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Shear Strength Evaluation of Directly and Indirectly Loaded Rectangular Recycled Self-Compacted Reinforced Concrete Slender Beams Using Experimental and Finite Element Analysis

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Abstract

This study presents an experimental and numerical evaluation of the shear behavior of recycled aggregate concrete beams without transversal reinforcement. These beams were manufactured as self-compacted concrete with the use of both natural aggregate and recycled aggregate. The beams were subjected to direct and indirect loading conditions. The mechanical properties of four mixes were characterized in terms of compressive strength, splitting tensile strength, and elastic modulus. The experimental results showed that the shear capacity of recycled aggregate concrete is lower than those made with natural aggregate. The experimental shear capacities of the tested beams were compared with ACI318M-14 and relevant research studies in the literature. The ratio of experimental shear stress divided by the root square of concrete compressive strength $(vexp/\sqrt{fc`})$, which indicates the ability of diagonally cracked concrete to transmit tension and shear. was remained for all configurations greater than 0.17, which is the minimum value recommended by the ACI318-14. Results from

nonlinear finite element models were compared with the experimental data, and good agreement was achieved.

Keywords: *Rectangular Beam, Mechanical Properties, Finite Element Method (FEM), Shear Strength, Indirect Load, and Recycled Aggregate Self-Compacted Concrete (RASCC)*

1. Introduction

Despite extensive research to understand the mechanism that governs the shear failure of slender concrete beams, and after almost 70 years of research, it is still unsolved. although those studies have proposed a large number of models, most of those models were developed by fitting existing experimental data. Thus, they are either empirical or Semi-empirical. The lack of a clear adequate theory for shear is more evident in codes of practice worldwide. For example, the American Concrete Institute Code (ACI 318-14), employs more than forty empirical equations to assess the shear strength of concrete structures in general [1]. Each of which is fitted for a specific concrete member or a certain condition. The present work aims to characterize the effect of recycled aggregates on the shear capacity of reinforced concrete beams. The shear failure is one of the most critical failure modes for reinforced concrete structures especially for members without transversal reinforcement. The maximum shear strength depends on many parameters such as the concrete compressive strength (f_c`), the longitudinal reinforcement ratio (ρl), the shear span to depth ratio (a/d), the aggregates size (dg), and the transversal reinforcement ratio. The shear span-to-depth ratio is mainly depending. Research programs have been conducted in the literature on the shear capacity and the shear failure mechanisms of beams made with concrete incorporating mainly recycled coarse aggregates. Alhussein and Khudhair (2020)[2] studied, reinforced

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concrete rectangular deep beams cast with self-compacted concrete (SCC) which contains recycled concrete as coarse aggregate (RCA) were tested under directly and indirectly loading conditions. In the experimental work, fifteen deep beams were investigated, the first parameter considered in this study was the shear span to effective depth (a/d) ratio. The other variable is the replacement ratio by which the normal coarse aggregate is replaced by RCA. The beams were cast without the use of shear reinforcement. During the tests, the response of the beams including the cracking load, the ultimate load, concrete strain, and mid-span deflection were recorded. Test results indicate that the presence of RCA caused a reduction in the values of cracking and ultimate loads. For instance, the cracking load was reduced by 9%, 23%, and 50% and the ultimate load was reduced by 2%, 23%, and 25% as RCA replacement increased by 25%, 50%, and 75% respectively for a/d ratio equals 1.0. Further, by increasing the a/d ratio, the ultimate load was decreased due to the lower contribution of arch action shear transfer in the beam with a higher (a/d) ratio. Gonzalez et al. (2007) [3] tested eight beams made from two concrete compositions with a 3% longitudinal reinforcement ratio. The first mixture was based on natural aggregates, while the second was formulated with 50% recycled coarse aggregates. The beams were tested in four-point bending up to shear failure with $a/d = 3.3$. Their study's results showed no significant difference between recycled aggregate concrete *(RCA),* and natural aggregate concrete (*NAC*) beams. They observed notable splitting cracks and tension reinforcement, especially for RAC beams without transversal reinforcement. Gonzalez et al. (2009) [4] repeated the previous study by adding 8% silica fume to the mix designs. They observed that the addition of silica fume mitigated the splitting cracks along with the tension reinforcements. Etxeberria et al. (2007) [5] tested twelve beams made from four concrete formulations of the same class of compressive strength with different coarse recycled aggregate replacement ratios (0%, 25%, 50%, and 100%). Also, beams prepared with a longitudinal reinforcement ratio ranged from 2.92% to 2.95% and tested with $a/d = 3.0$. The results showed that the effect of recycled aggregates on the shear strength depends on the substitution ratio and especially for beams without shear reinforcement.

Chkheiwer A. H. (2013) [6] studied the influence of mixed proportions on the properties of fresh and hardened SCC. The effect of coarse aggregate content, limestone powder (LSP) to total powder ratio, and water/powder ratio on the SCC's fresh and hardened properties were considered. Different test methods were adopted to determine the properties of fresh SCC, such as slump flow, T500, V-funnel, L-box, and sieve segregation tests. The results showed that it is possible to produce SCC from locally available materials, which satisfy this type of concrete. Moreover, it can be clarified that the produced SCC is sensitive to the amount of coarse aggregate, amount of fillers, superplasticizer dosage, and water/powder ratio. The second part of the Chkheiwer study investigated the influence of the type of concrete (SCC and NC) and three values of compressive strength on the flexural and shear behavior of reinforced concrete beams and the punching shear of slabs. SCC slender beams failing in shear showed higher ultimate loads than the NC beam for different concrete compressive strength. While for deep beams failing in shear, both types of beams showed almost the same ultimate loads.

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2. Significance of Work-Study

Although slender beams are widely used in construction projects, the evaluation of ultimate strength still has many uncertainties about understanding the behavior and the failure mechanism of these elements. There are a limited number of research studies in the literature that have been conducted to investigate the response of beams made with self compacted RA concrete. Thus, this research tries to fill the gap and presented a thorough study that includes an experimental, code-based, and FE evaluation to investigate the responses of such elements. Furthermore, studying the effect of indirect loading conditions on the response of RASCC beams are less documented even in international standards like ACI. Therefore, this research presents a detailed study and offers an experimentally-quantified approach that can be utilized to implement indirect loading conditions on the response of beams. Another importance of this research which is characterized by the combination of using slender beams prepared using RCA as a partial replacement of NA and utilizing the SCC technique. This combination produces more sustainable structural elements and eliminates construction efforts.

3. Materials Properties

The experimental work carried out in the laboratory of the civil engineering department, University of Basrah. The mechanical properties of all types of concrete used in this work, control specimens were casting from the same mixes used for the SCC beams. Three mechanical properties were evaluated here: compressive strength (f_c\`), splitting tensile strength (ft), and modulus of elasticity (E_c). Three cubes and two cylinders were used for each mixture. The mechanical properties of concrete mixes are presented in Table1.

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Table 1: Properties of Hardened RASCC

Mix ID	*RCA (%)*	f'_c (MPa)	ft (MPa)	E_C (MPa)
1	0	33.8	3.96	27510
2	25	32.3	3.94	26950
3	50	29.5	3.43	25740
4	75	26.0	3.12	24110

4. Beams Details

The experimental program consists of casting and testing of six, directly and indirectly, loaded simply supported reinforced concrete beams. These beams were designed to fail in shear with a/d ratio (3.0). Four mixes were prepared with different RCA replacement ratios (0%, 25%, 50%, and 75%). Figures (1 – 4) show the details and nomenclature of tested beams.

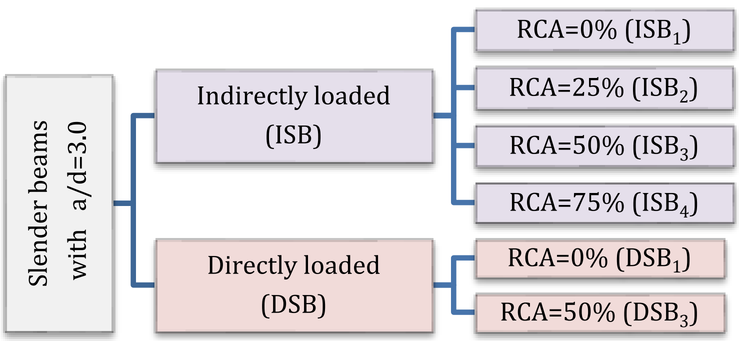

Figure 1: Nomenclature of Tested Beams

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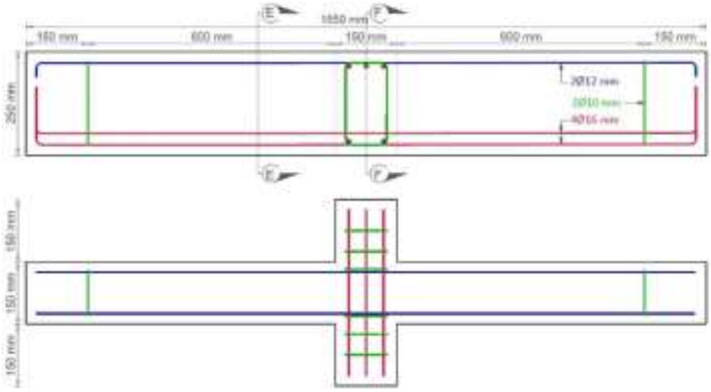

Figure 2: Indirectly Loaded Slender Beams

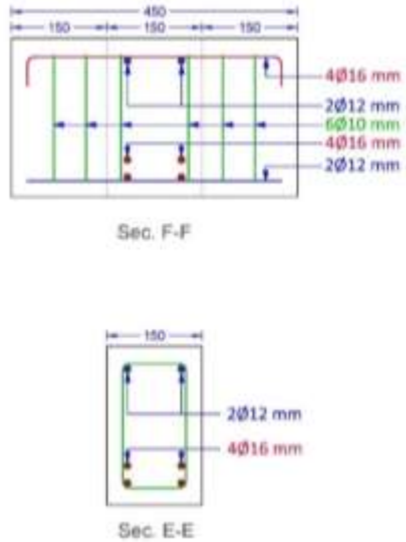

Figure 4: Cross-Section of Tested Beams

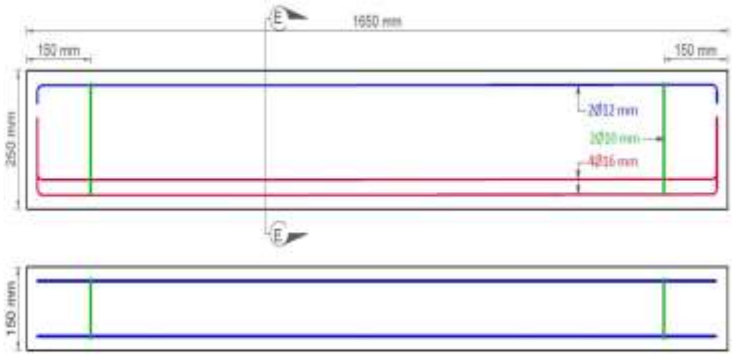

Figure 3: Directly Loaded Slender Beams

5. ACI 318-14 Shear Strength Evaluation

During the 1950s, the American Concrete institute launched a significant effort to develop the design provisions that existed at that time. This effort led to introduce to the 1962 shear design proposal, which still up to date, forms the base for the current ACI 318-14 [7]. To calculate the shear strength of any concrete member without web reinforcement, ACI 318-14 [7] recommends using a general formula as follows:

$$Vc = 0.17\lambda\sqrt{fc`}\, b_w\, d \qquad (1)$$

Where b_w; d represents the width and depth of the section respectively, λ is a factor to account for the density of concrete, while $\sqrt{fc`}$ is the compressive strength of concrete. As shown, $Eq.(1)$ accounts for neither the size effect nor the influence of longitudinal reinforcement. However, ACI 318-14 [7] gives a more accurate method to calculate the shear strength. The method consists of three different formulas in which the shear strength of the member shall be the lesser of them.

$$Vc = \left(0.16\,\lambda\sqrt{fc`} + 17\rho_w \frac{V_u\,d}{M_u}\right) b_w\,d \tag{2}$$

$$Vc = \left(0.16\,\lambda\sqrt{fc`} + 17\rho_w\right) b_w\,d \tag{3}$$

$$Vc = 0.17\lambda\sqrt{fc`}\,b_w\,d \tag{4}$$

Where Vu, Mu represent the ultimate shear and moment at the section considered respectively, ρ_w is the ratio of longitudinal reinforcement. Also, since $Mu = V_u a$, it is possible to replace the term $(V_u d/M_u)$ in Eq. (2) by the term (a/d), where a is the shear span of the member.

6. Experimental results

The results of hardened concrete tests are presented in Table 1. The test results showed that the compressive strength, modulus of elasticity, and splitting tensile strength were decreased by the increase of the RCA replacement ratio. For instance, increasing the RCA replacement ratio to 25%, 50%, and 75%; the compressive strength decreased by (4%, 13%, and 23%). The splitting tensile strength decreased by (1%, 13 %, and 21%) respectively. Also, it can be observed that the modulus of elasticity was reduced with the increased RA ratio by (2%, 6%, and 12%) respectively . This reduction is due to the composition of recycled aggregate, which

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generally consists of considerable amounts of porous old mortars that form zones of weakness in the concrete composite.

When the comparison is made between the surface of the fraction of both natural aggregate (NA) and recycled aggregate (RA), it can be noted that the failure in the NA took place along with the interface between aggregate particles and cement mortar. In contrast, in RCA, the plane of failure passes around or through aggregate. This failure leads to a somewhat sudden collapse of concrete, which means that the self-compacted concrete with RA was more brittle than the mixture with NA . Shear failure modes of beams are given in Figure 5, which reveals that; First, In terms of crack formation and propagation, the behavior of natural concrete beams and recycled concrete beams is practically identical. Second, failure is occurred by the formation of diagonal tension failure mode. Finally, The number of flexural cracks increases slightly when recycled aggregates are used.

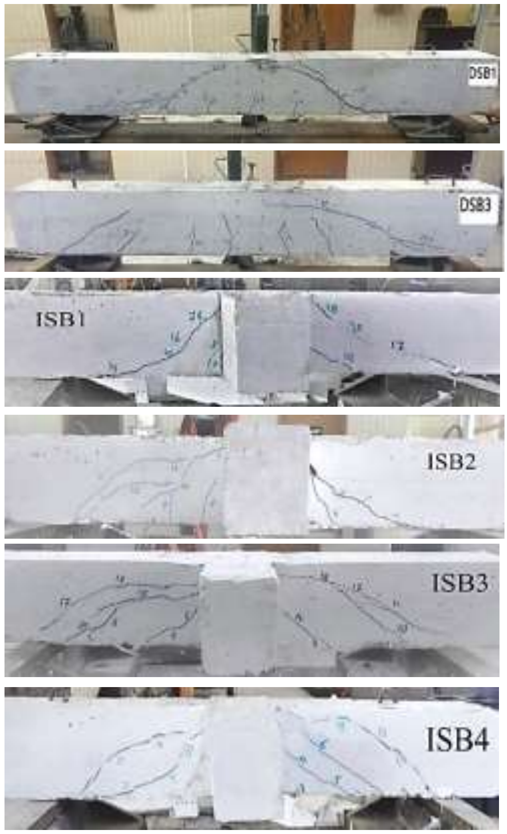

Figure 5: Crack Patterns of the Tested Beams

It was found that the surface failure of NCA beams occurs at the interface between the aggregates and the mortar, whereas it happens by the rupture of recycled aggregates for RCA. It was noticed that RCA's fracture surface is smoother than the fracture surface of NCA, which means that the contribution of bridging and the branching phenomenon is reduced in the stress transfer between the two lips of the crack. [8]. The microscopic observations showed that the crack of RCA is broader than NCA beams.

7. Load–Deflection Curves

Load–deflection curves for all tested beams are presented in Table 2, Figures 6, and 7. The behavior is linear before the beginning of the first flexural crack, visibly with a smaller slope for the RAC beams. After the early cracking, the RAC beams show flatter slopes compared with the NAC. Many researchers reported larger deflection in RAC beams than NAC [4, 9, and 10]. The difference was attributed to a lower flexural stiffness of RAC due to the reduced elastic modulus and tensile strength. The reduction in the tensile strength can be understood by the decrease in the resistance when recycled aggregates are used, while the reduction in the elastic modulus can be attributed to several effects such as the lower elastic modulus of recycled aggregates, porosity, connectivity between the pores, and finally, the quality of the used aggregates [8 and 11]. The service load defines in Table 2, was taken as 65% of ultimate loads as a recommendation by [10 and 12].

Table 2: Deflection of Investigated Beams

Beams	Service load (kN)	Deflection at Service Load (mm)	The measured deflection at 100 kN Load (mm)
Indirectly Slender Beams			
ISB1	169	9.66	3
ISB2	124	5.88	4
ISB3	117	6.36	5
ISB4	72	3.72	8
Directly Slender Beams			
DSB1	195	11.5	4

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DSB3	130	7.4	5

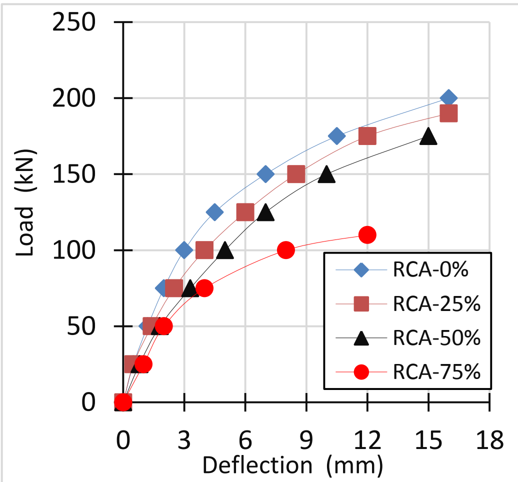

Figure 6: Comparison of Load-Deflection Curves with the Effect of RCA

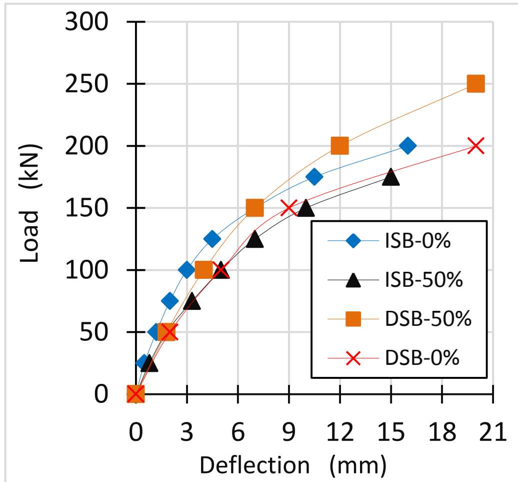

Figure 7: Comparison of Load Versus Mid-Span Deflection Curve for DSB and ISB Beams

8. Analysis of the Ultimate Strength

From the experimental data, the cracking load P_{cr}, and ultimate load P_u, of all tests are summarized in Figures 8, 9, and Table 3. As shown in Table 3, the ratios of inclined cracking to ultimate load for indirectly loaded beam (ISB) is ranged from 33% to 53% with an average value of 45% and standard deviation of 8%, while for directly loaded are 43% and 55% for 0% and 50% RCA, respectively.

The diagonal cracking loads and the ultimate loads decreased with an increase in the RA ratio increase, as appeared in Figures 8 and 9. The beams with indirect load; ISB2, ISB3, and ISB4 where the replacement ratio is 25%, 50%, and 75% respectively; the cracking load was decreased by 20%, 52%, and 60% respectively and the ultimate load was reduced by 27%, 31%, and 58% respectively. While the directly loaded beam with 50% RCA

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experimental a reduction by 15% in cracking load and 33% for the ultimate load. The decrease in ultimate loads of the beams is attributed to the reduction in compressive strength by increasing the RCA.

The shear stress was calculated using the following equation; $vexp = V_u/(bd)$. The experimental shear stress, $vexp$, has been divided by the root of the compressive strength, $fc`$, as $(vexp/\sqrt{fc`})$ which indicates the ability of diagonally cracked concrete to transmit tension and shear. The ratio remains for all configurations greater than 0.17, the minimum value recommended by the American Standard ACI-318-2014 [7].

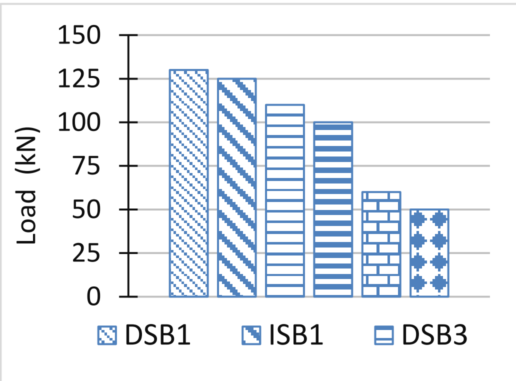

Figure 8: Comparison of Inclined Cracking Loads for DSB & ISB with RCA%

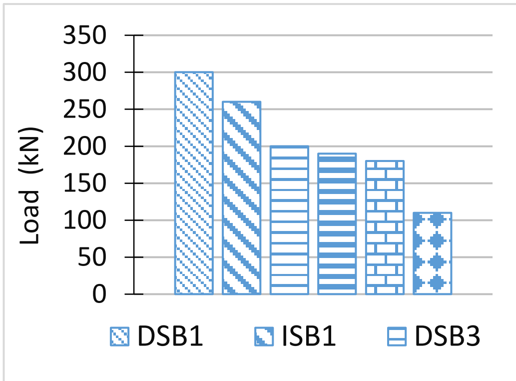

Figure 9: Comparison of Ultimate Loads for DSB &ISB with RCA%

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Table 3: Inclined Cracking and Ultimate load For Directly and Indirectly Loaded Beams

Beams	a/d	RCA (%)	Load (kN)		$\frac{Pcr}{Pu}$ (%)	The reduction due to the increase of RCA %		$Vexp = \frac{Pu}{2}$ (kN)	$vexp = \frac{Vexp}{bd}$ (MPa)	$\frac{vexp}{\sqrt{fc'}}$
			Pcr	Pu		Pcr %	Pu %			
Indirectly Loaded										
ISB1	3.0	0	125	260	48	-	-	130	3.85	0.66
ISB2		25	100	190	53	20	27	95	2.81	0.49
ISB3		50	60	180	33	52	31	90	2.67	0.49
ISB4		75	50	110	45	60	58	55	1.63	0.32
Directly Loaded										
DSB1	3.0	0	130	300	43	-	-	150	4.44	0.76
DSB3		50	110	200	55	15	33	100	2.96	0.54

9. Finite Element Modeling

9.1 ANSYS Program

Expermintal result were compared with the results given by ANSYS academic V15,[13] was included. Concrete is modeled using an eight-node element SOLID65, which has 3 degrees of freedom at each node being translated in x, y, and z directions. In addition to cracking and crushing, concrete may also undergo plasticity with Drucker–Prager failure surface. Cracking is treated as a smeared band rather than discrete discontinuities and modeled by an adjustment of material properties. The concrete material is assumed to be initially isotropic, but for the overall behavior up to failure, linear and nonlinear material properties, as well as the failure surface, are required. Additional concrete material data, such as shear transfer coefficients, uniaxial tensile cracking stress, uniaxial crushing stress, biaxial crushing stress, and stiffness multiplier for cracked tensile condition, are input in the data table to calibrate the failure surface. Element LINK180 was used for steel reinforcement modeling. It is a three-dimensional (3D) spar element with 3 degrees of freedom at each node being translated in x, y, and z directions. The element cannot carry bending loads, while creep, plasticity, rotation, large deflection, and large strain capabilities are included. For rebar reinforcement, an elastic, perfect plastic material model was adopted. Poisson's ratio was set to 0.3, whereas elastic modulus and yield stress were set equal to experimental values. The full bond used to model the bond between steel rebar and concrete [8]. For the supports and the loading plates, the SOLID185 element was used. It is defined by eight nodes having 3 degrees of freedom at each node being translated in the nodal x, y, and z. It is capable of plasticity, hyper elasticity, stress stiffening, creep, large deflection, and large strain capabilities. Full and one half of the beam was modeled, taking into account the symmetry of both geometry and loading. The total load was applied through a series of load steps, and the analysis type was set to small displacement static, where large deformation effects are ignored. The sparse direct solver based on the immediate elimination of equations was used to solve the model. The iterative process of the Newton–Raphson method was used to solve the nonlinear equations of the model and the line search tool. In this study, trial meshes were selected to find the more suitable mesh size for the control beam specimens with RCA 0%. An appropriate mesh density of 25 mm was selected when any increase in the mesh density became ineffective on the

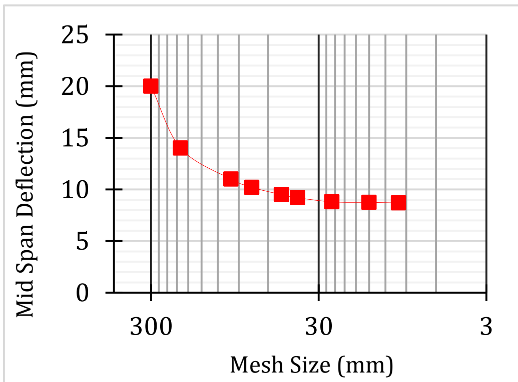

Figure 10: Deflection at Midspan VS Mish Size by ANSYS

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accuracy of the results. Figure 10 shows the mid-span deflection with the trial meshing of FE by ANSYS.

9.2 ABAQUS Program

Detailed 3D finite element models of RC beams were implemented in this study. The FE models were created using FE commercial software package ABAQUS [14] to predict ultimate shear capacity and comparing the results with experimental data. FE models were refined and calibrated using experimental data to extend them to predict other parameters that were not measured during the experimental program. The models were subjected to direct and indirect loading to evaluate their shear capacity. The geometry of the FE models was taken from experimental tests. All lab-measured dimensions, reinforcement details were used in FE models as shown in Figures 2,3, and 4. The mechanical properties of concrete adopted in FE models were taken according to the experimental program. Table 1 presents the properties of the concrete used in this study with Poisson's ratio of 0.2 for concrete under uniaxial compression. Concrete Damage Plasticity model (CDM) was used to model concrete behavior. While the uniaxial stress-strain relation for steel bars was idealized as a bilinear curve, representing elastic-plastic behavior with strain hardening, as shown in Figure 11.

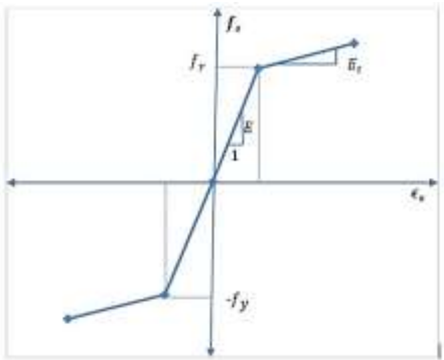

Figure 11: Stress-Strain relation of Steel reinforcement

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In the numerical simulation, a three-dimensional eight-node linear brick and reduced integration with hourglass control solid element (C3D8R) are employed to represent the concrete specimen. While a three-dimensional, truss elements with 2-node first order (T3D2– Truss) are used to model the steel reinforcements. Many research studies in the literature have recommended using these elements to model the tested beams [10, and 15]. A typical configuration of FE models for directly and indirectly loaded beams is presented in Figure 12. The beams were directly and indirectly loaded as in the experimental tests which are shown in Figure 13.

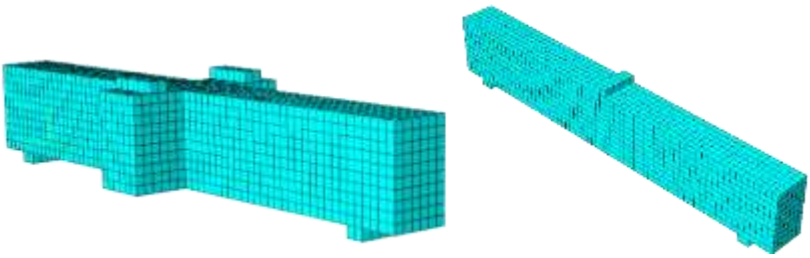


Figure 12: The FE Models Configuration

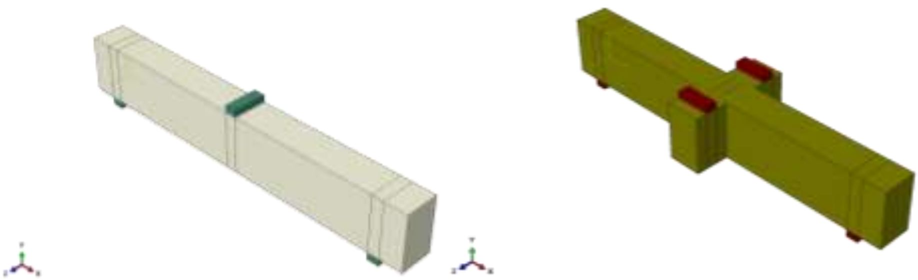
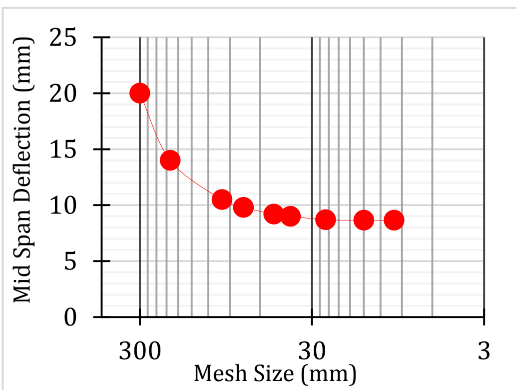


Figure 13: Testing Procedure of the Directly and Indirectly Beams

Mesh size is an important driver of model accuracy as well as solution time as the computational costs increase as the mesh is refined. Mesh



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size was the key parameter for calibration of the FE models to determine the mesh density that gives convergent results with reasonable computation time. Mesh refinement was addressed by running the model with a different mesh size and studying its effect on the midspan deflection. Mesh size versus midspan deflection is presented in Figure 14. The optimum mesh size of 25mm gives good FE results with reasonable computational time.

10. Comparison with Experimental Results

Validation of the proposed model, a comparison with experimental results and finite element model was performed. Load versus mid-span deflection curves from numerical analysis and experiments for RASCC beams are shown in Figures 15 and 16. The comparison of curves indicates that both experimental and numerical results are in good agreement .

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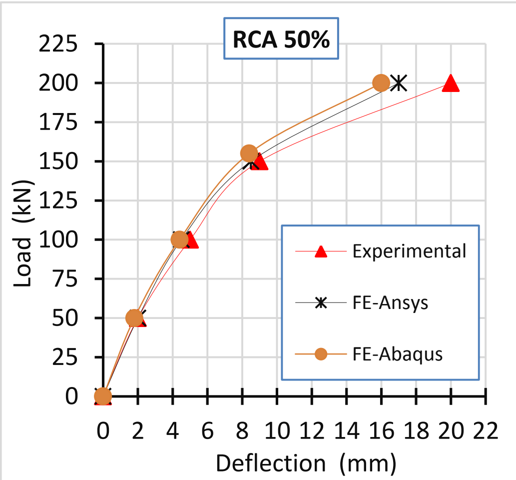

Figure 15: Load Midspan Deflection Curves for Indirectly Loaded Beams

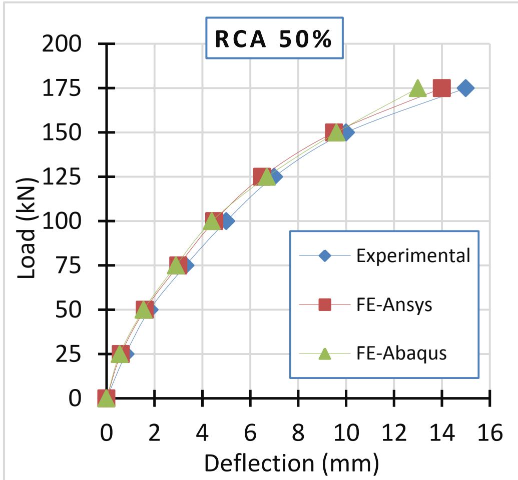

Figure 16: Load Midspan Deflection Curves for Directly Loaded Beams

The curves of FE load-deflection differ slightly from the experimental curves; this because of the, existince of the micro-cracks in the real concrete specimens formed through drying shrinkage in the concrete and beam handling. On the other hand, the FE models do not deal with micro cracks. Second, the bonding of concrete and steel as perfect in the FE model. However, this assumption is not valid for the experimental tests. Furthermore, the appearance of the cracks reflects the failure modes for the specimens.

The comparison between the experimental shear strength and the predicted resistances for all tested beams are presented in Table 4 and Figure 17. It can be seen that the predictions yielded by zsutty equations and ACI ordinary method are more conservative than FE models. Likewise, FE results were closed with experimental findings .

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The typical deflected shapes estimated by the programs to the mid-span deflection at the centerline of cross-section for the direct and indirect loading were depicted in Figures 18 and 19.

Table 4: The ultimate strength of experimental, Zsutty, ACI-14, and FE results

Beam SN	Exp (kN)	Zsutty (kN)	A1 (%)	ACI (kN)	A2 (%)	ANSYS (kN)	A3 (%)	ABAQUS (kN)	A4 (%)
Indirectly Loaded RCASCC without Stirrup									
ISB1	260	47.2	-82	33.35	-87	265	2	270	4
ISB2	190	46.6	-75	32.63	-83	210	11	200	5
ISB3	180	45.2	-75	31.20	-83	190	6	200	11
ISB4	110	43.3	-61	29.23	-73	120	9	125	14
Directly Loaded without Stirrup and RCA%									
DSB1	300	39.40	-87	33.35	-89	305	2	310	3
DSB3	200	37.7	-81	31.20	-84	211	5	220	10

A2 = $\frac{ACI_{pu}-Exp}{Exp}$ A3 = $\frac{ANSYS_{pu}-Exp}{Exp}$ A1 = $\frac{Zsutty_{pu}-Exp}{Exp}$ A4 = $\frac{ABAQUS_{pu}-Exp}{Exp}$

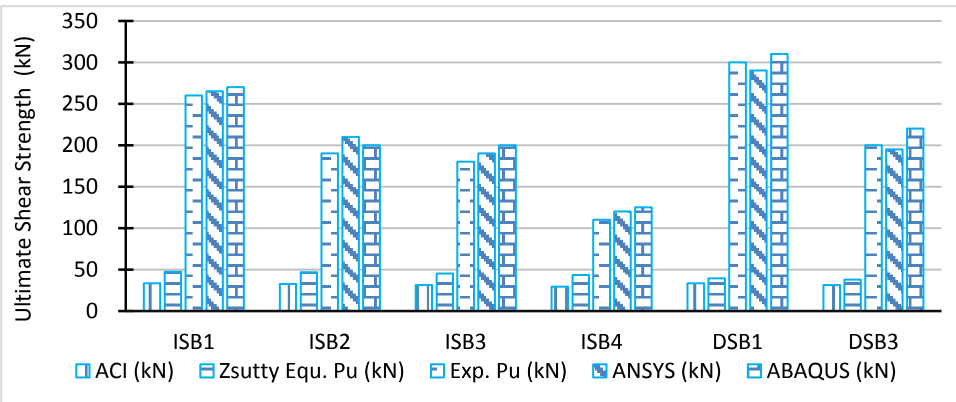

Figure 17: The comparison of the ultimate capacity of experimental, Zsutty, ACI, and FE results

The crack patterns predicted by the finite element analysis and the failure modes of experimental specimens agree well. The programs record a crack pattern at each applied load step. In general, flexural cracks occur early at mid-span. Vertical flexural cracks spread horizontally from the mid-span to the support. When the applied loads are increasing. At a higher applied load, diagonal cracks appear. Increasing applied loads induces additional diagonal and flexural cracks. Finally, compressive cracks appear at nearly the last applied load steps.

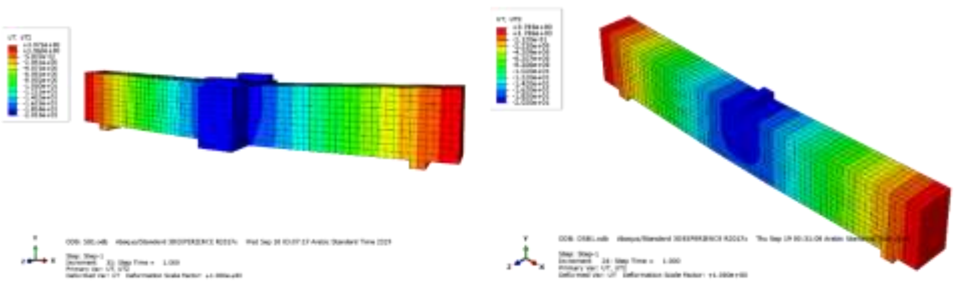

Figure 18: Deflected Shape Of Directly And Indirectly Loaded Beams By ABAQUS Program

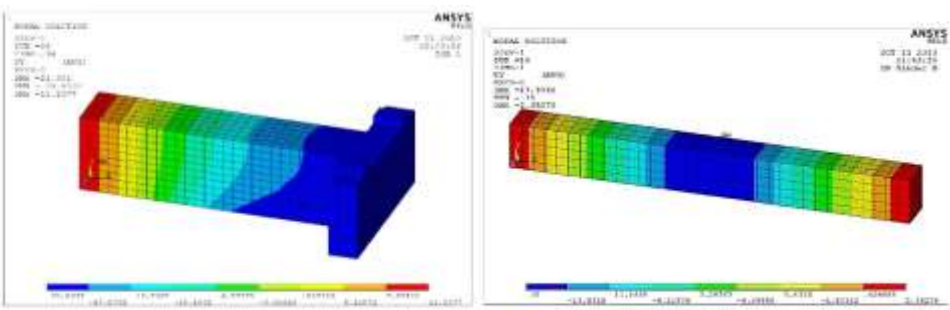

Figure 19: Deflected Shape Of Directly And Indirectly Loaded Beams By ANSYS Program

11. Summary and Conclusions

In this research, the shear capacity of directly and indirectly loaded slender beams was investigated. These beams were made with different percentages of recycled aggregate as a replacement of normal aggregate. Two directly loaded slender beams with 0% and 50% replacement ratio were tested and compared with four indirectly loaded slender beams made with 0%, 25%, 50%, and 75% aggregate replacement ratio. All beams were cast without shear reinforcement. Based on the results of the study, the following conclusions can be drawn :

All tested beams showed identical behavior in terms of crack propagation regardless of the RCA replacement ratio. Beams with recycled aggregates developed less shear capacity compared with beams constructed with natural aggregates. The decrease in ultimate loads of the beams is attributed to the reduction in compressive strength by increasing the RCA. For beams under indirectly loaded, $IRSB_2$, $IRSB_3$, and $IRSB_4$, where the replacement ratio is 25%, 50%, and 75% respectively; the ultimate load was decreased by 27%, 31%, and 58% respectively. While the directly loaded beam with the replacement ratio is 50%, the ultimate load was decreased by 33%.

The ratios of inclined cracking loads to ultimate loads (P_{cr}/P_u) for indirectly loaded beams were range from 33% to 53%. The (P_{cr}/P_u) for directly loaded beams were 43%, and 55% with replacement ratio of 0% and 50%, respectively.

The shear stress was calculated using the experimental data then divided by the root square of the compressive strength, $fc`$, as ($vexp/\sqrt{fc`}$), which indicates the ability of diagonally cracked concrete to transmit tension and shear. The ratio remains for all configurations higher than 0.17 which is the value recommended by the ACI318-14 .

In general, the response of the beams extracted from the finite element models showed good agreement with the corresponding experimental results. However, the finite element models show a slightly stiffer response in both linear and nonlinear portions which could be due to the variation in the material composition which is normally assumed to be homogenous in the

FE models. For example, the ultimate loaded was higher than the experimental result by 7%.

Mid-span deflection from experimental data was lower than those predicted by FE models with an average of 6%. The ultimate capacity of beams predicted by Zsutty and ACI318-14 for ordinary beams is lower than the experimental results for both directly and indirectly loaded beams.

In general, the comparison between using different softwares is that; ABAQUS was easier in meshing the models. For same mesh size, ABAQUS was faster than ANSYS in running and yielding the results. However, ANSYS provides an easy to visualize crack patron compared with ABAQUS.

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حساب مقاومة القص عملياً وبإستخدام تقنية العناصر المحددة للأعتاب الخرسانية النحيفة المستطيلة والمصبوبة بإستخدام خرسانة ذاتيَّة الرص وركام معاد تدويره والتي يتم تحميلها بشكل مباشر وغير مباشر

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إن الهدف الرئيس لهذه الدراسة هو اجراء دراسة مختبرية وتحليلية تخص السلوك الانشائي للأعتاب الخرسانية المسلحة النحيفة بمقطع عرضي على شكل مستطيل، والمعرضة لأحمال بشكل مباشر وغير مباشر، باستعمال خرسانة ذاتية الرص(SCC) ، التي تحتوي على قطع صغيرة من الخرسانة المتصلبة المعاد تدويرها كركام خشن (RCA) . تم دراسة الخواص الميكانيكية لأربع خلطات من حيث مقاومة الانضغاط ، مقاومة الشد (الانشطار) ومعامل المرونة. أظهرت النتائج التجريبية (العملي) أن مقاومة القص للخرسانة المعاد تدويرها أقل من تلك المصنوعة من الركام الطبيعي. تمت مقارنة مقاومات القص التجريبية للأعتاب المفحوصة مختبريا مع ACI318M-14 والدراسات البحثية السابقة، تم حساب إجهاد القص باستعمال النتائج المختبرية ثم قسّم على الجذر التربيعي لمقاومة الانضغاط ، $fc`$ ، مثل $(vexp/\sqrt{fc`})$ مما يشير إلى قدرة الخرسانة المتشققة قطريًا على نقل اجهادات الشد والقص. بحيث تبقى النسبة لجميع العتبات أعلى من 0.17 ، وهي القيمة الدنيا الموصى بها من قبل ACI318-14. تمت مقارنة النتائج من نماذج العناصر المحددة غير الخطية مع النتائج التجريبية العملية، وتم التوصل إلى توافق جيد.