NUMERICAL MODELING OF 3D BEAM-COLUMN JOINTS

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Abstract

Past earthquakes have highlighted the seismic vulnerability of non-seismically designed reinforced concrete (RC) frame structures worldwide. It is documented that the mechanics under seismic loads and the poor reinforcement detailing of beam to column connections may result in highly inelastic joint shear distortions thus resulting in non-ductile response of the entire structure. Even though seismic behavior of beam-column joints has been in the research focus for past few decades, most of the work is restricted to the planar (2D) joint sub-assemblies due to limitations of the test setup and laboratories. However, in a real structure, beams frame in more than one direction in a joint core and in most of the cases, a monolithic slab also passes through the beams. Therefore, in order to holistically and realistically capture the seismic behavior of beam-column joints of non-seismically designed RC structures, it is important to consider the influence of the presence of transverse beams and slabs as well. Currently, very limited and insufficient information on the seismic performance of non-seismically designed 3D beam-column joints with slabs is available in literature.

In the present paper, a numerical study based on the 3D finite element modeling of joint sub-assemblies of non-seismically designed structures subjected to cyclic loads is performed and discussed. Concrete is modeled using 8-node solid elements with the microplane model with relaxed kinematic constraint as the consitutive law, in which the material is characterized by the relation between the stress and strain components on planes of various orientations. Smeared crack approach is used to represent damage and fracture. To assure mesh objectivity, the principle of local crack band method is implemented into the finite element code. Reinforcing steel is modeled using 1D 2-node truss elements with tri-linear uniaxial stress-strain relationship. A discrete model comprising zero thickness spring elements is applied to simulate bond between steel and concrete.

First, the numerical approach is validated against an experiment on a 3D beam-column joint reported in the literature. Subsequently, the finite element model is extended to different 3D configurations which incorporate transverse beams and slab framing into the joint. The results are evaluated in terms of load-displacement behavior and crack patterns (damage) in the joint sub-assemblies. The initiation of diagonal cracking in the joint and ultimate shear strength of exterior and interior space frame joints is evaluated and compared to existing shear strength models which are based on the average plane stress plane strain approach. The effect of column axial load is not considered in the current study. The attained numerical results presented herein blatantly underline the need to consider the presence of transverse beams and slab to realistically assess the joint shear behavior under cyclic excitations. The simulations form the basis of a test program which will be pursued by the authors in near future.

Keywords: 3D frame joints; finite element analysis; joint failure; transverse beam and slab; non-seismic design

1. Introduction

When it comes to the seismic behavior of reinforced concrete (RC) moment frame structures, there exists consensus along the scientific community that one main reason for their vulnerability against earthquake loads originates in the weak performance of the beam-column connections. More specifically, the high gradient of the moment diagram across the joints due to the lateral loads results in high shear stresses. Thus, once the connecting link between the beams and column fails, there is a high chance that this might lead to an overall non-ductile response followed by a relatively immediate collapse of the structure.

Most of the existing RC structures worldwide have been built prior to the advent of the corresponding seismic design codes. In such cases, RC frame structures were designed to resist code-specified lateral forces through bending of the vertical and horizontal members without considering any deformability and ductility requirements [1]. Those kind of structures are categorized into the earthquake risk buildings (ERBs) and are associated with poor reinforcement detailing and lack on capacity design philosophy [2]. Commonly, the beam-column joints are provided either with insufficient or no transverse hoops at all, while the anchorage of the longitudinal beam reinforcement into the joints designed for vertical loads loses its integrity when the structure is subjected to cyclic horizontal excitations. In order to realistically predict the seismic performance of the non-seismically designed (NSD) structures and subsequently to retrofit adequately if required, the non-linear behavior of both exterior and interior beam-column connections must be necessarily taken into account [3].

Over the past decades, high impetus has been put on the examination of the behavior of 2D exterior beam-column joint sub-assemblies under reversed cyclic loading. Park (2010) [4] collected a database of over 60 tests of unreinforced exterior (2D) and corner joints and listed the following failure modes: (1) joint shear failure without beam longitudinal reinforcement yielding (JS), (2) joint shear failure with beam longitudinal reinforcement yielding (BJ), (3) beam flexural failure (BY), (4) column flexural failure (CY), (5) beam reinforcement pull-out failure (BP) and (6) anchorage failure. The bulk of the experimental database showed brittle joint shear failure modes (JS and BJ) due to the absence of transverse reinforcement in the joint core. However, most tests reported in literature are restricted to planar (2D) sub-assemblies due to limitations of test setups and laboratories, which actually do not correspond to the applications in reality, where beams frame in more than one direction in a joint core and in most of the cases, a monolithic slab also passes through the beams. As a corollary, the models to predict the joint shear behavior proposed by various researchers (e.g. Lowes et al. (2003) [5], Altoontash (2004) [6], Shin and LaFave (2004) [7], Sharma et al. (2011) [8]) do not incorporate the presence of transverse beams and slab. Due to the scarcity of data on 3D beam-column joint sub-assemblies, more information is needed to investigate the influence of out-of- plane members on the joint shear strength in order to holistically and thus realistically assess the overall behavior of the NSD structure when subjected to lateral loads. This paper aims to shed more light on the behavior of different 3D joint configurations in terms of joint shear strength, stiffness and ductility by conducting a numerical study. First the numerical modeling approach is validated against the results of an existing test from the literature and then the same modeling approach is used to study the influence of the presence of a transverse member and the slab on the behavior of exterior and interior beam-column joints.

1.2 Role of transverse members

It is well documented in the literature that the 2D exterior sub-assemblies suffer from distinct diagonal cracking in the joint panel when exposed to lateral cyclic loads. However, because concrete failure is associated with internal cracking and increase in volume, it is suggested that transverse beams and slab passing through the joint will help to confine the concrete in the joint and thus induce higher joint shear resistance. On the other hand, the stresses developed in the slab reinforcement will add to the longitudinal tensile forces in the beam bars acting in the joint by means of torsion and weak axis bending of the transverse beams. This leads to an overall increase in the joint shear demand.

Among the current design guides for well-detailed beam-column joints, which recommend limiting the stress in the joint core, ACI352R-02 [9] explicitly considers the confinement by providing γ-values for different joint classifications when calculating the nominal joint shear resistance. On the other hand, guidelines for the assessment of NSD joints can be found in FEMA 356 [10] as well as in CEB Bulletin 240 [11]. Comparing ACI 352 with FEMA 356 shows that the joint shear strength of non-seismically designed joints is considered as half of the strength for the corresponding seismically designed joint. However, this criteria may not always be on the conservative side as shown by Sharma and Hofmann (2016) [12].

In CEB Bulletin 240, the joint shear strength of exterior joints without transverse reinforcement is based on the critical principal tensile stress in the concrete of the joint core as recommended by Priestley (1997) [2].

As for the slab participation in practical earthquake design, Pantazopoulou et al. (2001) [13] indicates that neglecting the slab contribution in the lateral load resistance results in the gross underestimation of the beam flexural strength under hogging moments and consequently disturbs the whole strength hierarchy of the adjoining members. Furthermore, it is stated that slab participation is a drift-controlled problem. The equivalent effective width of slab b_{eff} given in the ACI 352R amendments is obtained from experiments of sub-assemblies at a 2% lateral drift [13]. Within this effective width, the strain in slab reinforcement is assumed to be of same value as the one in the web of the beam at the same distance of the neutral axis.

With respect to the evaluation of the influence of transverse beams and slab, most tests have been conducted on well-designed beam-column joints, whereas the database on NSD assemblies is rather limited. In the following, some experiments on NSD configurations are illuminated.

Hanson and Connor (1967, 1972) [14, 15] conducted experiments in which both interior and exterior joint sub-assemblies without transverse reinforcement in the joint were subjected to cyclic shear forces on the beam tips, whereas the column was axially loaded. The beam top and bottom longitudinal reinforcement terminated with a 90° bend into the joint. While the planar NSD exterior joint suffered from a brittle joint shear failure, in the confined case of one unloaded stub (corner joint) and two unloaded stubs (edge joint), the yield moment of the beam was reached and an enhanced ductility was observed. Regarding the interior joints, where beam reinforcement was anchored straight through, the ultimate flexural capacity has been attained as well. Interestingly, it was found that both top and bottom beam reinforcing bars were in tension through the width of the column.

Gokgoz (2008) [16] investigated unreinforced joints where the top beam reinforcement terminated with a 90° bend and the bottom reinforcement bars with a straight anchorage length of 6 inches. Compared to the exterior 2D case, by adding two transverse stubs on the joint, the joint shear strength showed an increase only for the downward loading of the beam, while during the upward loading, the test specimen exhibited similar capacity. In the case of cast-in-situ slab, the capacity as well as the stiffness for the downward loading further increased, which indicated that the flexural capacity of the beam has augmented.

Rossetto et al. (2017) [17] conducted experimental and numerical investigations on the influence of transverse beams and slab on typical pre-1970 designed interior joints. It was observed that the slab had a noticeable effect on the curvatures on the beam to joint interface. Indeed, the curvature graph plotted over the drift was unsymmetrical when the slab was present. Moreover, the maximum value of the curvature was much lower in comparison with a planar interior joint. It was emphasized that simplifying structures as 2D frames and using an effective flange width to account for the slab contribution without considering the confinement effect of transverse beams and slab may give flawed predictions with respect to the structural assessment of the building.

Park (2000) [4] carried out cyclic load tests on four full-scale unreinforced corner beam-column joint specimens with slab. The investigated parameters were the joint aspect ratio (ratio of beam height to column height) and the amount of longitudinal reinforcement in the beam, which terminated with a 90° bend in the joint. Slab top reinforcement was anchored with a 90° bend in the orthogonal beam as well, while bottom reinforcement was straightly anchored with a length of 6 in. It was observed that the joint shear strength decreases with higher aspect ratio and that the effective slab with is smaller for higher longitudinal beam reinforcement amount. In addition, loading of the beam incurred torsional cracks in the transverse beam.

Kam et al. (2010) [18] performed tests on two 2/3-scaled 2D (one-way) and two 3D (two-way) exterior beam-column joints with and without slab under unidirectional and bidirectional quasi-static lateral loading. The joints were devoid of hoops, whereas plain round bars with 180° end hooks were provided into the joint. It was confirmed that the slab and transverse stubs contribute to an increased lateral capacity. He explains this fact through by the slab induced torsion twist in the unloaded beam, which in turn adds confinement to the joint. Moreover, it was concluded that the slab participation is based on the stiffness of the spandrel beam and that the effective slab with is lower than in the case of well-detailed joints.

The aforementioned three-dimensional experiments are associated with costly and intricate test set-up configurations. On the numerical front, finite element (FE) code MASA [19] has proven its efficacy in analyzing structural sub-assemblies under cyclic loads [3, 20-24]. FE program MASA is shortly presented and used in the framework of this study.

1.3 Organization and scope

The present work comprises a numerical study to investigate successively the influence of transverse beams and slab on the shear strength of non-seismically designed beam-column joint sub-assemblies. The investigated joints can be depicted in Fig.1. The numerical results are evaluated in terms of capacity and crack propagation. First, the finite element model is shortly discussed and then validated against a 3D edge joint with slab subjected to cyclic loads. Note that this study solely treats of uni-directional loading on the main beam tips, while the transverse stub(s) remain unloaded.

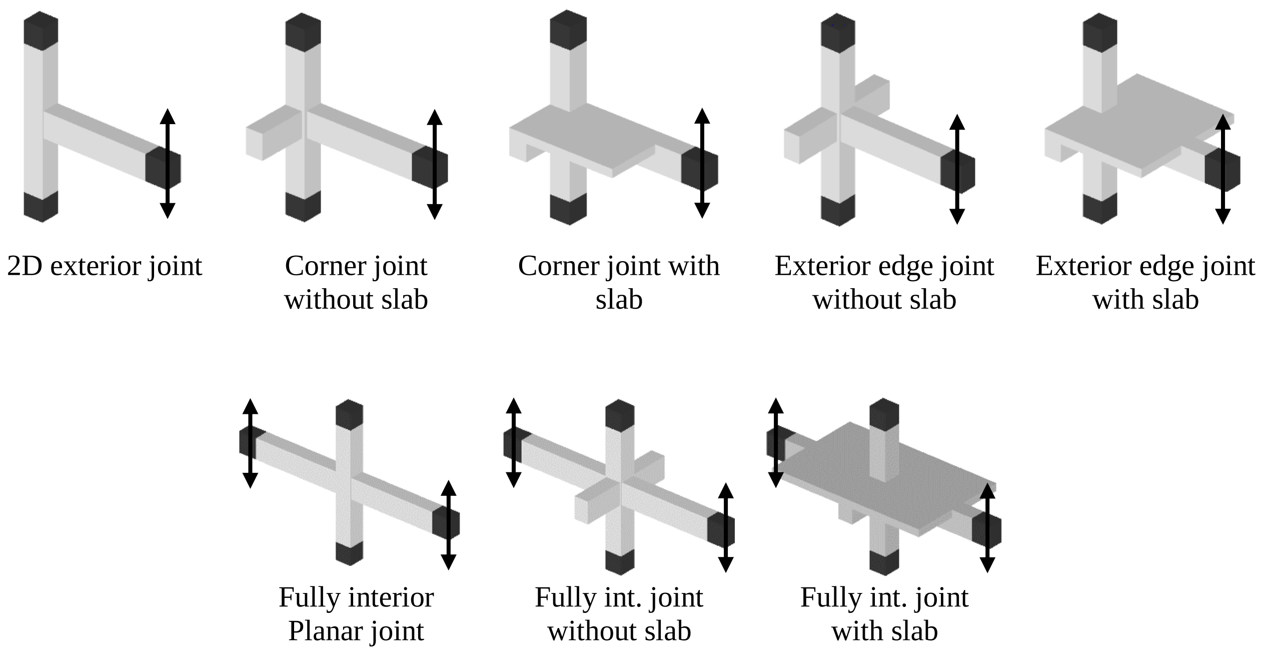


Fig. 1 – Overview of the investigated beam-column joints

2. Finite Element Model

2.1 Finite Element Code MASA

Finite element code MASA (Macroscopic Space Analysis) has been developed at the Institute of Construction Materials, University of Stuttgart [19]. Constitutive law for quasi-brittle material concrete is the microplane model with relaxed kinematic constraint, in which the material is characterized by the relation between stress and strain components on planes of various orientations. Concrete is meshed with hexahedral elements. To assure mesh objectivity and to limit the localization of strain, the crack band method is used. When referring to the crack pattern of the FE model in the course of this paper, the red color corresponds roughly to the critical crack width of 0.3 mm.

Reinforcing steel is modeled using 1D two-node truss elements with tri-linear uniaxial stress-strain relationship. The discrete model by Lettow (2007) [25] comprising zero thickness spring elements is utilized to simulate bond between steel and concrete and shown in Fig.2 and Fig.3. The corresponding implemented bond-slip cyclic relationship adopted by Eligehausen et al. (1983) [26] is presented in Fig.3.

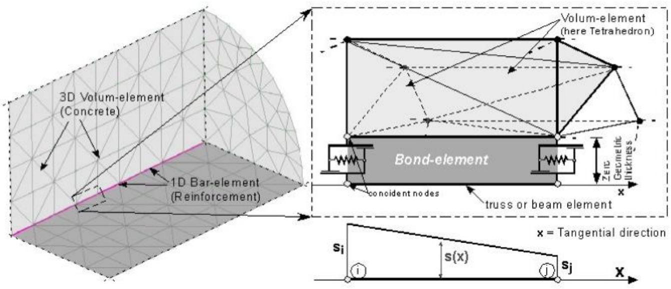
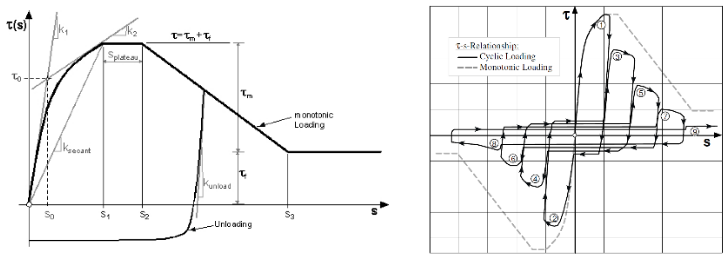


Fig. 2 – Bond model implemented in MASA [25]



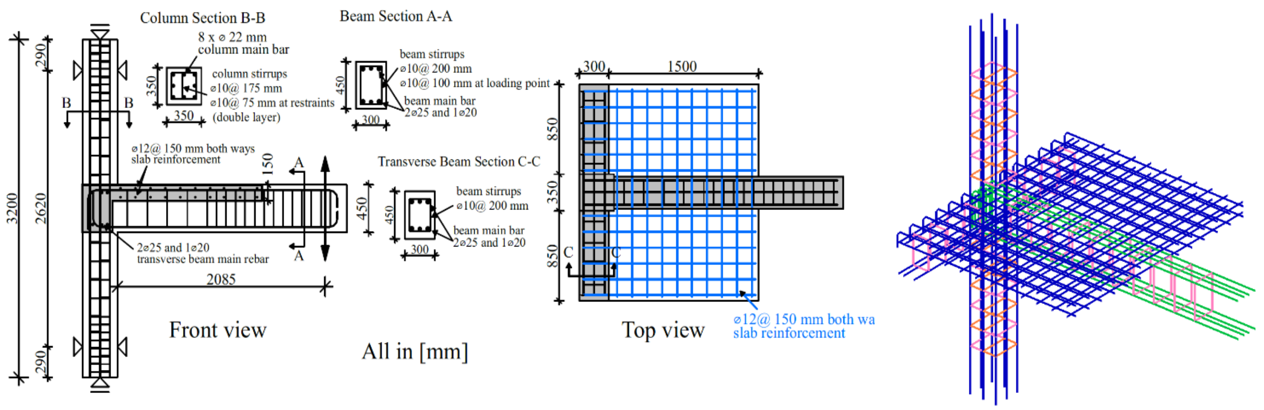
(a) (b)

Fig. 3 – (a) Monotonic bond-slip relationship [25] and (b) bond-slip cyclic relationship [26]

2.2 Validation of the numerical model

The numerical model is validated against an exterior edge beam-column joint sub-assembly tested by Marchisella et al. (2019) [27], including a floor slab and transverse beams. The specimen represents a non-seismically designed joint with no transverse hoops and widely spaced stirrups in beam and column. Main (loaded) beam and transverse beams are 300 mm wide and 450 mm deep. Main top and bottom longitudinal reinforcement consists of two Φ25 and one Φ20 millimeter reinforcing bars, which terminate with a 90° bend into the joint. Same reinforcement arrangement in the transverse beams just passes straight through the column. The 150 mm thick slab is provided with double layer of Φ12 millimeter bars (both ways) at top and bottom with a spacing of 150 mm. Normal strength concrete C20/25 and B500 steel was used in the experiment.

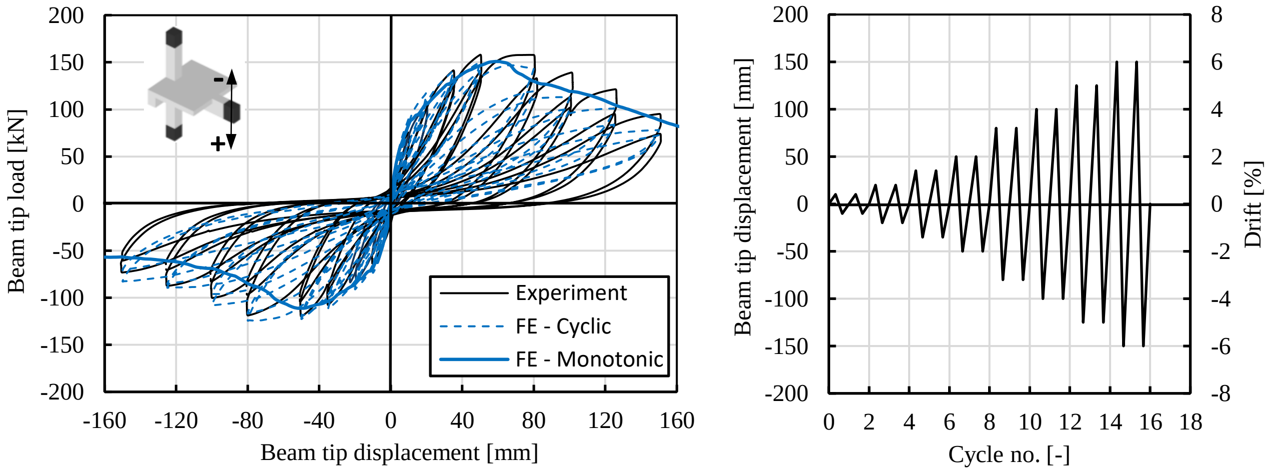
The exact material data can be depicted in the reference [27]. The geometry, reinforcement detailing and reinforcement cage of the finite element (FE) model is graphically shown in Fig.4. In order to use vertical symmetry, the beam reinforcement in the model was modified to four bars with the total area equal to the area of two Φ25 and one Φ20 bars. The concrete elements near the supports and applied load are given linear-elastic stress-strain characteristics to avoid unrealistic local damage.



(a) (b)

Fig. 4 – (a) Geometry and reinforcement details of experiment and (b) reinforcement cage of FE model

The exterior edge joint is hinge supported at the top and bottom of the column, whereas the transverse beams are free. The drift is defined as the ratio between beam tip displacement and the sum of cantilever length and half of the column depth. Test protocol as well as the experimental and numerical results are shown in Fig.5. Note that the positive displacement corresponds to the slab acting in tension. It can be seen that the numerical model is capable of satisfactorily reproducing both strength and global hysteretic behavior of the test specimen. Moreover, the fact that the monotonic load-displacement curve describes the envelope of the cyclic hysteretic loops is substantiated by the numerical results as well.



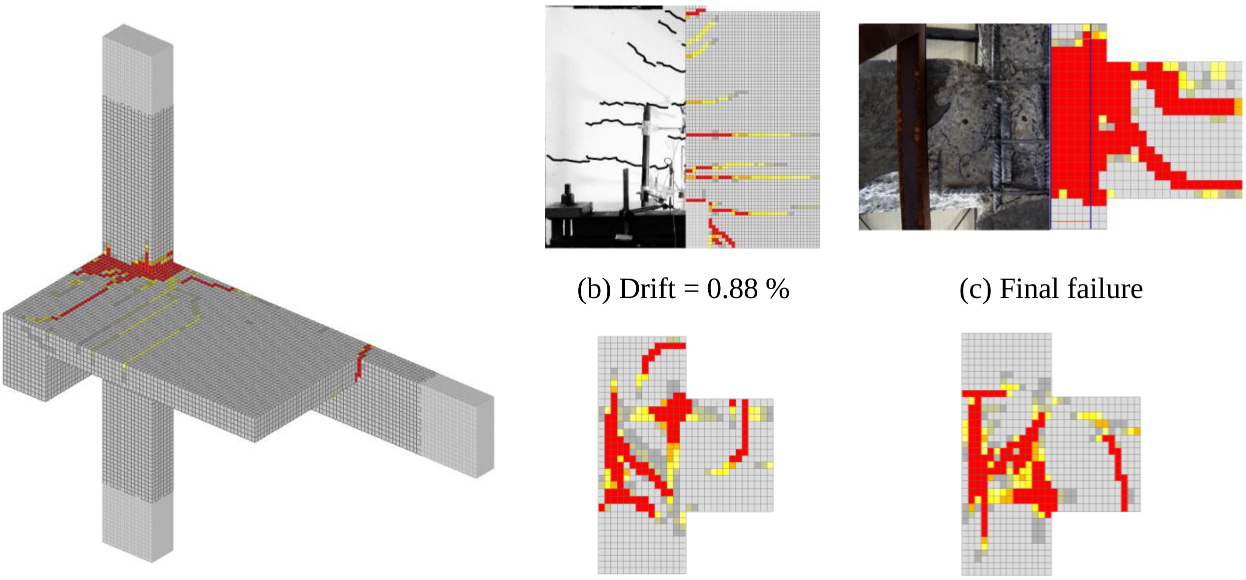
(a) (b)

Fig. 5 – (a) Validation of the FE model and (b) test protocol

In the experiment, the maximum beam tip load was 158 kN for downward loading and 120 kN for upward loading, respectively. From the numerical analysis, a maximum load of 150 kN and 120 kN has been attained. The results show that the contribution of slab to joint shear is significant.

A global view of the final numerical crack pattern is provided in Fig.6a. The first four cycles are associated with the development of flexural cracks for downward and upward loading, respectively (Fig. 6b). Interestingly, from the initiation of the cycle in which the peak load is attained onwards, no more flexural cracks do emerge. Furthermore, the suggestion by Marchisella et al. [27], namely that a X-shaped crack pattern is formed in the joint region is corroborated by the finite element analysis. Fig.6d indicates detrimental diagonal cracks in the joint vicinity at the symmetry plane when the peak load is reached for downward and upward loading, respectively. It has been observed in the numerical analysis that the crack widths of the diagonal cracks in the joint keep increasing continuously. The vertical cracks at the back of the column are attributed to the horizontal dilatation of the joint due to shear, which causes spalling of the concrete cover.

It is important to note that the slab gives rise to torsional cracks in the transverse beams. In the top view of the FE model in Fig.6b, torsional cracks evolve from the column at a 45 degree angle. Fig.6c shows the failure surface on the back side of the beam-column joint, where the inclined torsional crack along the transverse beam can be seen. The numerical crack pattern is in concordance with its experimental counterpart.



(a) (symmetry used) (b) Drift = 0.88 % (c) Final failure (d) At peak load (Drift = 2.21%)

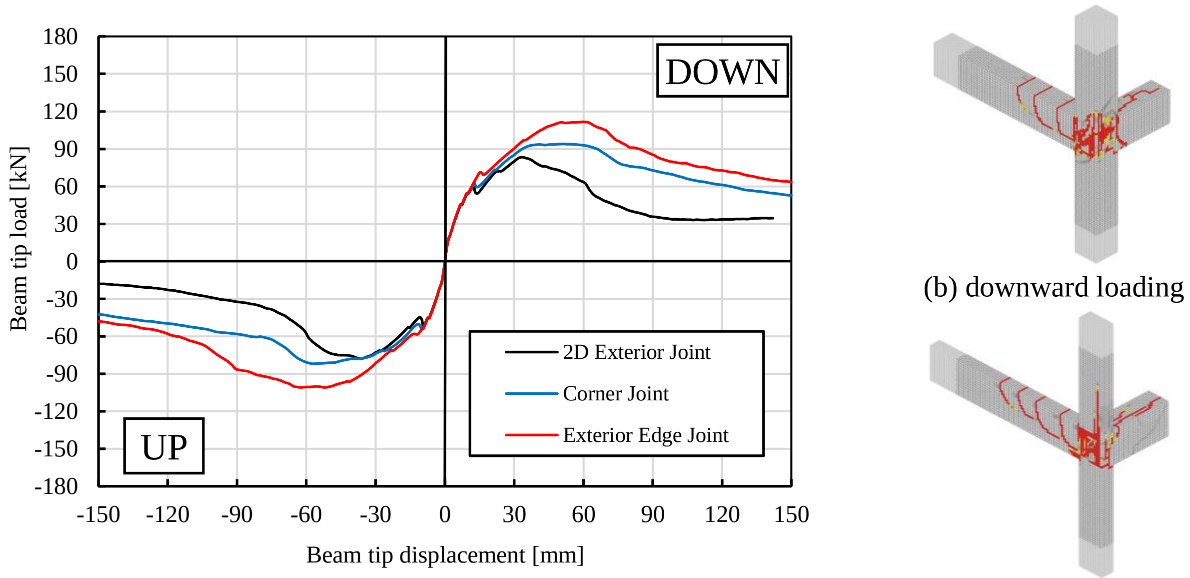
Fig. 6 – Validation of the FE model (a) final crack pattern (b) top view of test specimen, (c) failure surface on back side of joint and (d) diagonal cracking in joint region

3. Numerical study

Based on the findings discussed in the previous chapter, the paper now proceeds to the numerical study to further extend the knowledge of the three-dimensional behavior of beam-column joints. The investigated models have been shown in Fig. 1. It is important to note that only the monotonic envelopes are presented because the scope of this study is to evaluate the joint shear strength. Geometry, reinforcement detailing and boundary conditions are similar to the earlier validated exterior edge joint with floor slab.

3.1 Exterior beam-column joints

In Fig.7a, the load-displacement curves for downward and upward loading of the main beam of the exterior joint configurations without slab are illustrated. The first cracks are flexural ones along the loaded beam, followed by longitudinal splitting cracks into the joint due to the high bond stresses. The first drop of the curve is attributed to the first diagonal crack in the joint. However, the load is still increasing, until the ultimate strength of the beam-column joint is reached. This hardening behavior is owed to the fact that the longitudinal reinforcement terminates with a 90° bend into the joint, which results in the stabilization of the compression strut [2]. Because the longitudinal reinforcement arrangement of the beam is symmetric, the models show similar behavior in terms of strength and diagonal joint cracking for positive and negative beam tip displacement. Notwithstanding, inspection of the curve reveals slightly lower values in the case of upward loading, because the bottom longitudinal rebar has its bent before the top rebar, hence resulting in a higher aspect ratio.

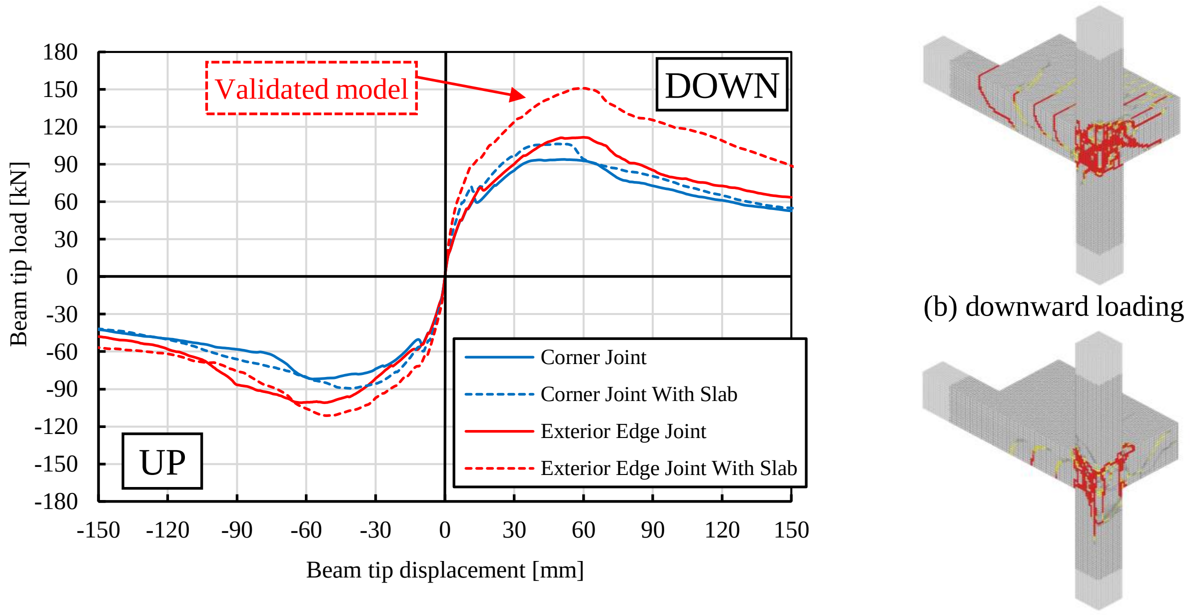


(a) (b) downward loading (c) downward loading

Fig. 7 – Influence of transverse beams on exterior joint – (a) load-displacement curves (b) final failure of corner joint and (c) final failure of exterior edge joint (half model, symmetry used)

Regarding the confining effect, the curves clearly illustrate that the ultimate load increases by adding one (corner joint) and two transverse beams (edge joint). Apparently, the transverse members do not enhance the stiffness. In addition, the 3D joints show a gentler post-peak behavior compared to the 2D beam-column joint. Nevertheless, also in the 3D configurations, severe diagonal cracking in the joint panel is observed, which results in an overall brittle failure. The attained peak loads for the 2D, corner and edge joint for downward loading are 83 kN, 93.5 kN and 112 kN, respectively. The final crack pattern for the downward loading is illustrated in Fig.7bc.

If we now turn to the corner and edge joint incorporating a monolithic slab, inspection of the load-displacement curves in Fig.8a reveals that both strength and stiffness have increased, namely for both loading directions. Particularly for the case when the slab acts in tension, the beam tip peak load of the edge joint increases from 112 kN to 150 kN and for the corner joint from 93.5 kN to 106 kN, respectively. Obviously, the torsional crack on the back side of the transverse beam has generated (Fig.8b), which is not seen in the corner joint without slab (Fig.7b). Interestingly, when the beam tip is subjected to negative displacement, the maximum stress in the beam longitudinal bars is roughly same for the corner joint without and with slab. But, due to the higher compression width coming from the slab, the neutral axis has shifted up. As a result, the peak load experiences an increase from 82 kN to 89 kN.



(a)

(b) downward loading

(c) upward loading

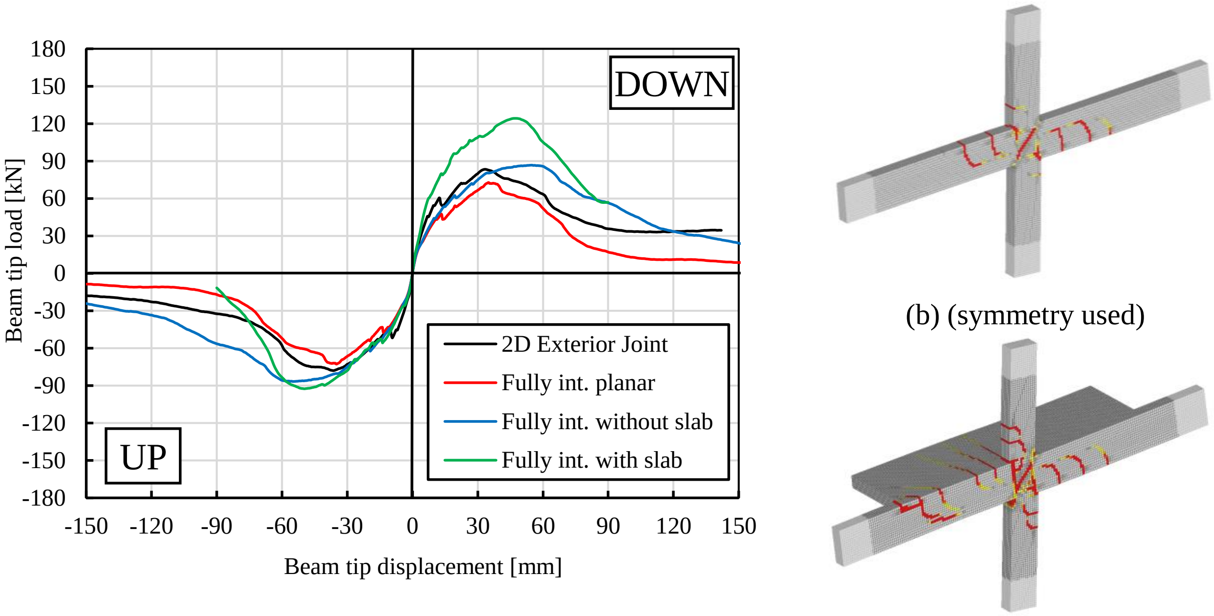
Fig. 8 – Influence of slab on exterior joint – (a) load-displacement curves (b) final failure of corner joint for downward and (c) upward loading

3.2 Interior beam-column joints

The main difference between the interior and exterior joints is that the loaded beam runs through the joint. Consequently, the lateral expansion of the joint, which initiates the failure of the diagonal strut, is considerably restrained, particularly when the main reinforcement of such beams is anchored through and past the joint [13]. Sharma (2013) [3] assigns the double critical tensile stress value for interior joints in comparison to the 2D exterior joints. Concurrently, the shear demand is also higher, because the displacement loads on either beam tip ends are of opposite sign. Indeed, same conditions do apply in this study. Both tips of the main loaded beam are exposed to equal magnitude but opposite direction.

According to the just described facts, the peak load of such interior joints is expected to be close to the one of the exterior 2D joint. The load-displacement curves are highlighted in Fig.9. The maximum load for the 2D exterior and fully interior planar joint without transverse beam is 83 kN and 73 kN, respectively. Adding a transverse beam which passes through the joint (fully int. without slab), the attained peak load is 87 kN, which equals to an increase of 20%. If we now turn to the fully interior sub-assembly with monolithic slab, the green curve reveals the different behavior for positive and negative displacement loading. Note that in all models, severe diagonal cracking in the joint is observed, which results to a final joint shear failure. The crack pattern of the interior planar and interior joint with slab at peak load is shown in Fig.9bc.

For all investigated interior joints, both top and bottom longitudinal reinforcement bars of the loaded beam are in tension through the whole width of the column. This finding accords to the observations made by Hanson and Connor (1972) [15].



(a)

(b) (symmetry used)

(c) (symmetry used))

Fig. 9 – Fully interior beam-column joints – (a) load-displacement curves (b) final failure of planar interior joint and (c) final failure of interior joint with slab

4. Summary and conclusions

In the present paper, a numerical study on the 3D behavior of non-seismically designed beam-column joint sub-assemblies has been conducted. The purpose was to determine the influence of the transverse beams and slab on the joint shear strength. A summary of the numerical results is presented in Table 1. The table provides the values of beam tip load, displacement, drift, horizontal joint shear force and normalized principal tensile stress value for the first shear crack and ultimate shear strength of the joint. The vertical and horizontal joint shear forces are calculated considering global equilibrium of the sub-assembly as well as internal equilibrium of the member at the beam-joint and column-joint interface. Dividing the shear forces by the effective joint area, we get the vertical and horizontal joint shear stresses v_{jv} and v_{jh},, which in turn give rise to diagonal compressive and tensile stresses p_c and p_t.

For the case of 2D exterior joints having beam reinforcement bent into the joint, which is also adopted in this study, Priestley (1997) [2] suggests the critical values of principal tensile stresses as $p_t = 0.29\ (f_c')^{0.5}$ for the first shear crack in the joint and $p_t = 0.42\ (f_c')^{0.5}$ for ultimate joint shear strength. As for the 2D interior joints, Sharma (2013) [3] proposes the double joint shear resistance, that is, $p_t = 0.58\ (f_c')^{0.5}$ for first shear crack and $p_t = 0.84\ (f_c')^{0.5}$ for ultimate strength. Note that those values apply to an aspect ratio equal to 1. For a direct comparison to the values given in Table 1, the reader should take into account the effect of the aspect ratio on the joint shear strength. However, further information on the effect of the aspect ratio on 3D joint configurations is needed.

The present study confirms previous findings and contributes additional evidence with respect to the confining action of transverse members on the joint. The results indicate that out-of-plane members such as beams and slab lead to higher principal tensile stress values for initiation of joint cracking and ultimate joint shear strength. It has been demonstrated that the overall joint behavior of 3D beam-column configurations deviates from the planar ones.

Table 1 – Summary of numerical results

	First joint shear crack				Ultimate joint shear strength			
Type of joint	Beam tip load V_b [kN]	Joint shear force V_{jh} [kN]	Beam tip displacement δ_b [mm] (Drift [%])	Principal tensile stress $[\frac{p_t}{\sqrt{f_c'}}]$	V_b [kN]	V_{jh} [kN]	δ_b [mm] (Drift [%])	$[\frac{p_t}{\sqrt{f_c'}}]$
2D exterior down / up	60 / 52	308 / 267	12.3 (0.55) / 9.3 (0.41)	**0.26 / 0.23**	83 / 78	428 / 402	33.5 (1.49) / 36.5 (1.62)	**0.36 / 0.34**
Corner without slab down / up	62.5 / 54	321 / 277	12.9 (0.57) / 9.6 (0.43)	**0.27 / 0.24**	93.5 / 82	480 / 423	40.5 (1.8) / 46 (2.04)	**0.41 / 0.36**
Corner with slab down / up	72 / 59.6	375 / 301	11.4 (0.51) / 9.9 (0.44)	**0.32 / 0.25**	106.3 / 89.4	554 / 451	45 (2) / 41.5 (1.84)	**0.48 / 0.38**
Ext. edge without slab down / up	71.2 / 58.4	366 / 300	15.3 (0.68) / 12 (0.53)	**0.31 / 0.25**	112 / 100	570 / 512	60 (2.67) / 56 (2.49)	**0.48 / 0.44**
Ext. edge with slab down / up	94 / 71.6	490 / 361	14.5 (0.64) / 11.5 (0.51)	**0.42 / 0.31**	150 / 120	783 / 606	50 (2.2) / 50 (2.2)	**0.67 / 0.51**
Fully int. planar	47	483	13.2 (0.59)	**0.39**	72.6	747	35 (1.56)	**0.61**
Fully int. without slab	62	638	19.5 (0.87)	**0.52**	86.6	891	55 (2.44)	**0.73**
Fully int. with slab down / up	80 / 58.6	701 / 715	13.2 (0.59) / 15.6 (0.69)	**0.58 / 0.57**	124 / 91	1120 / 1085	46 .5 (2.07) / 45.5 (2.02)	**0.93 / 0.87**

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