Computers and Concrete, *Vol. 16, No. 1 (2015) 163-178*
DOI: http://dx.doi.org/10.12989/cac.2015.16.1.163

Study on the behavior of beam-column connection in precast concrete structure

Marcela N. Kataoka[*1], Marcelo A. Ferreira[2a] and Ana Lúcia H.C. El Debs[1b]

[1]University of São Paulo, Engineering School of São Carlos, Structural Department, Av. Trabalhador Saocarlense, nº 400, CEP: 13566-580, São Carlos, SP, Brazil
[2]Federal University of São Carlos , Department of Civil Engineering, Rodovia Washington Luis, km 235, CEP: 13565-905, Monjolinho, São Carlos, SP, Brazil

(Received July 29, 2014, Revised February 15, 2015, Accepted July 12, 2015)

Abstract. Due to the increase of the use of precast concrete structures in multistory buildings, this paper deals with the behavior of an specific type of beam-column connection used in this structural system. The connection is composed by concrete corbels, dowels and continuity bars passing through the column. The study was developed based on the experimental and numerical results. In the experimental analysis a full scale specimen was tested and for numerical study, a 3D computational model was created using a finite element analyze (FEA) software, called DIANA. The comparison of the results showed a satisfactory correlation between loading versus displacement curves.

Keywords: semi-rigid connection; beam-column connection; precast concrete structure; numerical analysis; finite element method

1. Introduction

The studies on the behavior of precast concrete structures are very important for the modernization of the Civil Construction, mainly to improve the quality, productivity and to promote the rationalization on sites.

After the Second World War, the precast concrete structures were so much used to reconstruct the Europe. In this period, new methods and constructions techniques which emphasize the rationalization and the productivity became necessary to boost the prefabrication. The large-scale production and the few available workers were the main reasons for the development of the precast concrete structures.

The most important difference between precast concrete structures and the conventional reinforced concrete structures is the presence of connections. Therefore, the study of the connections behavior stands out in the field of precast concrete structures. The connections behavior has an important role because it is responsible for transmission and redistribution of

[*] Corresponding author, Ph.D., E-mail: kataoka@sc.usp.br
[a] Ph.D., E-mail : marcelof@ufscar.br
[b] Ph.D., E-mail : analucia@sc.usp.br

http://www.techno-press.org/?journal=cac&subpage=8 ISSN: 1598-8198 (Print), 1598-818X (Online)

stresses. In general, the connections between precast elements do not behave exactly as they are considered in the structural analysis. The designers consider that the connections allow or prevent entirely the relative displacements between the connected elements. What happens is that the connections have an intermediate behavior and is called semi-rigid. However, the development of this type of bending resistant connection is extremely important to enable the use of precast concrete system in multistory buildings.

The connections are composed by discontinuous regions that can mobilize displacements and stresses came from elements connected to it. Thereby, there is a redistribution of these stresses over the structure interfering with the performance of the connection. The beam-column connections which are designed to transmit bending moments must provide strength, stiffness and ductility.

According to PCI (2001), the strength of the structure should not be determined by the capacity of the connection, but the mechanism of failure should occur in the structural elements. In addition, according to the FIB (2003), the bending resistant connections must be detailed such that a ductile failure occurs, and the limit of the connection capacity cannot be governed by shear, short welding lengths or other similar details that can lead to fragility. Many of the prescriptions that are behind of these requirements have over the years of study in the field.

The bending resistant connections have been defined as “semi-rigid” in the study of precast concrete structures since the 1980s. This term has been used since 1930 to designate connections in steel structures and now it is becoming common among researchers in the field of precast concrete structures. The concept of semi-rigid connection and its behavior are included in various codes and procedures manuals of design, such as PCI Manuals (Precast Concrete Institute) in the United States. The study on the connections is a major research priorities established by the PCI.

In England, the University of Nottingham has been chosen as “reference center” for testing the beam-column connections, and the City University (London) was responsible for researching in the field of analysis of precast structures with semi-rigid connections. The main results of this research can be found in conferences organized by COST-C1. Both programs of research, PCI and COST-C1, constitute a large experimental data base for the study on behavior of connections in precast concrete structures.

Several parameters were varied and analyzed in researches of COST-C1 (1996), such as the type of shear connector and typology of beam-column connection (endplate, angles). The tests were intended to obtain data to the achievement of moment-rotation curve, allowing the analysis of connection stiffness. One interesting point that can be a reference for connections in precast concrete structures regards to the reinforcement details used in the slab in order to distribute the stresses around the column. According to Elliott (2002), researcher of University of Nottingham, continuity reinforcement must be used between structural elements, such as between beam and column, between slabs and panels, in order to avoid the progressive collapse in the case of accident. The British Standard 8110-1 (1997) also recommends the use of reinforcement in the connections with the same purpose.

Gorgun (1997), another researcher of University of Nottingham, conducted tests of beam-column connections with continuity reinforcement passing through the column and hollow core slab without concrete cover cast on site. The main conclusion of the study was that the continuity bars promoted the semi continuity behavior of the connection, presenting a good performance in terms of resistance and stiffness.

Chefdebien (1998) studied the behavior of two types of connections most used in France. In the procedure of design of these chosen connections, it was common the practitioners consider them as pinned and the results of this study proved the opposite; the connections had a semi-rigid behavior. The studied connections were comprised by support pad, bolts and concrete cover cast on site. To understand the influence of each component, a parametric analysis was carried out and it was possible to conclude that connections with flexible support pads and flexible vertical padding have lower strength and stiffness than the connections with rigid materials.

Elliott *et al*. (2003) other important aspect regarding to semi-rigid connections was analyzed. As might be expected, connections with internal and external columns have different behaviors and this fact was confirmed with the experimental study of Elliott *et al*. (2003). The results showed that connections with external columns had high initial stiffness followed by a ductile behavior and connections with internal columns presented slightly lower initial stiffness, but had higher plasticity moments.

The cracking in the connection region it is important in the study on the structure behavior. In PCI (1986) is summarized the results of several tests with different types of connections, and makes notations about the location of cracks and their potential causes. In the case of connections with continuity bars passing through the column and part of the beam cast on site for solidarization of the reinforcement, there are four possible ways for cracking

1. In the first mechanism the cracks are distributed along the section of the beam and not concentrated only at the interface between the beam and the column.
2. The cracks are concentrated in the beam-column interface.
3. The cracking occur in the area where the precast concrete and concrete cast on site have meet due to the lack of shear reinforcement.
4. The fourth mechanism of cracking occurs when there is a lack of dowel reinforcement on the beam reinforcement, contributing to the appearance of cracks in the corners of these elements.

The first mechanism is considered ideal due to indicate that the bending reinforcement is correct as well as the shear, which is important in the case of dowel failure.

Due to lack of normative prescriptions for the design of semi-rigid connections, Ferreira *et al*. (2007), using experimental results of semi-rigid connections tests, validate an analytical method to analyze the behavior of bending resistant connections based on the so called fixity factor α_R. This factor is a non-dimensional parameter that associates the rotational stiffness of the beam-column connection with the stiffness of the precast beam. This method helps the designers to select an appropriated connection to a specific structure.

In the research of Shariatmada and Beydokhti (2011) was tested a connection between precast beam and column built without the use of corbels. The improvement of the bending moment transfer to the column by the use of prestressed reinforcement was the way found by the researchers to improve the connections behavior. Hawileh *et al*. (2010) carried out a numerical and experimental study on the behavior of the connections involving prestressed reinforcement. The authors compared both results and they concluded that computer simulation is an economical option to analyze the behavior of connections.

Kaya and Arslan (2009) also analyzed beam-column connections with prestressed reinforcement. They noted that for different levels of prestressing applied, the connections presented satisfactory behavior. In the literature review was found that the use of prestressed reinforcement in precast concrete connections has been studied for a long time. The research of Saqan (1995) is one of these studies. Saqan (1995), a researcher of the University of Texas, tested various configurations of connections that would provide stiffness to the structure when subjected to earthquakes. The connections developed did not have concrete cast on site and should be economical and ductile. This research had an objective to increase the knowledge about the behavior of this type of connection in order to increase the application of precast concrete structures in the United States.

The dowel type of the connection is the most common all around the world. However, the knowledge about its seismic behavior was incomplete and poorly understood. To analyze the failure of dowel mechanism, Zoubek *et al*. (2013) created a numerical model in the FEA software and calibrated using the results of the experimental investigations. The most important observations of this research are that the failure mechanism is initiated by yielding of the dowel and crushing of the surrounding concrete was confirmed.

Innovative configurations of connections are been developed in order to make easier the assembly and to increase the stiffness. In Choi *et al*. (2013) a typology of beam-column connections using steel shapes as steel connector was tested. The shapes were casted with the structural elements (column and beam) and they are used to carry out the connections by bolts. In this type of connections there is not the presence of corbels and dowels.

2. Used methodology

The methodology adopted in this research involved two stages. First, validating a numerical model using experimental results, and second, performing a study on the connections behavior.

The considered experimental model consisted of a precast concrete beam-column connection subjected to monotonic loading. This connection was studied in Kataoka *et al*. (2012). The connection was composed by concrete corbels, dowels and continuity bars passing through the column by openings filled with grout. The consolidation of the connection was made with concrete cast on site.

The evaluation of the connection performance was made based on loading versus displacement curves and the stiffness of the connections. The pattern failure was also analyzed.

The main contribution of this research is to define a 3D finite element model capable to reproduce a beam-column connection in precast concrete structure and to carry out a study on its mechanical behavior in order to define the stiffness and strength.

3. Summary of the experimental study

The connection typology on study in this paper was chosen due to its easiness of execution and also for its wide use on sites. The tested prototype consists of a column with two corbels with two 20 mm dowels, simulating an internal column with two cantilevers beams. The beams had a precast part and another part cast on site. The part that was cast on site was carried out to consolidate the connection by the solidarization of the continuity bars which pass through the column. Fig. 1 illustrates the experimental model with indication of the continuity bars, column and beams.

The column used in the experimental model had 1400 mm of height and had rectangular cross section with 500 mm x 400 mm. The corbels had 400 mm x 400 mm x 250 mm and were precast with the column. The beams had a precast part of 400 mm of height and 270 mm of height of cast on site concrete. The compressive strength of the concrete used in the production of precast parts was 40 MPa while the cast on site had 25 MPa. The continuity bars were constituted by four 16 mm bars. Fig. 2 shows further details of the dimensions of the model and the location of the continuity reinforcement.

The assembly sequence of experimental model was as follows (Fig. 3)

1. Attaching the beams in the dowels, which were located in the corbels;
2. Filling the beam-column interface and the hole of the dowels with grout;
3. Placement of continuity reinforcement and filling the column hole with grout;
4. Bonding of strain gages;
5. Assembly of timber shapes;
6. Concrete casting.

In the test setup were used three reaction frames each one with a hydraulic jack. Two of them were placed at the end of the beams for monotonic load application. The third hydraulic jack was placed on the top of the column to apply a constant load of 170 kN during the test. This procedure was adopted in order to simulate the loading come from up floors and also to stabilize the model. Fig. 4 shows the configuration of the test setup.

In the test, a vertical loading was applied in the end of the beams producing negative bending moment on both sides of the column. The distance from the point of load application to the center of the connection rotation was 1.70 m. Under this point it was measured the vertical displacement of the beams.

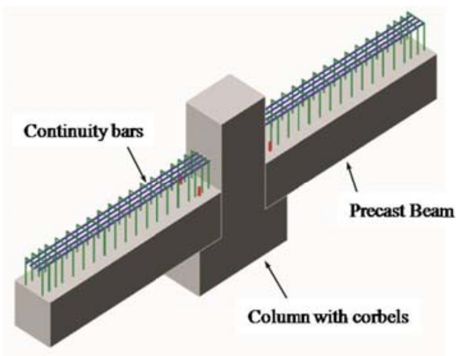

Fig. 1 Experimental model

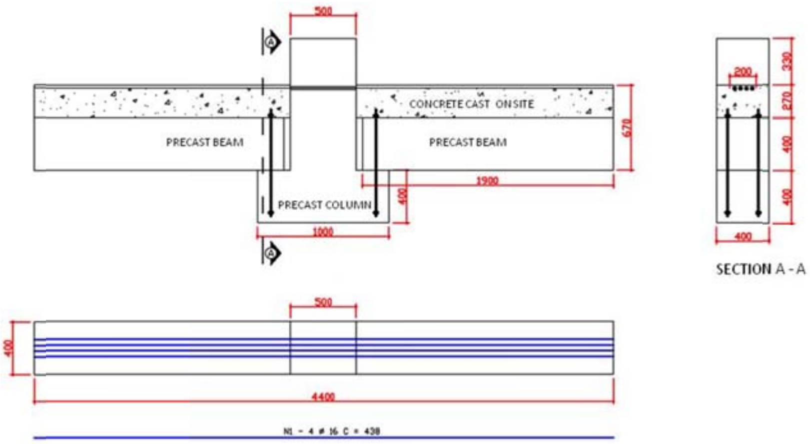

Fig. 2 Dimensions of the experimental model (Unit: millimeters)

Fig. 3 The assembly sequence of experimental model

Fig. 4 Test setup

4. 3D finite element model

The 3D finite element model created for this study replicates the beam-column connection of the experimental program of Kataoka *et al*. (2012). The software Midas FX+ was used to construct the geometry of the model and also to view the results (pre and post processing). The software DIANA was used to process the numerical model using the finite element method (FEM).

The construction of the numerical model began with the creation of the column with the corbels, after this the precast beams and the part of the beams which was cast on site were defined. The last stage of the construction of the geometry was the definition of the reinforcement of the structural elements. First the column reinforcement and dowels were created, after that were defined the continuity bars and the beams reinforcement. Fig. 5 illustrates each stage of the constructions of the numerical model.

4.1 Materials

4.1.1 Properties

The mechanical properties of the cast on site concrete were determined in the compression tests and regarding to the precast concrete, the adopted value was the one reported by the manufacturer company.

The yielding stress adopted for the steel bars used in the whole reinforcement of the model was 550 MPa. No tensile tests were carried out and this value was adopted based on the results of other tests that allowed to deduce that the steel bars used in Brazil has this average yield stress. Table 1 summarizes the mechanical properties.

4.1.2 Constitutive models

• Concrete

The constitutive model used for the concrete was suitable for brittle or quasi-brittle materials (CONCRETE AND BRITTLE MATERIALS). To characterize the distribution of crack was used the TOTAL STRAIN model, whose the advantage is the simple concept. The TOTAL STRAIN model can be represented by ROTATING CRACK MODEL or FIXED CRACK MODEL. In the numerical model created for this study was used the FIXED CRACK MODEL. The tensile concrete behavior was assumed as brittle and in compression was used an ideal elastic-plastic model.

• Reinforcement

The bolts, the slab reinforcement and the shear connectors were represented by REINFORCE, which is a tool of the software DIANA specific to simulate the behavior of steel bars. The finite element crossed by the REINFORCE is stiffened, which causes the same effect that steel bars cause in reinforced concrete structures. The plasticity models of Tresca and von Mises are applicable to steel elements because they are ductile materials. The model of maximum energy distortion of Von Mises was chosen for the reinforcement. This model admits that the maximum energy accumulated in the distortion of the material cannot be equal or greater than the maximum distortion energy for the same material in uniaxial tensile test.

Summarizing, METAL model was adopted with the criteria of Von Mises plasticity with IDEAL PLASTICITY, without consideration of the hardening or strain hardening. In the model of ideal plasticity, or also known as perfectly plastic, the material does not support efforts after reached the yielding stress.

• Interface

The DIANA has two families of interface elements: structural interface, for structural analysis, and structure-fluid interface, used for analysis of fluid and dynamic structure. These elements are usually used to analyze the contact between structural elements. The interface elements used in this study were structural interface. For the joints considered in numerical models, the interface was represented by constitutive model for cracking, with discrete cracking and brittle behavior.

Finally, Table 2 shows the input properties of the materials constitutive models.

4.2 Finite element

Two types of finite elements were used to construct the mesh: elements of plane stress and interface elements. The plane state elements were used to represent the concrete, while the interface elements are used at the joint between the column and the beams.

The finite element used for concrete was the solid element HX24L. This element has eight nodes and three degrees of freedom per node. The interface element used was Q24IF, which has 4 + 4 nodes with three degrees of freedom. The illustrations of the two types of finite elements were obtained from TNO (2005) and are shown in Fig. 6.

4.3 Mesh and boundary conditions

Different mesh sizes were tested to determine the appropriate mesh that would provide accurate results with less processing computational time. The chosen finite element mesh was defined with 50 mm elements in size, totaling 11392 elements and 18713 nodes. Fig. 7 shows the used mesh with the details of the interfaces between column and beams and between precast and cast on site concrete which constitute the beams.

The boundary conditions adopted for the numerical model were the restrictions of the displacements on x, y and z directions in the base of the column, simulating the same conditions of the test. The loadings were introduced near the end of the beams; at 1700 mm from the connection. In the top of the column was applied a constant loading of 170 kN during the test in order to reproduce the loading came from up floor and also to stabilize de model. The scheme of the boundary conditions and the loading application is given in Fig. 8.

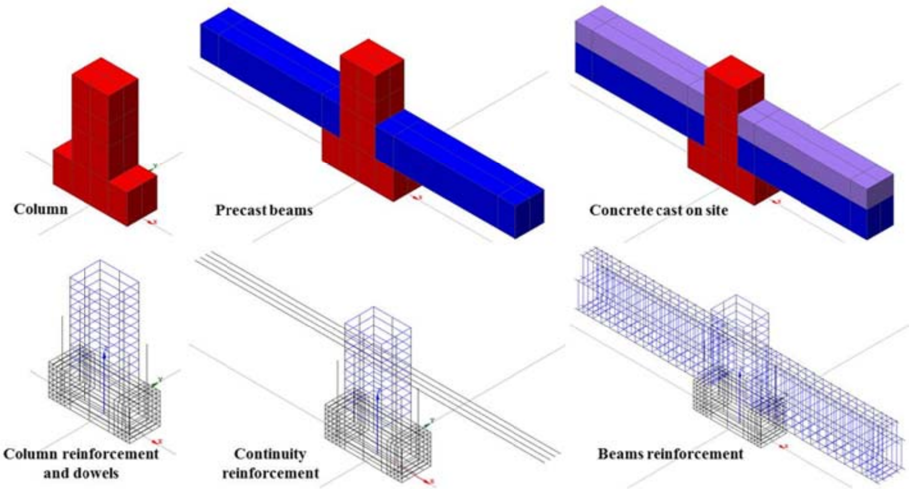

Fig. 5 Geometry of the numerical model

Table 1 Materials properties

	Yielding stress (MPa)	Compressive strength (MPa)	Young modulus (MPa)	Poison (υ)
Precast concrete	2.63	2.62	2.53	3.34
Concrete cast in site	0.041	0.369	0.123	0.290
Steel bars	23.39	23.24	22.55	23.63
Dowels	0.021	0.161	0.161	0.042

Table 2 Summarize of DIANA parameters for materials constitutive models

Parameter	Value
Concrete	
Total Strain model	FIXED
Tensile curve	BRITLLE
Compression curve	IDEAL ELASTIC-PLASTIC
Tensile fracture energy (G_f)	CEB Model Code 1990
Shear retention	0.01
Interface between beams and column	
Tangential Stiffness	10^{-1} *N/mm²/mm*
Normal Stiffness	10^{-1} *N/mm²/mm*
Interface between precast concrete and cast on site concrete	
Tangential Stiffness	10^{3} *N/mm²/mm*
Normal Stiffness	10^{3} *N/mm²/mm*
Steel	
Von Mises	IDEAL PLASTICITY
Reinforcement and dowels	
Von Mises	IDEAL PLASTICITY

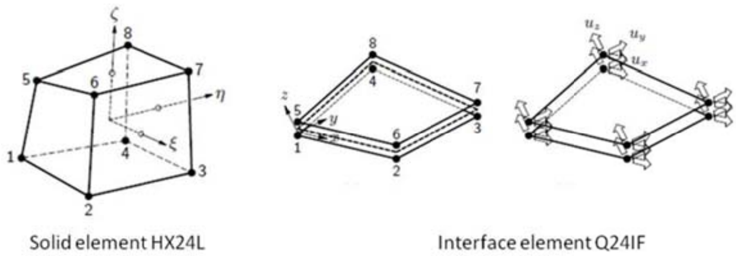

Fig. 6 Finite elements used in the numerical model (TNO, 2005)

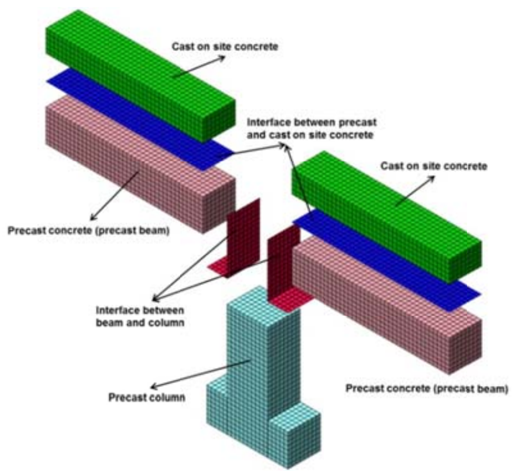

Fig. 7 Details of the mesh

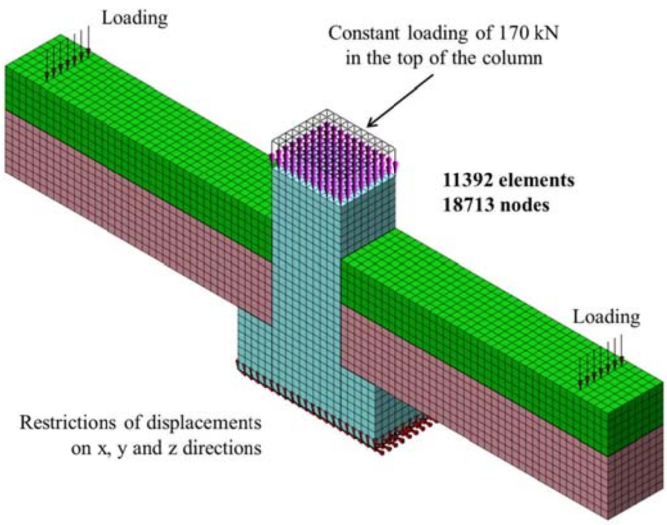

Fig. 8 Scheme of the boundary conditions and loading application

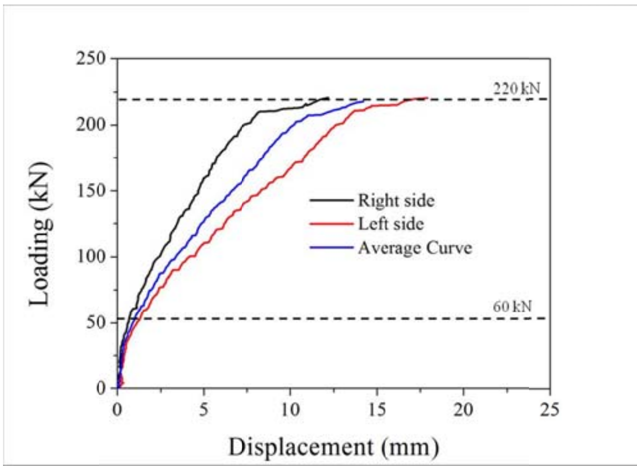

Fig. 9 Loading versus displacement curves of the experimental test

5. Results and discussion

5.1 Results of experimental analysis

The maximum loading achieved in the test was 220 kN, which caused a maximum displacement equal to 12.17 for the right side mm and 17.93 mm for the left side of the model as shown in Fig. 9. It is important to say that the application of the loadings on the beams was carried simultaneously.

During test, it was observed that on reaching 60 kN, the first cracks began to appear in of the region of the connection, and when the loading reached 77 kN, cracks began to propagate along the beam allowing to observe openings of 0.20 mm at 90 kN. The opening reached 0.30 mm when the loading was 100 kN. The failure of the connection was in the continuity bars which reached the yielding stress (550 MPa).

5.2 Results of numerical analysis

In Fig. 10 is indicated the loading which marked the beginning of the cracking, approximately 60 kN, when the loading versus displacement curve had its inclination changed, meaning the loss of stiffness. This point represents the end of the linear elastic regime. The model has remained the same cracking stiffness until the loading of 210 kN and developed up to that instant a vertical displacement of 10.92 mm, when the connection reaches the plasticity. The processing of the model loss the convergence when the loading was 219 kN.

The failure of the beam-column connection occurred when the continuity reinforcement reaches the yielding stress (550 MPa), as indicated in Fig. 11. The maximum vertical displacement of the beams was 14.37 mm for the final step of load corresponding to 219 kN, as shown in Fig. 12. Another important aspect that can be observed in Fig. 12 is the opening of the beam-column connection and also the differential displacement between the precast and cast on situ concrete as well as in the corbel support.

The maximum stresses in the concrete elements were analyzed too. The column, for the most part of it, showed tensile stresses, as shown in Fig. 13. In the face of the corbels where the beams were supported, compressive stresses were observed. In the region around the dowels the compressive stresses reached 53.37 MPa, it shows that the concrete was also in the failure process.

The beams, as well as the columns, also reached high levels of tensile stresses, indicating the presence of multiple cracks along their length (Fig. 13). In Fig. 14 is shown the distribution of stresses in the beams where the interface between the precast and cast on site concretes presents a stress concentration.

The behavior of the dowels was also observed in the numerical results. In the test of the connection, the dowels were not instrumented and it was not possible to study how they behave. The numerical simulation indicated that the dowels were so much requested due to the stresses reached was 235 kN, in the region near the corbels, as shown in Fig. 15. The yielding stress adopted for the dowels in the numerical models was 250 MPa, as reported by the manufacturer of the threaded bars used. In Fig. 16 is presented the loading versus stress in the dowels curve which shows the behavior of this element during the loading.

5.3 Comparison of results

The comparison between the experimental and numerical results was performed using the loading versus displacement curves which is presented in Fig. 17. According to the curves, the initial stiffness of the experimental and numerical models was the same until 60 kN, when the cracking started. From this point, the connection stiffness decreased in both cases, starting a nonlinear stage of the connection behavior.

In order to compare the stiffnesses of the models, the secant stiffness for each one was determined. The connection of the experimental model reached the plasticity with loading of 207 kN and vertical displacement of 11 mm. The secant stiffness of the experimental model was 18.82 kN/mm. The numerical model presented a secant stiffness just 1.6% higher than the experimental model, equal to 19.12 kN/mm. The loading corresponding to this stiffness was 219 kN and the displacement was 11.03 mm. Table 3 shows the mentioned values.

The failure of the beam-column connection occurred due to the continuity bars achieve the yielding stress. Comparing the results of the stresses reached by steel bars which crossed the column, the values indicate similar behaviors at the beginning of loading, but the results drifted away when the loading reached a higher level, as shown in Fig. 18. This difference can have been caused by the simplifications adopted in modeling, as the perfect bonding between the reinforcement and the concrete. This fact is confirmed by the continuity bars of the numerical model be more requested than the bars of the experimental model, taking into account the same loading applied on the beams.

Table 3 Results of the loading versus displacement curves

Curves	Plasticity Displacement (mm)	Loading (kN)	k (kN/mm)
Average curve	11.00	207	18.82
Numerical curve	11.03	211	19.12

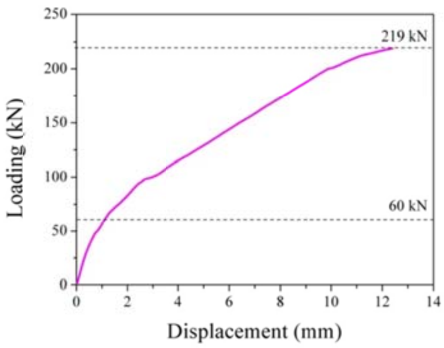

Fig. 10 Loading versus displacement curves of the numerical results

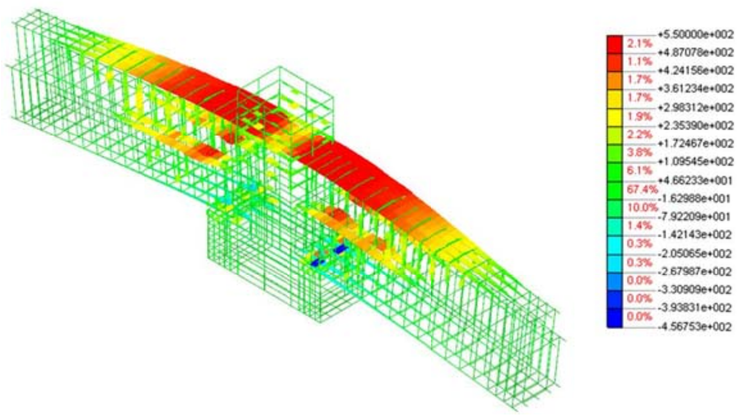

Fig. 11 Stress in the reinforcement of the numerical model

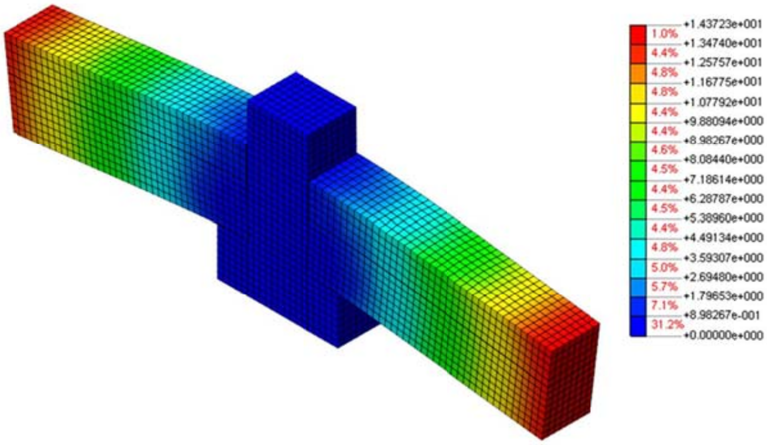

Fig. 12 Vertical displacement of the numerical model

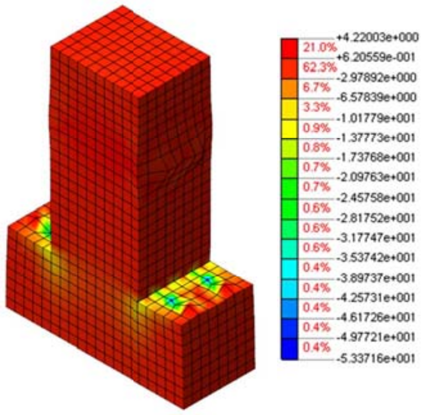

Fig. 13 Stress in the concrete column of the numerical model

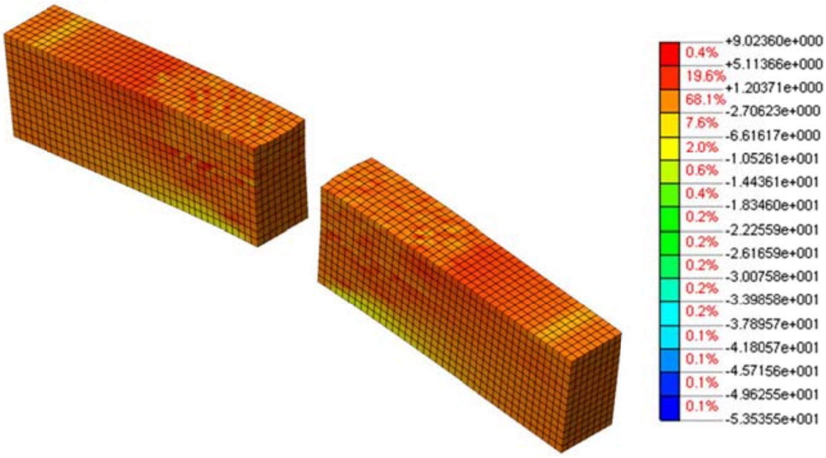

Fig. 14 Distribution of stresses in the concrete beams of the numerical model

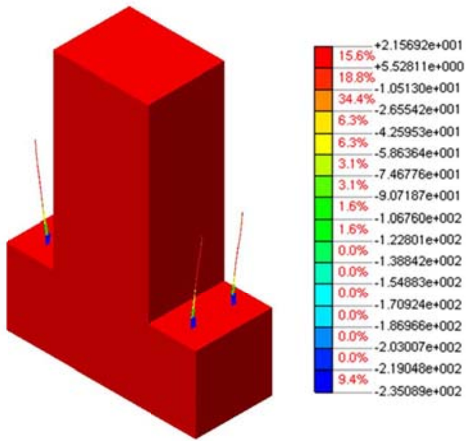

Fig. 15 Stress in the dowels of the numerical model

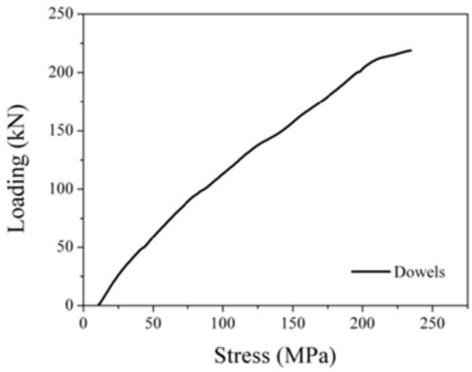

Fig. 16 Loading versus stress in the dowels curve

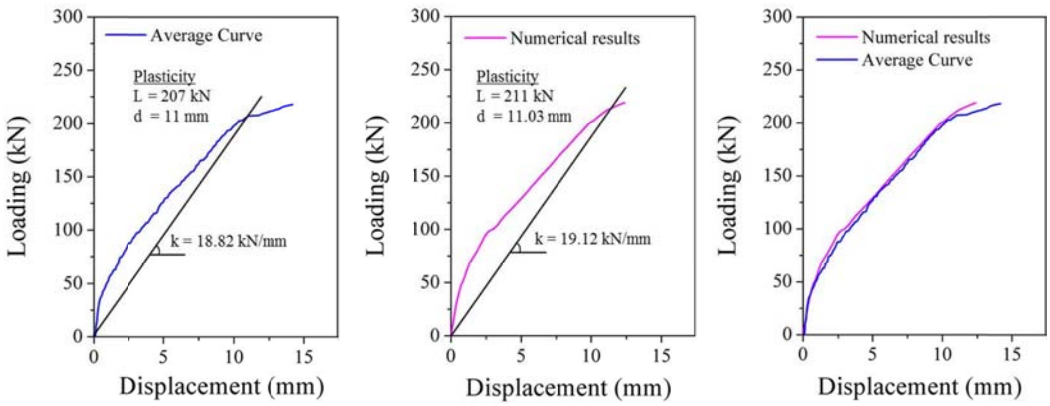

Fig. 17 Loading versus displacement curves of the numerical results

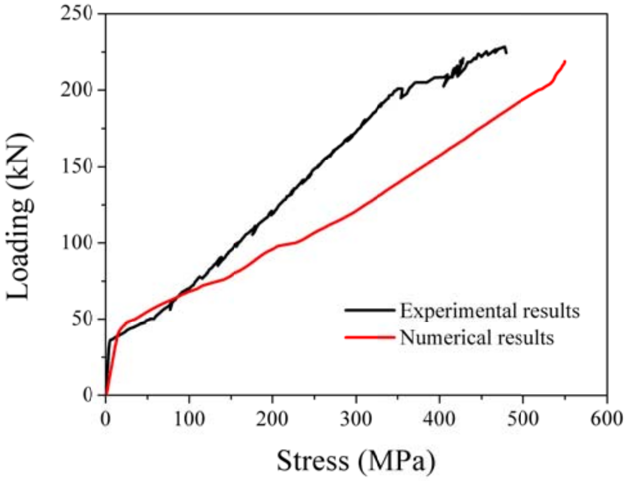

Fig. 18 Loading versus displacement curves of the numerical results

6. Conclusions

The numerical model using solid elements, interface and reinforced elements represented satisfactorily the experimental behavior of the beam-column precast concrete connection due to the good correlation between the loading versus displacement curves. Based on these results, the computational model created to represent the experimental connection is suitable to be used in advanced analysis.

The comparison of the results showed that the stiffness of the models was practically the same, presenting just 1.6% of difference. Also the strength capacity of the connections had similarities, the experimental model reached loading of 220 kN and the numerical model 219 kN. Regarding to the failure mode, both models had the continuity bars reaching their strength.

The use of numerical simulation in the study of this connection became possible to analyze the behavior of the dowels and also the behavior of the concrete elements, with the definition of the stresses in the beams and in the column. The values of stress indicated that dowels were so much requested during the loading (235 MPa), and for the ultimate resistance of the beam-column connection they almost reached the yielding stress (250 kN). Talking about the concrete elements, as observed in the experimental test, high levels of stress in the beams have been developed, generating cracks, especially at the interface between the beams and the column.

Acknowledgments

The authors would like to grateful the financial support from FAPESP, the laboratory of NETPRE (UFSCar) for performing the test and the computer laboratory of the Department of Structural Engineering of the University of São Paulo (USP) which granted the Diana software for the numerical simulation.

References

British Standard 8110-1 (1996), “Structural use of concrete. Part 1: Code of practice for design and construction”.

Chefdebien, A. (1998), “Precast concrete beam to column head connections”, *Control of the Semi-Rigid Behaviour of Civil Engineering Structural Connections*, COST-C1 International Conference, Liege, Belgium.

Choi, H.K., Choi, Y.C. and Choi, C.S. (2013), “Development and testing of precast concrete beam-to-column connections”, *Eng. Struct.*, **56**, 1820-1835.

COST-C1 (1996), European Cooperation in the Field of Scientific and Technical Research, “Semi-rigid behavior of civil engineering structural connections-Composite steel-concrete joints in braced frames for buildings”, Bruxelas, Luxemburgo.

Elliott, K.S. (2002), “Precast Concrete Structures”, Butterworkth-Heinemann, MA.

Elliott, K.S., Davies, G., Ferreira, M.A., Gorgun, H. and Madhi, A.A. (2003), “Can precast concrete structures be designed as semi-rigid frames? part 1-the experimental evidence”, *Electron. J. Struct. Eng.*, **81**(16).

Elliott, K.S., Davies, G., Ferreira, M.A., Gorgun, H. and Madhi, A.A. (2003), “Can precast concrete structures be designed as semi-rigid frames? part 2-analytical equations and column effective length factors”, *Electron. J. Struct. Eng.*, **81**(16).

FIB guide to good practice (2003), “Structural Connections for Precast Concrete Buildings”, *FIB: Commission C6: Prefabrication*.

Ferreira, M.A., Kataoka, M.N., Carvalho, R.C., Araujo, D.L. and Catoia, B. (2007), “Validation of analytical methods for the analysis of the flexural behaviour of precast beams with semi-rigid connections”. *Proceedings of the fib Symposium, Concrete Structures: Stimulators of Development*, Dubrovnik, Croatia.

Gorgun, H. (1997), “Semi-rigid behaviour of connections in precast concrete structures”, Ph.D. Thesis, University of Nottingham, United Kingdom.

Hawileh, R.A., Rahman, A. and Tabatabai, H. (2010), “Nonlinear finite element analysis and modeling of a precast hybrid beam-column connection subjected to cyclic loads”, *Appl. Math. Model.*, **34**(9), 2562-2583.

Kaya, M. and Arslan, S. (2009), “Analytical modeling of post-tensioned precast beam-to-column connections”, *Mater. Design.*, **30**(9), 3802-3811.

Kataoka, M.N., Ferreira, M.A. and El Debs, A.L.H.C. (2012), “A study on the behavior of beam-column connections in precast concrete structures: experimental analysis”, *Ibracon. Struct. Mater. J.*, **5**(6), 848-873

Precast Concrete Institute, PCI (1986), “Research Project nº ¼ - Moment resistant connections and simple connections”, Chicago, Illinois.

Precast Concrete Institute, PCI (2001), “Precast and Prestressed Concrete”, 5th edition, Chicago, Illinois.

Shariatmada, R.H. and Beydokhti, E.Z. (2011), “Experimental investigation of precast concrete beam to column connections subjected to reversed cyclic loads”, *Proceedings of the 6th International Conference on Seismology and Earthquake Engineering*, Tehran, Iran.

Saqan, E.I. (1995), “Evaluation of ductile beam-column connections for use in seismic-resistant precast frames”, Ph.D. Thesis, Faculty of the Graduate School of The University of Texas, Austin.

TNO Building and Construction Research (2005), “Diana User’s Manual”, Release 9, Delft, Netherlands.

Zoubek, B., Isakovic, T., Fahjan, Y. and Fischinger, M. (2013), “Cyclic failure analysis of the beam-to-column dowel connections in precast industrial buildings”, *Eng. Struct.*, **52**, 179-191.

AW