Journal of Engineering Sciences, Assiut University, Vol. 40, No 1, pp.121-146, January 2012

SHEAR BEHAVIOR OF REDUCED-WEIGHT REINFORCED CONCRETE BEAMS

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(Received November 27, 2011 Accepted December 27, 2011)

This paper presents an investigation to improve the understanding of the shear behavior of reinforced reduced-weight concrete beams made of light-weight expanded clay aggregate (LECA) as a partial replacement (by volume) to the normal-weight aggregates. Eleven reinforced concrete beams divided into two groups were fabricated and tested using the symmetrical two-point loads test. The tested beams consisted of seven reinforced reduced-weight concrete beams and four reinforced normal-weight control beams. The effects of several variables such as type of concrete according to its weight, shear span to depth ratio (a/d), concrete grade and the amount of stirrups were experimentally investigated. The behavior of the tested beams was analyzed in terms of mode of failure, load-deflection response, load-strains response, shear stress- shear strain relationships, first shear cracking loads, ultimate carrying capacity, stiffness and ductility. Furthermore, the test results were compared with the predictions using the Egyptian Code for Concrete Structures, (ECP 203). Despite the experimental results illustrated that the reduced-concrete beams were shown less load carrying capacity, stiffness and ductility than those of the comparative normal-weight concrete beams, the theoretical predictions using the Egyptian Code were quite conservative. This could be attributed to that the effect of arch action is still underestimated in the Egyptian Code.

KEYWORDS: *reduced-weight concrete beam; shear behavior; failure mode; first shear cracking load; ultimate load.*

INTRODUCTION

In concrete structures, the concrete imposes a huge amount of the total load of the structure. Lighter concrete offers design flexibility and substantial cost saving by providing less dead load, improved seismic structural response, low heat conductivity and lower foundation cost when applied to structures. In recent years, due to these advantages, there is an interest in production and investigation of the light or reduced-weight concrete. Many researchers such as Ilker and Burak, [1], Kilic et al, [2], Liu et al, [3], and Demirbog, [4], studied the mechanical properties, durability and thermal conductivity of the lightweight concrete. Kayali, [5], used fly ash light weight aggregate to produce light-weight high performance concrete. He reported that; concrete produced using these aggregates is around 22% lighter and at the same time

20% stronger than normal weight aggregate concrete. Also, drying shrinkage is around 33% less than that of normal weight concrete. On the other hand, Choi et al, [6], reported that the range of elastic modulus has come out as 24 –33 GPa, for light-weight concrete (LWC) with compressive strength more than 40 MPa, comparably lower than the normal concrete which possessed the same compressive strength. In addition, for LWC, different researchers, [7, 8 and 9], have proposed different relationships to estimate modulus of elasticity value from compressive strength and unit weight. However, these relationships very much depend on the type and source of the light-weight aggregate, since the light-weight aggregates are porous and have modulus of elasticity values lower than that of natural aggregate. Zhang and Gjorv, [10], reported also that the tensile/compressive strength ratio of light-weight high-strength concrete was lower than that of normal-weight high-strength concrete. On the other hand, Haque et al [11], carried out an experimental study and found that replacement of Lightweight fine aggregate with normal weight sand produces a concrete that is somewhat more durable as indicated by their water penetrability and depth of carbonation when concretes are of equal strength. Other research, [12], was carried out to investigate the autogenous shrinkage behavior of LWC. The wet light-weight aggregate provided an inner reservoir of water which caused contentious curing and hence, prevent the autogenous shrinkage.

However, although it was found that light-weight concrete (LWC) has good insulation and mechanical properties; it still needs further investigations of its structural behavior for use as structural members. Delsye et al, [13], presented an experimental investigation consisted of testing of 6 under-reinforced beams to study the flexural behavior of reinforced light-weight concrete beams produced from oil palm shell (OPS) aggregates that was produced from Malaysia. All OPS concrete beams showed typical structural behavior in flexure. OPS concrete beams showed also a good ductility behavior. The beams exhibited considerable amount of deflection, which provided ample warning to the imminence of failure. Other researchers, [14], presented an investigation of the flexural behavior of reinforced light-weight concrete beams made from light-weight expanded clay aggregate (LECA). Nine reinforced concrete beams were fabricated and tested using the symmetrical two-point loads test. Based on the experimental results, the ultimate moment of beams made with LECA lightweight concrete could be predicted satisfactorily via the equations provided by the ACI 318 building Code. For preventing the brittle failure of LECA beams, it was suggested that the maximum section bars of the ACI code should be reduced. Another study (Alengaram et al), [15], showed that, flexural behavior of reinforced palm kernel shell light-weight concrete beams closely resembles that of equivalent beams made by normal-weight concrete. On the other hand, Experimental results of a study made by Jumaat et al, [16], mentioned that the shear capacities of oil palm shell foamed concrete (OPSFC) beams without shear links were higher than those of normal-weight concrete beams and exhibit more flexural and shear cracks.

Nevertheless, there is a lack in the knowledge about the structural behavior of the light-weight concrete when used in structural members. Previous researches indicated also that the properties of light-weight concrete depend on the type of its lightweight aggregates. Therefore, the structural behavior of light-weight concrete members may vary according to the type of the used light-weight aggregates. Furthermore, the interlocking of the aggregates possesses a huge impact on the concrete shear strength as well as the shear behavior and shear capacity of the reinforced concrete beams.

Accordingly, the current research aims to investigate the shear behavior of reinforced reduced-weight concrete beams made with light-weight expanded clay aggregates (LECA), which is one of the widespread light-weight aggregates, as a partial replacement to the normal weight aggregates. Eleven beams; were fabricated and tested through the current experimental work for understanding the shear behavior of the reduced-weight reinforced concrete beams. The effects of several variables such as concrete weight, concrete grades, shear span to depth ratio (a/d) and the amount of stirrups were experimentally investigated. The test results are analyzed to demonstrate the effects of these considered variables on the tested reduced-weight concrete beams as well as the normal-weight concrete beams. Moreover, the test results were compared with the predictions using the Egyptian Code for Concrete Structures, (ECP-203), [17], for examining the shear design equations in predicting this type of reinforced concrete beams.

EXPERIMENTAL PROGRAM

To achieve the main aim of the current study, an experimental program consisted of fabricating and testing of eleven reinforced concrete beams was designed. Seven reinforced concrete beams contain light-weight expanded clay aggregates (LECA) as a partial replacement (by volume) to the normal weight coarse and fine aggregates with a percentage equals 50%. The unit weight of this type of concrete ranged between 1830 kg/m^3 to 1890 kg/m^3. The other four beams were cast with normal-weight concrete which contained normal-weight coarse and fine natural aggregates to be used as control specimens.

Materials and Concrete Mixes

Four concrete mixes were designed in the current research. Two mixes of them (mixes No. 1 and 2) possessed normal unit weights (control mixes) while the other two mixes (mixes No. 3 and 4) possessed reduced unit weights. Two intended compressive strengths; 30 MPa (for mixes 1 and 3) and 40 MPa (for mixes 2 and 4) were aimed. Table (1) shows the details of these four mixes. The used cement was Ordinary Portland Cement type CEM I – 42.5 complied with the Egyptian Standard. In the reduced-weight mixes, silica fume having a silica content of 96.5%, a specific gravity of 2.15 and specific surface area of 20000 cm^2/gm was used as a partial replacement to the cement. Silica fume was added to replace 10% of the cement content in mix 3 and 20% in mix 4. Local dolomite crushed stone size 10 mm and natural sand were used as coarse and fine aggregates, respectively, in mixes 1 and 2. While, in the reduced-weight mixes (mixes 3 and 4), coarse and fine light-weight expanded clay aggregates (LECA) were used as partial replacements to the normal- weight coarse and fine aggregates, respectively, with a percentage equals 50% (by volume). The used coarse LECA possessed a volume weight equals 600 kg/m^3 and a specific weight equals 1.0, while the fine LECA possessed a volume weight equals 1100 kg/m^3 and a specific weight equals 1.6. In addition, a high range water reducing and set retarding concrete admixture of modified synthetic dispersion basis (complies with ASTM C 494 Type G and BS 5075 Part 3) was used in the designed reduced-weight mixes for reducing the amount of the mixing water. The used dosage of the admixture was 2% of the binder materials. It must be mentioned that the amounts of water listed in Table (1) included the absorbed water by the coarse and fine aggregates. Finally, it should be mentioned also that the workability of the designed four mixes was adjusted to be maintained at the same level of workability. Slump tests were carried out on the fresh concretes and all mixes recorded slump values equal 70 mm ± 5 mm.

Table (1): Mix Proportions of Concrete Mixes

Mix No.	Type of Concrete	Cement (Kg/m^3)	Silica Fume (Kg/m^3)	Coarse Agg. (Kg/m^3)		Fine Agg. (Kg/m^3)		Water (Lit/m^3)	Admix. (Kg/m^3)
				Dolomite	LECA	Sand	LECA		
1	Normal Weight	350	---	1224	---	612	---	195	---
2	Normal Weight	440	---	1164	---	582	---	205	---
3	Reduced Weight	315	35	612	204	306	184	185	7.0
4	Reduced Weight	352	88	582	194	291	175	195	8.8

Details of the Test Beams

A total number of eleven reinforced beams in two groups (A and B) were fabricated and tested in the current study. Group A consists of beams B1 to B6 with intended concrete compressive strength 30 MPa, while group B consists of beams B7 to B11 with intended concrete compressive strength 40 MPa. All of the beams were 2000 mm long, 1800 mm span, 150 mm wide and 300 mm total deep, with an effective depth equals 275 mm. The main tensile reinforcing bars for the beams were 3 Φ 12 (high tensile steel 400/600) while the compression reinforcement of the whole beams was 2 ϕ 8 (mild steel 280/420). The shear reinforcements (stirrups) were used with diameter 6 mm (mild steel 280/420) at a spacing of 200 mm for beams B2, B4, B5, B8, B10 and B11 and at a spacing of 100 mm for beam B6. The other beams -B1, B3, B7 and B9- were fabricated without shear reinforcements. The main properties of the used steel bars were listed in Table (2). The geometrical and reinforcement details of the tested beams were shown Figure (1). The beams were cast in steel moulds as shown in Figure (2). Six standard cubes 150x150x150 mm and six standard cylinders of 150 mm diameter and 300 mm height were cast with the test beam as control specimens to determine the actual concrete compressive strength, splitting strength and static modulus of elasticity of each beam.

Table (3) presents the group number, the beam identifications and the main characteristic values of the tested beams. In group A; beams B1 and B2 were cast with mix 1 (normal-weight concrete) while beams B3, B4, B5 and B6 were cast with mix 3 which of reduced-weight concrete. Similarly, in group B; beams B7 and B8 were cast with mix 2 (normal-weight concrete) while beams B9, B10 and B11 were cast with mix 4 which of reduced-weight concrete. In addition, the shear span was 330 mm (shear span to depth ratio = 1.2) for all beams except beams B5 and B11 which had shear span equal 600 mm (shear span to depth ratio ≈ 2.2).

Table (2): Properties of Steel Reinforcement

Type / Size	Mild Steel	Mild Steel	High Tensile Steel
Diameter (mm)	6	8	12
Actual Cross Sectional Area (mm^2)	28.69	50.80	112.4
Weight / Unit Length (kg/m')	0.225	0. 399	0.882
Yield Strength (N/mm^2)	332.3	307.7	443.6
Ultimate Strength (N/mm^2)	506.6	437.7	676.4
Elongation (%)	25.9	28.9	13.2

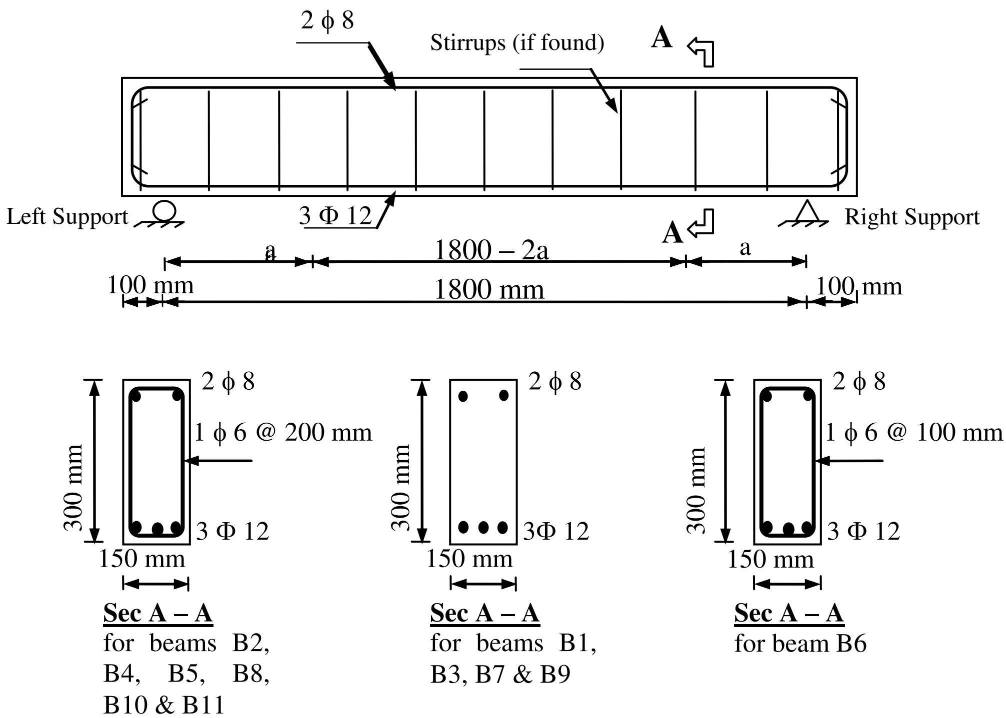


Figure (1): Geometrical and Reinforcement Details of the Tested Beams

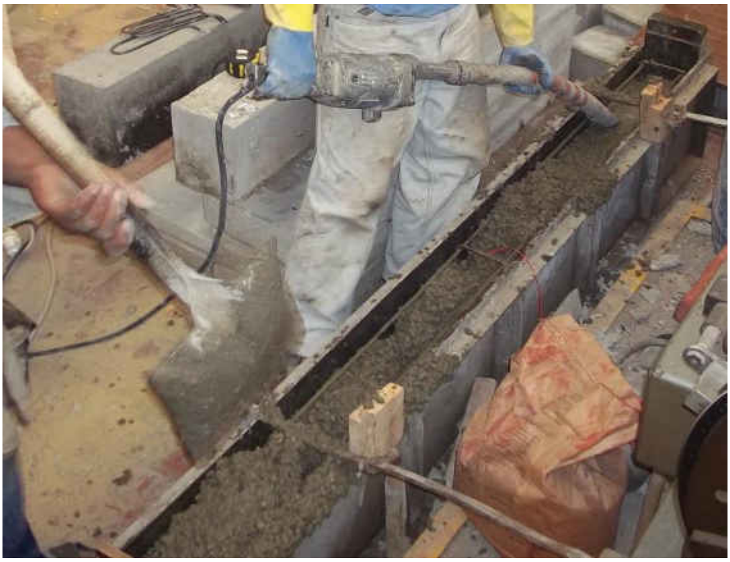


Figure (2): Casting and Compaction of the Test Beams in their Steel Moulds

Table (3): Main Properties of the Test Beams

Group	Beam Ident.	Shear Span to Depth Ratio	Stirrups / m'	Type of Concrete	Intended Concrete Grade, MPa
A	B1	1.2	0	Normal-weight	30
	B2	1.2	5ϕ6	Normal-weight	30
	B3	1.2	0	Reduced-weight	30
	B4	1.2	5ϕ6	Reduced-weight	30
	B5	2.2	5ϕ6	Reduced-weight	30
	B6	1.2	10ϕ6	Reduced-weight	30
B	B7	1.2	0	Normal-weight	40
	B8	1.2	5ϕ6	Normal-weight	40
	B9	1.2	0	Reduced-weight	40
	B10	1.2	5ϕ6	Reduced-weight	40
	B11	2.2	5ϕ6	Reduced-weight	40

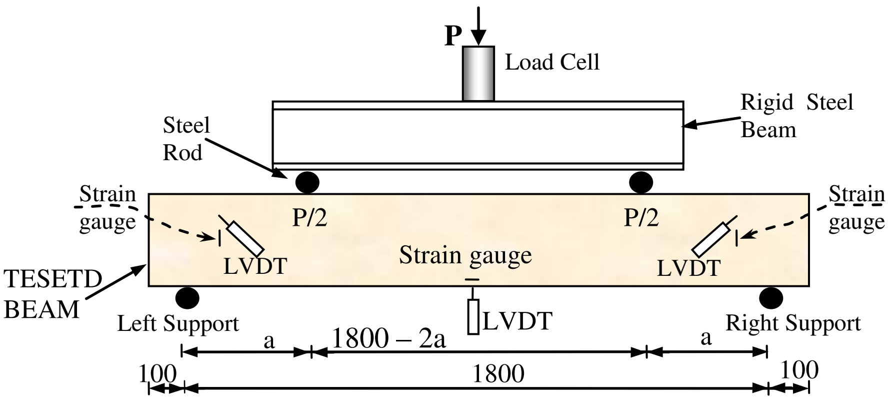
The unit weight was determined for the standard cubes before testing. The unit weight ranged from 2330 kg/m^3 to 2360 kg/m^3 for the normal-weight concrete mixes (mixes 1 and 2). On the other hand, the unit weight ranged from 1830 kg/m^3 to 1890 kg/m^3 for the reduced-weight concrete mixes (mixes 3 and 4). This means that the reduced-concrete in the current research was lighter than the normal-weight concrete by about 20%: 21%.

Due to the inherent higher total moisture content of the reduced-weight concrete, it does not need to water curing. Therefore, the beams and their control specimens were cured in ambient air in the laboratory until the testing day. Testing of beams was conducted at the age of about 55 to 65 days.

Instrumentation and Testing

The tests were performed using a 5000 kN hydraulic compressive machine. A 2000 kN load cell was used to measure the applied load and the readings were recorded automatically by means of a data acquisition system.

The mid-span deflection was measured for the tested beams using linear variable displacement transducer (LVDT). Strains were measured at the mid-span of the tensile steel by using 10 mm electrical strain gauges. Other two electrical strain gauges were mounted on the vertical leg of the second left and right stirrups. Other two LVDTs were attached in the maximum left and right shear regions at an angle of 45°. The strain gauges and LVDTs were also connected to the data acquisition system. Figure (3) illustrates a schematic of the loading setup and instrumentation of the tested beams. Also, Figure (4) presents a general view of the test setup.



*All dimensions in mm

Figure (3): Test Setup and Instrumentation of the Tested Beams

Figure (4): General View of the Test Setup

As shown in Figures (3) and (4), each beam was acted upon by symmetrical two vertical concentrated loads. The spacing between the two loads was 1140 mm in all beams except beams B5 and B11 which was 600 mm.

The measurements and observations were determined at each recorded load level. The test was continued after the ultimate load in order to assess the post peak behavior of the tested beams.

TEST RESULTS AND DISCUSSION

Results of Compressive Strength, Splitting Strength and Modulus of Elasticity

Table (4) illustrates the results of the compression, splitting and modulus of elasticity tests of the control specimens (cubes 150 x 150 x 150 mm for compressive strength and cylinders 150 mm diameter and 300 mm height for splitting strength and modulus of elasticity) which were cast with the test beams. These control specimens were tested in the same day of testing of their beams. It must be mentioned that each value listed in Table (4) is the average of the test results of three specimens.

Table (4): Actual Compressive Strength, Splitting Strength and Modulus of Elasticity of the Control Specimens of the Test Beams

Beam Ident.	Type of Concrete	Intended Concrete Grade, MPa	Actual Comp. Strength, MPa	Actual Splitting Tensile Strength, MPa	Actual Ec, MPa
B1	Normal-weight	30	34.8	2.56	27583
B2	Normal-weight	30	32.2	2.18	29073
B3	Reduced-weight	30	33.3	Not available	Not available
B4	Reduced-weight	30	32.0	1.89	18918
B5	Reduced-weight	30	35.2	2.43	19099
B6	Reduced-weight	30	32.7	2.31	18335
B7	Normal-weight	40	39.8	3.11	35910
B8	Normal-weight	40	42.8	3.63	38474
B9	Reduced-weight	40	40.8	2.60	19333
B10	Reduced-weight	40	39.2	2.55	19073
B11	Reduced-weight	40	39.7	2.71	19246

The average of compressive strength of beams B1 and B2 (in group A), which were made of normal weight concrete, mix 1, was 33.5 MPa while the average of splitting strength was 2.37 MPa, i.e. the splitting strength was about 7.1% the compressive strength. Moreover, the average static modulus of elasticity for these two beams was 28328 MPa, i.e. the static modulus of elasticity for this type of normal concrete equals 4894 $\sqrt{f_{cu}}$. This means that equation (2-1) in the Egyptian Code for Reinforced Concrete Structures, (ECP 203) [17], $Ec = 4400\sqrt{f_{cu}}$, is conservative. On the other hand, for the reduced-weight concrete beams, B3, B4, B5 and B6, of the same group, A, which had the same intended f_{cu}, the average compressive strength was 33.3 MPa. Also, the average of splitting strength was 2.21 MPa, i.e. the splitting strength was about 6.6% of the compressive strength. This means that the tensile/compressive strength ratio for the reduced-weight concrete was lower than that of normal-weight concrete. Furthermore, the average static modulus of elasticity for these beams was 18784 MPa, i.e. the static modulus of elasticity for this type of reduced-weight concrete equals $3255\sqrt{f_{cu}}$. Such results indicated that both of the splitting strength and static modulus of elasticity of the reduced-weight concrete mix of group A were smaller than those of the normal-weight concrete that possessed the same compressive strength. In addition, equation (2-1) in the Egyptian Code [17] can not be applied in the case of reduced-weight concrete.

Similarly, in Group B, the average of compressive strength of beams B7 and B8, which were made of normal-weight concrete, mix 2, was 41.3 MPa while the average of splitting strength was 3.37 MPa, i.e. the splitting strength was about 8.2% the compressive strength. Moreover, the average static modulus of elasticity for these two beams was 37192 MPa, i.e. the static modulus of elasticity for this type of normal concrete equals $5787\sqrt{f_{cu}}$. This means that Equation (2-1) in the Egyptian Code for Reinforced Concrete Structures, (ECP 203), [17], $Ec = 4400\sqrt{f_{cu}}$, is much conservative in this case. For the reduced-weight concrete beams, B9, B10 and B11, of the same group, B, which had the same intended f_{cu}, the average compressive strength was 39.9 MPa. Also, the average of splitting strength was 2.62 MPa, i.e. the splitting strength was about 6.6% of the compressive strength. Such result indicated that the ratios between the splitting strength and compressive strength for the reduced-weight concrete for the two mixes, 3 and 4 were equals. On the other hand, the average static modulus of elasticity for these beams was 19217 MPa, i.e. the static modulus of elasticity for this type of reduced-weight concrete equals $3042\sqrt{f_{cu}}$.

The above results indicated that, the reduced-weight concrete showed smaller tensile strength and static modulus of elasticity than those of normal-weight concrete. In group A, the splitting tensile strength of the reduced-weight concrete was about 93% of that of normal-weight concrete, while the static modulus of elasticity of the reduced-weight concrete was about 66% of that of normal weight concrete.

The trend that was observed in group A was more pronounced in group B. The splitting tensile strength of the reduced-weight concrete was about 78% of that of normal weight concrete, while the static modulus of elasticity of the reduced-weight concrete was about 52% of that of normal weight concrete.

Such results indicated that the static modulus of elasticity of the reduced-weight concrete is much less than that of normal-weight concrete which possesses the same grade. Moreover, this reduction was more pronounced in higher strength concretes.

Results of the Tested Beams

Results of the tested beams are presented; analyzed and discussed in this section. Topics to be covered include the mode of failure; the load-deflection relationships; the load-strain relationships; the shear stress-strain relationships, the cracking load, the ultimate load, the stiffness and ductility of the tested beams. Table (5) lists the cracking loads, the ultimate loads and the shown failure modes of the tested beams. The cracking loads corresponded to the appearance of first shear crack, while the ultimate loads are the maximum loads recorded during the tests. The tested beams showed different structural behavior according to the studied key variables.

Table (5): Cracking Loads, Ultimate Loads and Failure Modes of the Tested Beams

Group	Beam ident.	First shear crack (kN)	Ultimate load (kN)	Mode of failure
A	B1	97.8	247.2	Shear Failure
	B2	144.6	249.4	Shear Failure
	B3	130.2	231.7	Shear Failure
	B4	150.7	233.3	Shear Failure
	B5	115.0	148.9	Flexural Tension Failure
	B6	134.4	286.9	Flexural Tension Failure
B	B7	124.4	266.8	Shear Failure
	B8	154.0	285.4	Shear Comp. Failure
	B9	140.4	255.3	Shear Failure
	B10	164.7	260.4	Shear Failure
	B11	120.0	157.0	Flexural Tension Failure

Modes of Failure

Figures (5) and (6) illustrate the appearance of the tested beams of groups A and B, respectively, after loading. Also, Table (5) listed the shown failure modes of the different beams. As shown in the Figures and the Table, the cracking behavior and mode of failure of the tested beams followed different trends based on the studied key variables.

Figure (5): Failure Shapes of Beams of Group A

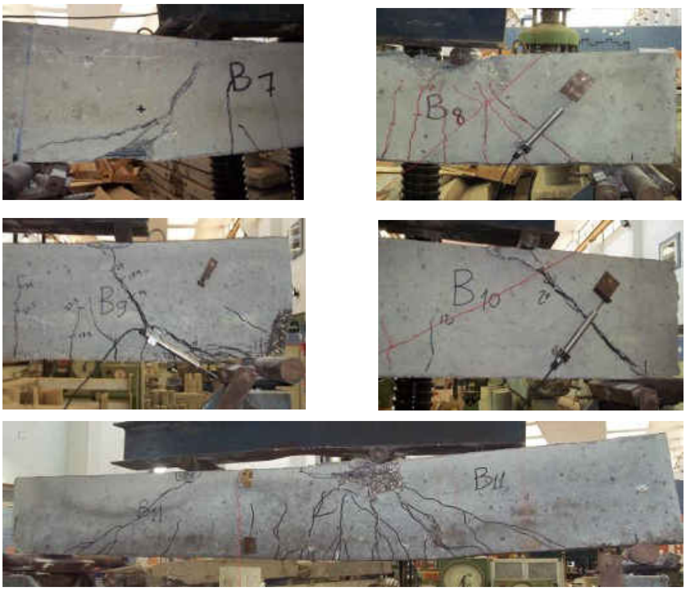


Figure (6): Failure Shapes of Beams of Group B

In general, in beams of a/d =1.2, the low a/d guaranteed that a significant amount of loading would transfer in arch action. The load path for the arch action is a direct load transfer from the loading point to the support that would result in less deflection. Beams without shear stirrups or with low amount of stirrups showed similar cracks and failure modes of diagonal tension failure. At first, fine tension cracks in the mid span zone appeared then the diagonal cracks were observed.

After development of the shear cracks in the normal weight concrete beams, B1, B2, B7 and B8, expected instantaneous failure did not occur due to the arch action which prevent the sudden failure and sustain the applied load. In addition, buckling in the compression reinforcement occurred at the loading point in B8 at later stages after the failure load. Shear cracking loads of the reduced-weight concrete beams, B3, B4, B9 and B10, were greater than those of the comparative normal-weight concrete beams, see Table (5). This means that, due to the high brittleness of the reduced-weight concrete beams, no enough warning could be obtained before failure. Furthermore, due to the high brittleness of the reduced-weight concrete beams, the presence of arch action could not prevent the instantaneous failure, i.e. sudden drop in the applied load was observed in beams B3, B4, B9 and B10. In general, beams with stirrups experienced the formation of fine flexural cracks in the mid-span region. In beam B6 that was provided with closely spaced stirrups, the shear reinforcement attracted more loading to transfer in beam action. In other words, the stirrups improved the shear capacity, promoted the beam action, attracted greater tensile stress in the web, prevented shear cracking to develop and prevented the sudden failure mode. Hence, despite the appeared fine shear cracks, B6 failed in flexural tension then crushing of concrete in the compression zone at the loading points.

On the other hand, in beams of a/d ≈ 2.2, B5 and B11, greater shear span to depth ratio promotes the beam action, especially in the presence of stirrups, and reduces the arch action. As a result, regardless the preceded appeared fine diagonal cracks, these beams failed in flexural tension and yielding of bottom reinforcement occurred. After that, excessive loading yielded crushing of the concrete in the compression zone.

Load – mid-span Deflection Records

The mid-span deflection due to the short-term loading of the beams of groups A and B are presented in Figures (7) and (8), respectively.

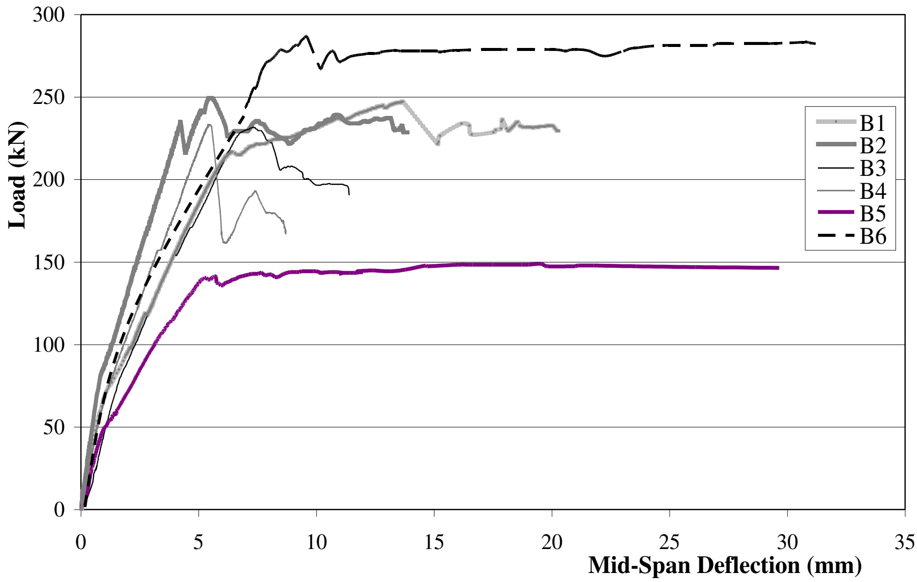


Figure (7): Load – Mid-span Deflection Relationships of Beams of Group A

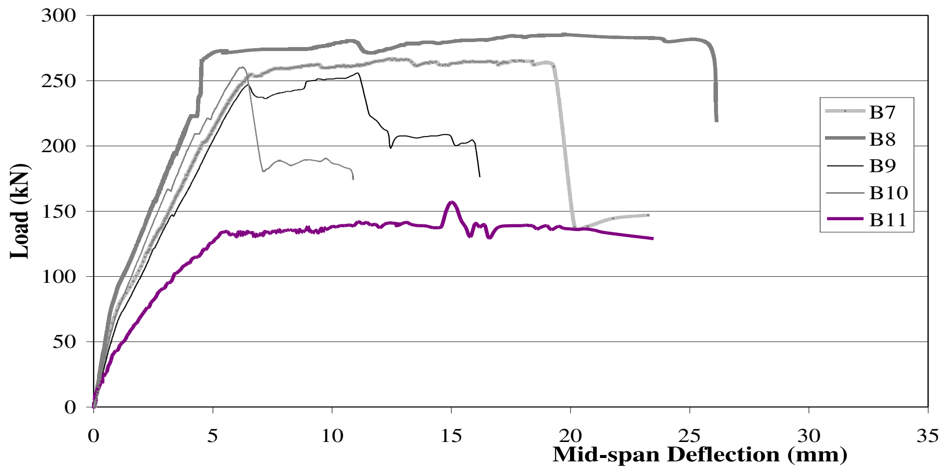


Figure (8): Load – Mid-span Deflection Relationships of Beams of Group B

Load-deflection curves of beams without shear reinforcement were basically linear up to failure. When shear reinforcements were provided, the load-deflection response was slightly curved after cracking. Also, beams with shear reinforcement were slightly stiffer than beams without shear reinforcement. Increasing the amount of stirrups (in B6) changed the behavior of the beam to fail in flexural tension mode. This beam showed higher stiffness, higher load carrying capacity and greater ductility than the corresponding beams (B3 and B4).

On the other hand, the normal-weight concrete beams were shown stiffer than the reduced-weight concrete beams. Also, the load carrying capacities of the reduced-weight concrete beams were less than those of the corresponding normal-weight concrete beams. Furthermore, reduced-weight concrete beams lost their strength faster than the comparative normal-weight concrete beams, i.e. behind the ultimate loads, the normal-weight concrete beams could sustain greater applied loads than those of the comparative reduced-weight concrete beams. This means that the reduced-concrete beams were shown less ductility in terms of deflection at failure.

Increasing a/d to 2.2 (beams B5 and B11) changed the behavior of the beams to fail in flexural tension mode. These beams showed less stiffness, less load carrying capacity and greater ductility.

Shear Stress – Strain Response

The nominal shear stress, q, developed in the tested beams at the critical shear zones can be estimated based on the recorded load as in Equation (1),

$$q = \frac{0.5 \text{ x } P}{b \text{ x } d} \qquad \text{Equation (1)}$$

Where: P is the applied load; b and d are the width and effective depth of the tested beams, respectively, [b=150 mm and d=275 mm].

On the other hand, the shear strain can be explained by the shear angle, γ, which can be estimated for the tested beams from the measurements of the diagonal LVDTs, Δ_d, as shown in Figure (9).

$$\gamma = \frac{[\sqrt{(D + \Delta_d)^2 - x^2}] - y}{x}$$

$$x = y = \frac{D}{\sqrt{2}}$$

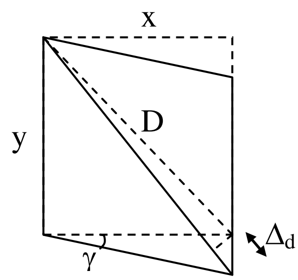


Figure (9): Geometry of Shear Strain in the Beams

Table (6) listed the shear stress and the corresponding shear angle of the tested beams of a/d =1.2 at the cracking load level and the ultimate load level. Moreover, Figures (10) and (11) illustrate the shear stress – shear angle relationships of these beams of groups A and B, respectively.

Table (6): Experimental Shear Stresses and Shear angles at Cracking and Ultimate Levels

Group	Beam ident.	First Cracking Level		Ultimate Level	
		Shear Stress (MPa)	Shear Angle (rad)	Shear Stress (MPa)	Shear Angle (rad)
A	B1	2.37	0.00016	5.99	0.02432
	B2	3.51	0.00013	6.05	0.01235
	B3	3.15	0.00045	5.62	0.02409
	B4	3.65	0.00101	5.66	0.00878
	B6	3.26	0.00099	6.95	0.00651
B	B7	3.02	0.00026	6.47	0.02380
	B8	3.73	0.00013	6.92	0.01991
	B9	3.57	0.00050	6.19	0.02175
	B10	3.99	0.00226	6.31	0.01073

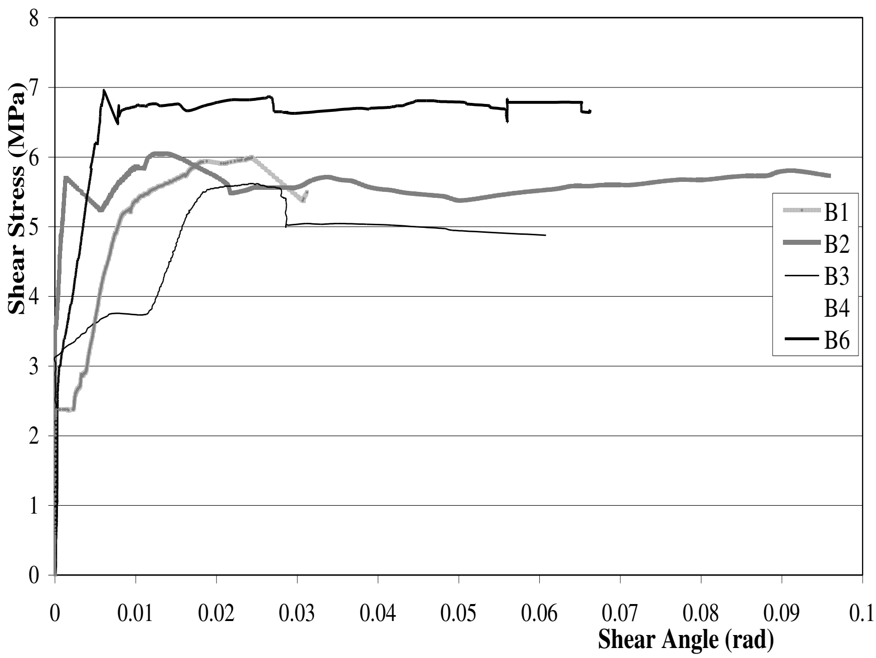


Figure (10): Shear Stress - Shear angle Relationships of Beams of Group A

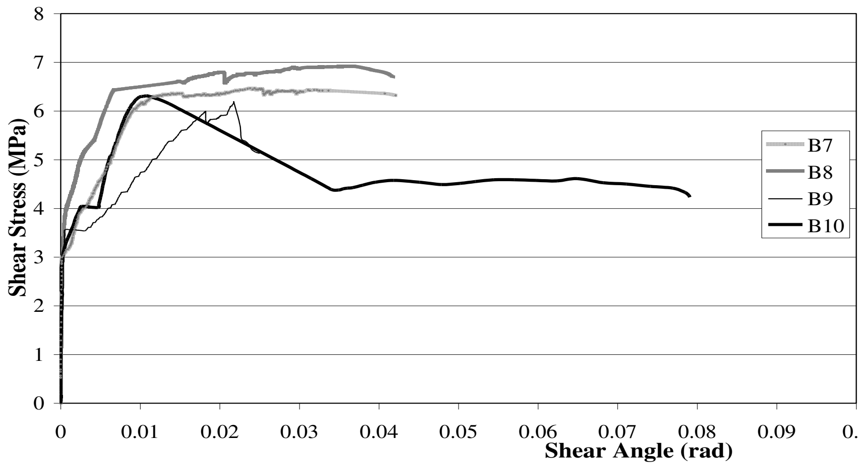


Figure (11): Shear Stress - Shear angle Relationships of Beams of Group B

It can be observed from Table (6) and Figures (10) and (11) that the shear angle was very small before cracking stage. Once shear cracks developed, a rapid increase in the shear angle occurred. Beyond the ultimate load, reduced-weight concrete beams showed faster drop in the shear strength than the drop shown in the normal weight concrete beams. Such result agreed with the shown sudden failure in the reduced-weight concrete beams. Furthermore, referring to Table (6), for the normal and reduced-weight concrete beams, it can be noticed that beams without shear reinforcement showed larger shear angles at the ultimate levels. This indicated that providing shear reinforcement in the tested beams can significantly decrease the occurred shear strain.

Load – strain Records

The readings of the strain gauges in the longitudinal tensile bars at the mid-span point and the stirrups in the shear zones were obtained for all beams. The load –longitudinal tensile bars strain relationships for beams in groups A and B are illustrated in Figures (12) and (13), respectively.

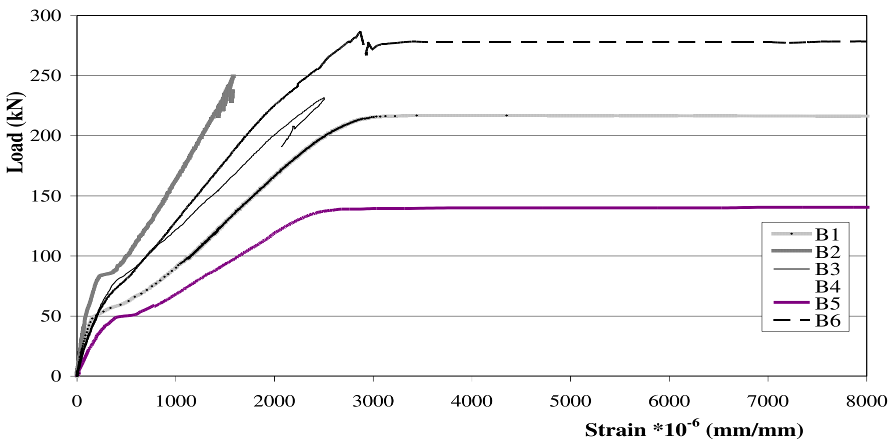


Figure (12): Load – Tension Steel Strain Relationships of Beams of Group A

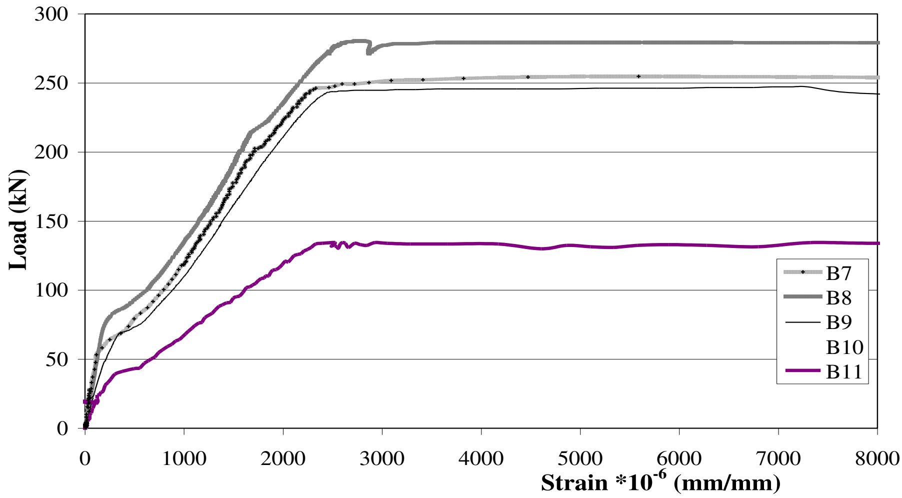


Figure (13): Load – Tension Steel Strain Relationships of Beams of Group B

In beams of a/d = 1.2 in the two groups, as mentioned before, due to the low value of shear span to depth ratio, the load path in the beams without shear reinforcement transferred in the arch action. Hence, the longitudinal bottom reinforcement acts as a tie. Therefore, despite the beams without shear reinforcement failed under shear, the longitudinal tension steel reached yield just before to the ultimate load, i.e. the strain values were more than 2200 x 10^{-6} mm/mm. Providing shear reinforcement promoted the beam action, therefore the longitudinal bottom bars did not yield in beams of low amount of stirrups, except B8. In beam B8, the high value of its concrete compressive strength and modulus of elasticity guaranteed that a significant amount of loading transfer in arch action, regardless of whether shear reinforcement was provided or not. As a result, in this beam, the longitudinal bottom reinforcement acted as a tie and yielded at the ultimate stage. Increasing the amount of stirrups (in B6) promoted also the beam action and changed the behavior of the beam to fail in flexural tension mode. As a result, the longitudinal bottom reinforcement yielded before the ultimate stage, since the beam was under reinforced.

On the other hand, increasing a/d to 2.2 (beams B5 and B11) changed the behavior of the beams to fail in flexural tension mode. In these beams, the longitudinal bottom reinforcement yielded also before the ultimate stage because the section was designed as under reinforced section.

Furthermore, the load–stirrups strain records for the beams with stirrups were obtained. Before shear cracking, the recorded strain values in the stirrups were almost zero. After that, the strain values increased as the applied load increased. Moreover, the recorded strain values of beams which failed in shear reached the yield value just before the ultimate loads (the yield strain value for stirrups equals 1660 x 10^{-6} mm/mm). On the other hand, beams which failed in flexure showed low values of strain in their stirrups (beams B5, B6 and B11). Figure (14) illustrates the relationships between the applied load and the strain values of the stirrups in the shear zones of beams B4 and B6 as examples for two tested beams which referenced shear and flexural failure, respectively.

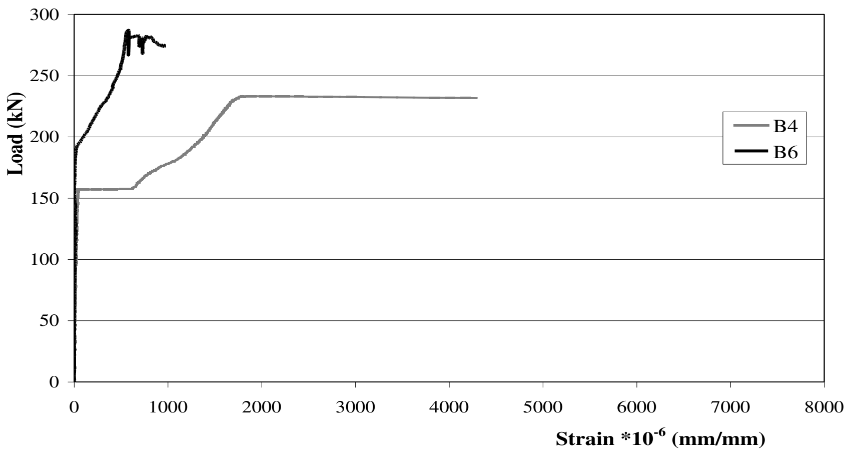


Figure (14): Load –Stirrups Strain Relationships of Beams B4 and B6

Effect of Key Variables

Based on the presented test results, an assessment was carried out for the effects of the key variables considered in the current research on the structural behavior of the tested beams. The investigated variables included weight of concrete, shear span to depth ratio, concrete grade and the ratio of stirrups. Such effects could be obtained by referring to Tables (5) and (6) as well as Figures from (5) to (14).

Effect of weight of concrete

Comparisons between the results of beams of normal-weight concrete and beams of reduced-weight concrete that possessed the same a/d ratio, the same reinforcement and the same concrete grade were carried out here. Therefore, the results of beams B1 and B2 that were made of normal-weight concrete in group A were compared with the results of beams B3 and B4 that were made of reduced-weight concrete, respectively. Moreover, the results of beams B7 and B8 that were made of normal-weight concrete in group B were compared with beams B9 and B10 that were made of reduced-weight concrete, respectively. However, the following remarks could be deduced:

- Due to the high brittleness of the reduced-weight concrete beams B3, B4, B9 and B10 experienced the sudden shear failure. Just after ultimate, these beams lost their strength quickly.
- The reduced-weight concrete beams showed less stiffness (the slope of the ascending part of the load–deflection curve) than the normal weight concrete beams.
- Insignificant reductions in the ultimate loads were observed in reduced-weight concrete beams if compared to the normal-weight concrete beams. In group A, the ultimate load of B3 was 93.7% of the ultimate load of B1 and the ultimate load of B4 was 93.5% of the ultimate load of B2. Also, in group B, the ultimate load of B9 was 95.7% of the ultimate load of B7 and the ultimate load of B10 was 91.2% of the ultimate load of B8.
- On the contrary, the reduced-weight concrete beams recorded higher cracking loads than those of the normal-weight concrete beams, especially in beams without shear reinforcement. In group A, the cracking load of B3 was 132.9% of the

cracking load of B1 and the cracking load of B4 was 104.2% of the cracking load of B2. Also, in group B, the cracking load of B9 was 112.9% of the cracking load of B7 and the cracking load of B10 was 106.9% of the cracking load of B8. Such results indicated that no enough warning could be obtained before failure in the reduced-weight concrete beams.

Effect of shear span to depth ratio

Comparisons between the results of beams B4 and B5 in group A and beams B10 and B11 in group B were carried out. Each of these two beams possessed the same properties except the shear span to depth ratio (a/d equals 1.2 for B4 and B10 and equals 2.2 for B5 and B11). Increasing a/d changed the failure mode for the tested beams from shear failure in beams B4 and B10 to tension flexural ductile failure in beams B5 and B11.This increase in the a/d promoted the beam action and decreased both cracking and ultimate loads of the tested beams. The ultimate load of B5 was 63.2% of the ultimate load of B4. Also, and the ultimate load of B11 was 60.3% of the ultimate load of B10. In addition, based on the load – mid-span records, the recorded deflection in beams of a/d = 1.2 were less than those of beams of a/d ≈ 2.2 at the same loading level. This can be attributed to the load bath for the arch action which is a direct load transfer from the loading point to the support that would result in less deflection in beams of lower a/d. Such results indicated that the increase in the shear span to depth ratio decreased the stiffness of the tested beams.

Effect of concrete grade

Comparisons between the results of the beams in group A and the similar beams in group B can give the effect of grade of concrete on the tested beams. The comparisons were carried out for five pairs of beams [(B1, B7), (B2, B8), (B3, B9), (B4, B10) and (B5, B11)]. Each of these two beams possessed the same properties except the grade of concrete. Figures (15) to (19) show the load – mid-span deflection relationships for these pairs of beams, respectively.

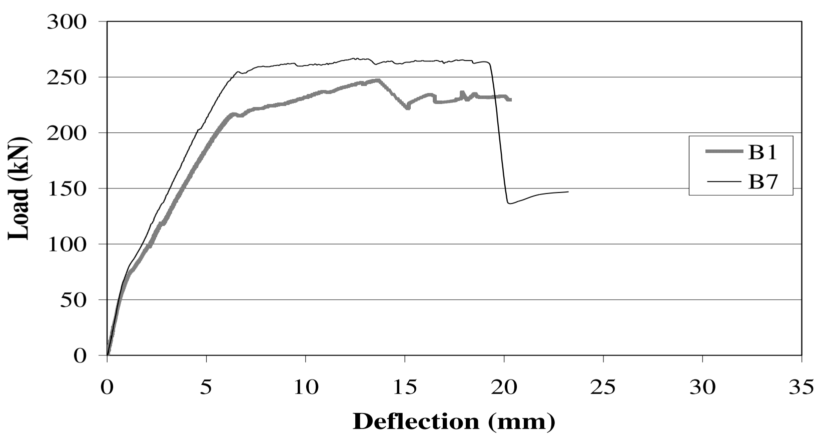


Figure (15): Effect of Grade of Concrete on the Load – Mid-span Deflection Relationships for Beams B1 and B7

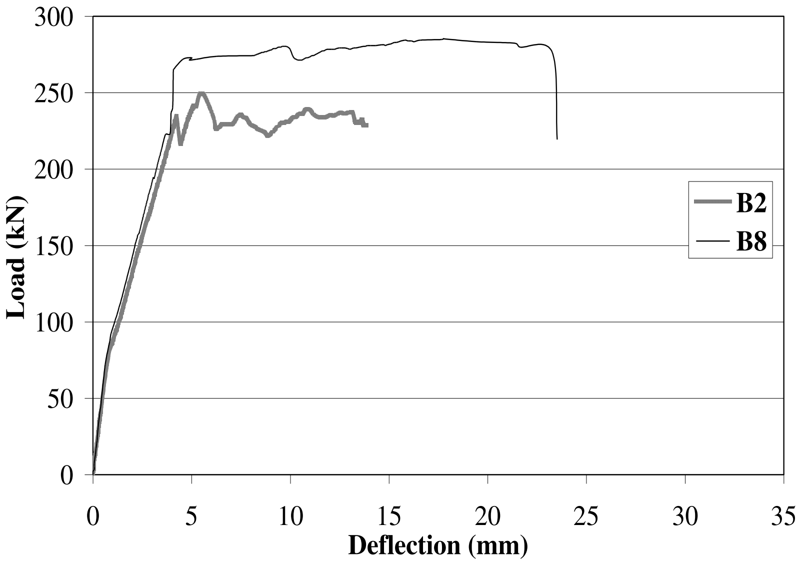


Figure (16): Effect of Grade of Concrete on the Load – Mid-span Deflection Relationships for Beams B2 and B8

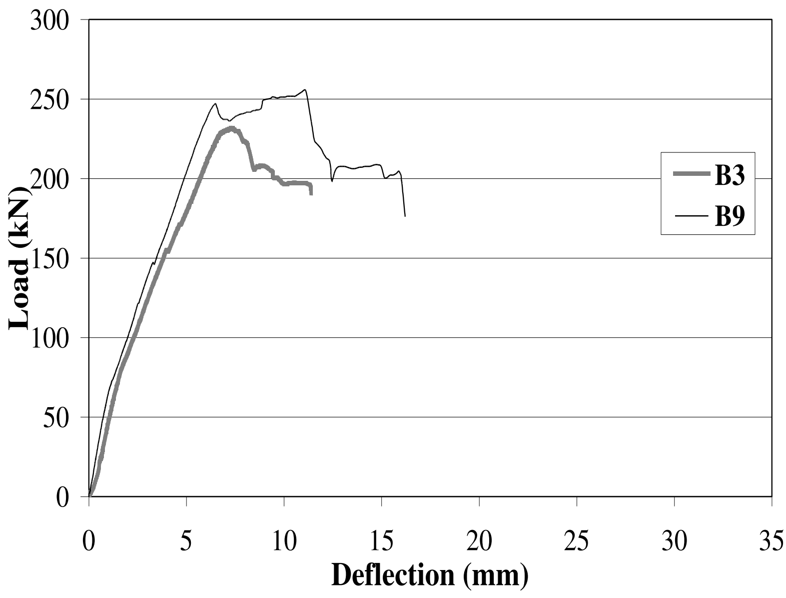


Figure (17): Effect of Grade of Concrete on the Load – Mid-span Deflection Relationships for Beams B3 and B9

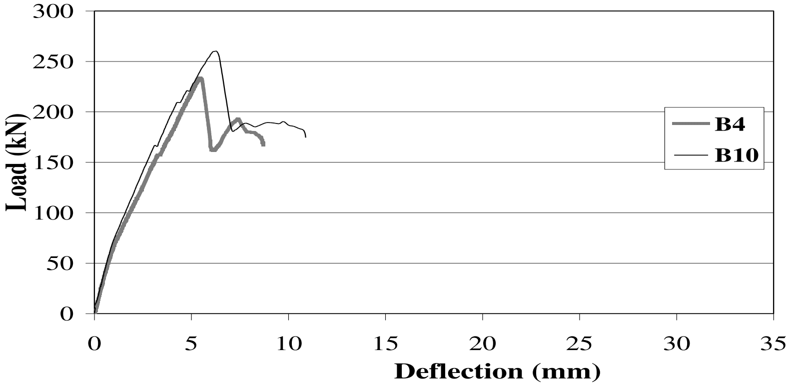


Figure (18): Effect of Grade of Concrete on the Load – Mid-span Deflection Relationships for Beams B4 and B10

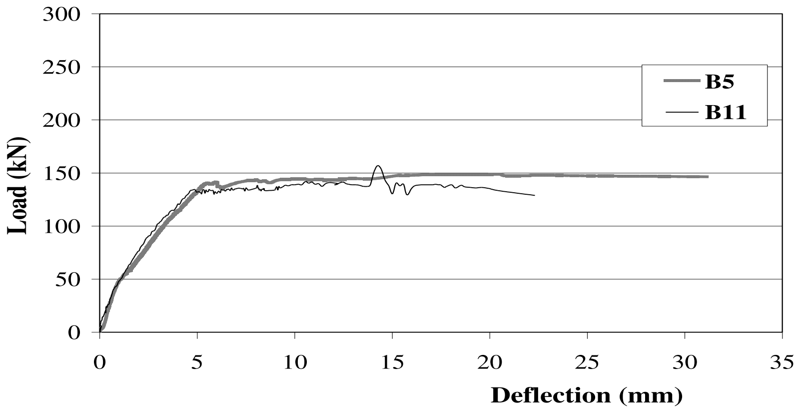


Figure (19): Effect of Grade of Concrete on the Load – Mid-span Deflection Relationships for Beams B5 and B11

It can be observed from the previous Figures that increasing the concrete grade from 30 MPa to 40 MPa caused an increase in the cracking load ranged from 4.4% (between beams B5 & B11) to 27.2% (between beams B1 & B7). Also, this increase in the concrete grade caused another increase in the ultimate capacity ranged from 5.4% (between beams B5 & B11) to 14.4% (between beams B2 & B8). This means that the increases in the cracking and ultimate loads of beams which failed in flexure were insignificant. The results indicated that, in the reduced-weight concrete beams, the impact of concrete grade on the cracking and ultimate loads was lower than that of normal-weight concrete beams. Moreover, as shown from the Figures, the observed enhancement in the stiffness of the tested beams due to the increase in the concrete grade was insignificant.

Effect of shear reinforcement

Comparisons between the results of beams which possessed the same properties except the shear reinforcement (the amount of stirrups) were carried out here. Therefore, comparisons in group A were discussed for both of beams (B1 & B2) and beams (B3, B4 & B6). Furthermore, comparisons in group B were also discussed for both of beams (B7

& B8) and beams (B9 & B10). As illustrated above, the shear span to depth ratio for all of these beams was 1.2, therefore, beams without shear reinforcement or with low amount of stirrups acted as arch with a tie. Providing the shear reinforcement in both normal and reduced-weight concrete beams can significantly decrease the occurred shear strain and increased the cracking loads. Moreover, increasing the amount of stirrups (in B6) promoted the beam action and changed the behavior of the beam to fail in flexural tension mode.

COMPARISONS WITH PREDICTIONS USING ECP – 203, [17]

Table (7) compares the experimental results for the tested beams with the predictions obtained using the Egyptian Code for Concrete Structures, (ECP-203), [17]. All of the design safety factors were taken as unity when using the ECP equations. The theoretical shear capacity and flexural capacity of all beams were listed in Table (7). Moreover, the ratios between the experimental and theoretical ultimate loads for the tested beams were also presented in Table (7).

Equation (2) is used to predict the shear strength of concrete beams with stirrups. In this Equation the ultimate shear strength of concrete beams depends only on the concrete strength and the amount of stirrups. This Equation is used for concrete with compressive strength up to 60 MPa. This Equation considers that the ultimate shear strength of concrete beams is resisted by the nominal shear strength of stirrups and half of the nominal shear strength of the concrete. In addition, the ultimate shear strengths of the tested beams were re-calculated according to Equation (3) using the full nominal concrete shear strength, see Table (7).

It should be mentioned that the Egyptian Code neglects the effect of the weight of concrete and the actual concrete modulus of elasticity; therefore, these equations were applied on the normal-weight concrete beams as well as the reduced-weight concrete beams.

In addition, the Egyptian code mentioned that if a/d ≤ 2; it is allowed to reduce the shearing force by multiplying it by the value a/2d. However, the shear stress before reduction should not be higher than $0.7\sqrt{f_{cu}/\gamma_c}$ (this condition was put for design, and hence, it was neglected here because the calculations were carried out at failure).

$$q_u = \left(\frac{q_{cu}}{2} + q_s\right) = \left(0.12\sqrt{\frac{f_{cu}}{\gamma_c}} + \frac{nA_s\left(\frac{f_y}{\gamma_s}\right)}{b\,s}\right) \qquad \text{Equation (2)}$$

$$q_u = (q_{cu} + q_s) = \left(0.24\sqrt{\frac{f_{cu}}{\gamma_c}} + \frac{nA_s\left(\frac{f_y}{\gamma_s}\right)}{b\,s}\right) \qquad \text{Equation (3)}$$

Where,

q_u is the ultimate shear strength, MPa

q_{cu} is the nominal shear strength of concrete = $0.24\sqrt{f_{cu}/\gamma_c}$, MPa

q_s is the nominal shear strength of stirrups, MPa
n is the number of branches of the stirrups
A_s is area of one branch of the stirrup, mm^2
s spacing between stirrups, mm
b beam width, mm
f_{cu} is the cube concrete compressive strength, MPa, see Table (4).
f_y is the yield strength of shear reinforcement, MPa, see Table (2).
γ_c is strength reduction factor for concrete *(will be taken here =1)*
γ_s is strength reduction factor for steel *(will be taken here =1)*

As a result, the ultimate shear load can be calculated as shown in Equations (4) & (5):

If $a/d > 2$ $\quad P_u/2 = q_u * bd$ $\quad$ Equation (4)

If $a/d \leq 2$ $\quad P_u/2 = (2d/a)(q_u * bd)$ $\quad$ Equation (5)

Where, P_u is the ultimate shearing Load, N

Table (7): ECP -203 Predictions versus the Experimental Values

Beam ident.	Theoretical Values				Experimental Values		Experiment / theoretical	
	Theoretical Ultimate Shear Load, kN		Theoretical Ultimate Flexural Load, kN	Theoretical Mode of Failure	Experimental Ultimate Load, kN	Experimental Mode of Failure		
	Total q_{cu}	$q_{cu}/2$					Total q_{cu}	$q_{cu}/2$
B1	194.7	97.3	235.3	Shear	247.2	Shear	1.27	2.54
B2	274.7	181.0	234.5	Shear or Flexural	249.4	Shear	0.91	1.38
B3	190.4	95.2	234.9	Shear	231.7	Shear	1.22	2.43
B4	274.1	180.7	234.4	Shear or Flexural	233.3	Shear	0.85	1.29
B5	169.9	109.0	129.5	Shear or Flexural	148.9	Flexural Ten.	1.15	
B6	363.5	269.1	234.6	Flexural Ten.	286.9	Flexural Ten.	1.22	
B7	208.2	104.1	237.0	Shear	266.8	Shear	1.28	2.56
B8	303.3	195.4	237.6	Shear or Flexural	285.4	Shear Comp.	0.94	1.46
B9	210.8	105.4	237.2	Shear	255.3	Shear	1.21	2.42
B10	294.0	190.7	236.8	Shear or Flexural	260.4	Shear	0.89	1.37
B11	177.2	114.8	130.4	Shear or Flexural	157.0	Flexural Ten.	1.20	

For beams without stirrups, B1, B3, B7 and B9, the calculated shear capacities using the total values of nominal shear strength of concrete were significantly less than the calculated flexural capacity. This means that shear failure is the theoretical governing mode of failure in these beams, which relates well with the experimental results. All the theoretical predictions are quite conservative, especially in the calculations of the normal-weight concrete beams. This means that the effect of arch action is still underestimated in the Egyptian Code. It can be observed also that the theoretical flexural capacities of these beams were less than the experimental ultimate loads. Nevertheless, flexural failure did not occur. Such results can be referred also to the large amount of load that transferred in the arch action.

Calculations of beams B2, B4, B8 and B10 which possessed low amount of stirrups and a/d = 1.2 showed that the values of nominal shear strength of concrete were significantly less than the calculated flexural capacity if half of the nominal shear strength of the concrete was taken into consideration. But if the full nominal concrete shear strength was taken into account, the calculated flexural capacity would be slightly less than the shear capacity. Hence, the theoretical governing mode of failure in these beams may be shear failure (in case of $q_{cu}/2$) or flexural failure (in case of q_{cu}). The low value of a/d and the low amount of stirrups promote the arch action; therefore, flexural failure did not occur also in these beams. Since these beams actually failed in shear, the ratios between the experimental and theoretical ultimate shear loads were calculated for the two assumptions (full q_{cu} and half of q_{cu}). The results indicated that Equation (2) gave conservative values when taking $q_{cu}/2$ into account, but Equation (3) was unsafe when taking full q_{cu} in the calculations.

In addition, calculations of beams B5 and B11 which possessed low amount of stirrups and a/d ≈ 2.2 showed that the values of nominal shear strength of concrete were slightly less than the calculated flexural capacity if half of the nominal shear strength of the concrete was taken into consideration (Equation (2)). But if the full nominal concrete shear strength was taken into account, (Equation (3)), the calculated flexural capacity would be significantly less than the shear capacity. Hence, the theoretical governing mode of failure in these beams also may be shear failure (in case of $q_{cu}/2$) or flexural failure (in case of q_{cu}). The increased value of a/d promotes the beam action; therefore, flexural failure occurred in these beams. Also, these beams actually failed in flexure; therefore, the ratios between the experimental and theoretical ultimate flexure loads were calculated. The results indicated that the experimental ultimate loads were bigger than the calculated loads by 15% and 20% for B5 and B11, respectively. Such results indicated that, despite the increasing in the shear span to depth ratio and the appeared flexural failure mode, there is a mild part of the applied load still transfer in the arch action.

On the other hand, increasing the amount of stirrups in B6 attracted the load to transfer in the beam action and then promoted the beam action. Moreover, extra shear stiffness was provided by closed stirrups. As a result, the behavior of the beam changed to fail in flexural tension mode. Table (7) showed that the calculated flexural capacity was significantly less than the calculated shear capacity. This insures that flexural failure is the theoretical governing mode of failure in this beam, which relates well with the experimental results. The ratio between the experimental and theoretical ultimate loads was 1.22. A result indicated that, despite the increasing on the shear reinforcement, there is a mild part of the applied load still transfer in the arch action which can be referred to the low value of a/d.

CONCLUSIONS

Based on the results of the current experimental work in, the following conclusions could be drawn:

1. The tensile/compressive strength ratio for the reduced-weight concrete was lower than that of normal-weight concrete.
2. The static modulus of elasticity of the reduced-weight concrete is much less than that of normal-weight concrete which possesses the same grade.
3. The first shear cracking loads of the reduced-weight concrete beams were greater than those of the comparative normal-weight concrete beams, i.e. no enough warning could be obtained before the failure of the reduced-weight concrete beams.
4. The reduced-concrete beams showed a slight reduction in the load carrying capacity, stiffness and ductility when compared to the normal-weight concrete beams.
5. Despite the presence of arch action, the reduced-weight concrete beams of low value of a/d and low amount of stirrups showed instantaneous modes of failure.
6. Increasing the shear reinforcement improved the shear capacity, promoted the beam action, attracted greater tensile stress in the web, prevented shear cracking to develop, decreased the shear strain and prevented the sudden failure mode.
7. Increasing the shear span to depth ratio promoted the beam action, decreased the cracking and ultimate loads and stiffness and increased the ductility of the reduced-weight concrete beams.
8. The observed enhancement in ultimate carrying capacity of the reduced weight-concrete beams due to the increase in the concrete grade was lower than that of the normal-weight concrete beams.
9. Although the ultimate carrying capacities of the reduced-weight concrete beams were less than those of the normal-weight concrete beams, the theoretical predictions obtained using the Egyptian Code for Concrete Structures, (ECP 203), were quite conservative. This could be attributed to that the effect of arch action is still underestimated in the Egyptian Code.

ACKNOWLEDGMENT

The author is grateful to Eng. Shady Mohamed and the staff of "Building Materials and Quality Control Laboratory" of Housing and Building National Research Center for their thankful effort through the experimental work of this research.

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سلوك القص للكمرات الخرسانية المسلحة خفيفة الوزن

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أستاذ مساعد بمعهد بحوث المواد وضبط الجودة بالمركز القومي لبحوث الأسكان والبناء – القاهرة – جمهورية مصر العربية

يتناول البحث الحالي دراسة السلوك الإنشائي للكمرات الخرسانية المسلحة خفيفة الوزن المصنوعة باستحدام ركام الليكا الخفيف كإحلال جزئي للركام الطبيعي (بالحجم) تحت تأثير إجهادات القص. وقد تضمن البرنامج العملي اختبار إحدى عشرة كمرة خرسانية مسلحة بينها سبع كمرات خفيفة الوزن بينما تم صب الأربع كمرات الباقية بالخرسانة ذات الوزن العادي لاستخدامها كعينات تحكم. ولقد تم إجراء الاختبارات باستخدام حملين مركزين رأسيين ومتماثلين. وقد تم خلال البحث الحالي دراسة تأثير عدد من المتغيرات مثل وزن الخرسانة والنسبة بين بحر القص وعمق الكمرة ورتبة الخرسانة بالإضافة إلى كمية الكانات في الكمرة. ولقد استعرض البحث الحالي تحليل تفصيلي لنتائج الاختبارات في ضوء متغيرات الدراسة. وتم عقد العديد من المقارنات التي تناولت شكل الكسر والعلاقة بين الحمل المؤثر على الكمرة والترخيم الحادث بها وكذلك العلاقة بين الحمل والانفعال الحادث بكلٍ من صلب التسليح الرئيسي والكانات في منطقة القص كما تم دراسة شكل العلاقة بين إجهادات وانفعالات القص وقيمة أول حمل قص يسبب تشرخ الكمرة بالإضافة إلى قيمة حمل الكسر (أقصى حمل تتحمله الكمرة المختبرة) وجساءة الكمرة وممطوليتها.

كما تناول البحث مقارنة نتائج الاختبارات بالكود المصري لتصميم وتنفيذ المنشآت الخرسانية رقم 203 إصدار 2007. وبالرغم من أن نتائج الاختبارات المعملية أظهرت أن الكمرات الخفيفة تتحمل حمل أقل من الكمرات عادية الوزن وكذلك كانت جساءتها وممطوليتها أقل من نظيرها في الكمرات عادية الوزن إلا أن الحمل المحسوب باستخدام معادلات الكود المصري كان دائماً أقل من النتائج المعملية. ويرجع ذلك إلى أن تأثير توزيع الإجهادات على شكل القوس مازال لايؤخذ كاملاً بالكود المصري.

<u>الكلمات الدالة</u>: الكمرات الخرسانية خفيفة الوزن، سلوك القص، شكل الكسر، حمل القص الذي يسبب أول شرح، حمل الكسر