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Outside fortified solid pillar section joint exposed to monotonic stacking

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ABSTRACT: The joints among pillars and segments are vital zones in a strengthened solid second opposing edge. The conduct of such joints incredibly impacts the strength and pliability of the general edge. In this examination, investigation of three-dimensional mathematical models of outside fortified solid bar section joints under monotonic stacking was performed utilizing the limited component ABAQUS bundle. Concrete and strengthening steel material nonlinearities, just as bond qualities between fortifying bars and encompassing cement were considered in the investigation. A parametric report including thirty joint models was led to look at the impact of con-crete strength, section pivotal burden, joint stirrups and state of the shaft top fortification on the pillar tip burden and relocation limits. The solid measurements and fortification of the stud-ied models were picked to guarantee the event of joint disappointment. The investigation indicated that the shaft tip extreme burden fundamentally expanded as the solid strength, segment hub burden and joint mix rups were expanded. Joints with L-molded pillar top fortifying bars showed comparable execution to those with U-formed bars. The utilization of straight bars for bar top support fundamentally decreased the bar tip.
KEYWORDS: Reinforced concrete; Beam-column joint; Finite element; Monotonic loading; Numerical analysis; Bond simulation

I. INTRODUCTION

The recent laboratory experiments indicated that unsatisfac- tory structural performance may result from the premature failure of beam-column joints as described by Scott (1992) [1]and by Parker and Bullman (1997) [2]. Therefore, a beam-column joint is a critical zone in reinforced concrete frames which should be designed to provide the sufficient ultimate strength and deformation capacity. Joints around the external border of a structure are more vulnerable tofail- ure than interior joints. The priority in static loading is strength then ductility, therefore, joints in an ordinary struc- ture are designed on the basis of strength to resist gravity and windloads.

There are several possible failure modes that can occur in joints. Such modes include column flexural failure, beam flex- ural failure, column shear failure, beam shear failure, joint shear failure and bond failure of reinforcement. Numerous numerical studies have been carried out on exterior beam col- umn joints under monotonic loading using different finite ele- ment programs. Baglin and Scott (2000) [3]carriedout numerical research using SBETA package to simulate exterior beam column joints subjected to monotonic loading and col- umn axial load. They made a validation against 19 tests per- formed by the authors [1,3]and by Hamil (2000) [4]. The validation specimens had varied beam top reinforcement con- figuration and concrete compressive strength. Additionally, different numbers of column stirrups in the connection zone were considered. The model took into consideration different types of failure mechanisms involving beam failure, joint diag- onal shear cracking, and bond slip. They performed a para- metric study involving column load value and different arrangement of joint stirrups. The authors concluded that increasing the column load to a value corresponding to an equivalent stress of 20 MPa ($f_{cu}/3$) increased the joint ultimate capacity, while exceeding this value resulted in the opposite.

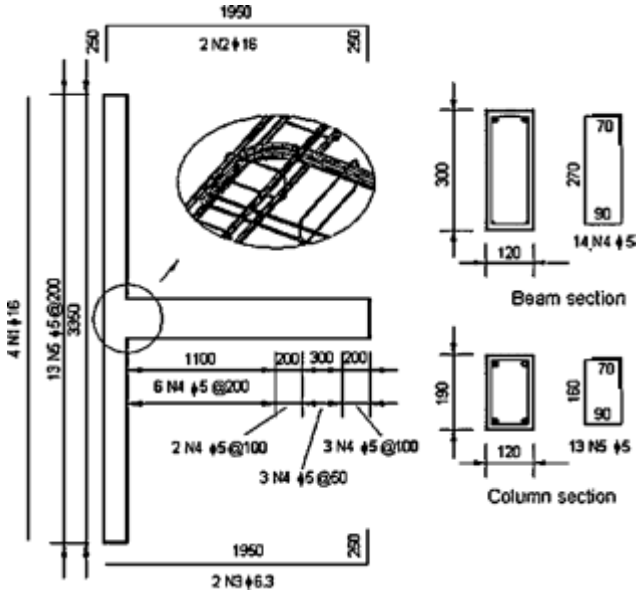


Fig.1 Layoutoftestspecimen,Haachetal.[12].

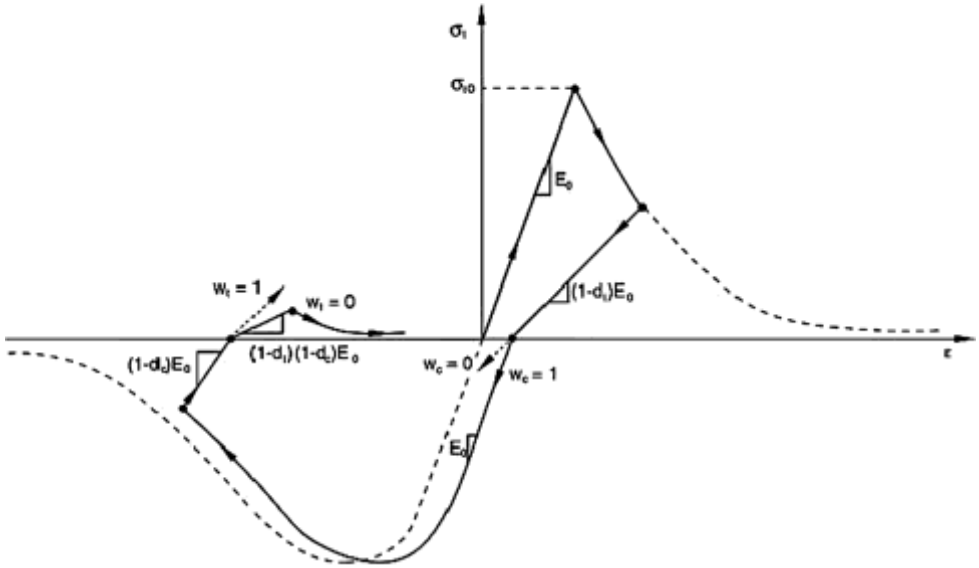


Fig.2 Concretestress-strainrelationwithloadingandunloading[13].

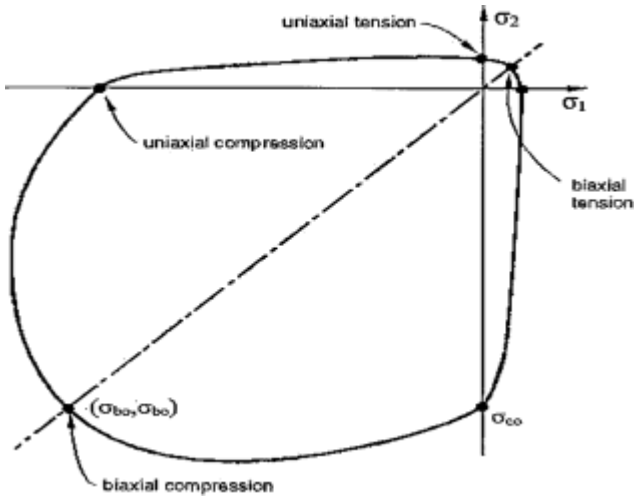


Fig.3 Concreteyieldsurfaceinplanestressby Kupfer[13].

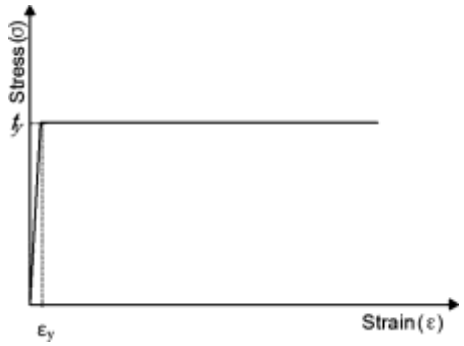


Fig.4 Typical stress-strain curve for steelreinforcement.

Moreover, the rise of axial load also improved the top beam bars anchorage within the joint as well as the joint stiffness. In addition, the more closely spaced stirrups within the joint increasedthejointultimatecapacitybylimitingtheextension of joint shear cracks as well as improving the contributionof thebent-downbarsofthebeambyincreasingtheconfinement aroundthem.

Bakir and Boduroglu (2002) [5] suggested a new empirical design equation for exterior joints subjected to monotonic loading based on 58 tests involving a large parametric study such as the beam reinforcement ratios, the stirrups of the joint core, the beam reinforcement detailing (U or L-bars anchorage), and the axial load stress. The authors concluded that increasing the beam reinforcement ratios and joint transverse reinforcement led to increasing shear capacity. In addition, U bar detailing ought not be utilized in monotonically loaded exterior beam column connection. On the other hand, using L bar detailing can change the failure mode from the beam fail- ure to joint shear failure. Furthermore column axial stress had no effect on ultimate joint shear capacity. Atta et al. (2004) [6]carried out finite element analyses using ANSYS to simulate the behavior of beam-column joint subjected to monotonic loading and column axial load. An extensive parametric inves- tigation was performed such as concrete grade, column axialforce, girder depth, column dimensions, joint transverse rein- forcement, and reinforcement anchorage. Theparametric

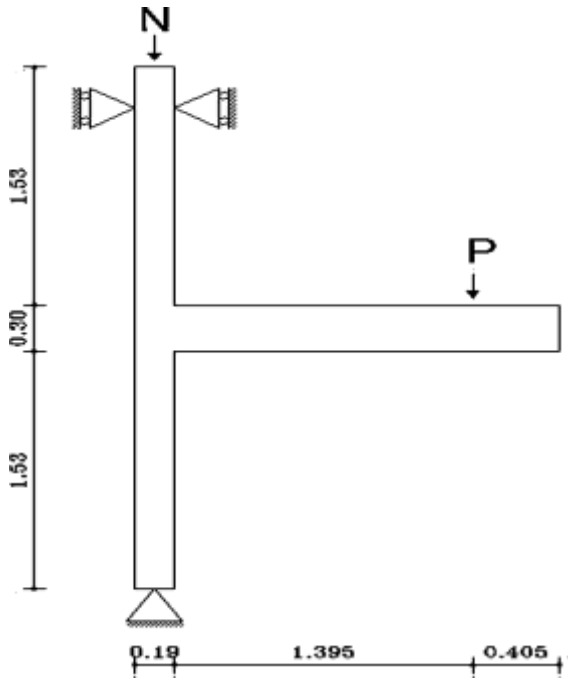


Fig.5 Loadingandboundaryconditions(dimsinmeters).

study confirmed that the increase of concrete grade increased the ultimate capacity. Moreover, the distribution of joint stir- rups had the most noticeable influence in improving the system stiffness and ultimate capacity and a more brittle failure mode was observed due to lesser sharing of joint stirrups. Hegger et al. (2004) [7] discussed the behavior of interior and exterior beam-column joints by using the finite element code ATENA. Two-dimensional plane stress analyses were carried out under monotonic loading only at the beam tip without axial column load. Beam failure and joint compression strut failure were observed. The parameters affecting the shear capacity were different for interior and exterior joints. In case ofexterior

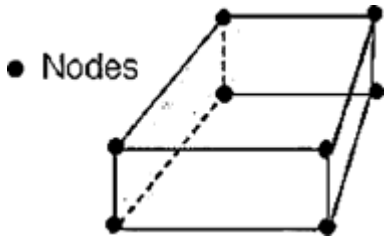


Fig.6 Typical 8 nodes linear brick element[13].

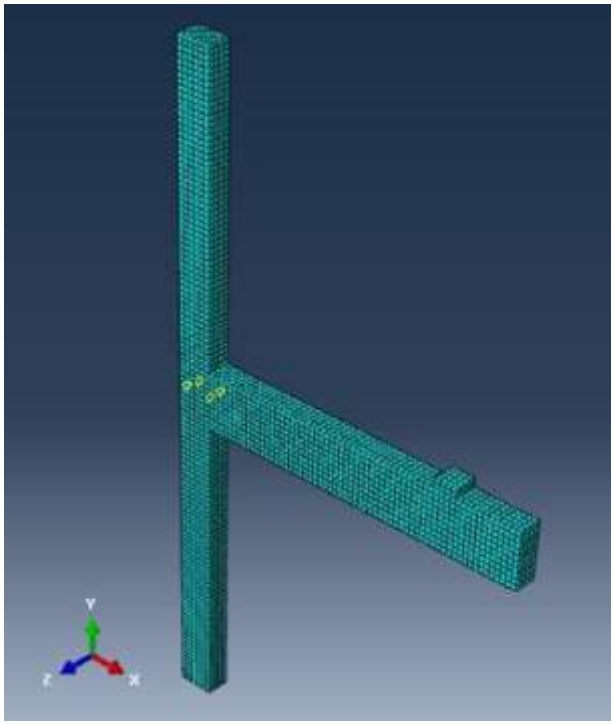


Fig.7 Numerical modelmesh.

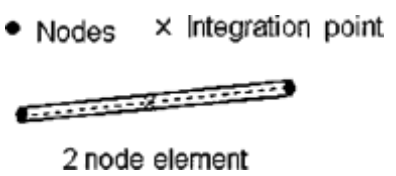


Fig.8 Typical 2 nodes linear beam element[13].

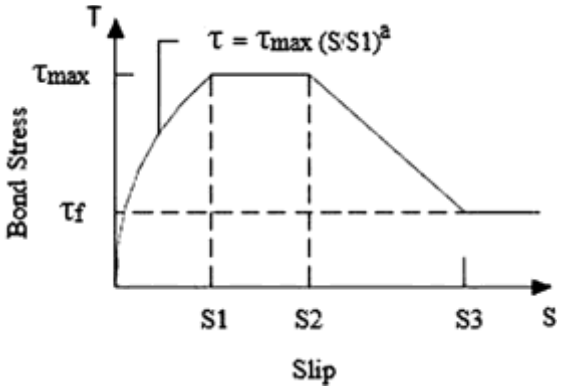


Fig.9 CEB-FIPMC90 model for bond-slip[14].

beam-column joint, the authors concluded that normal stresses of column higher than 40% of the concrete compressive strength reduce the joint shear strength.

Haach (2005) [8]conducted experimental tests of exterior beam-column joints under monotonic loading and column axial load. The tests contained three specimens with the same characteristics of geometry and material, with different axialload only. The authors used ABAQUS software to perform a numerical simulation. The numerical results were validated against experimental results and revealed good agreement specifically in reinforcement strains. The influence of the axial load level, axial load eccentricity, and joint stirrups ratio were investigated in a parametric study. The specimens collapsed due to shear diagonal cracking. The authors observed that higher column axial load made the joint stiffer and less ductile. Moreover, higher column axial loads generated better bond condition for the beam top reinforcement by increasing the compressive stresses of the anchorage. Patil and Manekari (2013)[9]carriedoutanumericalresearchtocomparebetween the behavior of corner and exterior connection subjected to monotonic loading using ANSYS finite element package. Dif- ferent parameters were analyzed such as maximum principle stress, minimum principle stress, displacement, different boundary conditions, and stiffness variation to make a com- parison study. Chaudhari et al. (2014) [10] performed a com- parison study between ABAQUS and ANSYS results on exterior beam column joint subjected to monotonic loading. The authors concluded that ABAQUS software gave more realistic findings than ANSYS. Tran and Hadi (2017) [11] con- ducted an analytical study on exterior beam-column joints sub- jected to quasi-static cyclic loading. The predicted shear strength of proposed model was compared with fiveanalytical

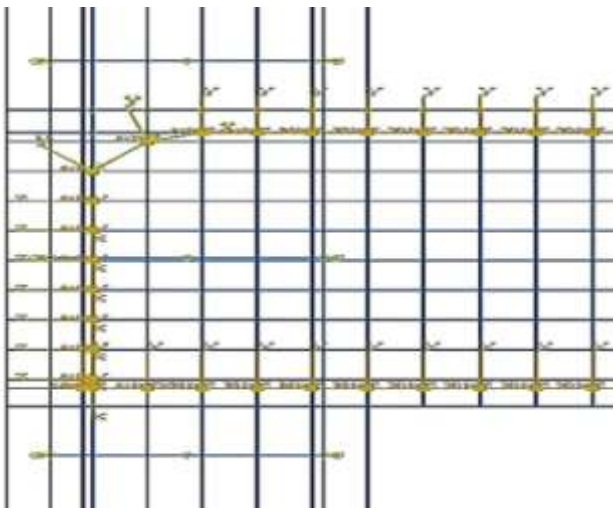


Fig.10 Beam top bar detail atjoint.

models and 142 test results to validate it. Three parameters involving concrete compressive strength, column axial stress level, and the vertical and horizontal amount of shear rein- forcement were considered to evaluate the proposed assump- tion. This assessment revealed that the suggested model can

efficiently anticipate shear strength.

The objective of the present work is to study the effect of axial load ratio, grade of concrete, number of joint stirrupsand type of beam top reinforcement anchorage on the strength and deformation capacity of exterior beam-column joints sub- jected to shear failure. For this purpose, thirty exterior beam-columnjointssubjectedtomonotonicloadingwerenumeri-

Table 1 Values of parameters for CEB-FIPMC90 model [14].

	Confined concrete			
	Good bond conditions	All other bond conditions	Good bond conditions	All other bond conditions
S1	0.6mm	0.6mm	1.0mm	1.0mm
S2	0.6mm	0.6mm	3.0mm	3.0mm
S3	1.0	2.5 mm	Clear rib spacing	Clear rib spacing
a	2.0 $\sqrt{f_{ck}}$	1.0 $\sqrt{f_{ck}}$	2.5 $\sqrt{f_{ck}}$	1.25 $\sqrt{f_{ck}}$
	0.15	0.15	0.40	0.40

f_{ck}: Characteristic concrete compressive strength

cally modelled and analysed using the finite element program ABAQUS.

II. NUMERICAL MODELLING BYABAQUS

1.1. Geometry

The considered exterior joint consisted of a beam portion anda column portion similar to the model adopted by Haach et al. [12], Fig. 1. The beam had a cross section of 300 mm 120 mm with 1800 mm overall length of can- tileveredportion. The column had a cross section of 190 mm 120 mm with a total length of 3350 mm. The beam top reinforcement was 2 bars of 16 mm diameter and bottom reinforcement was 2 bars of 6.3 mm diameter. The beam trans- verse reinforcement consisted of 5 mm diameter stirrups spaced at 200 mm as shown in Fig. 1. The column reinforce- ment was 4 longitudinal bars of 16 mm diameter and 5 mm diameter stirrups spaced at 200mm.

1.2. Materialmodel

The concrete damaged plasticity (CDP), which is an adjust- ment of the Drucker-Prager strength hypothesis, is used in ABAQUS [13]to define concrete materialas shown in Fig. 2. The basic constitutive parameters required to determine theCDPmaterialmodelinclude:thedilationangle(w)which is interpreted as the deformation capacity of concrete when subjected to shear stress and is usually taken between 30° and 40°, an intermediate value of 35° was considered in the presentstudywhichprovidedgoodcorrelationwithavailableexperimental results; the value of flow potential eccentricity(m) which was assumed 0.1; the default value of initialbiax- ial/uniaxial compressive yield stresses ratio (r_{bo}/r_{co}) which was taken equal to 1.16 as shown in Fig.3.

An elastic-perfectly plastic stress-strain curve was used to define steel material as indicated in Fig. 4. The yield stress (f_y) of beam top reinforcement and column reinforcement was 515 Mpa. The yield stress of beam bottom reinforcement and stirrups was 562.50 Mpa and 684.50Mpa, respectively.

1.3. Loading and boundaryconditions

The column was subjected to axial load as a pressure load at the top end according to a pre- specified value which was kept constant during the analysis. Each case was analyzed for a ver- tical displacement controlled loading that was applied at the end of the beam. The boundary conditions were set in the model as shown in Fig. 5. The column top end was roller, which had a horizontal restricted displacement U_x. The bot- tom of the column was hinged which was restrained in two degrees of freedom U_x and U_y.

1.4. The finite elementmesh

Three dimensional Solid 8-nodes brick element with reduced integration scheme (C3D8R) were used to model the concrete in space as shown in Fig. 6. A mesh size of 30 mm wasconsid- ered for overall elements in concrete part of beam and column as shown in Fig. 7. Three dimensional truss 2-nodes element (T3D2) were used to model the steel reinforcement in space as shown in Fig.8.

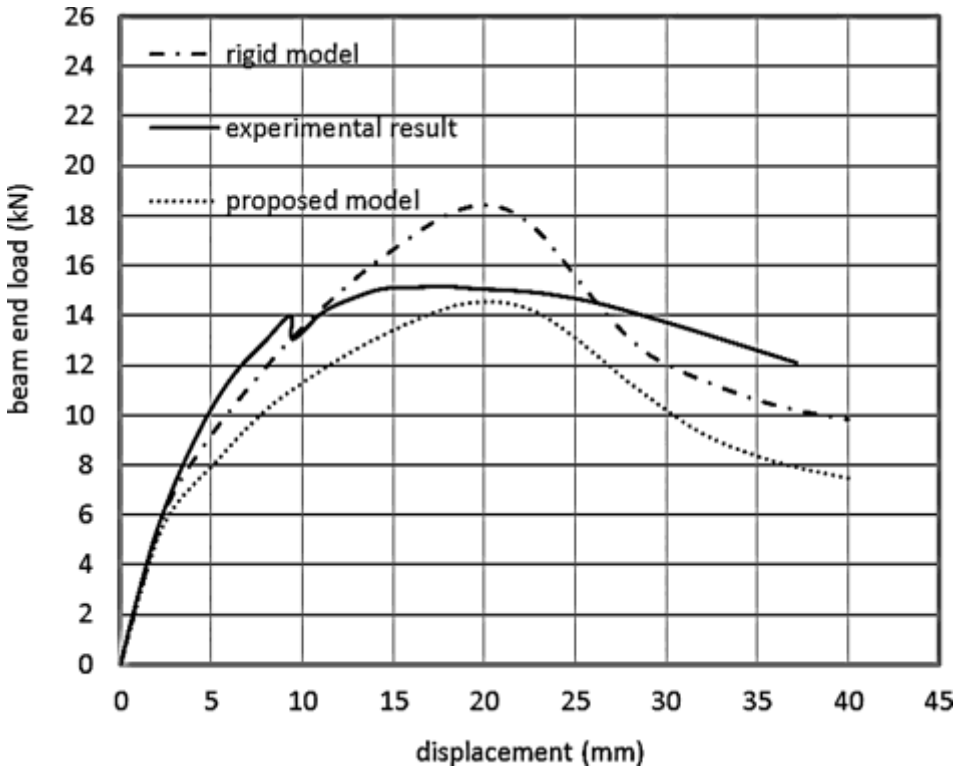


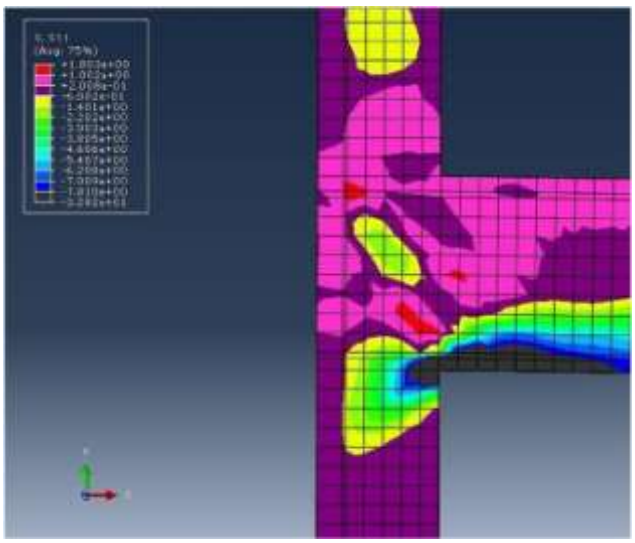
Fig.11 Verificationresults.

1.5. Connectors

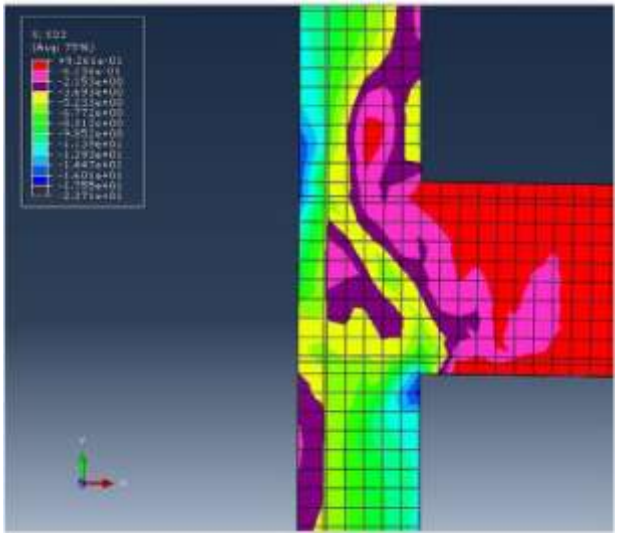
The bond slip between the concrete and steel reinforcement was introduced using connector elements. These connectors modeled the bond strength and the relation between bond stressandrelativedisplacementintheaxialdirectionbetween the steel reinforcement and the surrounding concrete. The bond law was based on the Euro-International Committee for Concrete (CEB) and the International Federation forPre- stressing (FIP) Model Code 1990 (CEB-FIPMC90) [14]as showninFig.9andTable1.Theselectedtypeofconcreteused in this model was confined concrete with good bond condi- tions. Fig. 10 shows the beam top bar detail atjoint.

1.6. Verificationstudy

The beam tip load-displacement curve obtained from the numerical analysis using ABAQUS was compared with the reported results from the experimental test which was carried



(a) S_{11} (Xdirection)



(b) S_{22} (Ydirection)
Fig.12 Concretestresses.

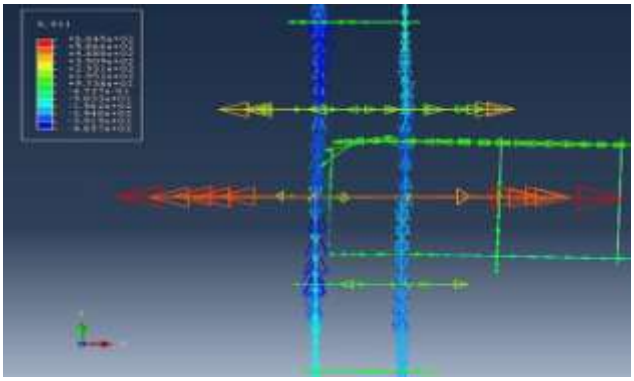
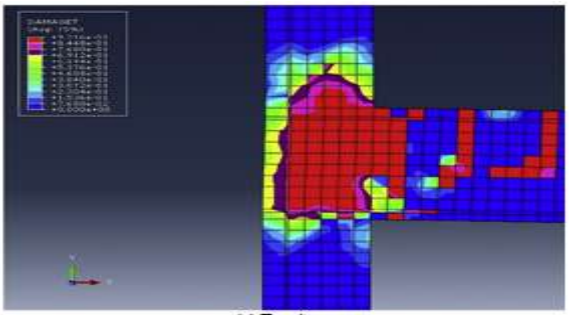
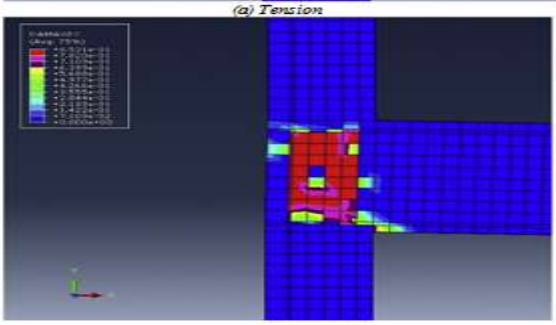


Fig.13 Reinforcing steelstresses.



(a) Tension



(b) Compression
Fig.14 Concretedamage.

out by Haach et al. (2014) [12]. To validate the precision of the proposed numerical model, specimen N400 was selected from the experimental database which was the control specimen. The cylinder compressive strength of concrete (f'_c) was taken equal to 20.53 Mpa. It may be visible that the overall trend of specimen obtained from the numerical simulation issimilar

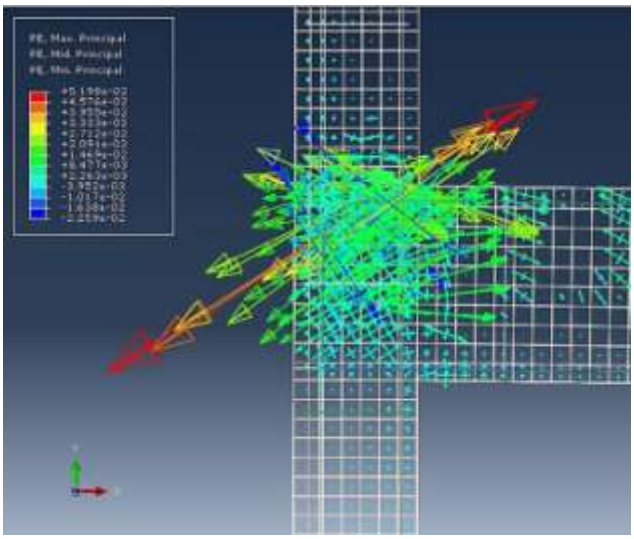


Fig.15 Plastic strain(PE).

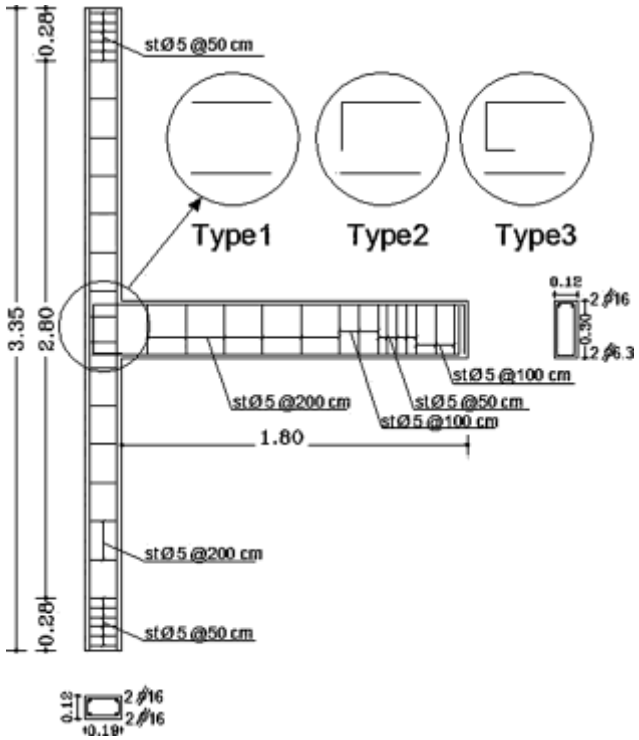


Fig.16 Reinforcementdetails.

Table 2 Parametric study database.

Joint No.	Compressive Concrete Strength (f'_c) Mpa	Axial Load Ratio (P/f'_cA_c%)	Column stirrups (n_s)	Type of Anchorage
S1	30	0.3	3	Type 2
S2	40			
S3	50			
S4	30	0	2	Type 1
S5				Type 2
S6				Type 3
S7			3	Type 1
S8				Type 2
S9				Type 3
S10			4	Type 1
S11				Type 2
S12				Type 3
S13		0.3	2	Type 1
S14				Type 2
S15				Type 3
S16			3	Type 1
S17				Type 2
S18				Type 3
S19			4	Type 1
S20				Type 2
S21				Type 3
S22		0.6	2	Type 1
S23				Type 2
S24				Type 3
S25			3	Type 1
S26				Type 2
S27				Type 3
S28			4	Type 1
S29				Type 2
S30				Type 3

to that of the experiment as shown in Fig. 11. In the proposed model, two cases for modelling the bond behaviour between reinforcement and the surrounding concrete were considered. The first case was a rigid model where full bond was assumed between reinforcement and concrete. The second case had a translator element connecting the reinforcement and the sur- rounding concrete which simulated the bond behavior.

The cracking load obtained from the numerical analysis using ABAQUS was 5.69 kN which was 7.97% higher than the experimental value. Similarly, the ultimate load provided from the numerical analysis was 14.49 kN which was 4.17% lower than the ultimate load reported from the experimental study. Fig. 12 describes the stresses of concrete. The stress of stirrupsinthejointreachedtheyieldstressasshownin Fig. 13, therefore the failure mode was joint shear failure due to diagonal tension which coincided with the experimental results. Fig. 14 shows the shape of concrete damage in tension and compression. Plastic strain, shown in Fig. 15, indicates the state and direction of concrete cracks that developed in the numericalmodelwhichagreeswiththeexperimentalresults.

III. PARAMETRICSTUDY

1.7. Studyprogram

A parametric study involving 30 exterior beam column joints wasconductedinthisresearchusingthefiniteelementpackage

Table 3 Numerical analysis results.

Joint No.	Strength capacity (kN)	Deformation capacity (mm)	Joint No.	Strength capacity (kN)	Deformation capacity (mm)
S1	19.90	23.15	S16	19.03	25.02
S2	21.75	21.23	S17	19.27	23.37
S3	24.82	22.58	S18	19.41	23.57
S4	16.27	28.45	S19	19.42	26.88
S5	16.62	24.51	S20	19.64	24.64
S6	17.37	26.15	S21	20.51	24.79
S7	17.61	26.93	S22	19.21	22.11
S8	18.02	25.23	S23	19.981	22.15
S9	18.16	26.05	S24	19.983	22.21
S10	17.87	26.13	S25	19.95	24.20
S11	18.58	25.47	S26	20.73	22.94
S12	18.64	25.86	S27	20.74	23.05
S13	18.28	24.87	S28	20.60	24.38
S14	18.46	22.63	S29	21.43	23.51
S15	18.58	23.89	S30	21.47	23.56

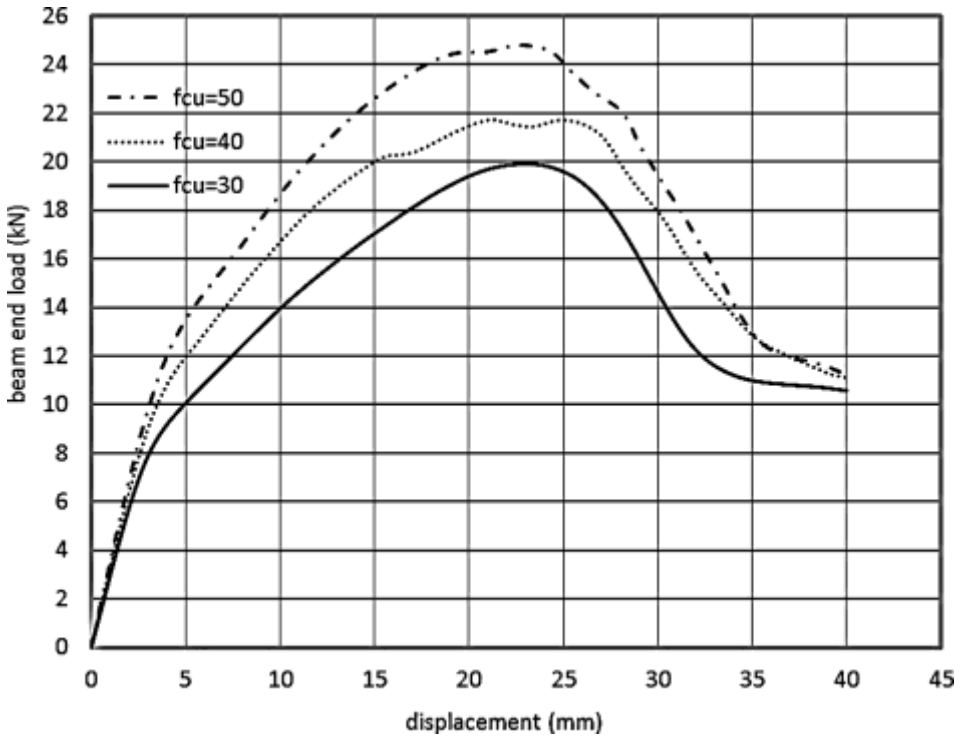


Fig.17 Load-displacement curve for S1,S2, and S3.

ABAQUS and the numerical model described above. The con- sidered parameters were the concrete compressive strength, the column axial load ratio, the number of column stirrups within the joint area and the shape of the beam top reinforcement embedment inside the column. The considered values for concretecompressivestrength(f'_c)were30MPa,40MPaand50MPa.Threevaluesforthecolumnaxialloadratio(P/f'_cA_c)wereconsidered;0,0.3and0.6,wherePisthecolumnaxial load,f'_cistheconcretecylindercompressivestrength,and$A_c$is the gross cross sectional area. The considered number of col- umn stirrups within the joint area was 2, 3 and 4 stirrups. Straight, L-shaped and U-shaped bars were considered for the embedment shape of top beam reinforcement inside the column and named Type1, Type2 and Type3 respectively. The considered parameters are summarized in Table 2. The concrete dimensions and reinforcement values for the beam and column were taken similar to the values considered by Haach et al. (2014) [12]. Fig. 16shows the reinforcement details and dimensions of the specimens with two stirrups within thejoint.

1.8. Results and discussion

The results of the ultimate loads and deformation capacity according to the conducted numerical analyses are shown in Table 3. The failure of all models considered in the parametric study was due to shear diagonal tension within the joint region. The effect of concrete strength on ultimate load is depicted in Fig. 17 which shows the load-displacement curves for models S1, S2 and S3. All three models had an axial load ratio of 0.3, three column stirrups within the joint and L- shaped beam top reinforcement. The figure shows that increas- ing the concrete strength lead to an increase in ultimate load.Fig. 18 shows the relation between the concrete strength and the ultimate load. As the concretestrength increased by 67%, from 30 MPa to 50 MPa, the ultimate load increased by 29%, from 19.27 kN to 24.82 kN. It could be observed that nonlinear relation existed between the concrete strength and the ultimate load which agrees with the typically nonlinear relation between the concrete shear and compressive strengths. Additionally, the difference between the ultimate load and concrete strength increase ratios might be attributed to the fact that the joint shear strength was only partially dependent on theconcretestrengthasthecolumnstirrupsalsohadaconsid-

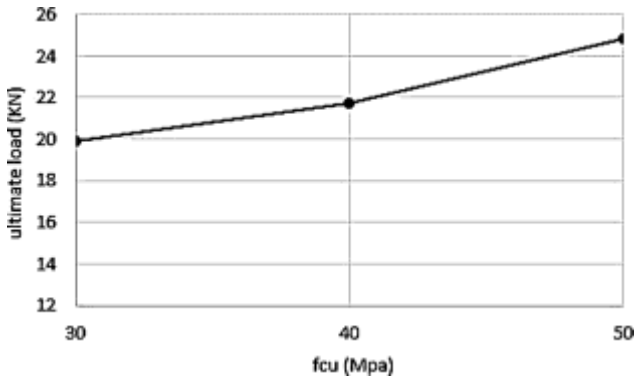


Fig. 18 Effect of concrete compressive strength (no. of stir- rups = 3, axial load ratio = 0.3, Type 2).

erable contribution. Table 3and Fig. 17also show that the beam tip displacement was reduced as the concrete strength was increased. This might be attributed to the increase in con- crete modulus of elasticity typically associated with the increase of concrete strength which resulted in a generally stif- fer model. The deformation capacity at the ultimate load

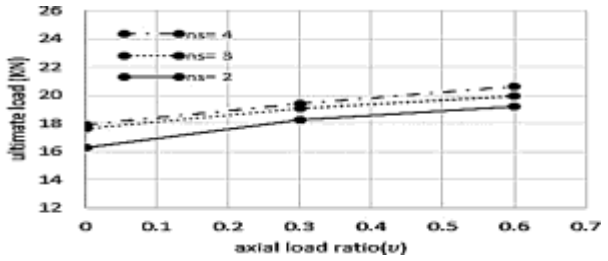


Fig.19 Effectofaxialloadratio(Type1).

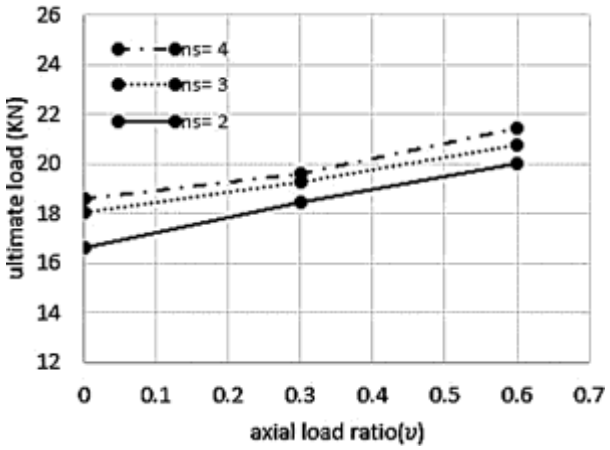


Fig.20 Effectofaxialloadratio(Type2).

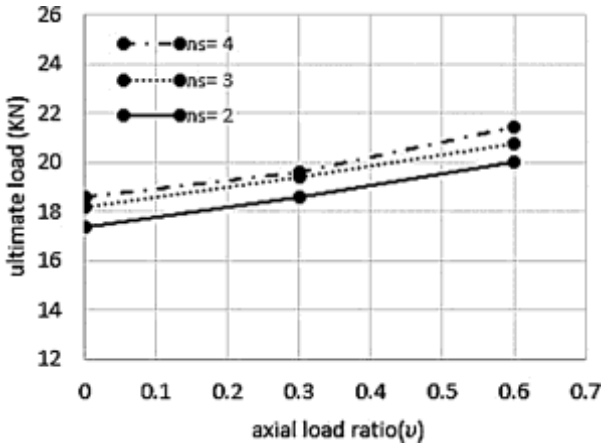


Fig.21 Effectofaxialloadratio(Type3).

decreased by 2.5% as the concrete strength was increased by 67%, despite the increase of the ultimate load by 24.7%.

The effect of the column axial load ratio is shown in Figs. 19–21 for beam top reinforcement shapes Type1, Type2 andType3,respectively.Eachofthethreefiguresshowsthe

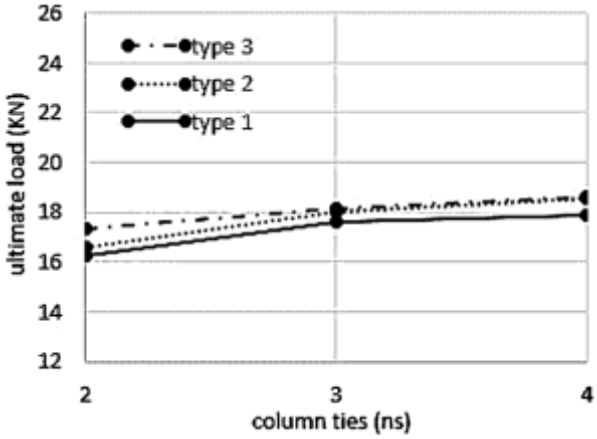


Fig.22 Effectofcolumnstirrups(AxialLoadRatio=0).

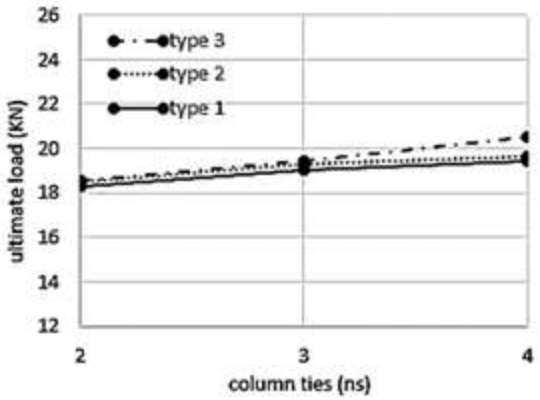


Fig.23 Effectofcolumnstirrups(AxialLoadRatio=0.3).

relation between the ultimate load and the column axial load ratio for the different cases of column stirrups numbers. It could generally be observed that increasing the column axial load resulted in an increase of the ultimate load. Applying axial load at the column top increased the ultimate load by average values of 8.48% and 15.67% for column axial load ratios of 0.3 and 0.6 respectively, compared with the corre- sponding cases without such load. The favorable effect of the column axial load could be attributed to reducing the tensile principal stresses due to jointshear.

Figs. 22–24 show the effect of column stirrups on the ulti- mateloadfor modelswithcolumnaxialloadratiosof0.0, 0.3 and 0.6, respectively. It could be observed thatincreasing

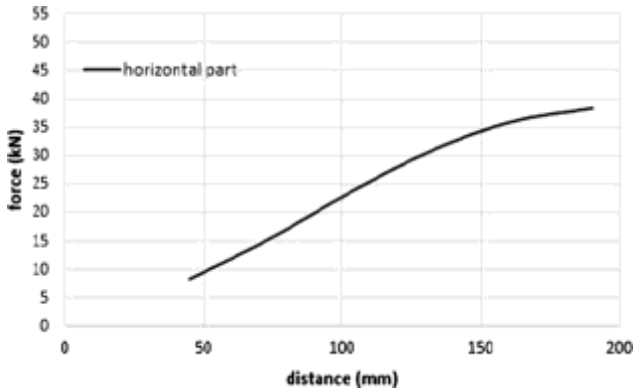


Fig.25 DistributionofsteelforceforType1.

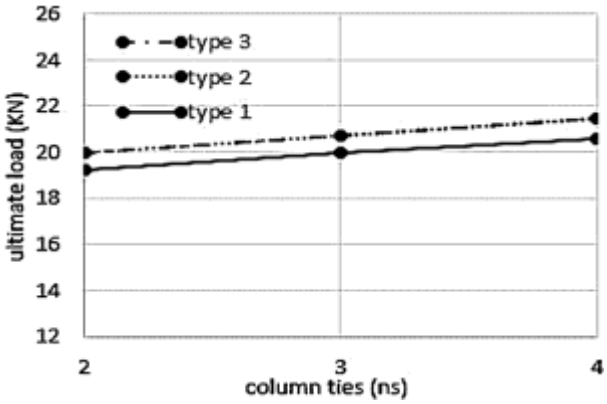
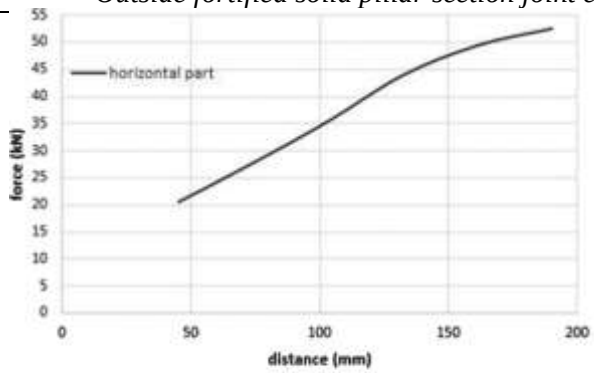
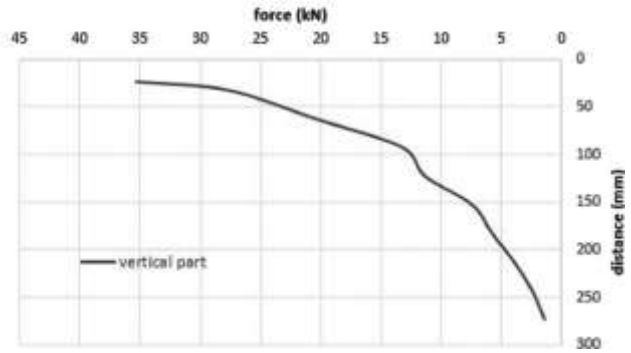


Fig.24 EffectofcolumnSTIRRUPS(AxialLoadRatio=0.6).



(a)Horizontal Part



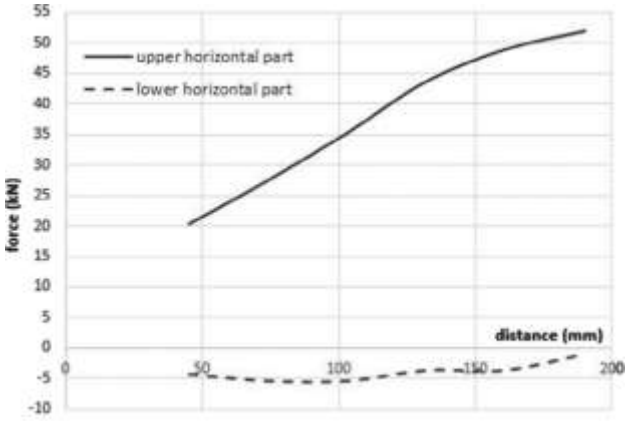
(b) Vertical Part

Fig.26 DistributionofsteelforceforType2.

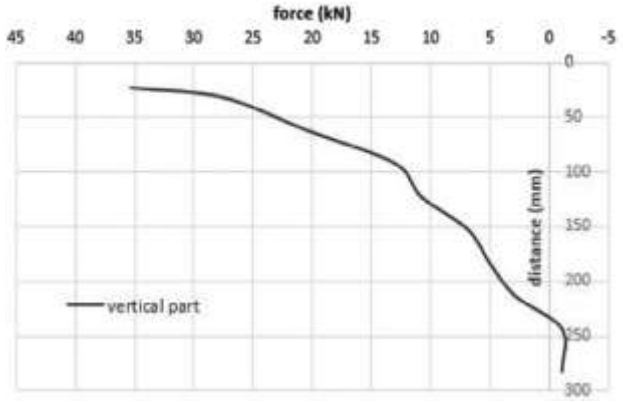
the number of column stirrups within the joint region favor- ably affected the ultimate load. Increasing the number of col- umn stirrups to 3 and 4 resulted in increasing the ultimate load by average values of 4.92% and 8.08%, respectively,compared withthecorrespondingcasesoftwocolumnstirrups.

The shape of the beam top reinforcement also affected the ultimate load of the considered models as shownin Figs. 22–24as well. The use of straight bars (Type1) resulted in the least ultimate loads compared with L- (Type2) and U- (Type3) shaped bars. This could be attributed to the additional confin- ing effect produced by the L- and U-shaped bars in addition to their contribution in resisting shear cracks in the joint region. The use of L- and U-shaped bars produced similar results as far as the ultimate load and deformation capacity were con- cerned. In average, the ultimate load was increased by 2.68% and 3.96% for models with L- and U-shaped bars respectively, compared with corresponding models with straightbars.

Figs. 25–27show an example of the distribution of steel force in the beam top reinforcement in the joint region at fail-ureforType1,Type2andType3,respectively.Thisforce dis- tribution is shown for models S13, S14 and S15 with concrete cylinder strength of 30 Mpa, axial load ratio of 0.3 and twojoint stirrups. The tensile force in the top bar for Type 1 almost diminished at the end of the bar while, for Type 2, a consider- able portion of the force existed at the end of the horizontal part and transmitted to the vertical part. The top bar tensile forceinType3wasalmostsimilartothatofType2forboth



(a)Horizontal Part



(b) Vertical Part

Fig.27 DistributionofsteelforceforType3.

the top horizontal part and the vertical part. No tensile force was transmitted to the lower horizontal part. This indicated that the L-shaped (Type 2) bars had an effective and beneficial contribution to the reinforcement anchorage compared with the straight bars. The U-shaped (Type 3) bars had an insignif- icanteffectonthetopreinforcementanchorage.

IV. CONCLUSIONS

A numerical investigation was performed to study the behav- ior of exterior reinforced concrete beam column joints sub- jected to monotonic loading. Thirty numerical models were analyzed using the finite element package ABAQUS to evalu- ate the effect of concrete strength, column axial load, joint stir- rups and shape of beam top reinforcement onthe joint ultimate load and deformation capacity. Based on the resultsof the conducted parametric study, the following conclusions could bemade:

1) Increasingthe concrete strength from 30 MPa to 50 MPa resulted in increasing the beam tip ultimate load by 24.7% and decreasing the deformation capacity at the ultimate load by2.5%.
2) For the studied joints, where the joint failure was due to shear diagonal tension, increasing the column axial load resulted in increasing the beam tip ultimate load. Aver- age ultimate load increase values of 8.48% and 15.67% were obtained for column axial load ratios of 0.3 and 0.6, respectively, compared with the corresponding cases of unloadedcolumns.
3) Increasing the column stirrups within the joint region favorably affected the beam tip ultimate load. Increasing the number of column stirrups to 3 and 4 resulted in increasing the ultimate load by average values of 4.92% and 8.08%, respectively, compared with the cor- responding cases of two columnstirrups.
4) The use of straight bars for beam top reinforcement resulted in generally lower ultimate loads than those obtained with L- and U-shaped bars. Similar joint behavior was demonstrated for the cases of using L- and U-shaped beam topreinforcement.

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