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Stud reinforcement in beam-column joints under seismic loads

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Abstract. Current codes recommend large amounts of shear reinforcement for reinforced concrete beam-column joints that causes significant bar congestion. Increase in congestion of shear reinforcement in joint core (connection zone), leads to increase accomplishment problems. The congestion may also lead to diameter limitations on the beam bars relative to the joint dimensions. Using double headed studs instead of conventional closed hoops in reinforced concrete beam-column joints reduces congestion and ensures easier assembly of the reinforcing cage. The purpose of this research is evaluating the efficiency of the proposed reinforcement. In this way, 10 groups of exterior beam-column joints are modeled. Each group includes 7 specimens by different reinforcing details in their joint core. All specimens are modeled by using of ABAQUS and analyzed subjected to cyclic loading. After verification of analytical modeling with an experimental specimen, 3D nonlinear specimens are modeled and analyzed. Then, the effect of amount and arrangement of headed studs on ductility, performance, ultimate strength and energy absorption has been studied. Based on the results, all joints reinforced with double headed studs represent better performance compared with the joints without shear transverse reinforcement in joints core. The behavior of the former is close to joints reinforced with closed hoops and cross ties according to the seismic design codes. By adjusting the arrangement of double-headed studs, the decrease in ductility, performance, ultimate moment resistant and energy absorption reduce to 2.61%, 0.90%, 0.90% and 1.66% respectively compared with the joints reinforced by closed hoops on the average. Since the use of headed studs reduces accomplishment problems, these amounts are negligible. Therefore, use of double-headed studs has proved to be a viable option for reinforcing exterior beam-column joints.

Keywords: exterior RC beam-column joint; ductility; performance; ultimate resistant moment; energy absorption

1. Introduction

After the earthquakes of Kobe (1995), Taiwan (1991), Turkey (1991) and Indonesia (2004), people in different parts of the world, have been more aware of outcomes of this natural disaster (Wong 2005). In the past, this issue has been noticed only in high-seismicity regions. The

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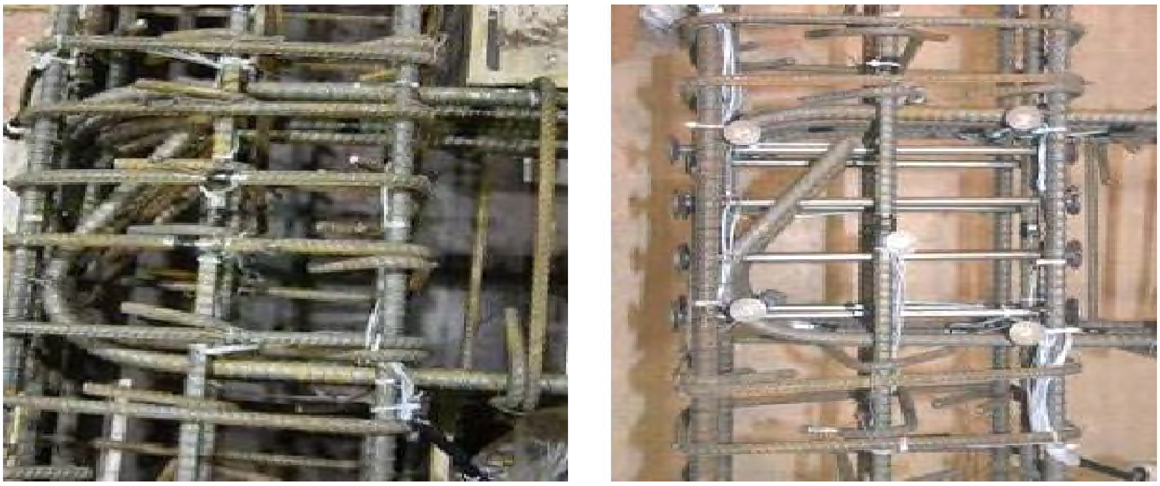
earthquake of 1991 in Australia, a relatively weak earthquake (M= 5.6), made 2.5 billion dollars damage. After this earthquake, more attention is devoted to the potential seismic hazard in regions with moderate seismicity (Ibrahim 2011). Cyclic behavior of structures is an important issue in seismic analysis and design. This behavior is influenced by different parameters such as connections that are among the most critical parts of structures in energy absorption and depreciation of energy (Khalifa and Alnaji 2008). Reinforced concrete beam-column joints, particularly exterior ones, which have poorly detailed in joint core, frequently fail by diagonal tension cracking resulting from high shear forces when insufficient shear reinforcement is provided. Current codes recommend large amounts of shear reinforcement for reinforced concrete beam-column joints causing significant congestion (Paulay and Priestley 1992). Early studies of structural behavior have been performed by Hansen and Conner in Portland Cement Laboratories Hanson and Conner 2008). Since then this has been considered by researchers from Canada, Japan and New-Zealand. However the goals of these researches were different, but their main emphasis was ductile behavior creation and appropriate performance in cyclic loadings. Results of these researchers lead to collection of the first code for design of reinforced concrete connections (Ueda and Hawkins 1986). ACI-ASCE352 Committee published the first design recommendations in 1976. Simultaneously much more recommendations were published by different codes. More researches lead to modification of ACI-ASCE352 in 1985 and 2000 (Mostofinezhad and Sobhani 2003). One of the most common models that is used for analyzing exterior beam-column joints is strut and tie model (STM). This model can be used for calculating the loads acting in the truss members, calculating the needed amount of reinforcement required and to choose dimensions of the concrete struts. The provisions of ACI318-08 code (2008) and CSA A23.3-04 Standards (2004) for design and detailing of beam-column joints are based mainly on this model.

Other one of the basic methods in design of beams against shear and torsion is using of strut and tie model (STM). The concept of STM is first appeared about hundred years ago when Ritter and Mörsch introduced independently the truss analogy for shear design of beams (Tjhin and Kuchma 2002). Then Hwang and Lee proposed softened strut and tie model (SSTM) for prediction of shear resistance of reinforced concrete external beam- column joints (Hwang and Lee 1999; Hwang and Lee 2002). Just mentioned earlier, placing the hooks and bends within the external joints core, lead to congestion of the reinforcement, causing construction difficulties. A special type of reinforcement in external joints core, known as headed studs has been developed by Dilger and Ghali (1981) at the University of Calgary for reinforcing thin concrete flat plates against punching shear in areas around the columns. Fig. 1 shows a double-headed stud. The stem of double-headed studs is normally plain without deformations. The area of the head is 9 to 10 times the area of the stem. This ensures that full yielding of the stem develops, with negligible slip, immediately behind the head (Ghali and Dilger 1998).

Fig. 2 represents two joint cores that the first reinforced with closed hoops and the second reinforced with double headed studs. In Fig. 2, it is clearly observed that the use of double headed studs reduces congestion.

Fig. 1 A double headed stud



(a) (b)

Fig. 2 Joint core (a) Reinforced with closed hoops (b) Reinforced with double headed studs

The stud shear reinforcement has been used in flat slabs, footings, and raft foundations of hundreds of structures around the world. Double-headed studs have been proposed for many other applications. Several recent investigations have shown that stud reinforcement provides better confinement and also enhanced ductility of concrete elements. Tests have been carried out on corbels reinforced with double-headed studs as primary tension reinforcement (Birkle *et al.* 2002) and on I-beams by the studs used as web shear reinforcement (Gayed and Ghali 2004). Some tests have been conducted to demonstrate the effectiveness of double-headed studs in confining concrete in columns (Youakim 2002; Youakim and Ghali 2002; Youakim and Ghali 2003), use of double headed studs in shear walls (Mobeen *et al.* 2005) and use of headed reinforcement for provide anchorage length instead of standard hooks (Thompson *et al.* 2002). In this research the aim is to consider effect of amount and arrangement of double-headed studs on ductility, performance, ultimate strength and energy absorption. Therefore, 10 groups of exterior beam-column joints are modeled, so that each group includes 7 specimens by different reinforcing details in joint core. These specimens are modeled three-dimensionally by using of ABAQUS software (2008). The specimens are analyzed by non-linear analysis under seismic loading that is simulated by cyclic loading.

2. Specification of investigated samples

Because of dimensional limitations, specimens with smaller scales are usually used for experimental tests, but computer models can be modeled in the size of real joints usually used in structural application. As mentioned earlier, investigated samples in this research, include ten joint groups, namely G1 to G10, each containing seven samples with different reinforcing details in joint core. Dimensions of samples in each group are given in Table 1 and Fig. 3. The height of stories that these joints selected from them, is 3 meter and the length of beam span is 4 meter. In different stories, dimension of beams and columns are various.

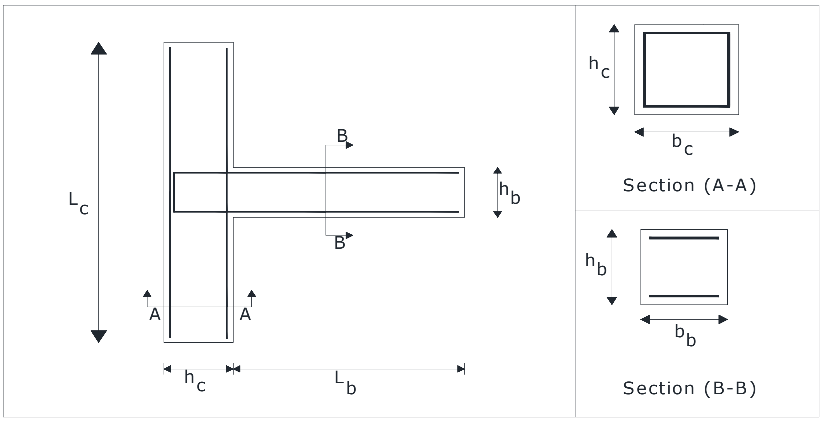


Fig. 3 Connection geometric specification

Table 1 Studied connection dimensions in each group

Group	L_c(mm)	L_b(mm)	b_c(mm)	h_c(mm)	b_b(mm)	h_b(mm)
G1	3000	2000	300	400	300	400
G2	3000	2000	400	400	400	400
G3	3000	2000	400	500	300	400
G4	3000	2000	400	500	400	500
G5	3000	2000	500	500	300	500
G6	3000	2000	400	600	400	600
G7	3000	2000	500	600	300	500
G8	3000	2000	500	600	400	600
G9	3000	2000	600	600	300	400
G10	3000	2000	600	600	400	500

Specimens in each group are named in the figure of S1 to S7. Reinforcing details for each specimen in its core is explained as follow:

Specimen S1: Specimen S1 is a control specimen designed as a shear-deficient specimen not provided with any transverse shear reinforcement in the joint. According to ACI-318-05 (2005), appropriate longitudinal reinforcement is between $0.35\rho_b$ to $0.40\rho_b$ that ρ_b is percent of longitudinal reinforcement in balanced section. Then, longitudinal reinforcement in the beam of desired joint is $\rho=0.35\rho_b$. Percent of longitudinal reinforcement in the column of desired connection is selected 2 percent. In other words, connection has low reinforcement level. Transverse reinforcement of joints is calculated based on the ACI318-05 (2005) requirements. These shear reinforcement are spaced in the shape of closed hoops. Eq. 1 represents equations for transverse reinforcement of beams.

$$\left(\frac{A_v}{s_v}\right)_{req}=\frac{\frac{V_u}{\Phi}-V_b}{F_y d_b} \qquad \left(\frac{A_v}{S_v}\right)_{\min}=\frac{1}{16}\sqrt{f'_c}\,\frac{b_b}{F_y}\geq\frac{1}{3}\frac{b_b}{F_y} \tag{1}$$

$$V_b=\frac{1}{6}\sqrt{f'_c}\,b_b d_b \qquad S_{\max}=Min\left\{\frac{d_b}{2},600\right\}$$

Where:

$(A_v/S)_{req}$ = Required respect of area of transverse reinforcement to their space from each other (mm)

$(A_v/S)_{min}$ = Minimum respect of area of transverse reinforcement to their space from each other (mm)

V_u and V_b = ultimate shear applied and shear capacity of concrete in beam section (N) respectively.

F_y and f'_c = yield steel stress in the beam bars and concrete compressive strength (Mpa) respectively.

b_b and d_b = beam width and beam effective depth (mm) respectively.

S_{max}= maximum space transverse bars in the shape of closed hoops (mm)

Eq. (2) represents equation for transverse reinforcement of columns.

$$\phi_L \le 32mm \Rightarrow use \qquad \phi_T = 10mm \qquad S_{\max} = \min\{16(\phi_L), 48(\phi_T), h_{\min}\} \tag{2}$$

In Eq. 2, Φ_L , Φ_T , h_{min} and S_{max} are diameter of longitudinal bars (mm), diameter of transverse bars (mm), dimension of smaller side of column section (mm) and maximum space transverse bars (mm), respectively.

Specimen S2: Specimen S2 is another control specimen similar to specimen S1. In this specimen, regions of beam and column that are close to joint core and also joint core are reinforced on the basis of ACI318-05 (2005) requirements. For this purpose, minimum length from two ends of column that transverse reinforcement as closed hoops must arrange by a minimum allowable area and maximum allowable space, are calculated by Eq. 3:

$$\begin{aligned} &L_0 = \max\left\{h, \frac{L_n}{6}, 45cm\right\} \qquad A_{sh} = 0.3(s)(b_c)(\frac{f'_c}{F_y})(\frac{A_g}{A_{ch}} - 1) \ge (0.09)(s)(b_c)(\frac{f'_c}{F_y}) \\ &S_{\max} = \min\left\{\frac{h_{\min}}{4}, 6\Phi_L, 100 + \frac{350 - h_x}{3}, 15cm\right\} \ge 10cm \end{aligned} \tag{3}$$

where: L_0= minimum length from two ends of column that should provide horizontal confinement (cm)

A_{sh}= minimum area of transverse bars in the figure of closed hoops in L_0 length (mm^2)

S_{max}= maximum space transverse bars in the figure of closed hoops in L_0 length (cm)

h and A_g = column height (cm) and total area of column (mm^2) respectively.

L_n and f'c = length of compressive member span (cm) and concrete compressive strength (Mpa)

S and F_y = space transverse bars (mm) and yield stress in the beam transverse bars (Mpa), respectively

b_c and A_{ch} = dimension of column's core (mm) and area of column's rectangular core (mm^2) respectively

h_{min}= dimension of smaller side of column section (cm)

h_x and Φ_L = maximum space between branches of hoops (cm) and diameter of longitudinal bars (cm)

Specimen S3: In specimens S3, horizontal and vertical double-headed studs were designed according to the Softened Strut-and-Tie Model (SSTM) (Hwang and Lee 1999; Hwang and Lee 2002). The forces in the horizontal and vertical ties, F_h and F_v, and in the diagonal strut of joint core, D, can be calculated from Eq. (4)

$$F_h = (R_h)(V_{jh}) \qquad F_v = (\frac{Rv}{\cot(\theta)})(V_{jh}) \qquad F_d = (\frac{Rd}{\cos(\theta)})(V_{jh})$$

$$R_h = \frac{\gamma_h(1-\gamma_v)}{1-(\gamma_h)(\gamma_v)} \qquad R_v = \frac{\gamma_v(1-\gamma_h)}{1-(\gamma_h)(\gamma_v)} \qquad R_d = \frac{(1-\gamma_v)(1-\gamma_h)}{1-(\gamma_h)(\gamma_v)} \tag{4}$$

$$\gamma_h = \frac{2\tan(\theta)-1}{3}, 0 \le \gamma_h \le 1 \qquad \gamma_v = \frac{2\cot(\theta)-1}{3}, 0 \le \gamma_v \le 1$$

where:
F_h and F_v = force in horizontal double headed stud and force in vertical double headed stud (N) respectively

Table 2 Area of double headed studs in different specimens (cm^2)

			G1	G2	G3	G4	G5	G6	G7	G8	G9	G10
S3	Horizontal	In-Plane	2.0106	3.0162	2.0106	3.0162	4.0216	3.0162	4.0216	4.0216	5.027	5.027
		out-plane	2.0106	3.0162	2.0106	3.0162	4.7124	3.0162	4.7124	4.7124	4.0216	4.0216
S4	Horizontal	In-Plane	4.7124	7.0686	4.7124	7.0686	9.4248	7.0686	9.4248	9.4248	7.854	9.4248
		out-plane	4.7124	6.7854	4.7124	6.7854	9.0472	6.7854	9.0472	9.0472	6.7854	9.0472
S5	Horizontal	In-Plane	4.7124	7.0686	4.7124	7.0686	9.4248	7.0686	9.4248	9.4248	7.854	9.4248
		out-plane	2.0106	3.0162	2.0106	3.0162	4.7124	3.0162	4.7124	4.7124	4.0216	4.0216
S6	Horizontal	In-Plane	2.0106	3.0162	2.0106	3.0162	4.0216	3.0162	4.0216	4.0216	5.027	5.027
		out-plane	4.7124	7.0686	4.7124	7.0686	9.0472	7.0686	9.0472	9.0472	6.7854	9.0472
S7	Horizontal	In-Plane	2.0106	3.0162	2.0106	3.0162	4.0216	3.0162	4.0216	4.0216	5.027	5.027
		out-plane	2.0106	3.0162	2.0106	3.0162	4.7124	3.0162	4.7124	4.7124	4.0216	4.0216
	Vertical	In-Plane	1.0053	1.5081	1.0053	1.5081	2.0108	1.5081	2.0108	2.0108	2.5135	2.0108
		out-plane	0	0	0	0	0	0	0	0	0	0

• Specimens S1 and S2 have no double headed studs.

• Specimens S3 to S6 have no vertical double headed studs

D and V_{jh} = force in diagonal strut of joint core and horizontal shear applied to joint (N) respectively
R_h, R_v, R_d and θ = distribution factors and the angle of inclination, respectively

By attention to obtained horizontal and vertical forces and yield strength of double-headed studs, the areas of the studs are calculated .Their stresses don't reach yield stress under the implied loads. In other words, area of double-headed studs should be enough to carry the force in horizontal and vertical ties. In this method, the area of double-headed studs in both in-plane and out-of-plane directions about 45-60 percent of the amount obtained from ACI318-05 (2005) provisions.

Specimen S4: In this specimen, all closed hoops of specimen S2 in joint core are replaced by double-headed studs in in-plane and out-of-plane directions, so that the area of double-headed studs are equivalent with the area of closed hoops calculated based on the ACI318-05 (2005)

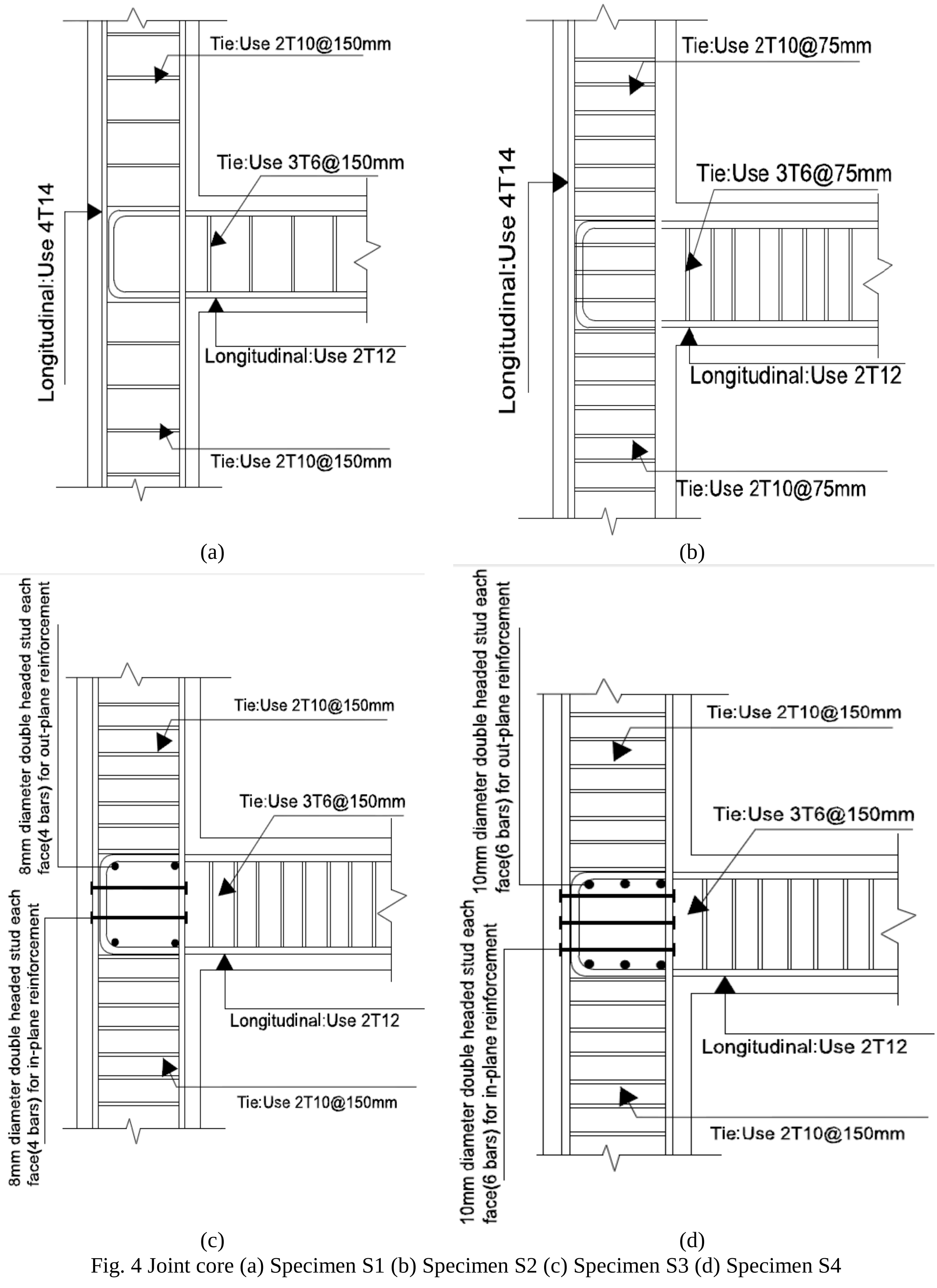
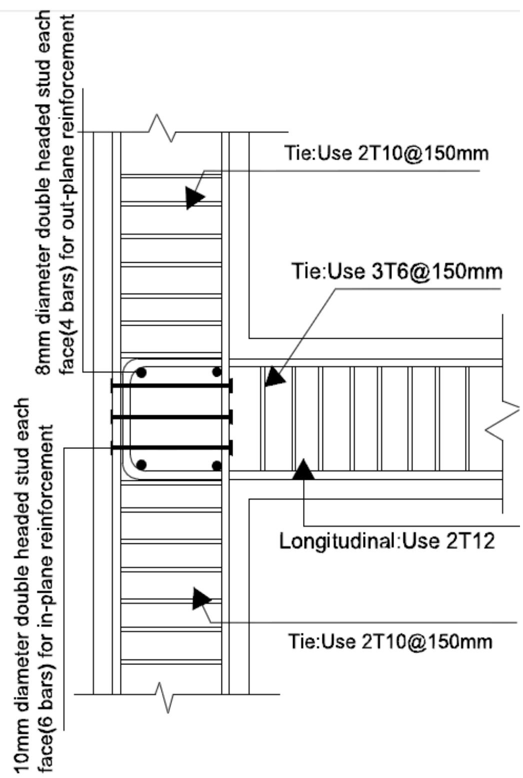
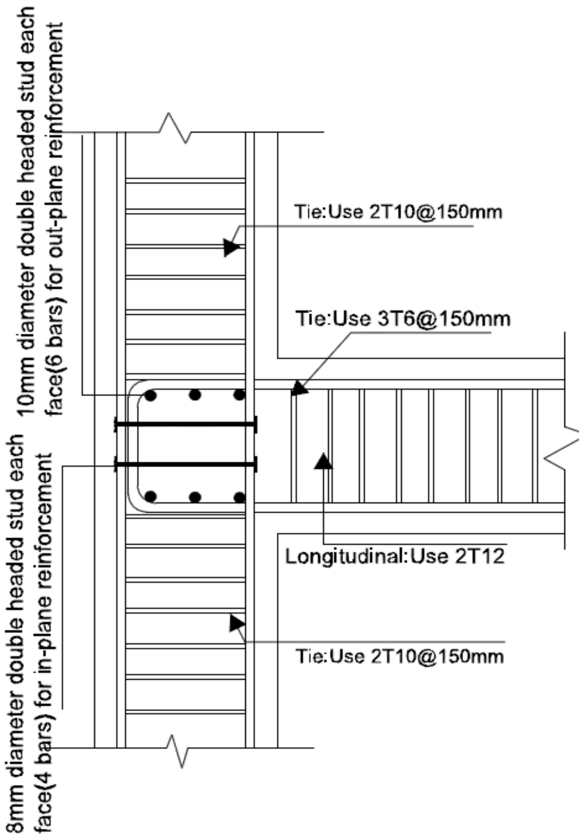


Fig. 4 Joint core (a) Specimen S1 (b) Specimen S2 (c) Specimen S3 (d) Specimen S4



(e)



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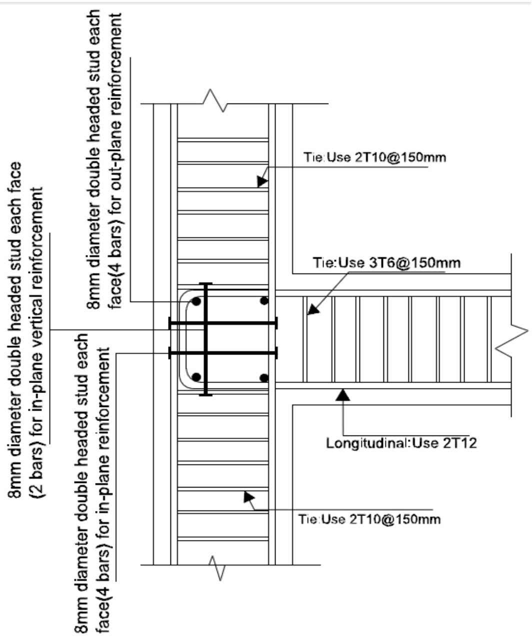

(g)

Fig. 4 Continued

provisions.

Specimen S5: This specimen is investigated in order to assess the effect of in-plane confinement on the behavior of the joint. For this purpose, this specimen has the same reinforcement as specimen S2, calculated based on ACI318-05 (2005) provisions, in in-plane direction. In out-of-plane direction, the specimen has the same reinforcement as specimen S3, calculated by using of the SSTM method.

Specimen S6:This specimen is investigated in order to assess the effect of out-of-plane confinement on the behavior of the joint. For this purpose, this specimen has the same reinforcement as specimen S3, calculated by using of SSTM method, in in-plane direction. In out-of-plane direction, the specimen has the same reinforcement as specimen S2, calculated based on the ACI318-05 (2005) provisions.

Specimen S7:This specimen was investigated in order to assess the effect of in-plane confinement in the vertical direction on the behavior of the joint. In this way, the specimen designed according to the same SSTM model as specimen S3, except that the vertical studs are provided in the joint core, so that the area of vertical studs is about 40-50 percent of the total area of horizontal studs in in-plane direction. The vertical studs are not centered in the joint core, but are placed at $\frac{1}{4}$ distance from the outer edge of the column. The purpose is to delay and control the diagonal cracks that occur at the column edge, triggering the joint failure. Table 2 shows the area of double-headed studs used in different specimens. Fig. 4 represents detail of reinforcement in joints core of specimens S1 to S7 in G1 group.

3. Modeling and analyzing the joints

Modeling and analyzing reinforced concrete connections is done by ABAQUS software. Details of these processes are as following:

3.1 Materials

For defining the concrete material in the software, concrete damage plasticity model is applied. This model is a synthetic model that is capable to consider fracture caused by pressure and tension in concrete simultaneously.

In concrete damaged plasticity model, most significant mechanisms of concrete fracture are tensile cracking and pressured crushing. To define yield surface in this model, it is required that its parameters be defined in the software (Thompson *et al.* 2002).

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E_C is concrete elasticity modulus in pressure that can be obtained by the Eq. (5) (ACI318 1995)

$$E_c = 4700\sqrt{f_c'}(Mpa) \tag{5}$$

Other parameters are as Eq. (6) (ACI318 1995, 2005)

$$f_t = 0.6\sqrt{f_c'(Mpa)} \qquad \varepsilon_{cr} = 0.0001 \qquad \varepsilon_{cr-u} \approx 10\varepsilon_{cr} = 0.001 \tag{6}$$

where:

f_t= stress equivalent concrete fraction in uniaxial tension(Mpa)

ε_{cr}= strain equivalent concrete fraction in tension

ε_{cr-u}= strain equivalent with tensile stress equal to zero, that obtained for over limit opening in crack span.

Poison ratio, concrete elasticity modulus and concrete compressive strength in the modeling of this research are as following:

Stress-strain diagram of concrete in tension and pressure is shown in Fig. 5

$\vartheta = 0.2 \qquad E_c = 257430 kg/cm^2 \qquad f'_c = 30 Mpa$

Parameters dt and dc in Fig. 5 are named damage parameters. These parameters determine concrete stiffness in different points in stress strain diagram. Maximum amount of these parameters is equal to 1, and in case that amount of them are assumed equal to zero, it means that Material stiffness in loading and unloading cycles is constant. Values of these parameters are

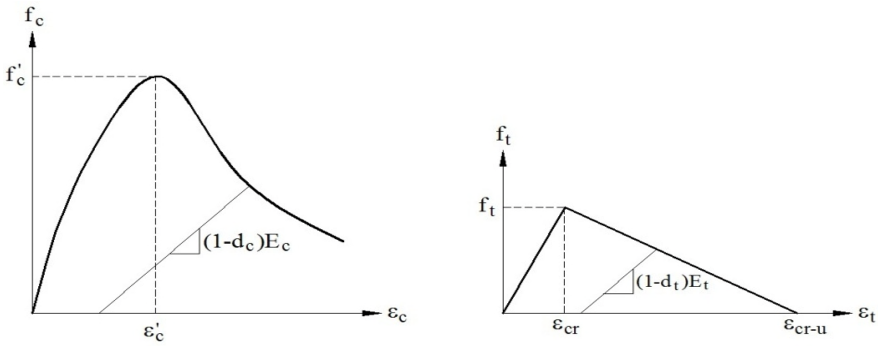

Fig. 5 Stress-strain diagram of concrete a) Behavior in pressure b) Behavior in tension

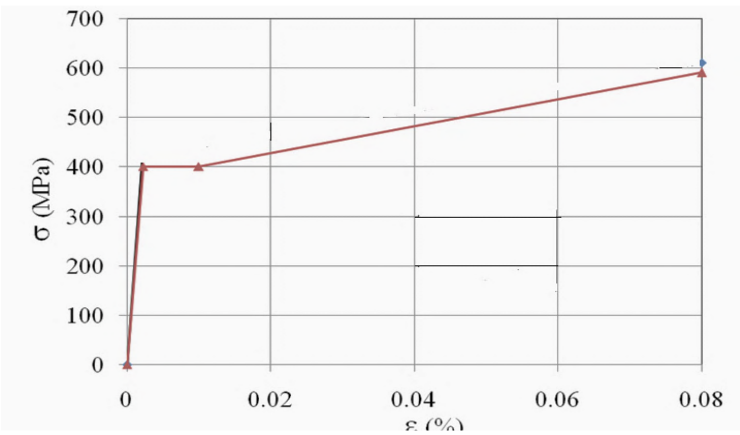

Fig. 6 Stress-strain diagram of steel

functions of geometry and type of model of reinforced concrete, and in different structures, they are determined from trial and error procedure and from comparison with experimental results or other reliable analysis. In this study, damage parameters of concrete are assumed equal to 0.5 and zero in compression and tension respectively. Steel rebar and double headed studs behavior is considered linear and with hardening. This behavior is equal in pressure and tension and materials elasticity modulus in unloading is assumed to be equal to primary elasticity modulus. In cyclic loading, it is abandoned from restricted fading that is created in stiffness and strength. Poison ratio and elasticity modulus of steel are assumed as following:

In this research, only one type of steel for all longitudinal and transverse bars with yield stress of 400 MPa is used (Fig. 6).

$$\vartheta = 0.3 \qquad E_S = 2\times10^6 kg\ /cm^2$$

For support plates and load transfer zones, rigid properties are used. For rigid materials used in this model, only elastic properties are defined as follows: $\vartheta = 0$ $\qquad E = 2\times10^9 kg/cm^2$

3.2 Analysis options

ABAQUS software can do various types of simulations that most significant of them are statical and dynamical analyses. In static analysis, long time response of structure against implied fores is obtained. In other cases, dynamical response of structure under implied forces is desired. Desired type of analysis in this research is General Static analysis that can calculate linear and nonlinear responses against implied forces (ABAQUS 2008).

3.3 Determination of the type of interactions

A lot of engineering problems contain contacts between two or several elements. In these problems, as two elements contact each other, a force is applied normal to the contact surface. In available models, two types of interactions are identified:

1. Interaction between steel surface with concrete surface, interaction between rigid surface with concrete surface and interaction between double-headed studs with steel bars. For description (definition) of interaction between steel and concrete surfaces, interaction between rigid and concrete surfaces or interaction between double-headed studs and steel bars, tie constraint is used. A tie constraint ties two separate surfaces together so that there is no relative motion between them. This type of constraint allows you to fuse together two regions even though the meshes created on the tied surfaces may be dissimilar (Li and Tran 2009).

2. Interaction between longitudinal and transverse rebars with concrete and interaction between double-headed studs with concrete. For description of interaction between longitudinal and transverse rebars with concrete and interaction between double headed-studs with concrete , embedded element technique is used. The embedded element technique is used to specify an element or group of elements that is embedded in “host” elements. In this method, the translational degrees of freedom of the embedded nodes are restricted (Li and Tran 2009).

3.4 Loading model

Imposed loads on specimens are represented in Fig. 1. Axial load of the column is selected so

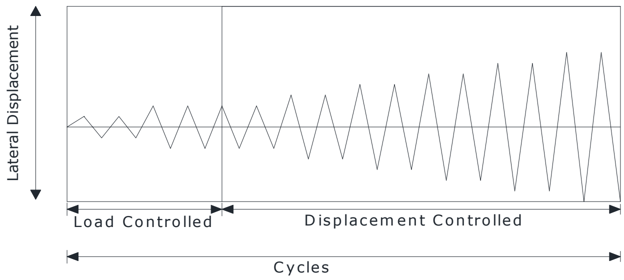

Fig. 7 Loading diagram

that desired joint would have best performance under the load. The value of this load is equal to $P_c = 0.2 f'_c A_g$, where (Sangjoon and Khalid 2012) where P_c, f'_c and A_g are Axial loading applied on column (N), concrete compressive strength (Mpa) and total area of column (mm^2), respectively.

As in practical applications, axial forces of beams are negligible, so no axial load is applied on beam in modelings. For simulation of earthquake loading, cyclic loading is used as shown in Fig. 7. First and second cyclic loadings are applied on the basis of load control and their amounts are $0.5P_y$ and P_y respectively. P_y is corresponding force at which longitudinal bars of beam yield. Next cycles have been applied based on displacement control and their amounts are Δ_y, $2\Delta_y$, $3\Delta_y$, $4\Delta_y$, $6\Delta_y$ and $8\Delta_y$ respectively. Δ_y is corresponding displacement at which longitudinal bars of beam yield (Shirazi 2011). This loading is applied on the beam as shear force so that each two consecutive cycles, involve equal P and equal Δ in load control and displacement control phases respectively. P_y and Δ_y are force and displacement corresponding to yielding point affected by monotonic loading, respectively. For determining yielding point of longitudinal bars in monotonic loading, the stress should be identified in critical element of longitudinal bars affected by tension. The first step in which yielding stress is reached, would be considered as yielding point, and corresponding force and displacement would be considered as Δ_y and P_y respectively.

3.5 Meshing of parts

To mesh the shell elements, following general points are considered:

1. The shape of elements is selected Quad-dominate. In this method, quadrilateral elements are used for meshing as possible. In regions that it is not possible to use quadrilateral elements, the meshing is done by triangle elements.

2. The structural meshing technique is selected in this research. This technique creats a regular meshwork by use of simple surfaces.

3. To determine type of elements, following general points are considered:

• The type of element is selected standard. Because this type of element contain spacious span of linear and nonlinear problem, such as statical and dynamical problems.

• The geometric order of element is selected linear.

• The family of element is selected plain strain. Plain strain enhances the convergence of answers in comparison with plain stress.

4. Dimension of meshworks are considered 5cm. In this state, there is a very suitable compatibility between analytical model and experimental results.

To mesh the wire elements, following general points are considered

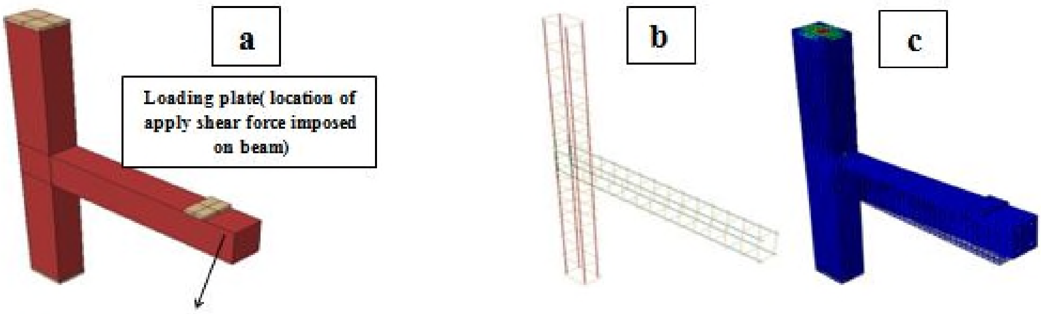

Fig. 8 3D model in ABAQUS (a) Concrete structure (b) Reinforcing cage in 3D model (c) Structure after loading and in time of collapse

Table 3 Specifications of the test specimen

	Beam	Column	
Section (cm)	15×20	15×20	
Length (cm)	100	150	
Longitudinal bars	2Φ14 (1%)	4Φ14 (2%)	
Transvrse bars	Φ8@80	out of joint core	Φ8@150
		in the joint core	Φ8@150

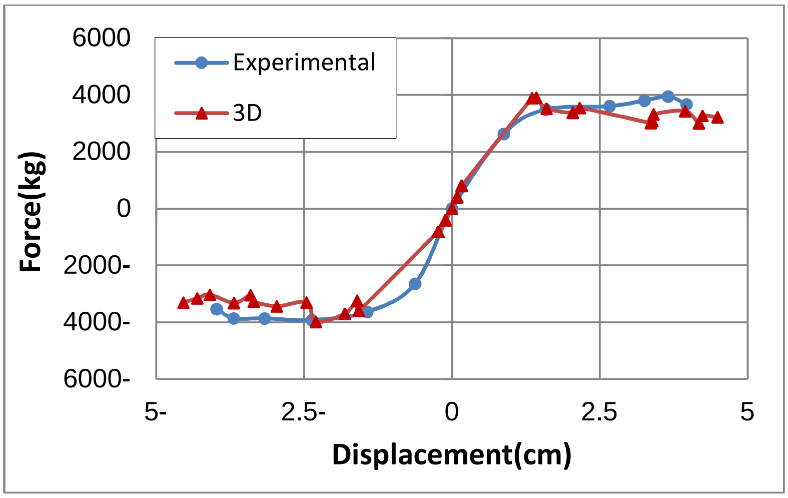

Fig. 9 Comparing enveloped diagrams of experimental and analytical results affected by cyclic loading

1- The type of element is selected standard.
2- The geometric order of element is selected linear.
3- The family of element is selected truss elements.
4- Dimension of meshworks are considered 5cm. In this state, there is a very suitable compatibility between analytical model and experimental results.

In Fig. 8 represent concrete joint, reinforcing cage and concrete joint after loading and collapse, respectively.

Table 4 Ductility of specimens in various groups

	S2	S4	S5	S6	S3	S7	S1
G1	3.3844	3.284	3.1462	2.9967	2.9035	2.7512	1.8702
G2	3.9332	3.8522	3.6970	3.5413	3.4509	3.3068	2.2960
G3	3.2113	3.0991	2.9511	2.8127	2.7362	2.6976	1.7549
G4	3.6574	3.5578	3.3571	3.2525	3.1704	3.0556	2.0523
G5	3.9026	3.8078	3.6910	3.4306	3.3688	3.2463	2.1901
G6	3.8142	3.7125	3.5578	3.4069	3.3203	3.2056	2.1504
G7	3.9247	3.8015	3.6820	3.5404	3.4463	3.3401	2.1764
G8	4.1669	4.1008	3.9166	3.7031	3.6643	3.5289	2.4011
G9	3.2650	3.1781	3.0648	2.9915	2.9154	2.8034	1.8639
G10	4.2563	4.1842	4.0026	3.8554	3.7912	3.5091	2.4174

4. Compatibility of analytical model and experimental results

Experimental model is a reinforced concrete beam-column external joint with specifications given in Table 3, that is tested at structural laboratory of Ferdowsi Mashhad university (Shirazi 2011). The model are tested under cyclic loading, and cyclic and envelope force displacement diagrams are obtained. The experimental model is simulated three dimensionally in ABAQUS software. Simulated model is analyzed under cyclic loading, and cyclic and envelope force displacement diagrams are obtained**.**

Compatibility of experimental and analytical models is tested by comparing their envelope force displacement diagrams as shown in Fig. 9. Very good conformity of these diagrams to each other is observable. To verify the compatibility of experimental and analytical models quantitatively, correlation coefficient of the envelope force displacement diagram is calculated.

This coefficient has a value of 0.9006, very close to 1. Thus, it can be verified that analytical and experimental models have very good agreement and that modeling is performed with a very good accuracy.

5. Behavior of external joints

Since in most of design codes, structural members of buildings, such as beams, columns and joints, are designed so that, under weak to medium earthquakes, remain in elastic behavior phase, and under strong earthquakes, experience inelastic behavior with providing enough ductility. In inelastic phase, input energy to the system is dissipated as hysteresis energy. Other control in design codes is preventing the structure from becoming unstable. In this research, four parameters: ductility, performance, ultimate strength and energy absorption, is selected to evaluate the behavior of joints.

5.1 Ductility

The philosophy of seismic design of concrete structures is to enable the members to tolerate earthquake cyclic loads through inelastic deformations. If members and joints can resist these deformations appropriately, the structure can dissipate input energy of cyclic loads and will absorb great amount of earthquake energy without loss of stability. Ductility is the ratio of joint ultimate displacement to joint displacement in yielding state and can be calculated by the Eq. 7 (Mahini and Ronagh 2010):

$$\mu = \Delta_u / \Delta_y \tag{7}$$

Δ_u and Δ_y = joint ultimate displacement and joint yield displacement, respectively.

In cyclic loading, ultimate limitation point is the point that falling of force displacement diagram starts at which. Corresponding displacement at this point is the ultimate displacement, Δu. Yield displacement, Δy, is defined as Eq. 8 in cyclic loading

$$\Delta_y = \frac{|\Delta_1| + |\Delta_2|}{2} \tag{8}$$

where Δ_1 and Δ_2 are displacements corresponding to loads P_y and $-P_y$, respectively. The value of P_y is determined in monotonic loading (Wong 2005).

Table 4 shows the values of ductility of specimens in various groups. Fig. 10 shows the variation of ductility versus changes in reinforcing detail in joints core in each group.

5.2 Performance of joints

In this study, performance is defined as the ratio of force that is applied on the joint in ultimate limitation state (that can be calculated from the results of FEM analysis) to force that is applied on the joint in the state of maximum nominal moment resistance, and can be calculated from Eq. (9) (Nilson 1973)

$$Z = (P_u)/(P_{max}) \tag{9}$$

P_u= shear force at ultimate limitation point3

P_{max}= shear force calculated based on common equations in calculate moment capacity of reinforced concrete beam on the basis of equations of ACI318-05 code

With this definition, joint strength by performance greater than 1 is more than calculated strength of the joint beam (adjacent beam) .In other words, this joint is so resistant that after formation of plastic hinge in the end of joint beam, yet it has capacity and due to stress and moment redistribution, is able to carry more loads. But performance less than 1 represents that before formation of plastic hinge in the end of joint beam, the fracture happens inside the joint region and the joint loss its strength (Mostofinezhad and Sobhani 2003).

In cyclic loading, ultimate limitation point is the point at which falling of force-displacement diagram starts. Corresponding force at this point would be the ultimate shear force, P_u (Wong 2005).

For determining P_{max}, as beam is assumed to have low reinforcement level, nominal moment capacity can be calculated by Eq. (8), and then with attention to loading arm, amount of shear force that is applied on beam is calculated by Eq. (10) (ACI318 2005)

$$\rho \langle \rho_b \Rightarrow \tag{10}$$

ρ and ρ_b = percent of longitudinal bars of beam and percent of longitudinal bars of beam in balanced state

Table 5 Performance of specimens in various groups

	S2	S4	S5	S6	S3	S7	S1
G1	1.7867	1.7699	1.7434	1.7277	1.7079	1.6974	1.5706
G2	1.8628	1.8478	1.8394	1.8250	1.8036	1.7949	1.5621
G3	1.7575	1.7359	1.7225	1.6990	1.6826	1.6642	1.5301
G4	1.9278	1.9191	1.9041	1.8908	1.8757	1.8676	1.6034
G5	2.1154	2.0916	2.0771	2.0681	2.0530	2.0387	1.7893
G6	1.9290	1.9159	1.9011	1.8955	1.8870	1.8671	1.6729
G7	1.8652	1.8441	1.8138	1.7999	1.7786	1.7693	1.5400
G8	1.8732	1.8616	1.8524	1.8405	1.8288	1.8192	1.5589
G9	1.7508	1.7342	1.7149	1.6969	1.6868	1.6663	1.4878
G10	2.1354	2.1155	2.0904	2.0218	1.9798	1.9616	1.7057

Table 6 Ultimate resistant moment of specimens in various groups (KN.m)

	S2	S4	S5	S6	S3	S7	S1
G1	330.0594	326.9522	322.058	319.1566	315.508	313.5509	290.1424
G2	433.803	430.307	428.339	424.988	420.012	417.977	363.773
G3	324.656	320.669	318.2	313.856	310.824	307.433	282.649
G4	711.373	708.156	702.625	697.738	692.148	689.177	591.677
G5	619.676	612.7	608.444	605.821	601.405	597.201	524.149
G6	1046.27	1039.12	1031.12	1028.06	1023.46	1012.67	907.375
G7	546.387	540.192	531.335	527.268	521.027	518.291	451.112
G8	1168.43	1161.18	1155.46	1148.05	1140.7	1134.73	972.395
G9	323.433	320.357	316.784	313.469	311.604	307.82	274.836
G10	787.9772	780.6334	771.3729	746.0574	730.5736	723.8388	629.4250

Table 7 Energy absorption of specimens in various groups (KN.m)

	S2	S4	S5	S6	S3	S7	S1
G1	10.3333	10.2179	10.1026	10.0199	9.7166	9.5833	8.2666
G2	14.3501	14.2259	14.0577	13.9119	13.7653	13.5411	11.211
G3	6.3959	6.1892	6.1124	6.0752	5.9691	5.9063	5.3355
G4	15.4565	15.3007	15.0006	14.6569	14.4752	14.394	11.8896
G5	10.1647	10.0564	9.8116	9.6948	9.55588	9.5067	8.005
G6	21.8959	21.4983	20.9919	20.7303	20.5403	20.0261	16.5677
G7	9.9569	9.7627	9.2792	9.1873	9.0821	9.0124	8.1367
G8	23.7985	23.4189	22.8091	22.6651	22.4128	21.9521	18.0799
G9	5.8489	5.7252	5.6449	5.5208	5.4892	5.4201	4.9019
G10	14.6536	14.4305	14.0818	13.8911	13.8231	13.5415	11.2833

M_n= nominal moment capacity of beam (N.mm)

F_y and f'_c = yield steel stress in the beam bars and concrete compressive strength (MPa)
b and d = width of beam section and effective depth of beam section (mm), respectively.

Table 5 shows the performance of specimens in various groups. Fig. 10 shows the variation of performance versus changes in reinforcing detail in joints core in each group.

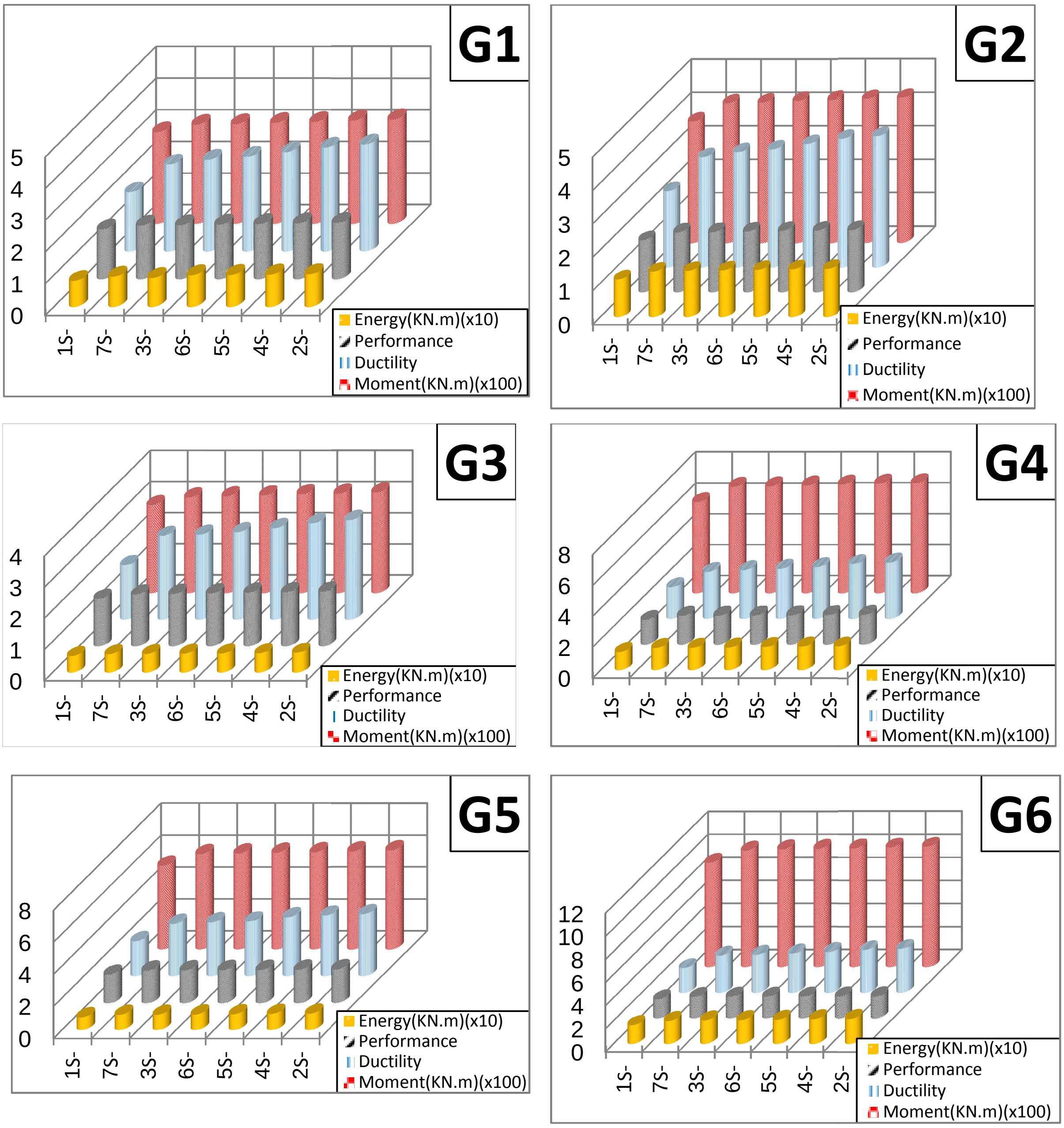

Fig. 10 Diagram of ductility, performance, ultimate resistant moment and energy absorption changes with changes in reinforcing details in joints core in each group

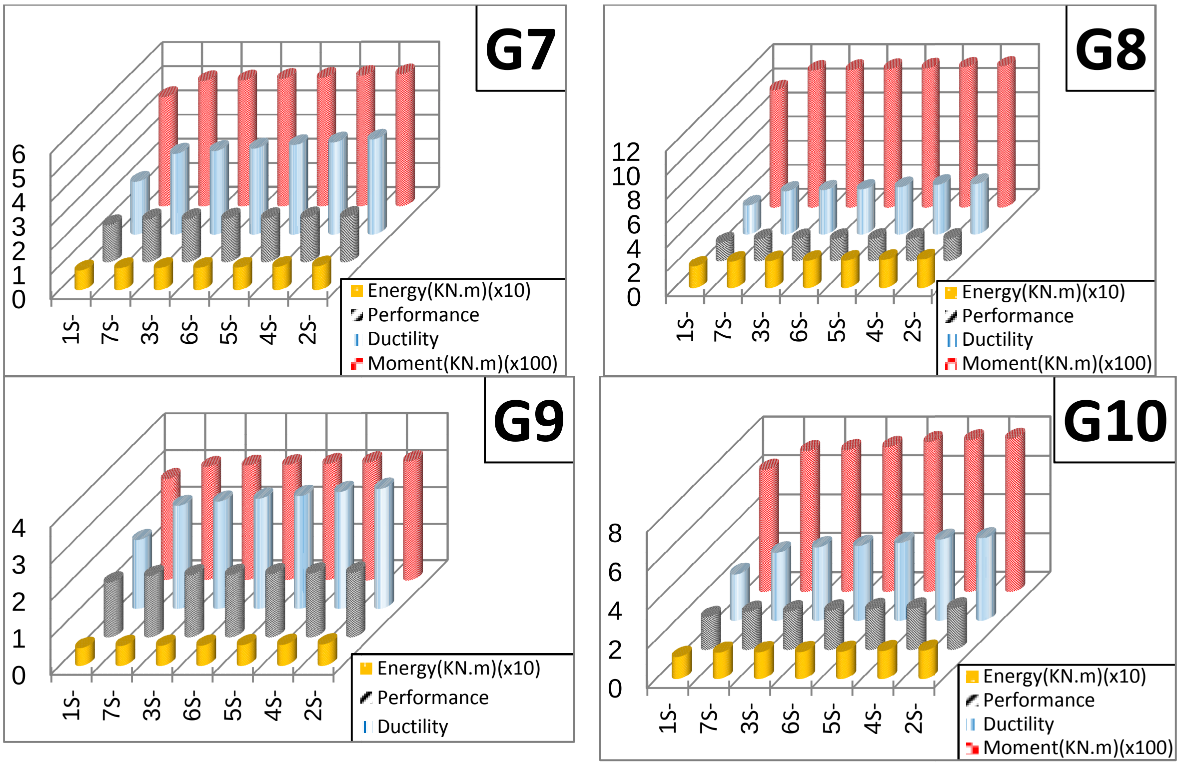

Fig. 10 Continued

5.3 Ultimate resistant moment of joints

Ultimate resistant moment of a joint is the most moment that can be tolerated by the joint set (Hanson and Conner 2008). In cyclic loading, ultimate limitation point is a point at which falling of force displacement diagram starts. Corresponding moment at this point would be the ultimate moment, Mu (Wong 2005). Table 6 shows the ultimate resistant moment amounts of specimens in various groups. Fig. 10 shows the variation of ultimate resistant moment versus changes in reinforcing detail in joints core in each group.

5.4 Energy absorption

In each joint, the area enclosed by force-displacement diagram until the ultimate limitation point, introduces the ability of material for energy absorption. By increasing the area enclosed by force-displacement diagram, the ability of material for energy absorption will be increased. In inelastic region, only a little part of saved energy in the joint is recoverable and most amount of the energy will be absorbed by means of permanent deformation of material (Ibrahim 2011). Table 7 shows energy absorption values of specimens in various groups. Fig. 10 shows the variation of energy absorption versus changes in reinforcing detail in joints core in each group.

According to Fig. 10, following points can be stated:

In each group of joints, specimen S2 (the control specimen that reinforced by closed hoops according to seismic designing equations of ACI318-05 code) represents maximum value of each one of above parameters and specimen S1 (the control specimen that is not provided with any transverse shear reinforcement in the joint core) represents minimum value of each one of above parameters. As expected before, behavior of specimen that is reinforced in joint core is several times better than behavior of specimen that is not provided with any transverse shear reinforcement in joint core. Based on the results, in different groups, ductility, performance, ultimate resistant moment and energy absorption in specimens S2 with respect to specimens S1 has increased 72 to 83 percent, 14 to 25 percent, 14 to 25 percent and 19.32 to 32.16 percent, respectively.

In specimens S3 to S7 that are reinforced by double-headed studs, specimen S4 represents the best behavior and its behavior is very similar to specimen S2 that is reinforced by closed hoops, so that ductility, performance, ultimate resistant moment and energy absorption in specimens S2 with respect to specimen S4 in different groups has decreased 1.6 to 3.6 percent, 0.45 to 1.25 percent, 0.45 to 1.25 percent, 0.87 to 3.34 percent, respectively. By attention to the fact that use of double headed studs in respect with closed hoops, decreases accomplishment problems, such as concrete placing, concrete vibration and accomplishment closed hoops and also clarify limitations on the beam bar sizes relative to the joint dimensions.Very little decreases in above parameters in specimen S4 with respect to specimen S2, against very considerable decreasing in accomplishement problems, is very insignificant and connivancely. By consideration of reduction trend in ductility, performance, ultimate resistant moment and energy absorption in specimens S4 to S6, it can be said that whatever area of horizontal double-headed studs in in-plane and out of plane direction be closer to the amounts obtained from ACI318-05 code, the joint would have better behavior. With comparing amounts of ductility, performance, ultimate resistant moment and energy absorption in specimens S5 and S6, it is observed that joint's behavior sensitivity in respect with amount of out of plane double-headed studs is more than joint's behavior sensitivity in respect with amount of in-plane double-headed studs, so that decrease of the area of double-headed studs in out of plane direction in respect with decrease of the area of double-headed studs in in-plane direction, has more effect on each one of above parameters. Placing enough double-headed studs in out of plane direction, has an important effect on improvement of joint's behavior. In specimens S3 and S7, which contain completely equal horizontal double headed studs in out of plane and in-plane directions, it is observed that addition of vertical double-headed studs in in-plane direction in specimen S7 has decreased the ductility, performance, ultimate resistant moment and energy absorption compared with specimen S3. It can be inferred that adding vertical double-headed studs in in-plane direction, has undesirable effect on joint's behavior.

6. Conclusion

Reinforcing the concrete external beam-column joint by closed hoops, because of congestion of bars, lead to significant accomplishment problems. Using double headed studs instead of closed hoops, with holding abilities of joints, lead to significant reduction in congestion and accomplishment problems. Following points can be stated based on the finding of the present research

1. As it was expected, those specimens that are not provided with any transverse shear reinforcement in the joint core, with respect to specimens that are reinforced in joint core by

different forms of reinforcement (closed hoops and double-headed studs) represents more undesirable behavior and their amounts of ductility, performance, ultimate resistant moment and energy absorption are less than those joints that are reinforced in joint core.

2. Among different specimens that are reinforced by double-headed studs, those specimens that are designed based on seismic design provisions of ACI318-05 code, have a very close behavior to those specimens that are reinforced by closed hoops with high congestion in joint core, so that in spite of significant reduction in accomplishment problems, amounts of ductility, performance, ultimate resistant moment and energy absorption have very little reduction.

3. Adding vertical double-headed studs in in-plane direction, has an undesirable effect on joint's behavior.With adding these vertical double-headed studs, amounts of ductility, performance, ultimate resistant moment and energy absorption have reduced.

4. Sensitivity of Joint's behavior with amount of out of plane double-headed studs is more than joint's behavior sensitivity with amount of in-plane double-headed studs, so that decreasing area of double-headed studs in out of plane direction with respect to decreasing area of double-headed studs in in-plane direction, has more effect on each one of above parameters. So, placing enough double-headed studs in out of plane direction, has key effect on improvement of behavior of joints.

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