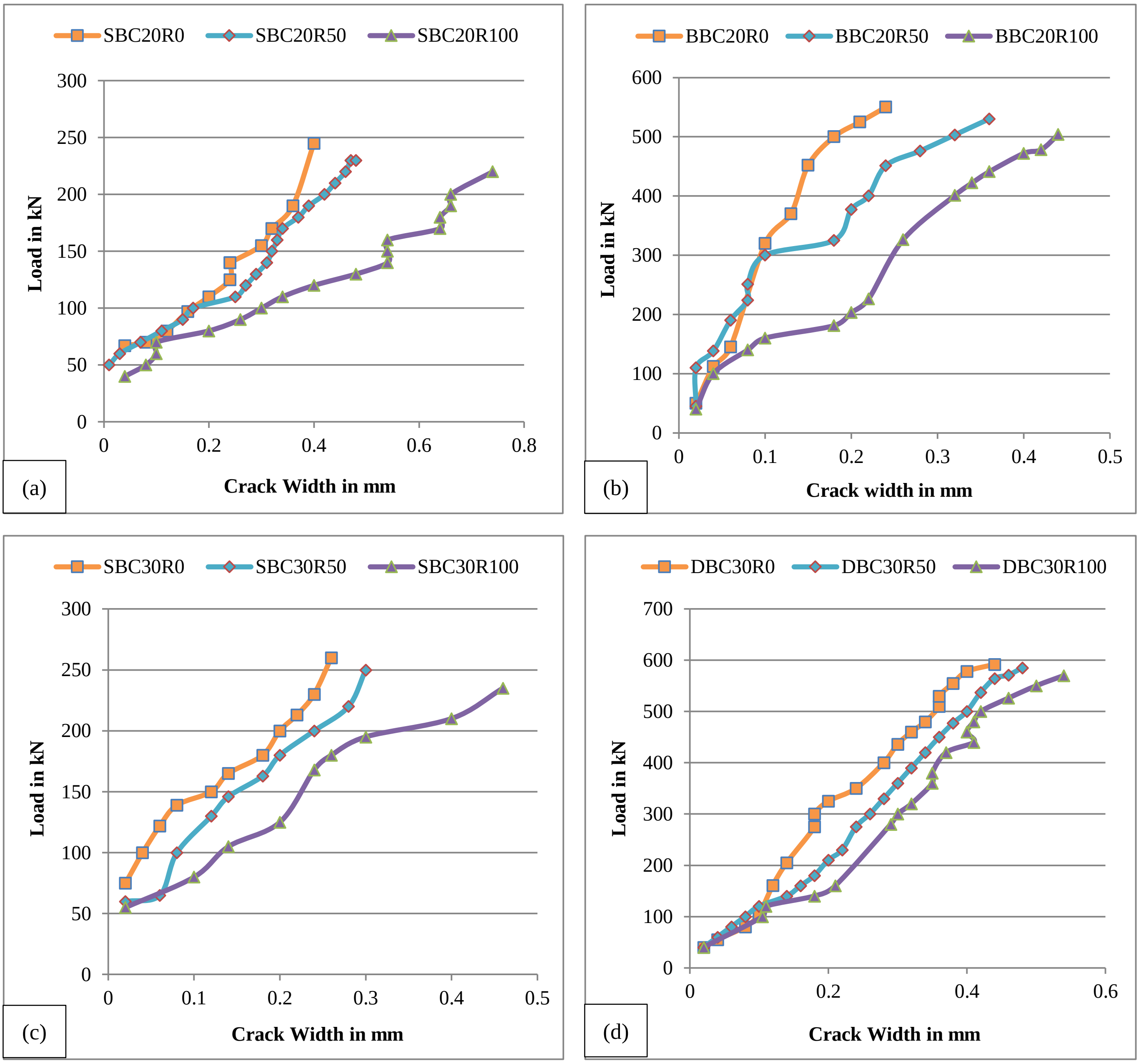

Figure 19. Crack width propagation: (a) Group A. (b) Group B. (c) Group C. (d) Group D

In fact, the crack propagation and its maximum values shows a noticeable degree of similarity. On the other hand, the role of RCA replacement is obvious and it is observed that this matter motivates further research to observe if there is a good degree of relation of such concern to other RCA considerations. Additionally, it is reported that the percent of first crack load to the final failure load is decreased as the degree of partial replacement of recycled coarse aggregate is progressed. Furthermore, the first crack occurs below the two concentrated point load and then propagated toward the mid span till the sudden failure occurs.

3.3. Load Deflection

The load mid – span deflection curves of the proposed beam specimens are illustrated in Figure 20.

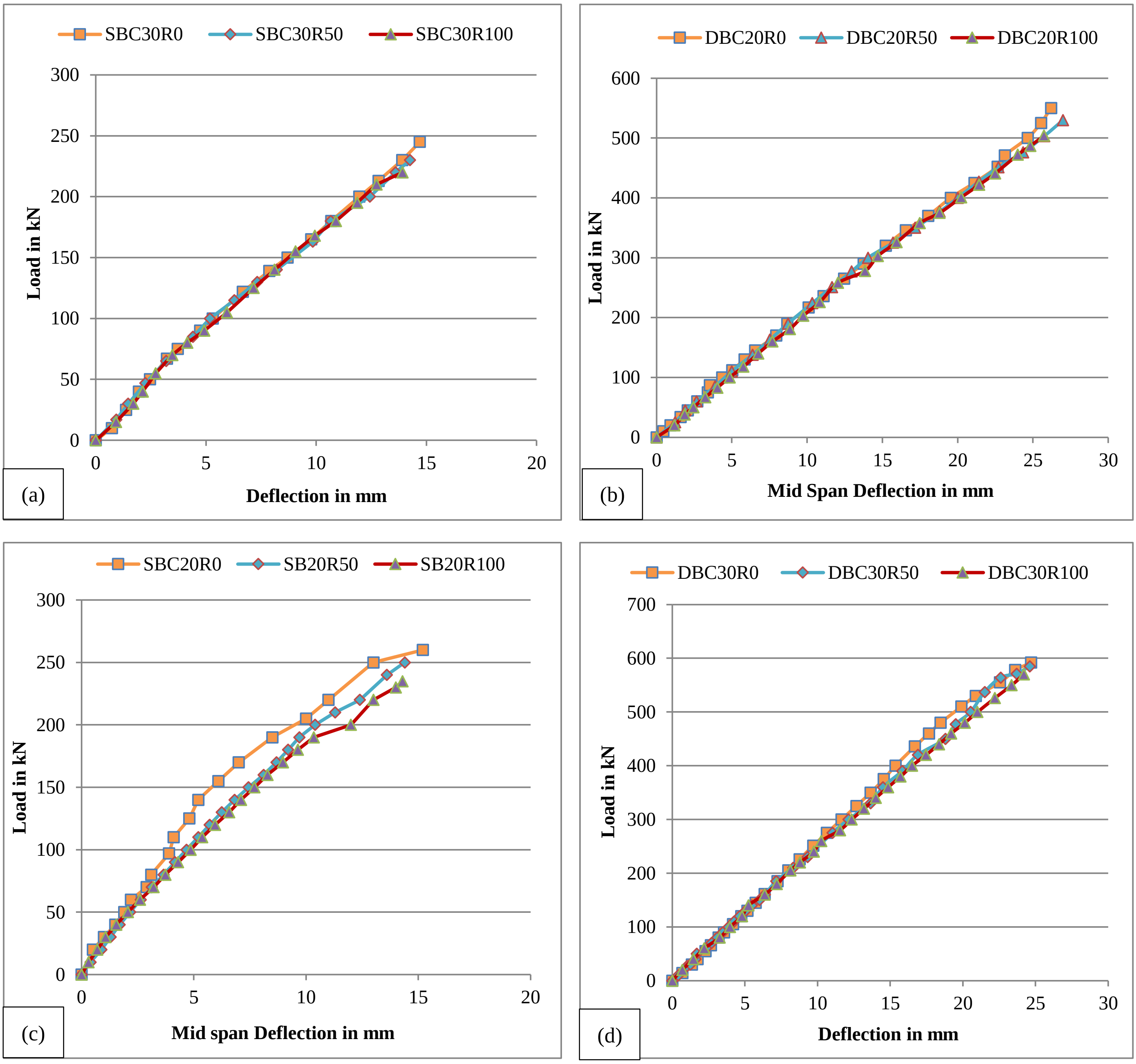

Figure 20. Mid – span deflection: (a) Group A. (b) Group B. (c) Group C. (d) Group D

It is observed throughout the mid span deflection curves that there is a usual trend to view a low level of divergence between the proposed degree of RCA replacement in doubly beams due to the design nature, in addition, the singly beams did not exhibit a uniform behavior with respect to of such divergence. Additionally, It can be noticed that there is no significant change between the proposed degree of RCA replacement in doubly beams whereon such beams seem to behave as linear elastic as these beams were inherently designed as compression failure flexural members.

3.1.4. Strain in Concrete Surface

Figure 21 shows the variation of the surface concrete strain for the defined specimens.

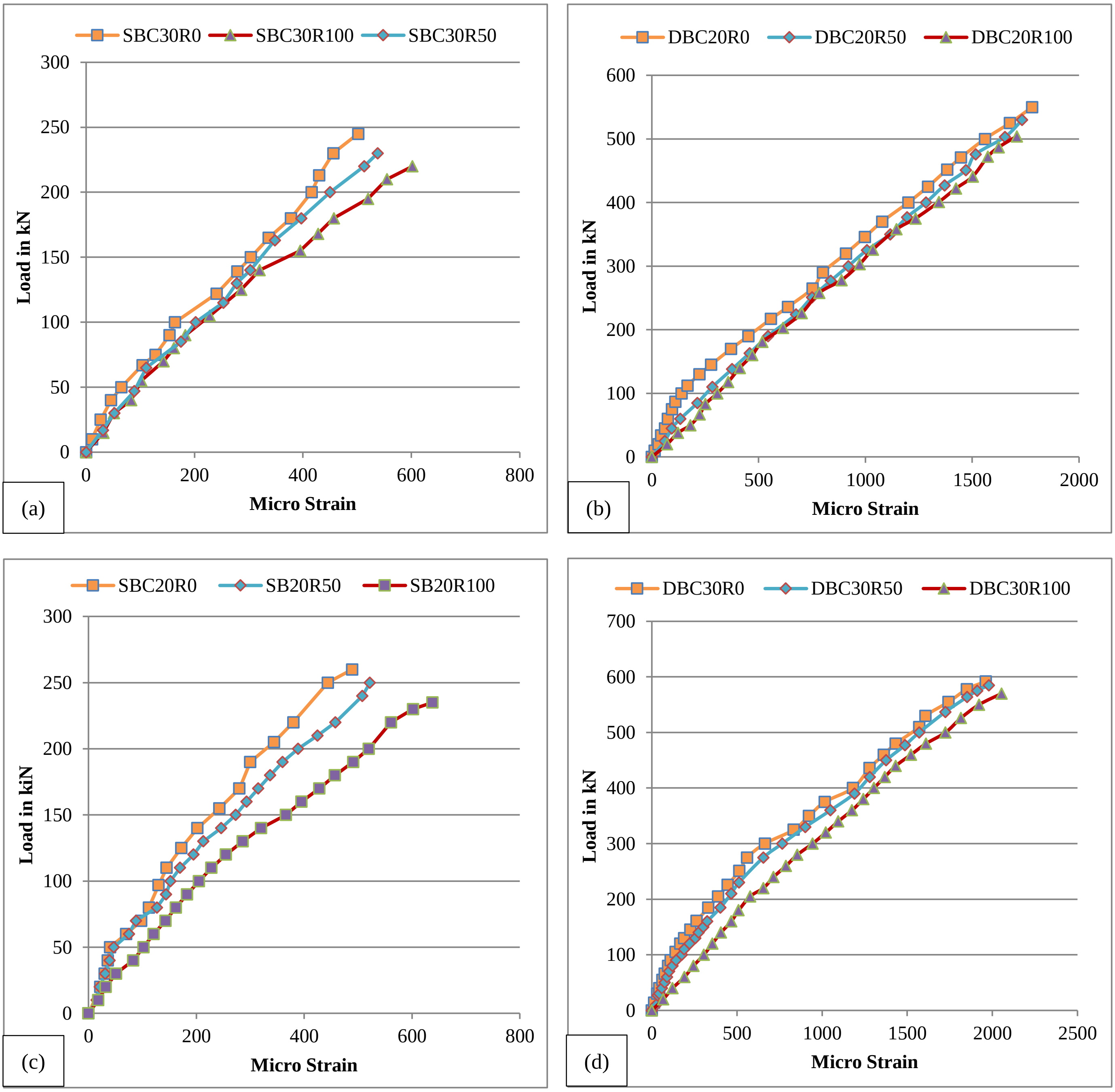

Figure 21. Surface concrete micro strain: (a) Group A. (b) Group B. (c) Group C. (d) Group D

Actually, it is so clear in Figure 21 that all the specimens behave approximately linear and the results of concrete surface micro strain illustrate a quite degree of uniformity. It is argued that this behavior can be ascribed also to the original variation in modulus of elasticity due to the degree of RCA replacement for the all the groups proposed in the present study. Moreover, the strain level in the doubly specimens exhibit higher levels than the singly due to its inherent design.

3.1.5. Tension Strain in Steel

Figure 22 shows the load versus tension reinforcement strain to the proposed beam specimens within the four groups while Figure 23 shows such variation in compression steel within groups B and D.

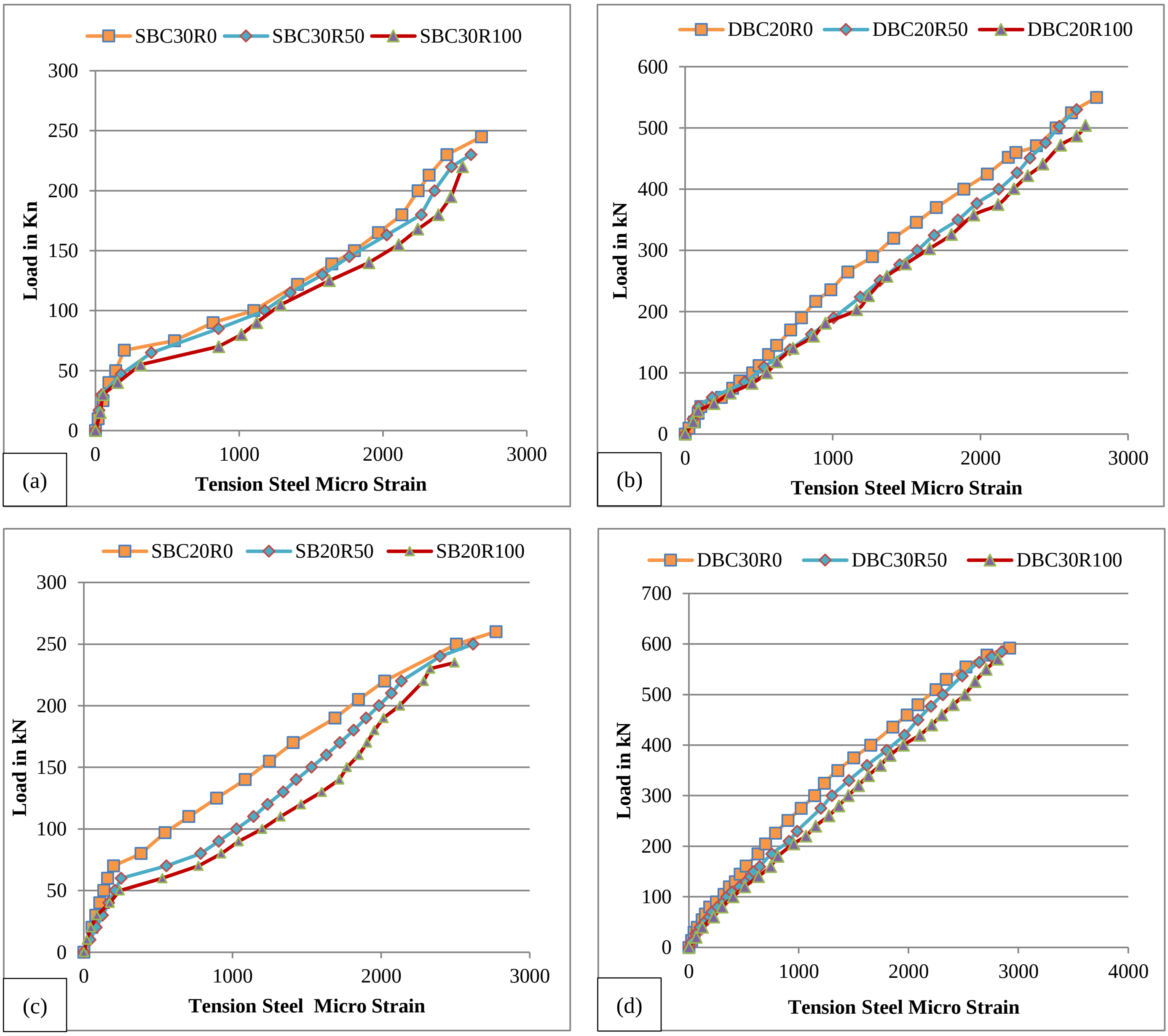

Figure 22. Tension steel micro Strain: (a) Group A. (b) Group B. (c) Group C. (d) Group D

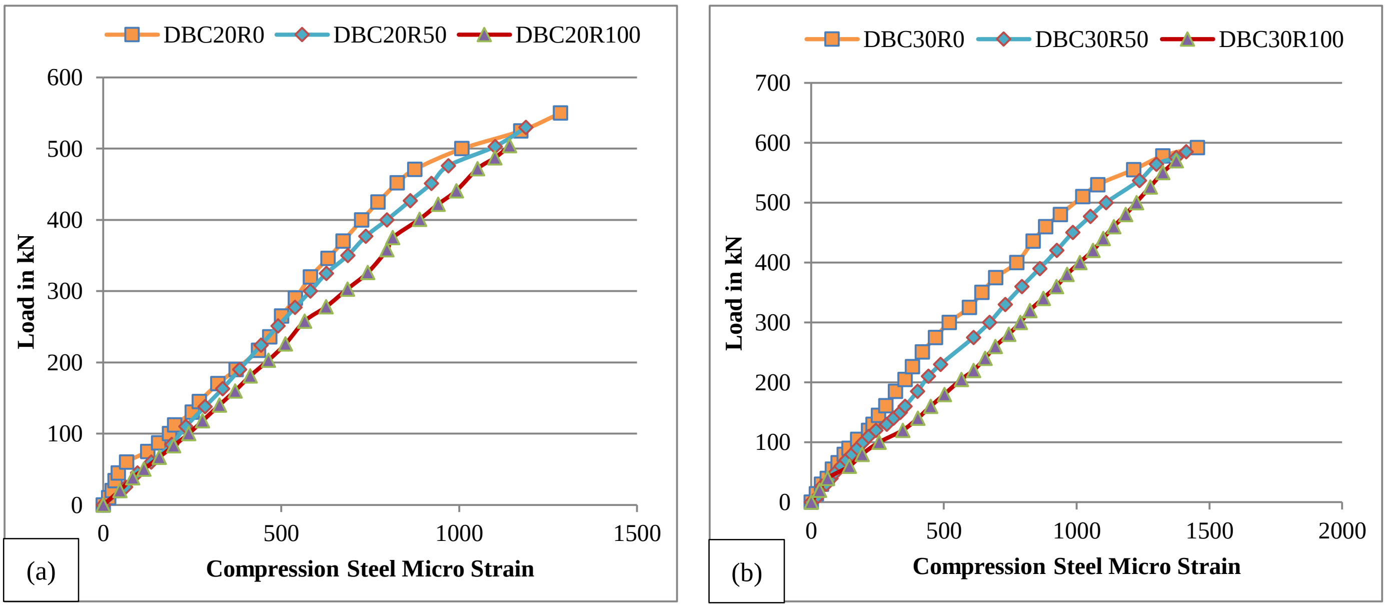

Figure 23. Compression steel micro Strain: (a) Group B. (b) Group D

In general, it can be observed that the load strain curves for tension steel are still can be a good indicator to the first crack occurrence for both singly and doubly beams, in addition, the degree of sharpness that accompanies the abrupt change of such occurrence seems higher in singly beams due to the inherent design circumstances. Although this curves were indicated that tension steel bars were almost reached its yield limit, the presence of the surrounding concrete and its confinement role makes this limits does not appear evidently.

Finally, it is observed also that the load limit at which the tension steel yielding occurs decreases as the degree RCA replacement progressed.

3.1.6. Bond Stress

Bond stress results of the beam specimens that included in this study are viewed in Figure 24.

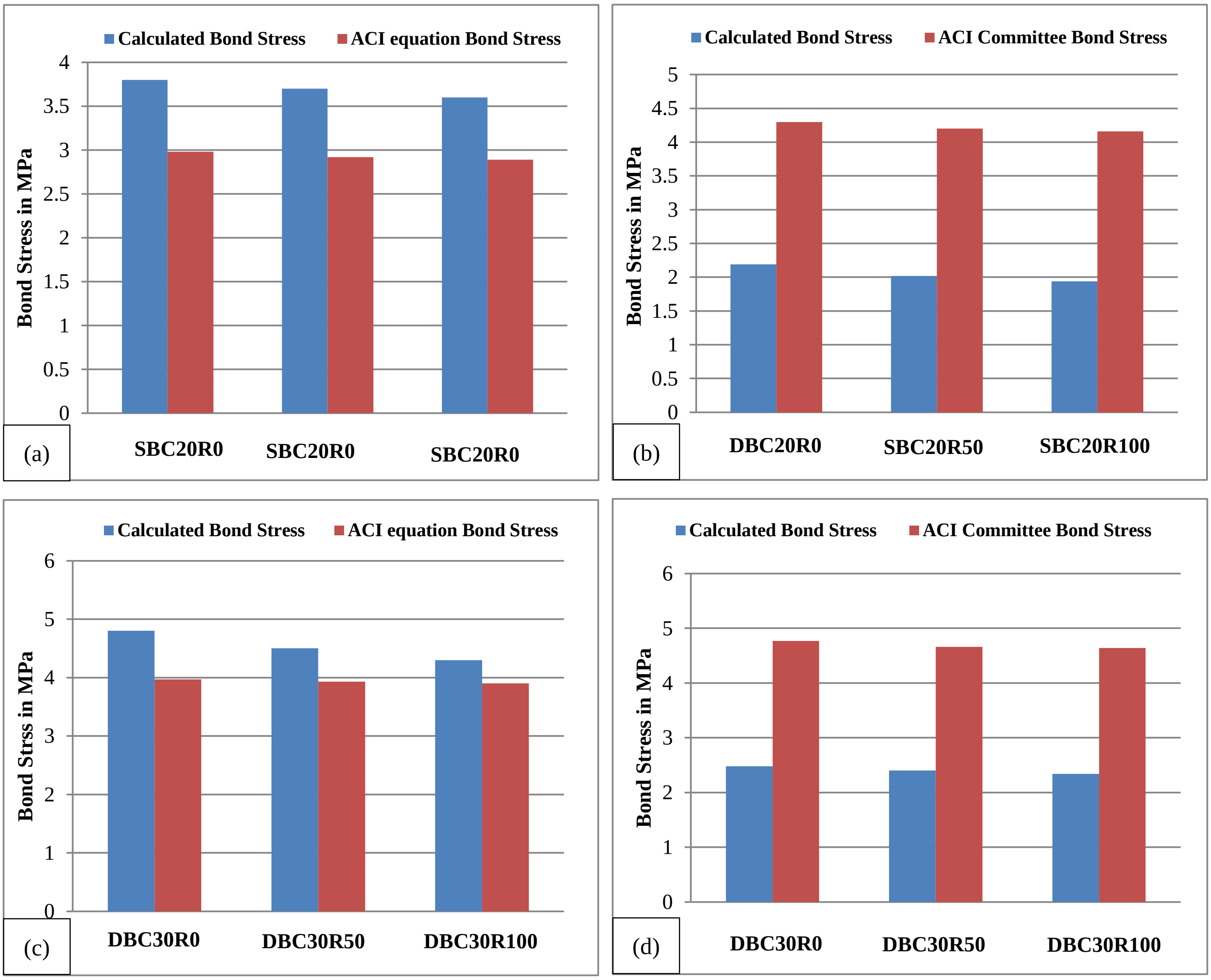

Figure 24. Bond stress: (a) Group A. (b) Group B. (c) Group C. (d) Group D

Actually, it's confirmed that increasing the degree of RCA replacement decreases the calculated bond stress for all specimens to a serious concern due to the existed decremented nature of strain of steel reinforcement at failure, bond stress levels in compression exhibits low limits if compared with tension due to load application mechanism dictated by the original design. Moreover, the calculated bond stress throughout the present study illustrate a noticeable degree of disparity to the ACI committee equation, more precisely, such degree of divergence is less in 30 MPa mix for both singly and doubly, however, it its believed that there is a need to further research to account if this approximation is existed with RCA concrete or not.

4. Conclusions

This study is aimed to investigate the behavior of lap splice in reinforced concrete beams, many related issues that indicate the general structural behavior of beams are also observed. All the conclusions are valid only to the materials and methods followed in this study, the conclusions are as follows:

- The progress of RCA degree of replacement decreases the bond strength in beam specimens.
- The beam splice specimens has a general trend to illustrate sudden failure due to the loss of ductility and the brittle nature of RCA concrete which is higher than the natural.
- The ACI committee approximation seems to have a need to be modified for being more sensitive to the degree of RCA replacement.
- Placing the lap splices is not recommended in the maximum moment area region in structural members, in addition, further results is needed to account the another locations within such members.
- Efforts are needed to use both continues bars in addition to lap splice at the same section to be studied and tested.
- Another series of research should be devoted to the lap splice behavior in RCA high strength and self - compacted concert built members.
- Scientific research programs should take into account the effect of the common concrete admixtures.
- FE programs should be used to simulate the behavior of beam specimens by inserting the desired stress strain behavior.
- Degree of relation between the preliminary concrete properties and the consequent bond taking high numbers of readings.

5. Conflicts of Interest

The authors declare no conflict of interest.

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