Original Research Article

Flexural Behavior of RC Beams by Using Agricultural Waste as a Cement Reinforcement Materials

ABSTRACT

Using replacement for cement to assure sustainability is critical as the raw materials (limestone, sand, shale, clayand iron ore) used in making cement are depleting. The use of agriculture waste byproducts to replace cement is becoming an economic option. Rice husk ash (RHA), rice straw ash (RSA) and wheat straw ash (WSA) which have pozzolanic properties areviable alternatives. A study on RHA, RSA and WSA were conducted to determine their suitability. From the various grades of concrete studiedshow that up to 15% replacement of OPC with RHA, RSA and WSA have the potential to be used as partial cement replacement (PCR), having good compressive strength, performance and durability.In particular, the utilization of as PCR material as PCR can contribute to sustainable construction.

Keywords: Concrete beams, Flexural Behavior, Cement reinforcements, Agricultural wastes

1. INTRODUCTION

Now, the use of rice husk ash (RHA) as a cement replacement material exists as a different development in concrete technology. So, it would also support to solve difficulties else come across in placing of the wastes as showed Chandra. Removal of husks was a large problem and open load heating is not suitable on environmental surroundings, and so the common of husk was presently gone into landfill.

Concrete, was generally applied material of construction in the world as showed by Glavind. While it was being used in construction industries all around the world nevertheless it has some inherited deficiency in terms of strength and durability; such deficiencies pay the way for its limitation in the field of repair and re-habitation of structures while Kurose discussed. Research is being carried out all around the world to improve such deficiency of the concrete as repair material. Performance of cement mortar and concretes can be proved with polymeric compounds Abdullah discussed this. Polymeric compounds which can improve limitations of cement mortar/concrete, mainly; Ohama and Fowler studied polymer latex/dispersions, Re-dispersible polymer powder, Water-soluble polymers and Liquid polymers.

Polymer modified concrete is a latest high performance material to a relative extent, which has extensive applications due to its advantages that make it to be more successful than cement concrete as Czarnecki and Broniewski studied. Most notably advantages put forth by the polymer concrete are: Splendid mechanical strength, rapid curing, having the property of making ingredients to stick together, able to resist the abrasion and weathering, water tightness and giving good results against thermal properties as Abdel-Fattah & El-Hawary. There are various areas to utilize polymer concrete/polymer modified concrete i.e. manufacturing of precast concrete beams etc; in water retaining structures; dams, dikes, reservoirs and piers; highway surfaces and bridge decks; additionally, to the petrochemical industry, underground constructions, roads surfaces and coating or repairing materials in the chemical and food industries as by Fowler Abdel-Fattah, El-Hawary Barbuta and Hing. Due to the special characteristics of PC/PMC that is also being used for repair and rehabilitation overlaying/underlying for bridge surfaces, stadium floors, laboratories, hospitals, factories and other structural system.

The main aim of this study was to present an investigation on the behavior of concrete beams produced from blending cement with RSA and WSA. The physical and chemical properties of RSA and WSA were first investigated and compared to the ordinary Portland cement (OPC). Mixture proportioning was performed to produce workability concrete with target strength of 30 MPa, 45MPa and 60 MPa for the three mixtures. A total mixture was casted to study the effect of OCM ordinary concrete mix, RSA and WSA on the properties of concrete beams and its compressive strength and the failure behavior.

2. EXPERIMENTAL STUDY

2.1. Concrete mix materials

2.1.1. Cement

Ordinary Portland Cement (OPC) was used for this study, the cement used in this study was the Egyptian Ordinary Portland Cement (OPC) CEM 42.50 N, which was manufactured locally and complies with the Egyptian specifications. The chemical analysis as well as the physical characteristics of the used cement as determined by given data showed its suitability for concrete works. The cement content was 350 kg, 500 kg and 600 kg per cubic meter of the mixtures. Table 1, shows the chemical characteristics of the OPC used.

TABLE 1 Chemical and physical properties of OPC

Characteristics		Measured values
Fineness cm^2/gm		4100
Soundness (expansion mm)		1.0
Initial setting time (minutes)		50
Final setting time (hours)		10
Crushing Strength	7 days	22
(MP_a)	28 days	25

2.1.2 Aggregates

The coarse aggregate which has been used for concrete mixes was natural gravel of 10 mm (passing 14mm sieve, retained on 10mm sieve, single sized) while Natural siliceous sand with a round particle shape and smooth texture with fineness modulus of (2.75). The grading curve lies between the upper and the lower limits of BS1377, BS812 requirements.

2.1.3 Mixing water and Super plasticizer

Clean drinking water free from impurities was used for mixing the concrete and in curing the specimens. The water cement ratio used is 0.40 which given in the concrete mix proportions. Super plasticizers the admixtures were used in order to sustain the workability of the concrete mix, to avoid any voids in the concrete specimens, and to keep its slump within the standard limits to achieve the intended characteristic strength of concrete. About (0.1% by weight of cement), of super-plasticizer was used. Super plasticizer's commercial name is SIKAMINT 163M as shown in Fig. 1.

FIG. 1 Super plasticizer admixtures.

2.1.4 Cement replacements

Wheat straw ash and rice straw ash are the main organically replacement used in concrete mix in this study. Wheat and rice straw sticks have golden yellow color, light weight and brittle structure. It is re-grinded to become a powder with a large surface area and higher smoothness in special grinding machine as in Fig. 2.

FIG. 2 Wheat and rice straw ash before and after grinding

2.2 Test specimens

In this investigation there are nine concrete beams divided into three groups. Each group has three group one of them is the control and the two others compared with it. All group compared together to explain the failure behavior. Each group has two types of cement replacement of wheat and rice straw ash with percentage of 15% which gives the greatest concrete strength. Three specimens were poured without adding any straw ash to cement as control beams with concrete strength of 30 MPa and longitudinal reinforcement two 16 mm diameter and two 10 mm diameter respectively and stirrups of 6 mm in meter. Another two beams were cast totally with straw ash volume ratio of 15% one of them with wheat straw ash WSA and the other with RSA rice straw ash with the same reinforcement. Two other groups of beams were cast totally three specimens the first was without replacement, the second was with WSA and the third with RSA with the same reinforcement but with concrete strength of 450 MPa. The third group like to the previous groups but with concrete strength of 60 MPa. The yield stress was measured to be 360 MPa. Two bars of 10 mm diameter were used as top reinforcement and 6 mm diameter stirrups @ 166 mm spacing were used as shear reinforcement. All beams cross-section was 200 mm height and 100 mm width but 1500mm span as shown in Fig. 3 and Table 2.

TABLE 2 Details of Tested beams

Group	Symbol	Ash content Vf %		Bottom RFT.	Top RFT.	Transverse RF. Bar size
Group 1 Fcu 30	B1-1 B2-1 B3-1	0 15 15		2Ø16 2Ø16 2Ø16	2Ø10 2Ø10 2Ø10	6Ø6/m' 6Ø6/m' 6Ø6/m'
Group 2 Fcu 45	B2-1 B2-2 B3-2	0 15 15		2Ø16 2Ø16 2Ø16	2Ø10 2Ø10 2Ø10	6Ø6/m' 6Ø6/m' 6Ø6/m'
Group 3 Fcu 60	B1-3 B2-3 B3-3	0 15 15		2Ø16 2Ø16 2Ø16	2Ø10 2Ø10 2Ø10	6Ø6/m' 6Ø6/m' 6Ø6/m'

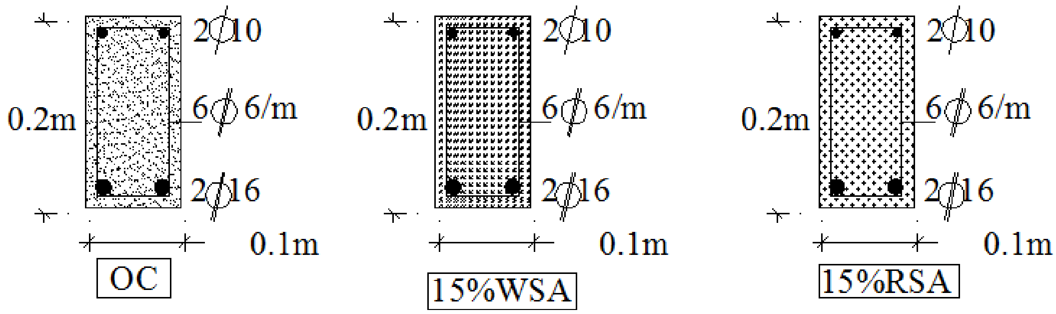


FIG. 3 Tested beams details

2.3 Test setup

The specimens were tested under in a machine of 1600 kN load capacity. Beams were simply supported over a span of 1200 mm. The load was distributed on two point load of 300 mm apart. The two loads are symmetrical to centerline of the beam. the edge dimension between the load plate and the nearest support is 450 mm. The specimens were tested under load control with the rate of 30–70 increments to failure. Deflection at the centerline was measured for every 0.2 kN increment of load using a linear variable differential transformers (LVDT) fitted at the center. The cracks during loading stages were cleared out and other observations were recorded at the failure as shown in Fig. 4 which shows crack pattern of beam B1 as shown in Fig. 4.

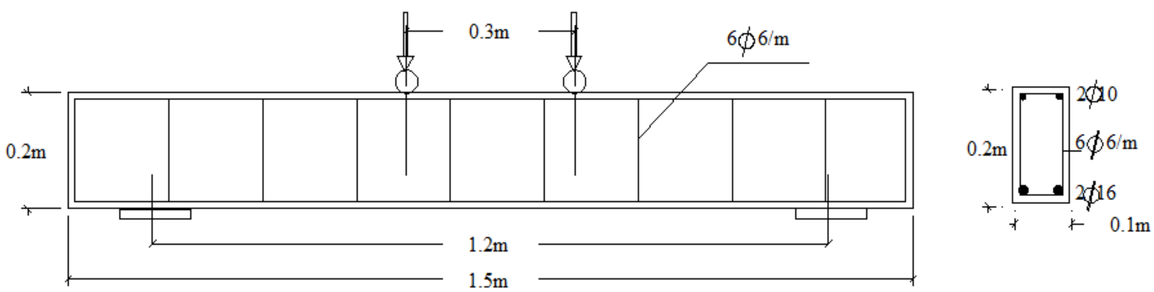


FIG. 4 Beam Test setup

3 EXPERIMENTAL RESULTS AND DISCUSSION

3.1 First crack and ultimate failure load

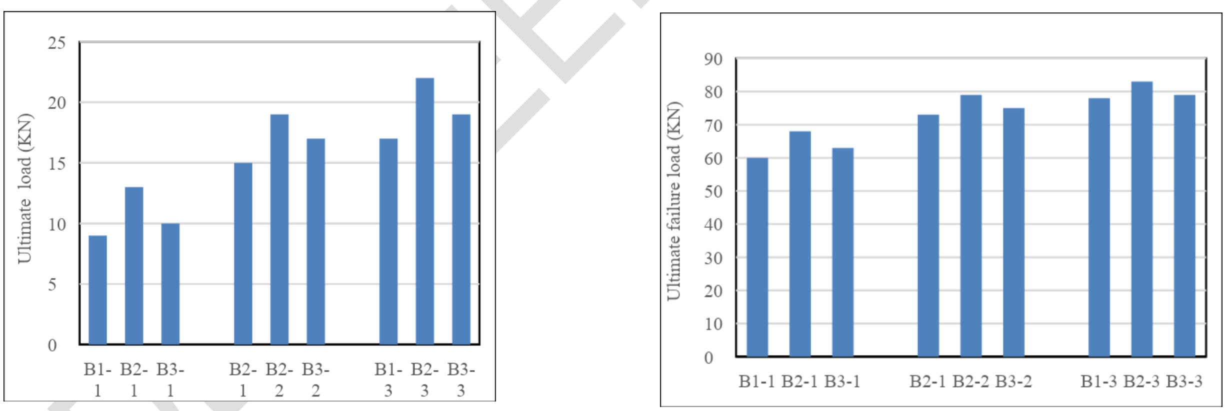
All tested beams in the three groups were observed under the experimental test till failure. Table 3 summaries the obtained and noticed results for the first crack loads, ultimate failure load and the deflection at ultimate failure load. The initial crack load increases from one specimen to other according to the type of cement replacement of wheat or rice straw ash and the grade of concrete. In the first group of concrete grade 30 MP_a, the beam B1-1 with ordinary concrete the first crack was at 9.0 kN but for B2-1 which use wheat straw ash with 15% as cement replacement. The first crack was at 13.0 kN but for B3-1 which use rice straw ash was at 10.0 kN. This delay in appearance of the first crack in B2-1 is due to the effect of using wheat straw ash which closed the cracks openings with respect to the rice straw ash. Also, the ultimate failure load increases from one specimen to another. The failure load of B1-1 was at 60.0 kN but for B2-1 was 68.0 kN and for B3-1 the failure load was 63.0 kN. Ash began to work during applying loads up to failure increase in ultimate load is very marginal compared to control beam B1-1 for group 1 as shown in (rewrite this sentence) Fig. 5.

In the second group which had a concrete grade of 45 MP_a, the initial crack loads and ultimate failure loads increase due to the increase of the used concrete grade and the percentage of used ash of wheat and rice as cement replacement. For the initial cracks load of B1-2, B2-2 and B3-2 were 15.0 kN, 19.0 kN and 17.0 kN respectively. This due to the best effectiveness of wheat straw ash in closing any cracks opening with respect to rice straw ash. This also affect in increasing the ultimate failure loads beyond the grade of concrete while the ultimate failure loads were 73.0 kN, 79.0 kN and 75.0 kN for B1-2, B2-2 and B3-2 respectively as shown in Table 3 and Fig. 5 and Fig. 6.

For the last group of concrete of 60 MPa which took the behavior of relatively high strength concrete and cement replacement ratio of 15% using wheat and rice straw ash. The obtained results were summarized in Table 3 as 78.0 kN, 83.0 kN and 79.0 kN for B1-3, B2-3 and B3-3 respectively as ultimate failure loads. For the initial cracks load of B1-3, B2-3 and B3-3 were 17.0 kN, 22.0 kN and 19.0 kN respectively which due to the used concrete strength and cement replacement ash. The increase in load carrying capacities when using ordinary material or cement replacement might be due to strain hardening and multiple micro-cracking behavior.

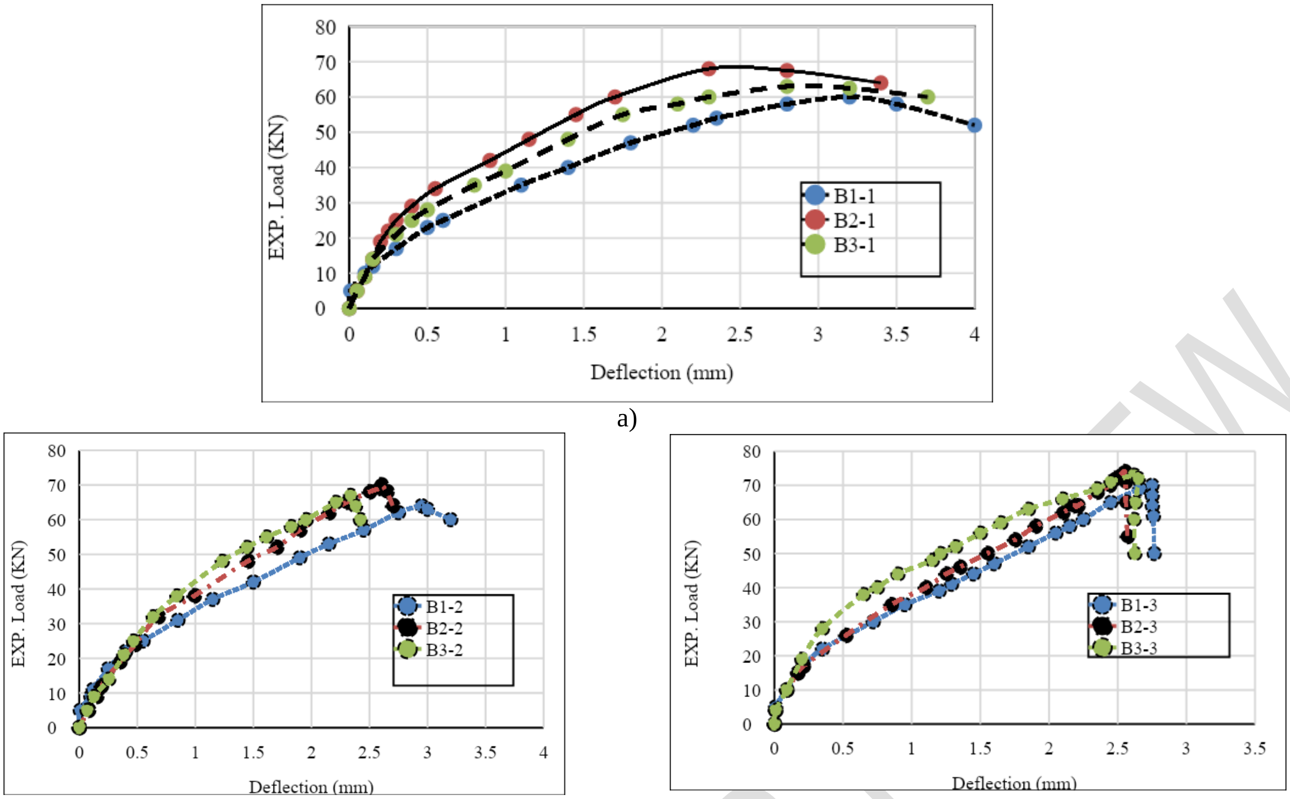
TABLE 3 EXPERIMENTAL RESULTS

Group	Sym bol	Ash content Vf %	Initial cracking Load (kN)	Ultimate failure Load (kN)	Max. deflection At mid span (mm)	Enhancemen t % In failure load	Enhancemen t % in ultimate deflection
Group 1 Fcu 30	B1-1	0	9	60	3.20	---	---
	B2-1	15	13	68	2.30	13.3	28.1
	B3-1	15	10	63	2.80	5.0	12.5
Group 2 Fcu 45	B1-2	0	15	73	2.9	---	---
	B2-2	15	19	79	2.3	8.3	21.0
	B3-2	15	17	75	2.7	2.8	7.0
Group 3 Fcu 60	B1-3	0	17	78	2.5	---	---
	B2-3	15	22	83	2.1	6.5	16.0
	B3-3	15	19	79	2.35	1.3	15.0



a) b)

FIG. 5 Comparison between ultimate failure loads for different beams specimens: a) initial cracking load; and b) ultimate failure load.



a)

b) c)

FIG. 6 Comparison between ultimate failures loads, a) group 1, b) group 2, c) group 3.

3.2 Mode of failure

Specimens were experimentally tested for flexure by applying two-point loading tests as shown in Fig. 4. The failure type was recorded as a flexure failure for all beams as in Fig.7. The beams showed initial cracking in the high bending moment region and then the cracks propagated in the vertical direction as the load was increased at approximately 70–85% of the ultimate load as mentioned previously. Material demonstrates strain hardening property, therefore, it could sustain tension load after cracking and it could be indicated that the load capacity would have increased. At the stage of ultimate failure, crush in compression region of concrete was observed in all specimens. Beams for Group 1 showed the same pattern of failure and the modes of failure are shown in Fig. 7

FIG. 7 Sample of cracks propagation and mode of failure

3.3 Deflection response

The experimental load to mid-span deflection curves are depicted in Fig 6. The plots represent the deflection value measured by the LVDT mounted at mid-span of beam. In general, the observed deflection was recorded for each beam as shown in Table 3. For the first group which has concrete grade of 30 MP_a, B1-1 has no cement replacement recorded the largest deflection value of 3.2 mm. for B2-1 and B3-1 the deflections were 2.3 mm and 2.8 mm respectively. The decrease in deflection value for B2-1 related to using of wheat straw ash as cement replacement which closed a lot of cracks occurred decreasing the deflection.

For the second group which has concrete grade of 45 MP_a, B1-2 has no cement replacement recorded the largest deflection value of 2.9 mm. for B2-2 and B3-2 the deflections were 2.2 mm and 2.7 mm respectively. The decrease in deflection value for B2-2 related to concrete grade and using of wheat straw ash as cement replacement which closed a lot of cracks occurred decreasing the deflection. This behavior was for the third group which used concrete of grade 60 MP_a as shown in Fig. 8.

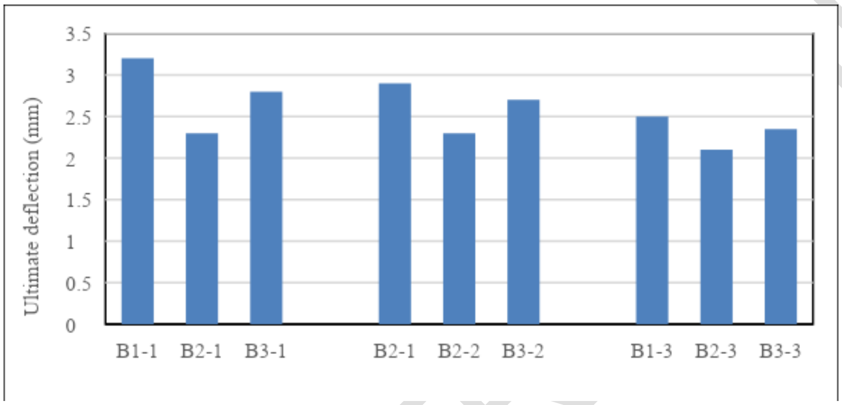


FIG. 8 Ultimate experimental deflection (mm) at mid span for different groups

3.4 Ductility factor parameter (DFP)

The ductility factor can be defined as the Percentage of the deflection at maximum load (**Δu**) to the deflection when steel RFT starts to yield (**Δy**). The values of (DFP) are depicted in Table 4. Concrete beams with cementreplacement material have more ductile behavior than the beam with ordinary cement. It was noted that in the case of wheat straw ash the ductility increases. Comparing the relative ductility factor to control beams of OC and RSA for Group A, the relative ductility factor increases by 9 and 5.3 % for 1.0% for the first group of concrete grade of 30 MP_a as in Table 4.However, when using a mix of both concrete grade of 45 MP_a and 60 MP_a the ductility factor enhancement increase to be 10.7, 8% for 1% in the second group but it was 12, 8.3% for 1% for the third group respectively which indicted the effect of using the wheat and rice straw ash as a ratio of cement content in concrete.

4. NON-LINEAR FINITE ELEMENT (FE) STUDY

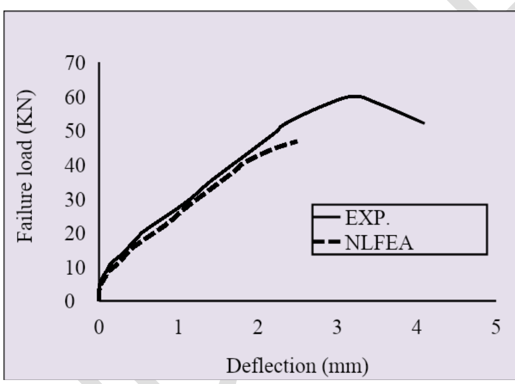
NLFE analysis was conducted to modeling the tested beams. ANSYS (ANSYS release 14.5) suit is used as the current procedure to conduct the analytical study. The load-deflection curve is the important aspect in verifying the specimen's behavior. It includes parameters such as ultimate loads, first cracking load, and maximum deflection. Therefore, comparing the loaded flexion curves extracted from analytical results with experimental curves as in Fig. 9. The concrete element was represented using solid 65 as 3D element with a volumetric ratio of rice and wheat straw ash. The reinforced steel is modeled using link-8 spare.

4.1 Finite element Model verification

Non-linear finite element model was conducted to compare the obtained experimental results of all beams. The first cracks started at early stage with respect to the experimental first crack where the group 1 first crack start at load of 7 kN for concrete grade of 30 MP_a, the second group first crack was at 11 kN but for the third group of concrete grade 60 MP_a it started at 13.0 kN. This due to recording the experimental first crack was by eye but in FEA it was micro cracks by programs. The cracks start to propagate in an upward direction through the beam depth. It can be observed from Table 5 that, reasonable agreement was achieved between the test results and the analytical results as will be discussed. The percentage of the predicted to experimental ultimate load for the beams ranged between 0.70 and 0.84 but for deflection it varied between 0.78 to 1 in different groups.

TABLE 4 EXPERIMENTAL DUCTILITY FACTOR PARAMETER

Group	Symbol	Ash content Vf %	Max. deflection Ultimate Δu	Max. deflection At yield load Δy	Ductility Factor Parameter Δu/ Δy	% of Enhancement In D.F.P increase
Group 1 Fcu 30	B1-1	0	3.20	2.40	1.33	1.0
	B2-1	15	2.30	1.58	1.45	9.0
	B3-1	15	2.80	2.0	1.40	5.3
Group 2 Fcu 45	B1-2	0	2.90	2.31	1.25	1.0
	B2-2	15	2.30	1.65	1.40	10.7
	B3-2	15	2.70	2.00	1.35	8.0
Group 3 Fcu 60	B1-3	0	2.50	2.10	1.20	1.0
	B2-3	15	2.10	1.56	1.35	12.5
	B3-3	15	2.35	1.81	1.30	8.3



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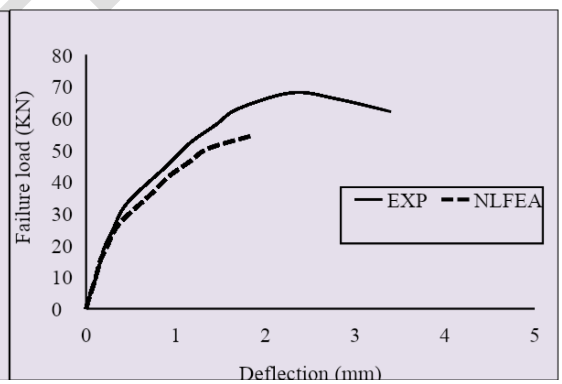

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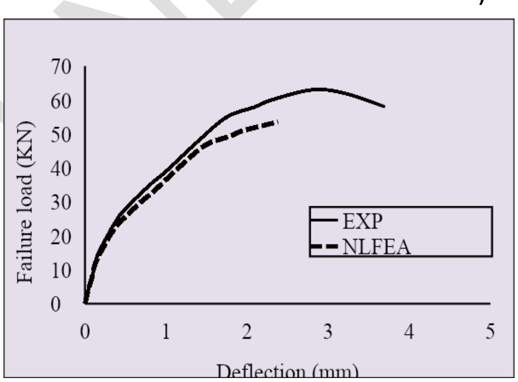

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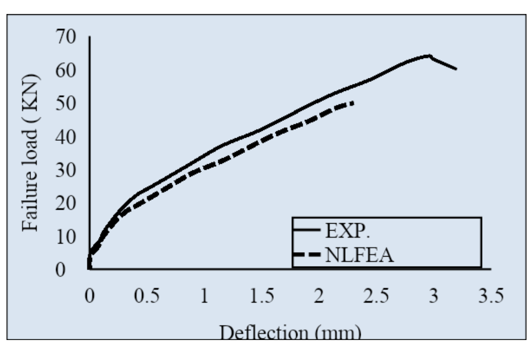

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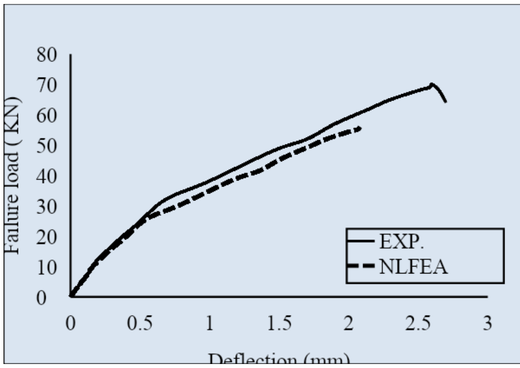
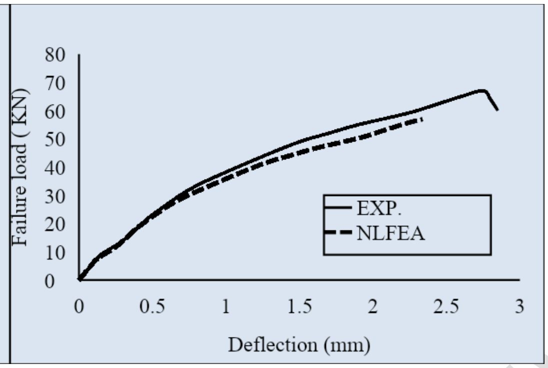


e)



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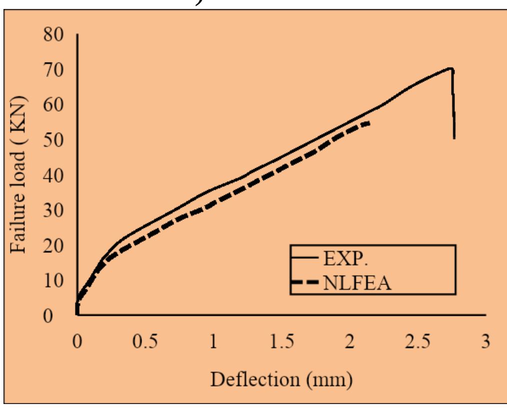

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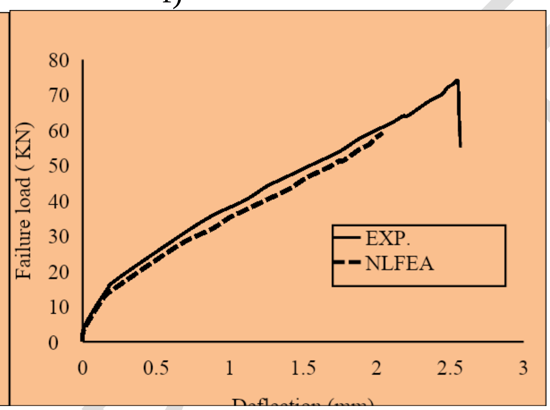

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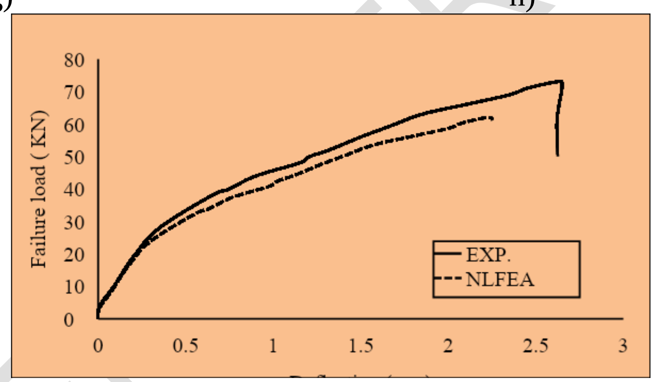


i)

FIG. 9 Comparisons between experimental and NLFEA load displacement curves; a) B1-1, b) B2-1; c) B3-1; d) B1-2; e) B2-2; f) B3-2; g) B1-3; h) B2-3; i) B3-3.

4.2 Load–displacement comparison

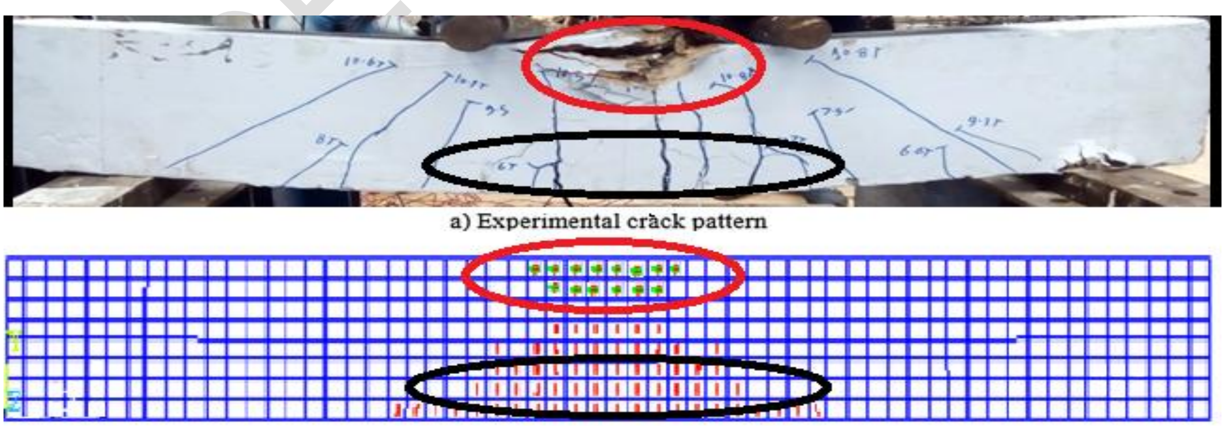
Fig. 9 showed the load-deflection curves for all beams in phase of experimental and NLFE obtained results. The recorded deflection for experimental and NLFE analysis showed a satisfactory agreement with respect to the deflection recorded for the control specimen for each group as in Fig. 9 and Table 5. For the first group 1, The recorded ratio between ΔNLFE / Δ Exp was 0.78, 0.78 and 0.85 for B1-1, B2-1, B3-1 respectively. but for B1-2, B2-2 and B3-2, these ratios were 0.79, 0.86 and 0.85 respectively. Also, for B1-3, B2-3 and B3-3, these ratios were 0.88, 1.00 and 0.79 respectively. These ratios showed that NLFE program provide a reasonable response in deflection as in Fig. 9.

TABLE 5 COMPARISON BETWEEN NLFEA RESULTS AND EXPERIMENTAL RESULTS

Group	Symbol	Ash content Vf %	Initial cracking Load (kN)	Ultimate failure NLFE Load (kN)	Ultimate failure Load (kN)	NLFE deflection At mid span (mm)	Max. deflection At mid span (mm)	Load (NLFE / EXP)	Deflection (NLFE / EXP)
Group 1 fcu 30	B1-1	0	7.0	46.8	60	2.5	3.20	0.78	0.78
	B2-1	15	7.0	54.4	68	1.8	2.30	0.80	0.78
	B3-1	15	7.0	53.5	63	2.4	2.80	0.84	0.85
Group 2 fcu 45	B1-2	0	11.0	49.9	73	2.3	2.9	0.70	0.79
	B2-2	15	11.0	56.0	79	2.0	2.3	0.71	0.86
	B3-2	15	11.0	56.9	75	2.3	2.7	0.76	0.85
Group 3 fcu 60	B1-3	0	13.0	54.6	78	2.2	02.5	0.70	0.88
	B2-3	15	13.0	59.2	83	2.1	02.1	0.71	1.0
	B3-3	15	13.0	61.2	79	2.3	2.35	0.78	0.79

4.3 Crack Patterns

Fig. 10 shows a comparison between the crack patterns experimentally and in NLFE analysis these cracks begins micro cracks and increased in length and width till failure that is agreed with El-Sayed et al.



a) Experimental crack pattern

b) NLFEA crack pattern

FIG. 10 Typical crack pattern beams; a) Experimental crack pattern; b) NLFE crack pattern

5. CONCLUSIONS

The investigations on the structural behavior of reinforced concrete beams with WSA & RSA under static loading conditions showed improved load carrying capacity for different mixes suggesting that it can be used as a potential cement reinforcement material. Also, experimental results revealed that 15% ideal ratio of WSA & RSA had improved flexural behavior.

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