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Behavior of Lightweight Concrete Columns Under Eccentric Loads (Parametric Study)

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ملخص البحث

يقدم هذا البحث تحليلا لسلوك الخرسانة الخفيفة الوزن عمليا ونظريا من خلال استخدام حبيبات البوليسترين كحل بديل جزئي لخفض وزن وحدة الخرسانة الجافة من ٢٣ كيلو نيوتن/ متر٣ إلى ١٨،١ كيلو نيوتن/ متر٣ و دراسة تاثير زيادة نسبة الحديد الطولي ومعامل النحافة لهذة الاعمدة.

Abstract

The previous experimental work conducted by the authors for reinforced lightweight concrete columns under concentric and eccentric loads is extended analytically in this paper using non-linear finite element analysis. A detailed parametric study is done to quantify the effect of the amount of longitudinal reinforcement, the amount of transverse reinforcement, and eccentricity ratio. Based on the analysis, understanding the mechanism of the reinforced lightweight concrete columns under concentric and eccentric loads.

Keywords: Columns; Confinement; Eccentricity; Lightweight concrete (LWC); polystyrene foam.

1. Introduction

Structural lightweight concrete mixtures can be designed to achieve similar strengths as normal weight concrete. The same is true for other mechanical and durability performance requirements. Structural lightweight concrete provides a more efficient strength-to-weight ratio in structural elements. In most cases, the marginally higher cost of lightweight concrete is offset by size reduction of structural elements, less reinforcing steel and reduced volume of concrete, resulting in lower overall cost could have impact on the design of the foundations. Use of reduced unit weight concrete could also lead to great advantages for the precast industry by reducing the transportation cost. Furthermore, the reduced mass will reduce the lateral load that will be imposed on the structure during earthquakes, hence simplifying and reducing the lateral load carrying system.

Y.C. Kan, L.H. Chen. C.H. Wu, C.H. Huang , T. Yen, and W.C. Chen (2012) studied the behavior of LWC Columns under axial load. This research aims to figure out the behavior of axial load and size effect of reinforced lightweight aggregate concrete column. Both normal weight aggregate concrete (NWC) and lightweight concrete (LWC) were used to cast three similar square columns with various sizes. The slenderness ratio of all columns is 21.6. Two concrete compressive strengths of 23 and

33 MPa were selected. Test results revealed that the failure model of LWC columns is similar to that of NWC column. The tension crack appeared at the middle part of both concrete columns. Under the same axial loading, the displacement of LWC column is larger than that of NWC column. The measured ultimate strengths of the small- and medium-size LWC columns are close to the computed values of ACI nominal strength indicating that the ACI equation of nominal strength is quite applicable for the strength prediction of small-size and medium-size LWC columns. In addition, the ratio of (ultimate strength / nominal strength) of LWC column decreases with the increase of column size; when the strength of concrete increases, the ratio of (ultimate strength / nominal strength) decreases. These results indicate that LWC columns have the incentive of size effect. Therefore, the size effect should be considered in the design of LWC columns.

M. R. Esfahani and A. Kadhkodaee (2008) studied strength and ductility of reinforced concrete columns made of lightweight aggregates under eccentric loading. In this research, the strength and ductility of reinforced concrete columns made of lightweight were tested. The shape and spacing of transverse reinforcement and compressive strength of specimens were varied. For all specimens, the eccentricity of axial loading was 60 mm. Test results showed that the confinement of transverse reinforcement has a positive effect on the ultimate strength and ductility. Test results have been compared with the provisions of ACI Code for normal strength concrete. The comparison shows that the reinforced concrete columns made of natural lightweight aggregates can be used in structures if they include appropriate transverse reinforcement and have a good mix design.

Shamim A. Sheikh and Ching-Chung Yeh studied tied concrete columns under axial load and flexure. Fifteen specimen (30.5 x 30.5 x 274) cm long reinforced concrete columns were tested under flexure to large inelastic deformations while simultaneously subjected to constant axial load. The main purpose of this research was to investigate the behavior of column sections confined by rectilinear ties. Major variables considered in this program included: (1) Distribution of longitudinal and lateral steel, including unsupported longitudinal bars and supplementary cross-ties with 90° hooks; (2) level of axial load (0.46 F’ ARgR, to 0.78 F’ ARgR) ; (3) amount of lateral steel (0.8% to 1.6% of core volume), and (4) spacing of ties (2- 1/8 in. to 6-13/16 in. [54-173 mm]). Test results indicate that a larger number of laterally supported longitudinal bars results in better performance of columns. Unsupported longitudinal bars and cross-ties with 90° hooks confine concrete effectively only at small deformations and result in rapid deterioration of column behavior at a later stage, particularly under high axial load levels. The amount of lateral steel and the level of axial load have significant effects on the column behavior.

- A_g = gross cross-sectional area of the column.

Hussein O. Okail (2008) explained a group of six medium scale beams coded N_1, L_1, L_2, L_3, L_4, and L_5, Beams had a rectangular cross-section of dimensions 150 mm x 300 mm over a total length of 3000 mm and a clear span of 2700 mm. Shear reinforcement was increased in the region outside the two concentrated loads near the support to clearly pronounces the parameters of the study, the flexural reinforcement ratio through beams L_1, L_4 and L_5, having reinforcement ratios of 0.80%, 1.35% and 1.90%, respectively, Finally, the amount of stirrups located at the constant moment zone is discussed through beams L_2, L_1 and L_3, having stirrups of zero, 5Φ8/m' and 10Φ8/m', respectively. In this respect and for ease and simplicity of calculations, the equivalent rectangular stress block [Whitney block, adopted in both ACI 318-02 (2002) and ECCS 203-01 (2001)] is used. The analysis also demonstrated that, for the latter block to have the well-established uniform, its stress block parameter β_1 should be 0.72 the neutral axis depth, c. for all other parameters we make this research.

2. EXPERIMENTAL PROGRAM USED IN MODELING VERIFICATION

This section describes the experimental work previously done by the authors, Maged Tawfik, Sherif Elwan, Hosam Seleem and Amr Abdelrahman (2017) which will be used for verification purposes.

2.1 Materials Used, Specimens Details and Test Setup

All specimens are made from one concrete mix with the proportion shown in Table 1. The target standard 28-days compressive cube strength f_{cu} = 38MPa, and according to the ACI the equivalent compressive cylinder strength, $f_c^{'}$ = 30 MPa. The results of testing cubes have satisfied the target strength. Ordinary locally available concrete constituent materials have been used to manufacture the test specimens. The used silica fume consists of very fine vitreous particles with a surface area on the order of 20000 m2/Kg, The normal silica fume range from 5 to 20 percent of Portland cement content. The used polystyrene foam is type of plastic produced from styrene. Polystyrene foam has an excellent resistance to moisture, imperiousness to rot, mildew and corrosion. The used super-plasticizer was of a liquid form under trade name, VISCOCONCRETE#3425 which is in compliance with ASTM C494, 2001of type V with dose of cement about 3%. It permits a reduction of 30% of the water content in concrete mixture.

Cement (Kg)	Silica Fume (Kg)	Coarse Agg. (Kg)	Sand (Kg)	Polystyrene Foam (Liter)	Super Plasticizer (Liter)	Water (Liter)	Fiber (kg)
450	**40**	**630**	**630**	**330**	**13.5**	**139**	**0.9**

Table 1: Mix design proportion /m^3 (Average Strength= 38 MPa)

The reinforcement used in the specimens consisted of longitudinal and transverse reinforcement, High grade steel for 10mm and 12mm diameter were implemented. The average tensile yield strength (f_y) was 460 MPa, the ultimate strength (f_u) was 530 MPa and the modulus of elasticity (E_s) 193 GPa. Mild steel for 8mm diameter was implemented with average tensile yield strength (f_y) was 280 MPa, the ultimate strength (f_u) was 460 MPa and the modulus of elasticity (E_s) 117 GPa.

Polypropylene MasterFiber®012(BASF product) is used by 19mm length for plastering at the rate of 0.9kg/m^3, with tensile strength 350N/mm^2.

Six specimens was divided into two groups with longitudinal reinforcement 4Φ10 with ratio to concrete cross section 0.8% used for all specimens as shown in Fig.1, The first group have stirrups Φ8 with ratio 0.4% and the second group have stirrups Φ10 with ratio 0.6%, All columns have square cross section with dimension 200x200mm and 1600mm height, Specimens have column head with dimensions 400x200mm and tapered depth from 100 to 200mm, as shown in Fig.1. The coding, concrete dimensions, reinforcement details, and eccentricity ratios of the aforementioned columns are also summarized in Table 2.

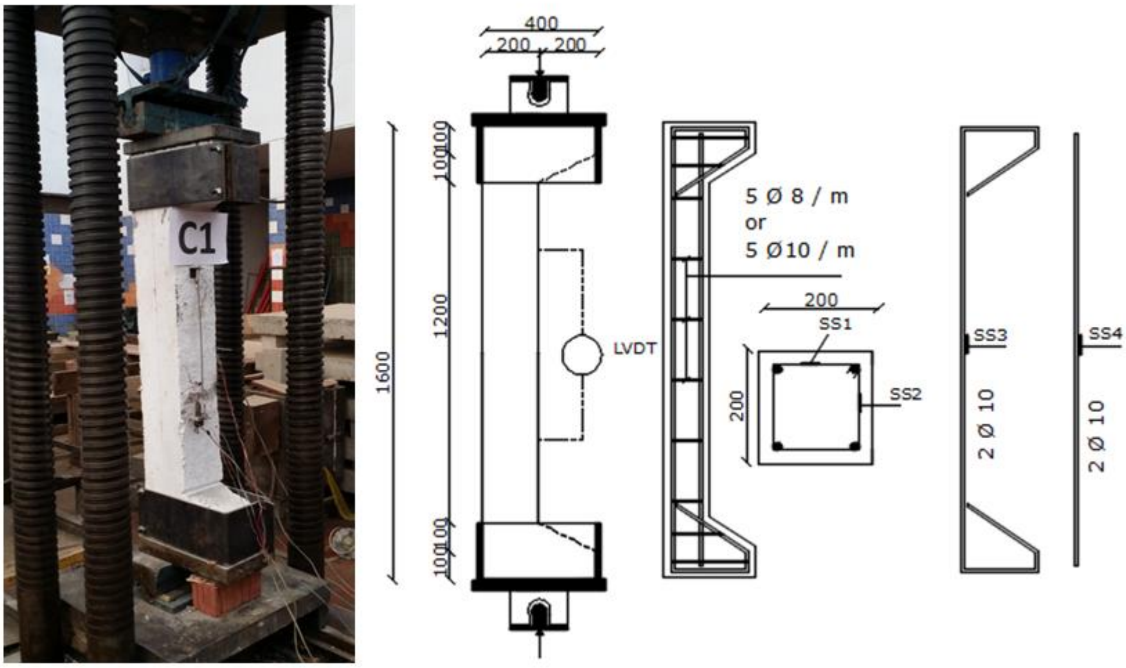


Fig.1: Test Setup of specimens

Table 2: Specifications of the tested columns.

Group	ID	Cross section (mm)	Specimens height (mm)	Longitudinal Reinforcement	Transverse Reinforcement	Eccentricity/ Thickness (e/t)
(A)	C1	200x200	1600	4 Φ 10	Φ 8@200mm	0
	C2	200x200	1600	4 Φ 10	Φ 8@200mm	0.2
	C3	200x200	1600	4 Φ 10	Φ 8@200mm	0.4
(B)	C4	200x200	1600	4 Φ 10	Φ 10@200mm	0
	C5	200x200	1600	4 Φ 10	Φ 10@200mm	0.2
	C6	200x200	1600	4 Φ 10	Φ 10@200mm	0.4

2.2 Experimental Test Results

Table (3) shows the experimental results of the tested specimens. For axially loaded columns (C1&C4) the initiation of cracking and crushing occurred at zone near the lower middle of column, the failure was associated with concrete crushing near the column ends, this may attributed to the high concentration of stresses near the column head, as shown in Fig.2.

Columns (C2&C3&C5&C6) tested under static monotonic uni-axial loading, the initiation of cracking and crushing occurred at the middle height of column, When the eccentricity ratio (e/t) increased the cracking width increased and the failure was crushing failure at the middle height of column with wide cracks in the tension side, and when we increased the transverse reinforcement ratio crack width decreased and late appearance of first crack in column, as shown in Fig.2.

Table 3: Summarized strains on specimens at first crack stage and maximum Load stage.

I.D	At First Crack					At Maximum Load				
	Crack Load (K.N)	Concrete Strain x10^{-6}	Long. Rft. strain at inner side x10^{-6}	Long. Rft Strain at outer side x10^{-6}	Long. Rft Strain at outer side x10^{-6}	Failure Load (K.N)	Concrete Strain x10^{-6}	Long. Rft. strain at inner side x10^{-6}	Long. Rft Strain at outer side x10^{-6}	Long. Rft Strain at outer side x10^{-6}
C1	960.1	-1018	-1200	-1150	294	1207	- 170	-168	-160	382.4
C2	473.4	-1080	-1000	-820	185	712.1	-210	-163	-121.5	350
C3	220.2	-1050	*****	670	176	367.3	-198	******	105	315.3
C4	1012	-1034	-1280	-1231	296	1350	-182	-181.8	-175.2	446.6
C5	501.8	-1112	-1150	****	195	780.7	-223	-172	******	390
C6	340.2	-1090	-1120	768	180	445.7	-205	-165	134	360

****** Strains cannot be determined because of strain gauges damage during casting

Colum (C1&C4) Colum (C2&C5**)** Colum (C3&C6**)**

Fig.2: Crushing of specimens under eccentric loading.

3 Non-linear Finite elements Analysis

3.1 Geometry

The same experimental setup was simulated in the numerical model to ensure full compatibility between tested and simulated columns.

3.2 Modelling

All of the specimens were simulated with ANSYS 15.0, which offers a series of very robust nonlinear capabilities for analysis. A solid element, SOLID65, is used to model the concrete in ANSYS. The element is capable of simulating plastic deformation, and cracking in three orthogonal directions. To model the non-linear behavior of concrete, ANSYS requires linear isotropic and multi-linear isotropic as well as some additional concrete material properties to simulate the real concrete behavior. The failure criterion of concrete was the William-Warnke five parameters model. Stress-strain relationship used for this study is based on work done by Kachlakev et al 2001. The peak strength f_c=30 MPa, initial Young's modulus E_c =19444 MPa. A LINK180 element is used to model the steel reinforcement. At each node, degrees of freedom are identical to those for the SOLID65. The column mesh was selected such that the node points of the solid elements will coincide with the actual reinforcement locations. The yield stresses for longitudinal and web reinforcement, are equal 460, and 280 MPa, respectively. The elastic modulus and poisson's ratio are equal 200000 MPa and 0.3 respectively. The solid element SOLID185 was used for the steel plates at loading points, steel plates added at the support locations to avoid stress concentration problems. The steel plates were assumed linear elastic materials with elastic modulus equal to 2000000 MPa, and Poisson's ratio of 0.3. Nodes of the solid elements (solid 185) were connected to those of adjacent concrete solid elements (solid 65) in order to satisfy the perfect bond assumption.

Fig.3 shows the finite element model of the square column.

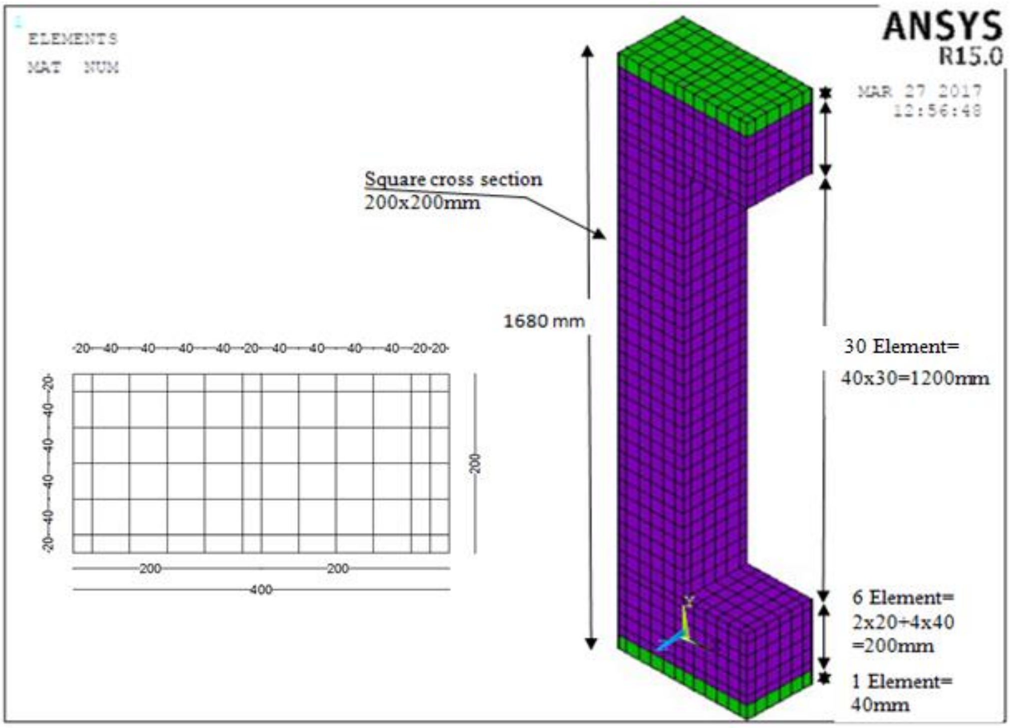


Fig.3: Finite element model of the square column.

3.3 Verification of the model

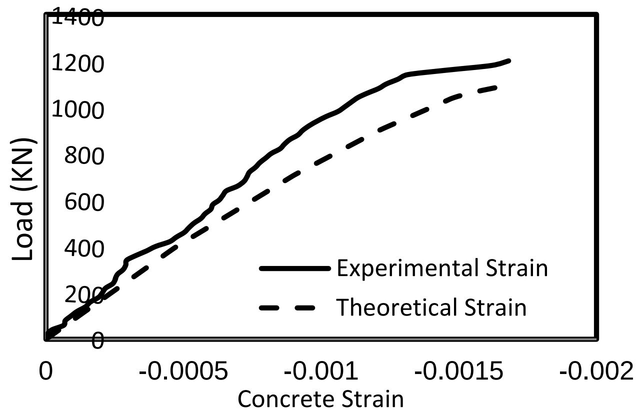
The goal of the verification of the finite element model is to ensure that the proposed elements, material properties, real constants and convergence criteria are adequate to model the response of the square lightweight concrete columns.

During the test procedure, axially loaded columns (C1&C4) the initiation of cracking and crushing occurred at zone near the lower middle of column, The failure was associated with concrete crushing near the column ends, This may attributed to the high concentration of stresses near the column head, Columns (C2&C3&C5&C6) tested under static monotonic uni-axial loading, the initiation of cracking and crushing occurred at the middle height of column, When the eccentricity ratio (e/t) increased the cracking width increased and the failure was crushing failure at the middle height of column with wide cracks in the tension side, and when we increased the transverse reinforcement ratio crack width decreased and late appearance of first crack in column.

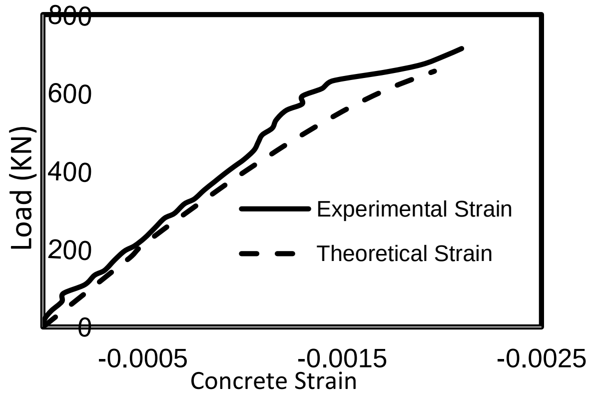
The columns were tested in monotonically increasing load until the ultimate load and subsequently total failure of the column occurred. Table 4 shows comparisons between the experimental results of failure load and the theoretical results of failure load from analytical model. Fig.4. shows the Load concrete- strain of both theoretical and experimental work. In all cases, the error has a minimum of -1.05% and a maximum of 9.1%.

Table 4: Summary of experimental and theoretical results

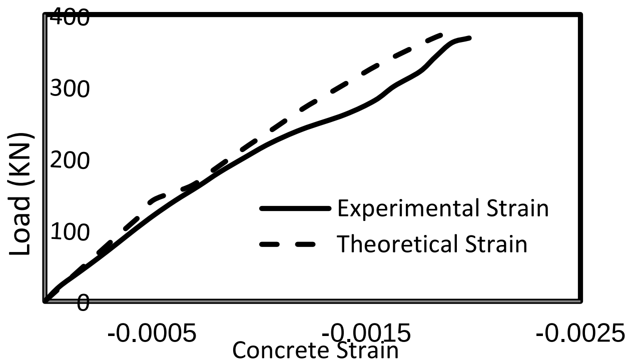
Group	Specimen	Failure Load (kN)		
		Experimental	Theoretical	% Diff.
(A)	C1	1200.00	1090.80	9.1
	C2	712.80	654.82	8.1
	C3	367.28	371.13	-1.05
(B)	C4	1349.77	1249.2	7.45
	C5	780.72	714.72	8.45
	C6	445.75	412.4	7.48



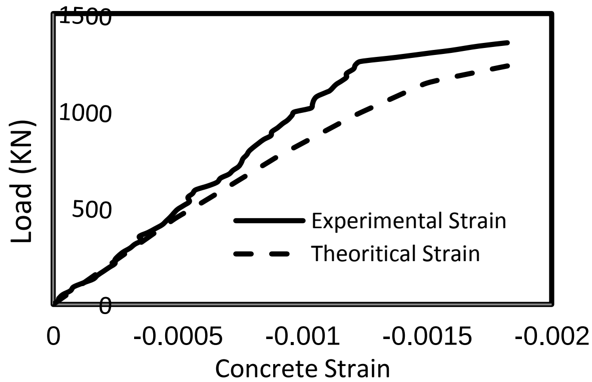
a) C1



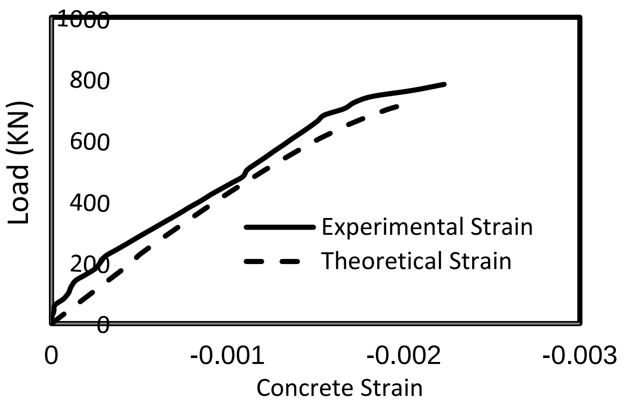
b) C2



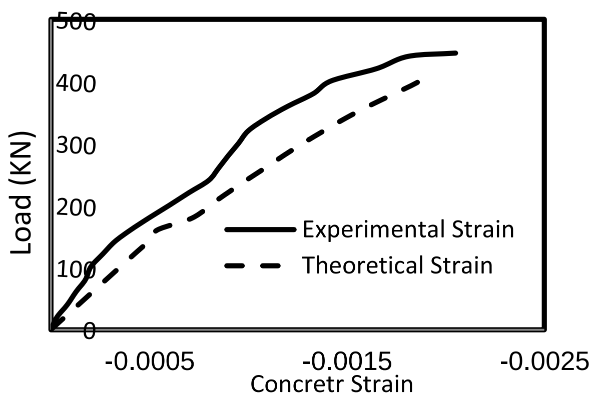
c) C3



d) C4



e) C5



f) C6

Fig.4: Load concrete- strain of both theoretical and experimental work

4 . The Parametric Study

Two parameters were taken into consideration through this study, the effect of slenderness ratio and longitudinal reinforcement ratio in square lightweight concrete columns.

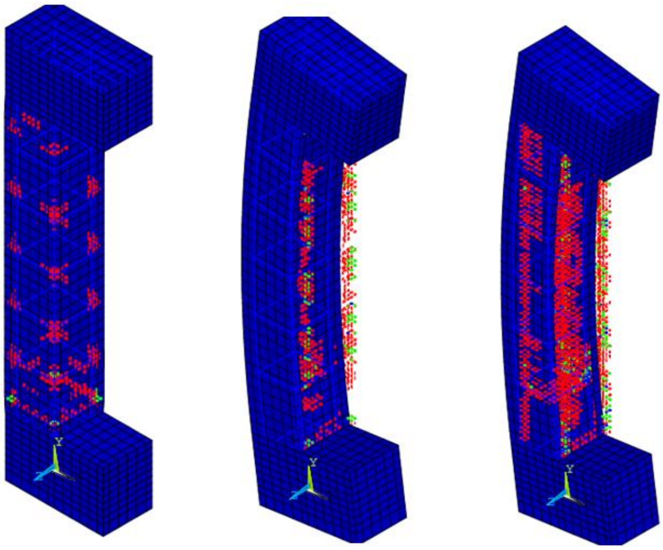
The first group varying in the longitudinal reinforcement ratios (0.8%& 1.1%& 2%) at different eccentricity ratios (with e/t=0, e/t=0.2, and e/t=0.4). The second group varying in the overall column slenderness ratio (6&15) at different eccentricity ratios. Table 5 shows the details of the parametric study series and the theoretical results from analytical model.

Table 5: Parametric study database of reinforcement lightweight concrete columns

#	ID	Eccentricity/ Thickness (e/t)	Specimens height (mm)	Slenderness ratio (h/t)	Transverse Reinforcement	Longitudinal Reinforcement
1	$C0/6/T_1/L_1$	0	1600	6	5Φ8/m (0.4%)	4Φ10 (0.8%)
2	$C0/6/T_1/L_2$	0	1600	6	5Φ8/m (0.4%)	4Φ12 (1.1%)
3	$C0/6/T_1/L_3$	0	1600	6	5Φ8/m (0.4%)	4Φ16 (2%)
4	$C0/15/T_1/L_1$	0	3400	15	5Φ8/m (0.4%)	4Φ10 (0.8%)
5	$C0/15/T_1/L_2$	0	3400	15	5Φ8/m (0.4%)	4Φ12 (1.1%)
6	$C0/15/T_1/L_3$	0	3400	15	5Φ8/m (0.4%)	4Φ16 (2%)
7	$C0.2/6/T_1/L_1$	0.2	1600	6	5Φ8/m (0.4%)	4Φ10 (0.8%)
8	$C0.2/6/T_1/L_2$	0.2	1600	6	5Φ8/m (0.4%)	4Φ12 (1.1%)
9	$C0.2/6/T_1/L_3$	0.2	1600	6	5Φ8/m (0.4%)	4Φ16 (2%)
10	$C0.2/15/T_1/L_1$	0.2	3400	15	5Φ8/m (0.4%)	4Φ10 (0.8%)
11	$C0.2/15/T_1/L_2$	0.2	3400	15	5Φ8/m (0.4%)	4Φ12 (1.1%)
12	$C0.2/15/T_1/L_3$	0.2	3400	15	5Φ8/m (0.4%)	4Φ16 (2%)
13	$C0.4/6/T_1/L_1$	0.4	1600	6	5Φ8/m (0.4%)	4Φ10 (0.8%)
14	$C0.4/6/T_1/L_2$	0.4	1600	6	5Φ8/m (0.4%)	4Φ12 (1.1%)
15	$C0.4/6/T_1/L_3$	0.4	1600	6	5Φ8/m (0.4%)	4Φ16 (2%)
16	$C0.4/15/T_1/L_1$	0.4	3400	15	5Φ8/m (0.4%)	4Φ10 (0.8%)
17	$C0.4/15/T_1/L_2$	0.4	3400	15	5Φ8/m (0.4%)	4Φ12 (1.1%)
18	$C0.4/15/T_1/L_3$	0.4	3400	15	5Φ8/m (0.4%)	4Φ16 (2%)

4.1 Results and Discussion

The finite element predicted failure loads, concrete strain at mid-height, and the maximum longitudinal and transverse steel strains for the parametric study database are displayed in Table 6, and Fig. 5 shows the cracks propagation before failure from finite element model.



a) e/t=0 b) e/t=0.2 c) e/t=0.4

Fig.5: Cracks propagation before failure from finite element model.

Table 6: Parametric study database-failure loads, and strains at failure load.

#	ID	Failure Load (K.N)	Concrete Strain	Long. RFT strain at compression	Long. RFT Strain at Tension	Trans. RFT. Strain
1	$C0/6/T_1/L_1$	1090.80	-0.001660	-0.001650	-0.001650	+0.000400
2	$C0/6/T_1/L_2$	1140.10	-0.001740	-0.001740	-0.001740	+0.000420
3	$C0/6/T_1/L_3$	1255.60	-0.001910	-0.001910	-0.001910	+0.000480
4	$C0/15/T_1/L_1$	828.94	-0.001260	-0.001252	-0.001252	+0.000278
5	$C0/15/T_1/L_2$	862.69	-0.001310	-0.001300	-0.001300	+0.000291
6	$C0/15/T_1/L_3$	950.78	-0.001400	-0.001400	-0.001400	+0.000307
7	$C0.2/6/T_1/L_1$	654.82	-0.001960	-0.001620	-0.000112	+0.000381
8	$C0.2/6/T_1/L_2$	670.33	-0.002010	-0.001680	-0.000113	+0.000405
9	$C0.2/6/T_1/L_3$	735.22	-0.002190	-0.001830	-0.000115	+0.000434
10	$C0.2/15/T_1/L_1$	459.00	-0.001370	-0.001228	-0.000100	+0.000253
11	$C0.2/15/T_1/L_2$	479.00	-0.001430	-0.001262	-0.000103	+0.000261
12	$C0.2/15/T_1/L_3$	543.69	-0.001550	-0.001330	-0.000109	+0.000285
13	$C0.4/6/T_1/L_1$	371.13	-0.001846	-0.001568	+0.001149	+0.000322
14	$C0.4/6/T_1/L_2$	399.95	-0.002020	-0.001650	+0.001212	+0.000352
15	$C0.4/6/T_1/L_3$	475.18	-0.002310	-0.001800	+0.001350	+0.000371
16	$C0.4/15/T_1/L_1$	285.82	-0.00148	-0.001210	+0.000878	+0.000249
17	$C0.4/15/T_1/L_2$	314.67	-0.00164	-0.001266	+0.000930	+0.000255
18	$C0.4/15/T_1/L_3$	373.81	-0.00197	-0.001360	+0.001040	+0.000260

4.2 Failure loads and strains compression

Table 7: Summarized percentage of changes in all parameters for tested columns.

Comparison Type	Group	Long. RFT Ratio	Percent of changing at Maximum Load%				
			Failure Load	Concrete Strain	Long. RFT strain at inner side (comp.)	Long. RFT Strain at outer side (tension)	Trans. RFT. Strain
Changing in longitudinal reinforcement ratios from 0.8% to 1.1%, and 2%	e/t=0	1.1%	4.52%	4.82%	5.45%	5.45%	5%
		2%	15.11%	15.06%	15.76%	15.76%	20%
	e/t=0.2	1.1%	2.36%	2.55%	3.70%	0.89%	6.30%
		2%	12.28%	11.73%	12.96%	2.68%	13.91%
	e/t=0.4	1.1%	7.77%	9.43%	5.23%	5.48%	9.32%
		2%	28%	25.14%	14.80%	17.49%	15.22%
Changing in column slenderness ratio from λ=6 to λ=15	e/t=0	0.8%	-24%	-24.10%	24.35-%	-24.35%	-30.50%
	e/t=0.2	0.8%	-29.90%	-30.10%	-24.19%	-10.71%	-33.59%
	e/t=0.4	0.8%	-22.99%	-19.83%	-22.83%	-23.58%	-22.67%

The reason for all variation in (the LWC strain, longitudinal reinforcement strain (inner side, outer side), and transverse reinforcement strain) is attributed to the fact that the changes in longitudinal reinforcement bar diameter between 10 mm (0.8%), 12 mm (1.1%), and 16 mm (2.0%) This increases in ratio of longitudinal reinforcement between the columns resulted to more confinement and it leads to increase the failure load, when the load increased the deformation increased and all strains increased. The changes in column height from short (λ=6) column to long column (λ=15). This increases in ratio of (λ) between the columns resulted to more buckling and it leads to reduce the failure load, when the maximum load reduced the deformation at maximum load reduced and all strains reduced.

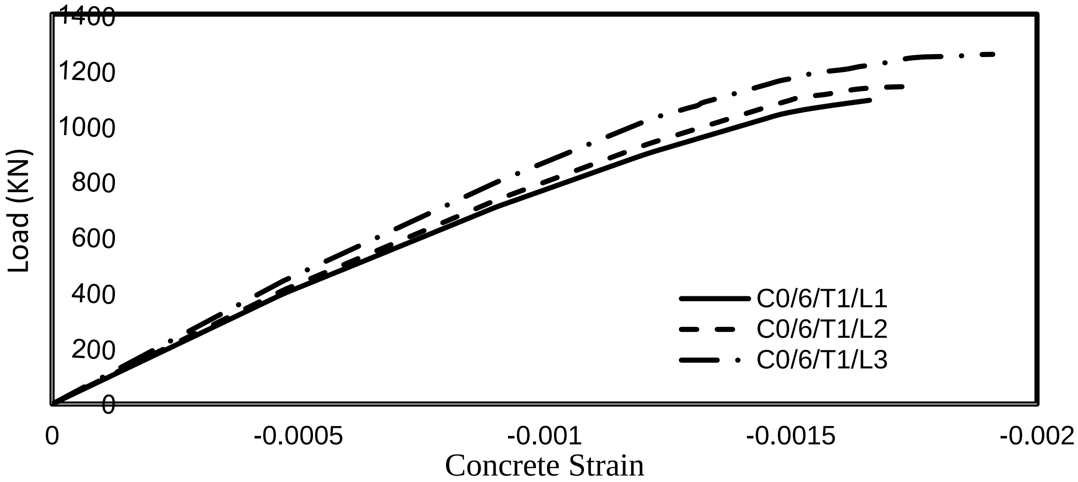


Fig.6: Load-concrete strain curve for columns under concentric load.

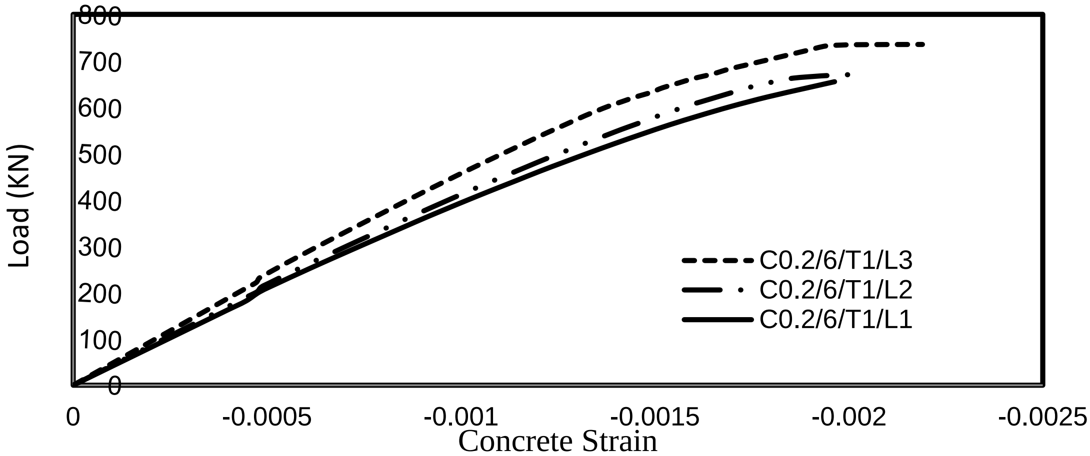


Fig.7: Load-concrete strain curve for columns under eccentric load (e/t=0.2).

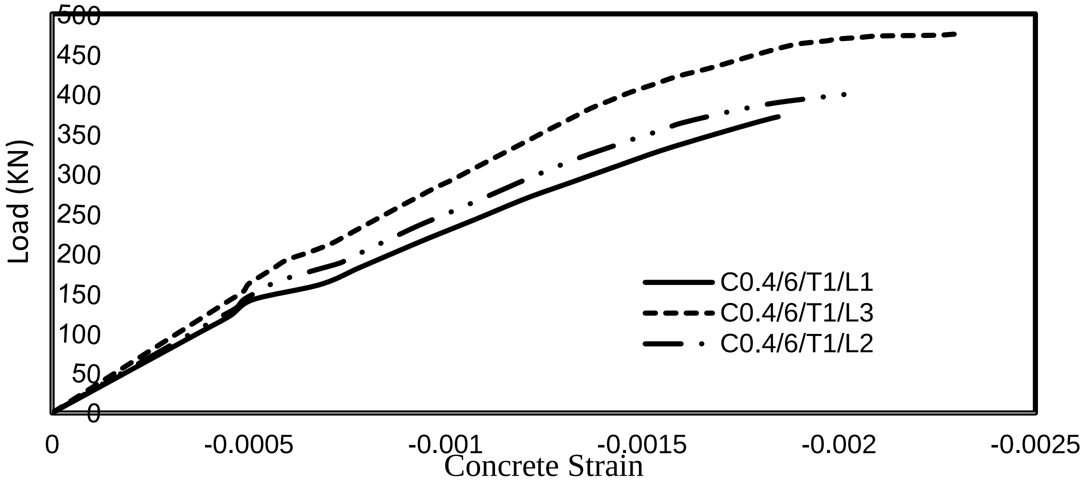


Fig.8: Load-concrete strain curve for columns under eccentric load (e/t=0.4).

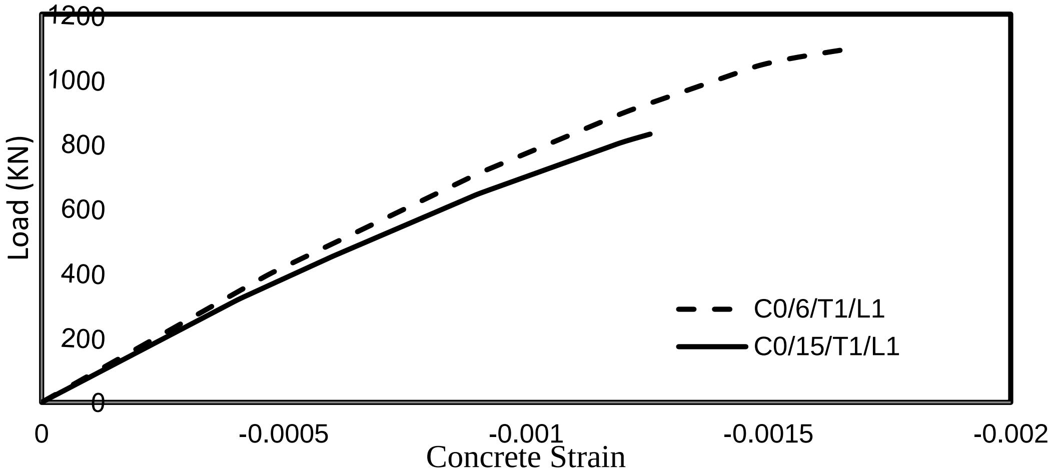


Fig.9: Load-concrete strain curve for columns under concentric load.

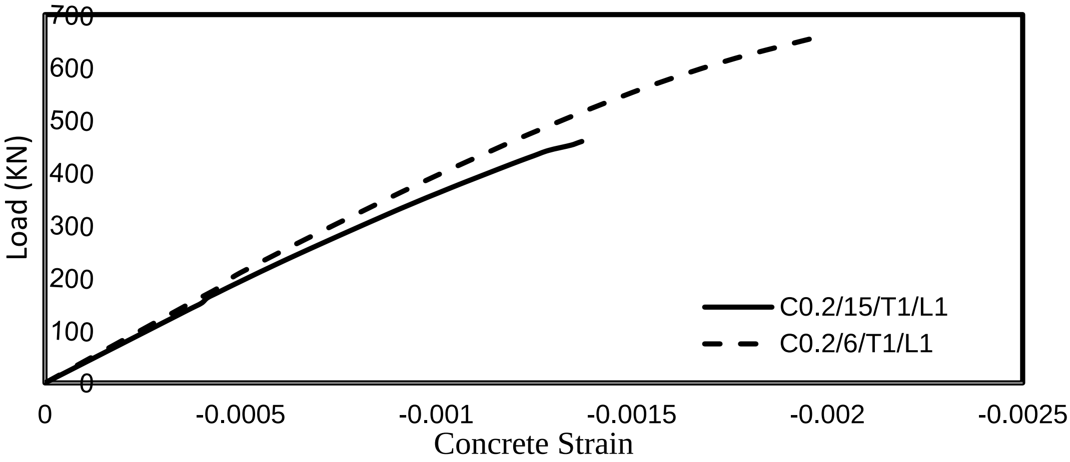


Fig.10: Load-concrete strain curve for columns under eccentric load (e/t=0.2).

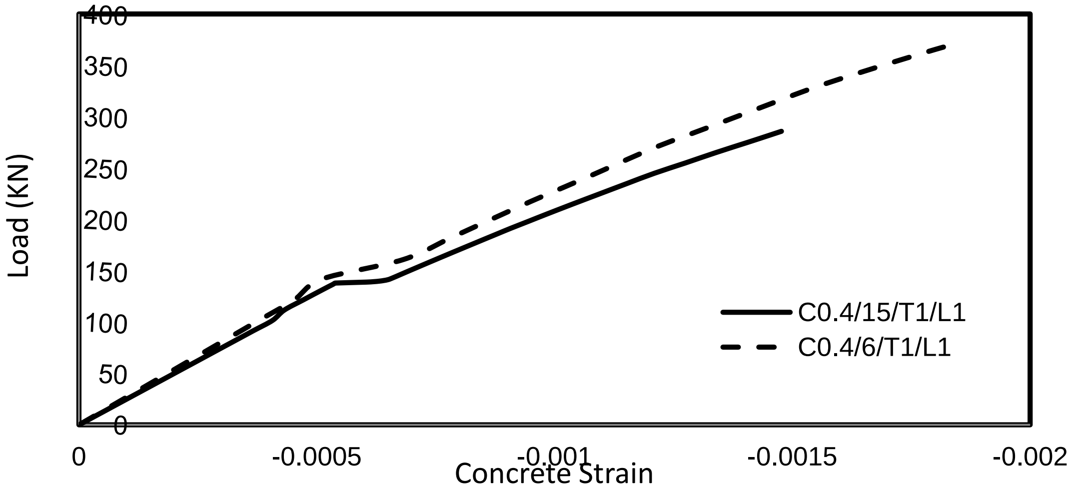


Fig.11: Load-concrete strain curve for columns under eccentric load (e/t=0.4).

5 . Conclusions

The non-linear finite element model was found to be capable of predicting the behavior of lightweight concrete columns under uniaxial eccentric loading. A good agreement was found between the experimental results of the work conducted by, Maged Tawfik, Sherif Elwan, Hosam Seleem and Amr Abdelrahman (2017) and theoretical results of the model along the loading history. A detailed parametric study was done to account for several possible two parameters were taken into consideration through this study, the effect of slenderness ratio and longitudinal reinforcement ratio in square lightweight concrete columns. Based on the results of the parametric study, the following conclusions could be drawn:

1. In case of increasing vertical reinforcement ratio (L) from 0.8% to 1.1% the failure load increased by 4.52%, 2.36%, and 7.77% for eccentricity ratio (e/t) of 0, 0.2, and 0.4 respectively. Also when increase vertical reinforcement ratio (L) from 0.8% to 2% the failure load increased by 15.11%, 12.28%, and 28% for eccentricity ratio (e/t) of 0, 0.2, and 0.4 respectively.
2. In case of increasing vertical reinforcement ratio (L) from 0.8% to 1.1% the concrete compressive strain increased by 4.82%, 2.55%, and 9.43% for eccentricity ratio (e/t) of 0, 0.2, and 0.4 respectively. Also when increase vertical reinforcement ratio (L) from 0.8% to 2% the concrete compressive strain increased by 15.06%, 11.73%, and 25.14% for eccentricity ratio (e/t) of 0, 0.2, and 0.4 respectively.
3. In case of increasing vertical reinforcement ratio (L) from 0.8% to 1.1% the longitudinal reinforcement strain at inner side increased by 5.45%, 3.7%, and 5.23% for eccentricity ratio (e/t) of 0, 0.2, and 0.4 respectively. Also when increase vertical reinforcement ratio (L) from 0.8% to 2% the longitudinal reinforcement strain at inner side increased by 15.76%, 12.96%, and 14.8% for eccentricity ratio (e/t) of 0, 0.2, and 0.4 respectively.
4. In case of increasing vertical reinforcement ratio (L) from 0.8% to 1.1% the longitudinal reinforcement strain at outer side increased by 5.45%, 0.89%, and 5.48% for eccentricity ratio (e/t) of 0, 0.2, and 0.4 respectively. Also when increase vertical reinforcement ratio (L) from 0.8% to 2% the longitudinal reinforcement strain at outer increased by 15.76%, 2.68%, and 17.49% for eccentricity ratio (e/t) of 0, 0.2, and 0.4 respectively.
5. In case of increasing vertical reinforcement ratio (L) from 0.8% to 1.1% the transverse reinforcement strain increased by 5%, 6.3%, and 9.32% for eccentricity ratio (e/t) of 0, 0.2, and 0.4 respectively. Also when increase vertical reinforcement ratio (L) from 0.8% to 2% the transverse reinforcement strain increased by 20%, 13.91%, and 15.22% for eccentricity ratio (e/t) of 0, 0.2, and 0.4 respectively.
6. In case of increasing slenderness ratio (λ) from 6 to 15 the failure load reduced by 24%, 20.9%, and 23%, the concrete compressive strain reduced by 24.1%, 30.1%, and 19.83%, the longitudinal reinforcement strain at inner side reduced by 24.35%, 24.19%, and 22.83%, the longitudinal reinforcement strain at outer side reduced by 24.35%, 10.71%, and 23.58%, and the transverse reinforcement strain reduced by 30.5%, 33.59%, and 22.67% for eccentricity ratio (e/t) of 0, 0.2, and 0.4 respectively.
7. We concluded from finite element results increasing in longitudinal reinforcement ratio from 0.8% to 1.1%, and 2% in lightweight concrete columns resulted to more confinement and it leads to increase the failure load and ductility, subsequently all deformation increased and it leads to increase all strains.

8. This increases in slenderness ratio (λ) from 6 to 15 between lightweight concrete columns resulted to more buckling and it leads to reduce the failure load, subsequently all deformation at maximum load reduced and it leads to reduce all strains.

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