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Numerical Investigationof RC Exterior Beam Colum Connections underMonotonic Loads

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Abstract:*A finite element model of reinforced concrete beam column connections (BCC) using ABAQUS program is presented. The concrete is discretisised into 4-node shell elements. Truss elements discretely model each single reinforcement bar. They arecoupled to the concrete elements using the embedded modelingtechnique. The concretedamage plasticitymodel of ABAQUS is used to describe the nonlinear material behavior ofconcrete and steel plates of supports under monotonic loading. Theparametric model is verified to experimental results taken from the literature.Eleven specimens were investigated in the experiment. Two specimens only were modeled. The effect of new parametric study was studied by ABAQUS. Change of column width, axial column load and concrete compressive strength were presented. It was found a close agreement between numerical and experimental results. Also, the column load has minor effect on the deformation of specimens. On other hand, as the column width and/or concrete compressive strength increased, the BCC capacity increased.*

Keywords*:Reinforced concrete beam column connections, shear joint failure, embedded modeling, ABAQUS, monotonic loading, column width, FEA, axial column load.*

I. Introduction

The beam column connections (BCC) are one of important structural elements in concrete structures. This is due to its construction difficulty and its manual design. In several structural collapses, the BCC was the main reason particularly, when the building subjected to earthquake. Researchers have observed four types of failures that can take place in BCC [(Kiran and Genesio, 2014)]. These modes of failure can be classifiedas: - shear failure in the joint, slippage of the beam main reinforcement bars, yielding of the beammain reinforcement(beam hinging) and yielding of the column longitudinal bars (columnhinging). Shear failure of BCCwas the main causeinfailure of several moment-resisting frame structuresduring recent earthquakes (Park and Mosalam, 2012).

The effect of axial load ratio on seismic behaviorof interior beam-column joints was studied by FU et al. (2000). The data of interior RCbeam-column sub-assemblage was investigated by Murakami et al. (2000). Thespecimens failed in joint shear.Theparameters affecting on joint shear capacity werestudiedin addition toashear equation wasinduced.Several researchers have simulated the behavior of the BCC using finite element analysis software such as ABAQUS, ANSYS and VECTOR2. A cyclic tests on hybrid fibre concrete structural walls wereproposed using ABAQUS and reported by Davide et al (2006). The effect of CFRP on RC beams behavior using ABAQUS was proposed by Yamane et al (2010). ABAQUS/Explicit (ABAQUS, version 6.12.3, 2012) was used to simulation of the current study.

In the present study, the finite element analysis (FEA) was used to model thebehaviour of beam column connections. For validation, thestudy was carried out ontwo specimens of BCC that had beenexperimentally tested and reported byMohamed et al. (2014). A parametric study including axial column load, column width and concrete compressive strength was induced.

II. Research Significance

The FEA using ABAQUS facilitatesthe design and analysis of RC beam column connections for researchers and designers. Through FEA, the compressive strength of concrete, the failure mode of joint, the crack pattern, effect of column width and effect of axial column load ratio can beobtained. Therefore, with the proposed FEA, it was possible for engineers to achieve a reasonable solution for BCC behaviour.

III. Experimental Results Considered

Experimental data was obtained from previous work by Mohamed et al. (2014). The research tested elevenRC exterior beam–column connections. This experimental program was tested under monotonic loading. All specimens had the same concrete dimensions. Each specimen had acantilever beam of 900 mm length. The high tensile steel (yield strength (f_y)= 400 MPa) was used for the top and bottomlongitudinal bars of the beam and used for the main longitudinal steel reinforcement of the column. The used stirrups for both beam and column were made from normal mild steel bars (yield strength (f_y)= 250 MPa). The diameter was 8 mm and the spacing between stirrups of column was 150mm and it was 100mm for the beam.The specimen Jo contained three stirrups of 8mm diameter at joint panel.For all specimens, the compressive strengthof concrete (f_{cu}) was 25 MPaandyoung'smodulusof steel was 200 kN/mm^2. The ultimate tensile strength of concrete was taken according to Egyptian code (2012) 0.6 $\sqrt{f_{cu}}$ N/mm^{2}because it did not mentioned in experiment.Figure.1 shows the concrete dimensions and reinforcement detailingfor the base control specimen.The loading direction on the beam end was acted at the lower side. A vertical load of 200 kN was applied at the upper end of column for all specimens.

The experimental program was consisted of three groups plus the base control specimen (Jo).The first group included three specimens: JI0,JI1, and JI2.This group studiedthe absence of the shear reinforcementat the beam–column connection.The specimen JI0, which considered in the current study, was without stirrups at joint panel. Specimens of both second group and third group were not represented in the current study. Only two specimens Jo and JI0 were investigated. Verification of experimental results using ABAQUS was obtained firstly then the parametric study was developed.

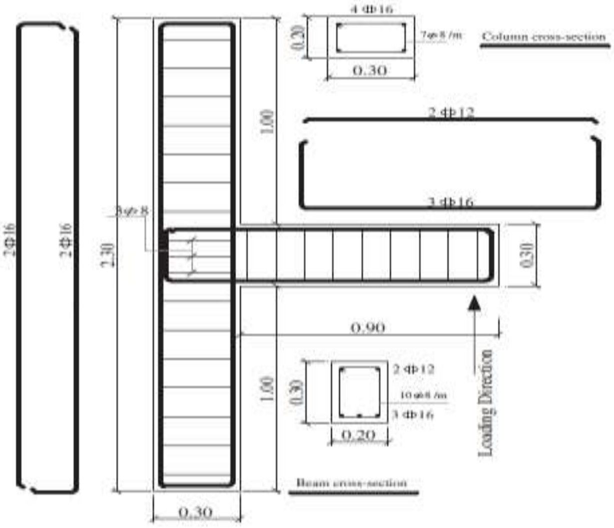


Figure.1 Concrete dimensions and reinforcement details for the base control specimen, J0 (Mohamed et al. (2014))

IV. Finite Element Analysis (FEA)

Finite element analysis was proposed to model the 2D nonlinear behaviour of the beam column connections. TheAbaqus/standard (ABAQUS, version 6.12.3, 2012) was used for the analysis.

4.1. **Modeling ofMaterial Properties**

4.1.1 Concrete

The concrete behavior was modeled by a plastic damage model.a two failure modes, tensile cracking and compressive crushing, were assumed in this model.The concrete damaged plasticity model in ABAQUS version6.12.3 was used to simulate theconcrete behavior for both the column and beam. The 2-D planar shell used for modeling of the RC beam column connection (BCC). The concretebehavior will be considered dependent on the embedded steel. The bond skidis modeled by embedded region. The boundary condition of the lower end and upper endof the column was simulated as hinged support and roller support respectively. The roller support was allowed the vertical movement when the axial column load applied. A vertical pressure load was applied on column to simulate the vertical load applied on column. A displacement of 40 mm was applied on free end of beam up word to simulate the applied load in test at same point.

The compressive strength, f_{cu} , was in the experimental work measured to be 25 MPa. Modulus of elasticity of concrete (E_c) and concrete tensile strength (f_{ct}) were calculated by Egyptian code (2012) in equation (1) and equation (2) respectively:

$E_c = 4400\sqrt{f_{cu}} = 22000$ MPa (1)

$f_{ctr} = 0.6\sqrt{f_{cu}} = 3$ MPa(2)

Wheref_{cu}, is given in MPa.

The stress–strainequations (3 and 4) proposed by Saenz (1964) was used to draw the compressive stress–strain curve forconcrete:

$$\sigma_c = \frac{E_c \varepsilon_c}{1 + (R + R_E - 2)\left(\frac{\varepsilon_c}{\varepsilon_0}\right) - (2R - 1)\left(\frac{\varepsilon_c}{\varepsilon_0}\right)^2 + R\left(\frac{\varepsilon_c}{\varepsilon_0}\right)^3} \quad (3)$$

$$\text{Where} R = \frac{R_E (R_\sigma - 1)}{(R_\varepsilon - 1)^2} - \frac{1}{R_\varepsilon}, \quad R_E = \frac{E_c}{E_0} \text{ and } \quad E_0 = \frac{f_{cu}}{\varepsilon_0} \quad (4)$$

and, $\varepsilon_0 = 0.0025$, $R_\varepsilon = 4$, $R_\sigma = 4$ as reported by Hu (1989). The stress–strain curve in compression for concrete is shown in Figure.2. Poisson’s ratio of concrete was taken 0.2.Same method was used to draw stress-strain curve of concrete when compressive strength equal 35 MPa(see in Figure.2).In addition, a linear function of softening has been considered in current study inorder to evaluate modeling of concrete in tension.The fracture energy method was used to specify the tension behaviour of concrete.G_f is the area under the softening curve and it was assumedequal to 164.8 J/m^2 (0.09 N/mm) and 192.3 J/m^2 (0.105 N/mm) for f_{cu}= 25MPa and f_{cu}= 35MPa respectively(seeFigure.3.).

For definition of the damaged plasticity model, some specificparameters which have to be defined to describe the non-linear behavior of concrete were assumed. Table 1 illustrates the values which are chosen in this study for the damage parameters. Coefficient of the dilation angle (β) is dependent on the concrete shear strength.Shear resistance of concretewasaffected by properties of aggregates and the age of the concrete.Magnitude of β was assumed because the experiment of Mohamed et al. (2012) not illustrated any information about the specifications of the aggregates andage of the concrete. According to Rasoul (2012), the dilation angle can vary from $0^0 \leq \beta \leq 56.3^0$. For normal concrete, theβ is often a reasonable assumption between 30^0 and 40^0. In this study, a parametric study has been performed for different dilationangles and tension softening function (see in Figure.3) to select an appropriate value for each of them incomparison with the experimental results.

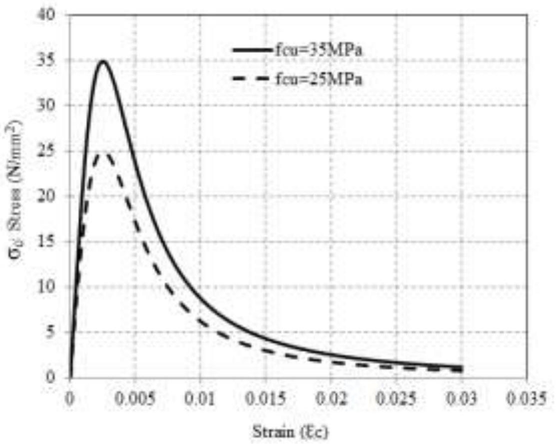


Figure.2 stress strain curve for concrete in compression.

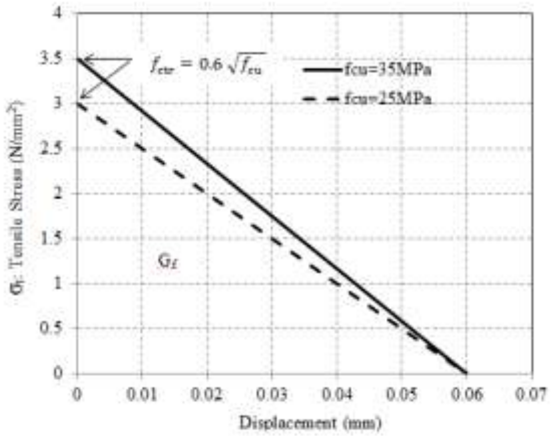


Figure.3Tensile stress deformation relationship for concrete.

Table 1: Concrete damaged plasticity parameters

Dilation angle (β)	Eccentricity	f_{b0} / f_{c0}	K	Viscosity parameter
35	0.1	1.16	0.67	1E-07

4.1.2 Steel Reinforcement

The used reinforcement was representedas an elastic material in tension and compression.The assumed stress-strain curve of the steel bars is shown in Figure 4.Steel reinforced bars in the concrete were modeled as one-directional strain elements (truss element) and were simulated by T2D2 (A 2-node linear 2-D truss).It was obtained from experiment that the elastic modulus of steel (E_s) was equal 200 GPa for all types of bars. For longitudinal bars in beam and column,theyield stress (f_y) was measured in theexperimental study and the value obtained was 400 MPa.While stirrups of both beam and column have a yield stress of 250MPa.These values were used in the FEA model. A Poisson's ratio of 0.3 was used for the steel reinforcement. Thebond between reinforcement bars and concrete was assumed asa perfect bond (embedded region). The material properties of reinforcement bars were assumed for the numerical simulationand it were listed in Table 2.

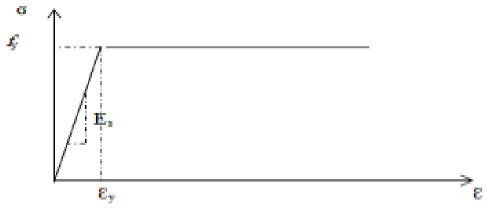


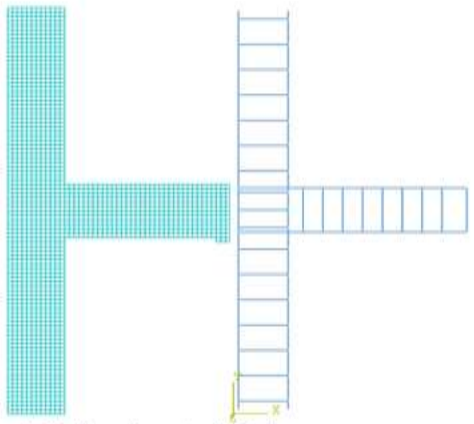
Figure.4. Stress strain behavior of steel.

Table 2: Material properties of reinforcement bars assumed for the numerical simulation.

a)D8mm bar	b)D16mm bar
*Material, Name= steel8 *Elastic , Type = isotropic 200000, 0.30 *Plastic, Hardening=combined, Data type=parameters 250, 7993, 175	*Material, Name= steel16 *Elastic , Type = isotropic 200000, 0.30 *Plastic, Hardening=combined, Data type=parameters 400, 7993, 175
c)D12mm bar	
*Material, Name= steel12 *Elastic , Type = isotropic 200000, 0.30 *Plastic, Hardening=combined, Data type=parameters 400, 7993, 175	

4.2. Numerical Analysis

2-Node linear elements (T2D2) were used for the reinforcedconcrete, reinforcement bars, and steel plates at supportsin this model. Figure 5 showsfinite element model of specimen for both concrete and steel.The boundary conditions and loading direction are illustrated in Figure6. A fine mesh is needed to obtain results of sufficient accuracy. The mesh size is 25mmx25mm for all elements whether concrete or reinforcement. The processor type used for this study was intel(R) Core (TM) i3-2370CPU @ 2.4GHz 2.4GHz.



a) Meshing of concrete b) Steel

Figure 5.Finite element model of specimen.

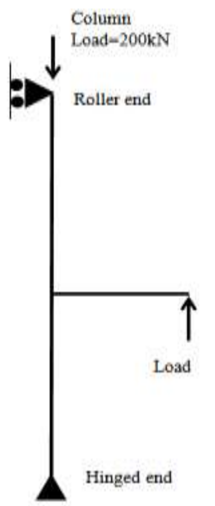


Figure 6 the boundary conditions and loading direction.

V. Results

5.1.FEAVerification Of P-Δ Curve Forcontrol Tested Specimens (J0, JI0).

The Load–deflection (P-Δ) curve defined as the relationship between the applied force on free end of beam and vertical displacement at same point (see in Figure. 6). For the base control specimen (J0) shown in Figure.1, the free end beam deflection measured experimentally illustrated in Figure.7. Also the connection between beam and column was reinforced by three stirrups of 8mm diameter. The control specimen JI0 of group I was without joint reinforcement. Figure.8 shows Load–deflection (P-Δ) curve of control specimen JI0 obtained in experiment. After modeling for two control specimen J0 and JI0, the results showed in Figure 7 and Figure 8 respectively. There was a good agreement between FEM and experimental results for the two control specimens J0 and JI0as shown in Figure 7 and Figure 8respectively. The good agreementindicates that the models used for concrete and reinforcement can capture the fracture behaviour well.There are several possible causes for the differences betweenthe experimental data and the finite element analysis. One is asfor the control beam, theassumed perfect bond between concreteand steel reinforcement. In addition to, the stress- strain curve of concrete was not obtained from experimental work but it was estimatedas mentioned previously in section 4.1.1 (see in Figure 2).

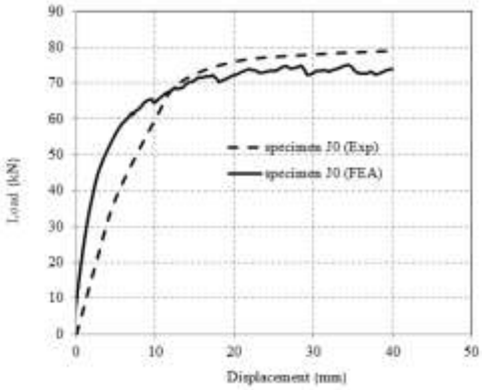


Figure .7 Load–deflection curve of Base control specimen J0, obtained by experiments and FEA (ABAQUS)

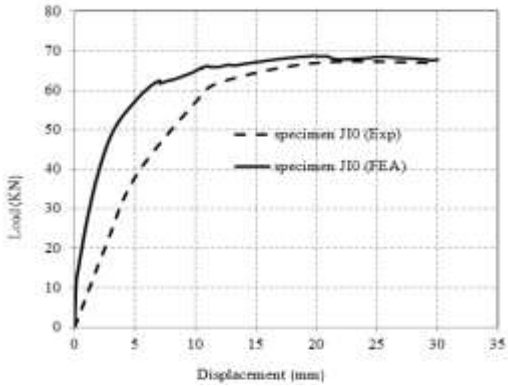
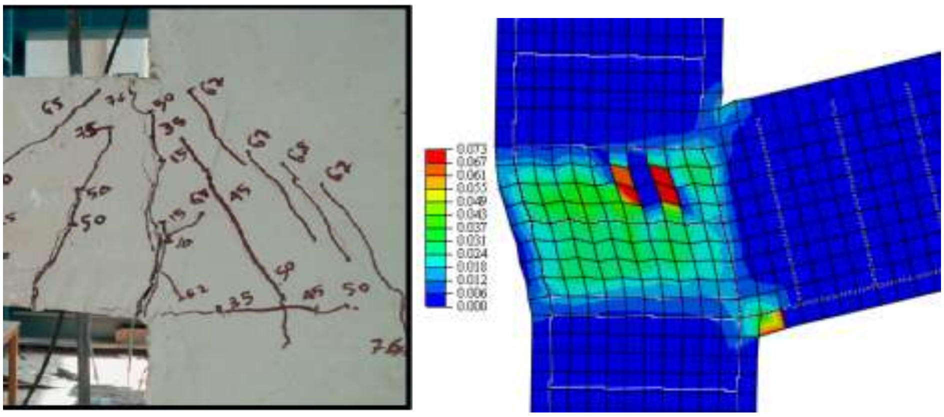


Figure.8Load–deflection curve of control specimen JI0 of group I (without joint reinforcement), obtained by experiments and FEA (ABAQUS)

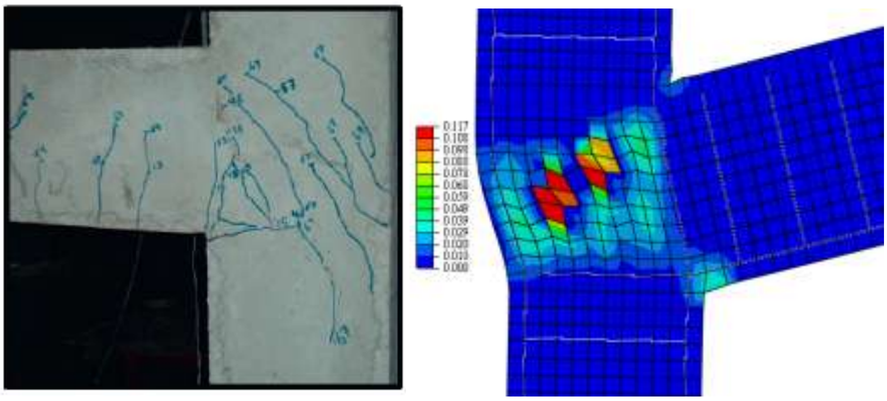
5.2. FEA verification of failure modes for specimens J0 and JI0.

The failure mode of base control specimen J0 and control specimen JI0 of group I using FEA just like in in the experiments and this is illustrated in Figure. 9 and 10 respectively. For specimen J0, with joint reinforcement of 3D8mm, the failure mode was beam flexure as shown in Figure.9. This might due to high resistance of the joint area. When the joint reinforcement of 3D8mm was removed in specimen JI0, the failure took place in the joint panel as shown in Figure.10. In the fact, the experimental work takes more time, effort, and high cost to reach conclusion. So, the modeling by ABAQUS program is useful and first to get accurate results in little time, and without cost. We could change joint reinforcement amount and joint reinforcement configuration and we studied its effect on the failure modes.



Specimen J0 (Exp) Specimen J0 (plastic strain-FEA)

Figure.9failure mode of Base control specimen J0, obtained by experiments and FEA (ABAQUS)



Specimen JI0 (Exp) Specimen JI0 (plastic strain-FEA)

Figure.10failure mode of control specimen JI0 of group I (without joint reinforcement), obtained by experiments and FEA (ABAQUS)

5.3 Numerical Parametric Study.

Section 5.1 and 5.2 studied simulation of P-Δ curves and failure modes of control specimensJI, JI0. It was obtained that the FEA results agreed with experiment results. So, three parameters were developed in the current study and its effect on load deformation behavior. The one is effect of axial column load while the second is effect of column width. The third is compressive strength of concrete.

5.3.1 Effect of Column Width on Load Deformation Behavior

For base control specimen J0, with joint reinforcement, the column width was 200mm equal to beam width. The curve of load deformation was drawn by FEA as shown in Figure. 11. The column width was changed from 200mm to 250mm to 300 mm. the applied axial column load in test was 200 kN or 0.133 $A_g f_{cu}$. So, the axial column load in FEA was dependent on cross-sectional area of column. On the other hand, the parameters of concrete compressive strength and column depth were constant. It must be noted that the column width have a big good effect on load deformation curve. As width of column increased, the capacity of beam column connection improved.

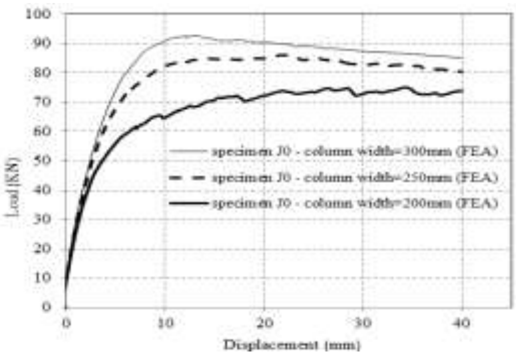


Figure.11 P-Δ curve of specimen J0 with different values of column width.

5.3.2Effect of Column Loadon Load Deformation Behavior.

Figure.12 shows P-Δ curve of specimen J0 in case of column width=250mm.Three values of axial column load of 200 kN, 250 kN, and 750 kN were applied. Although the value of column increased from 200 kN ($0.107A_g f_{cu}$) to 750kN ($0.40A_g f_{cu}$), the additional load at same displacement was not observed approximately. So, the axial column load hasnegligible effecton the behavior of the beam column connection as shown in Figure. 13. For assurance, the analysis were carried out using column width = 300 mm under column load of 240 KN and 300 kN. The results are shown in Figure. 13. The same conclusion was reachedas well.

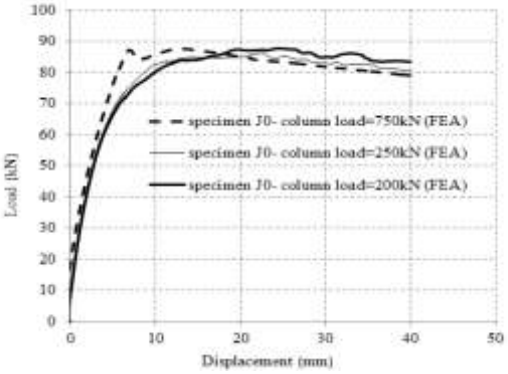


Figure.12 P-Δ curve of specimen J0 with different values of column load in case of column width=250mm.

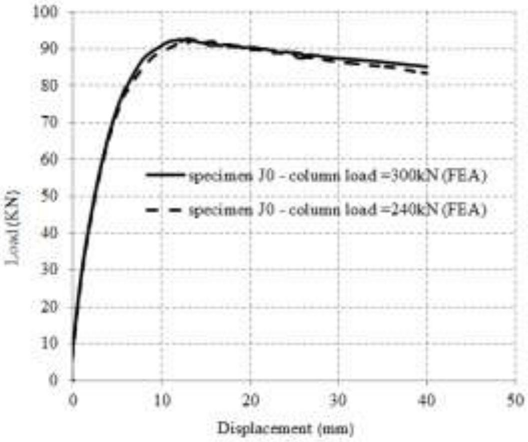


Figure13 P-Δ curve of specimen J0 with different values of column load in case of column width=300mm.

5.3.3 Effectof Compressive Strength of Concrete on load Deformation Behavior.

This section studied the effect of two degree of concrete 25 MPa and 35 MPa on the behavior of specimen J0. All details of specimen J0 showed in Figure.1. Only f_{cu} increased to 35 MPa. The stress strain curve of concrete with f_{cu}= 25 MPa was drown previously in figure 2. The concrete compressive strength affected on column capacity and beam flexure capacity hence affected on beam column connection. So, the compressive strength of concrete increased to f_{cu} = 35 MPa and the stress strain curve drown in figure 2(As mentioned in section 4.1.1). This curve used in simulation. Also Modulus of elasticity E_c and tensile strength f_{ctr} were calculated by Eq.(1) and Eq.(2) respectively. Figure.14showthe effect of compressive strength of concrete on behavior of beam column connection. It can be concluded that the compressive strength of concrete have a significant effect on the BCC behaviour. As compressive strength of concrete increased, the capacity of beam column connection improved.

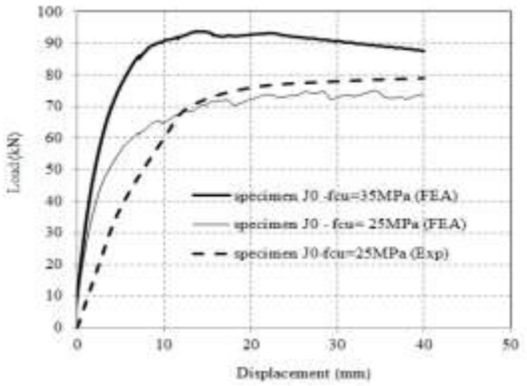


Figure14P-Δ curve of specimen J0 with different values of compressive strength of concrete.

VI. Conclusion

The beam column connection is the most important structural element in reinforced concrete buildings. In the present study, a numerical investigation to the experimental work on the behavior of the beam column connection under static load was carried out. The effect of column load, column width and concrete compressive strength was studied. Based on the results of this investigation, thefollowing conclusions or observations can be drawn:

1- There was a good agreement between finite element analysis (ABAQUS) and experimental results.
2- The failure modes of the beam column connection was classified to joint shear failure, slippage of the beam steel, beam flexure and yielding of the column main steel.
3- The beam flexure type of failure took place in case of specimens with joint shear reinforcement while the joint shear failure occurred in case of specimens without joint shear reinforcement.
4- The concrete compressive strength had a significant influence on the BCC behaviour.
5- Theaxial column load ratio had a negligible effect on the BCC behaviour.
6- The increase of column width in perpendicular direction on the beam improved the behavior of the beam column connection.

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