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# EXPERIMENTAL TESTS ON A PRECAST CONCRETE FRAME STRUCTURE SUBJECTED TO LATERAL LOADS

BY

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**Abstract.** The precast concrete frames subjected to lateral loads show a very different behaviour in comparison with cast in place ones, due to connection types between beams and columns. For a better understanding of the connections behaviour, experimental tests on real structures have to be performed. The paper presents a simulated seismic load test on a real precast concrete structure, scale 1:3, with two levels and a single opening, applying an incremental lateral load, acting in the longitudinal direction of the frame, in compliance with the testing procedure described by ACI T1.1-01. The tested structure consists of precast columns and beams with semi-rigid connections between them and cast-*in-situ* slabs at each level. The columns were fully supported at the base and the precast beams were connected with reinforced concrete slabs through the steel stirrups. The structure was subjected to several sequences of increasing displacement controlled cycles until the maximum allowable drift was achieved.

**Key words**: bolted connection; seismic design; semirigid joint.

## 1. Overview of the Experimental Program

The main objective of the experimental program was determining the behaviour of the precast reinforced concrete frames (PRCF) with semirigid

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joints subjected to lateral forces. As floor beams there were used reinforced concrete (RC) elements with cutting ends. The beam was supported by columns through a corbel, fixed with a vertical dowel and two reinforcing bars placed in the topping, Fig. 1. The entire program was supervised by Prof. PhD. Eng. Kiss Zoltán – director of the Structures Department from Technical University of Cluj-Napoca (TUC-N).

Fig. 1 – Beam-column connection (before and after topping was added).

The chosen solution for the tested structure is used more and more often by the civil engineers for multistory precast buildings placed in seismic areas. Previous experiments on precast frames, relatively few though, and the lack of expertise in designing such structures in regions with a risk of seismic hazard, keeps an open door in evaluating the capacity of semirigid joints to develop significant flexural moments.

So far, the behavior of such structures against vertical and horizontal forces (live loads, snow load, wind load, etc.) proved to be very good, that why a better understanding of the seismic response for this kind of structural systems turns out to be a necessity.

A two story precast structure with one bay was erected inside Central Laboratory at Faculty of Civil Engineering, Technical University of Cluj-Napoca. The assembling of the precast elements as well as the reinforcing and casting-in-place the monolithic ones took place between the 10$^{th}$ and 14$^{th}$ of December 2013. In the following time the installation of the acquisition data device took place. Equipments necessary for monitoring the displacements and deformations of the structure were installed, as well as two force transducers. The proper experiment started in the 7$^{th}$ of March and ended six days later when a 3.5% lateral drift was reached.

The tested spatial frame model was derived from a prototype bay of 6 × 9 m, two stories high ($H_{story1}$ = 4.00 m, $H_{story2}$ = 3.75 m) and with column`s cross section of 60 × 60 cm. Using the theory of similarity and a parameter of the geometry $\lambda$ = 3, the model's dimensions were established (Tables 1 and 2).

**Table 1**

*Elements Dimensions – Prototype versus Model*

| Structural element | Prototype element, [cm] | Model element, [cm] |
|---|---|---|
| Column | 60 × 60 | 20 × 20 |
| Beam (without topping) | 60 × 66 | 20 × 22 |
| Beam (with topping) | 60 × 90 | 20 × 30 |
| Slab | 24 | 8 |

The materials used were: for columns, beams and foundations a C40/50 concrete and reinforcement: S500C, except for the ∅6 stirrups where S255C was chosen; while for slabs a C30/37 concrete and S345C reinforcement.

Fig. 2 – Monitoring the displacements and deformations of the structure.

The measurement data was recorded both electronically, using a MGCplus data acquisition system connected to a personal computer, as well as analogical, writing down in tables the records for each of the loading stage. The software used was CatmanEasy/AP 3.0 developed by HBM Company. For measuring the necessary force for each imposed lateral displacement, two force transducers of 1 MN capacity (C6A HBM load cell type) were installed. To measure the displacements of the structure elements (columns, joints, beams, foundations), were used displacement transducers (HBM WA type of 100 mm and 300 mm) and clock comparators (Fig. 2).

**Table 2**
*Structure Geometry – Prototype versus Model*

| Parameter | Prototype structure, [m] | Model structure, [m] |
|---|---|---|
| Story height: level 1 | 4.00 | 1.33 |
| Story height: level 2 | 3.75 | 1.25 |
| Bay geometry | 6.00 × 9.00 | 2.00 × 3.00 |

The RC frame structure was gravitational loaded in two stages. In the first stage a $q_k$ = 5 kN/m$^2$ load was applied on each of the slabs, consisted of bags filled with gravel, 55 kg each. For this stage was considered the live load corresponding the service limit state (SLS), $\psi_0 q_k = 1.0 q_k$. In the second stage the vertical load was reduced to $\psi_2 q_k = 0.6 q_k$ = 3 kN/m$^2$, in order to fulfill the requirement specific to the combinations for seismic actions design. The building was considered belonging category D (shopping areas), according to EN 1990:2002.

Afterwards a number of 11 cycles of controlled horizontal displacement were applied on structure (Fig. 3), corresponding to drifts of 0.20%, 0.25%, 0.35%, 0.50%, 0.75%, 1.00%, 1.40%, 1.75%, 2.20%, 2.75% and 3.50%. Each cycle consisted of 3 steps with a displacement induced from the right side to the left (positive direction), and alternatively another 3 steps with a displacement applied from left side to the right. Also each step had 4 sub-steps in order to be

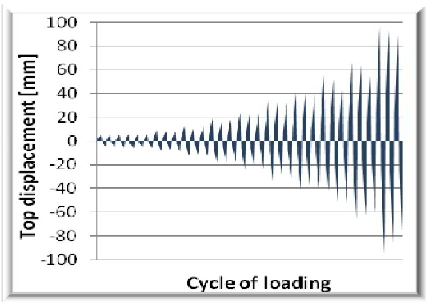

Fig. 3 – Variation of displacement at the top of the building.

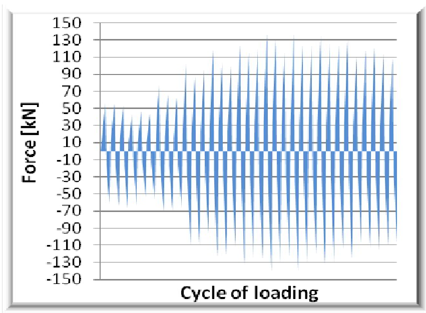

Fig. 4 – Variation of total force, $F_b$, applied on structure.

able to draw the hysteresis curves (force–displacement $P$ *vs.* $\Delta$) afterwards. The necessary lateral force induced to obtain the proper displacement increased steadily until in the 9th cycle, when a maximum force of $F_b$ = 140 kN was recorded, corresponding to a drift of 1.40% and a top displacement of 34 mm. Starting with cycle no. 10, the necessary force to reach predetermined target displacement started to decrease.

In determining the necessary lateral displacements to be imposed, the testing methodology described in the American Standard ACI T1.1-01 “Acceptance Criteria for Moment Frames Based on Structural Testing” was followed (ACI T1.1-01, 2001). The choice of using this standard was due to the lack of a Romanian testing procedure as well as the absence of a European methodology in order to test frame structure that don’t fulfill the prescriptions of the European Standard: EN 1998-1/2004 and Romanian seismic design code: P100-1/2013. Also ACI T1.1-10 is a commonly used standard abroad, offering the possibility to compare current results with old ones. For a substructure that doesn`t meet the prescription of ACI 318-99, chapter 21 (in many aspects similar with EN 1998-1/2004 and P100-1/2013), and wants to be designed for a seismic region, an experimental research is required for the substructure proposed as an earthquake resistant element. The specimen will have enough strength, stability, ductility as well as good seismic energy dissipation capacity. The testing platform used by te authors can be seen below in Fig. 5, while in Fig. 6 there is an image with the entire structure prepared for testing to lateral loading.

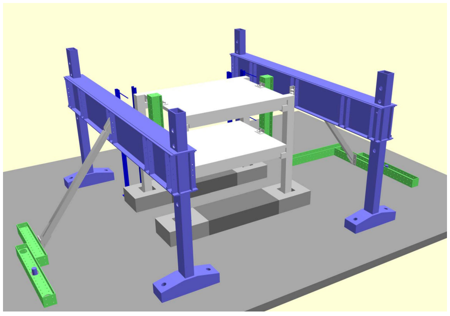

Fig. 5 – Axonometric view of the testing platform (3-D reproduction).

Fig. 6 – Axonometric view of the testing platform.

## 2. Results of the Experimental Tests

During the 1st and 2nd loading stages no crack could be seen, so it can be said the structure`s elements remained into the elastic domain. Afterwards a horizontal load, $F_b$ = 67 kN, was applied for obtaining a first drift of 0.2%, enough for the first cracks to be identified. In this cycle the crack`s maximum width was $w_{\max}$ = 0.12 mm. Considering the spatial frame could be divided into two plane frames, a convention was chosen: on one of the frames (columns: S1 and S2, beams: G1 and G3 – Fig. 7) to be monitored the displacements, while on the other frame (columns: S3 and S4, beams: G2 and G4) the cracks to be tracked in the ordered they appeared.

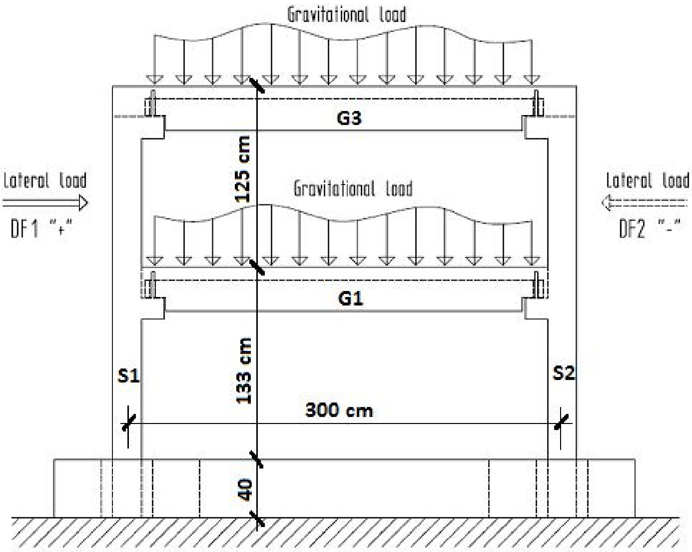


Fig. 7 – Specimen tested – elevation view.

After the appearance of the first crack, for each loading step, the evolution of all representative cracks was monitored. The width of the cracks was measured with an optical device, with an accuracy of measurement of 0.02 mm. Each of the cracks was monitored till $w_{\max}$ = 0.40 mm was reached, while a couple of it was tracked till the end of the loading-unloading cycles.

The first crack with a width of 0.40 mm was identified for a 0.25% drift, placed at the interface of the precast beam and the mortar and labeled as crack no. 15 (Fig. 8). The evolution of this crack reveals the transversal deformation of the vertical dowel. For each of the beam–column joints, the transversal deformation of the dowel was also measured through displacement transducer, marked in the experiment as: T.13, T.14, T.15 and T.16. The behavior of the dowel in relation to the lateral displacement of the building is revealed in Figs. 9 and 10. Using the data from the test, a particular eq. of the dowel's deformation have been determined. Using the results for all the 4 dowels monitored, a more general relation depending on the lateral drift was calculated – eq. (1). When on the end of the beam a negative moment appears, the reinforcement bars placed in the topping will become tensioned, and pulling apart of the precast element (beam and column) is prevented. But when a positive moment acts, the tensions developed are taking over by the bolt (which becomes subjected to shear with bending) with much greater deformation then in the case of a negative moment, at high drifts a gap is created between the precast elements. The transversal rigidity of the dowel being much smaller in comparison with the rigidity of the reinforcement bars when in tension, explain this differences in the behavior of the joint at moments of different signs.

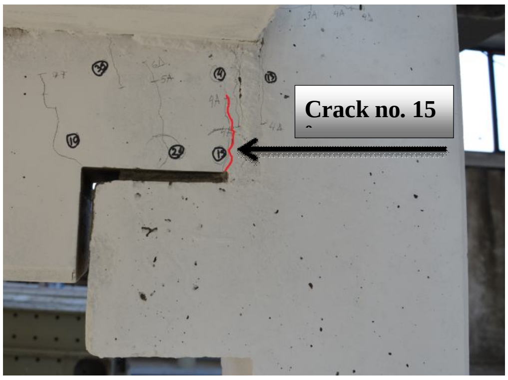


Fig. 8 – Specimen tested – elevation view.

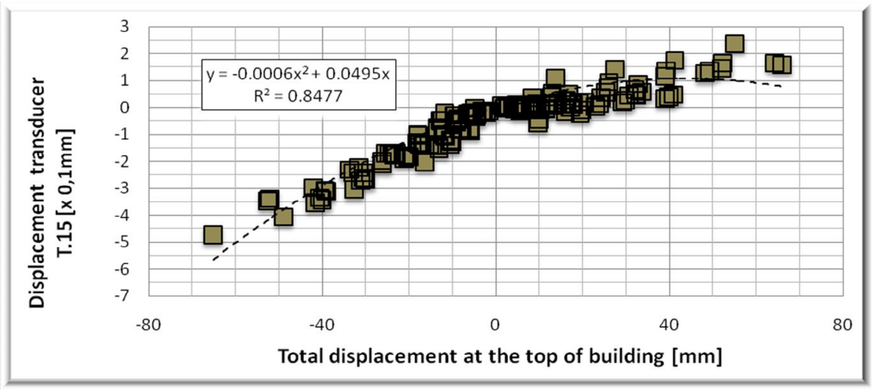


Fig. 9 – First story joint; the transversal deformation of the dowel/crack no. 15 *versus* Total Lateral Displacement.

If we study the graphics in Figs. 9 and 10 it can be noticed an important similarity between the behaviours of the two types of beam-column joints. Although in the joint at the first level the gap between elements was filled with expansive high strength mortar, while in case of the second floor joint the entire connection was cast-in-place, both had similar deformations.

If we denote by $\Delta_d$ the transversal deformation of the vertical dowel and with $\Delta_s$ the story drift, then according to the experimental test, for the studied connection we can say:

$$\Delta_d = -937.50\Delta_s^2 + 47.25\Delta_s \text{ mm.} \quad (1)$$

On the other hand, the cracks developed in the elements, not in the gap between them, reached $w_{max}$ = 0.30 mm at a drift of 1.00% and $w_{max}$ = 0.40 mm at a drift of over 1.40%. The evolution of one of these cracks, placed at the base of the column S3, can be seen in Fig. 11.

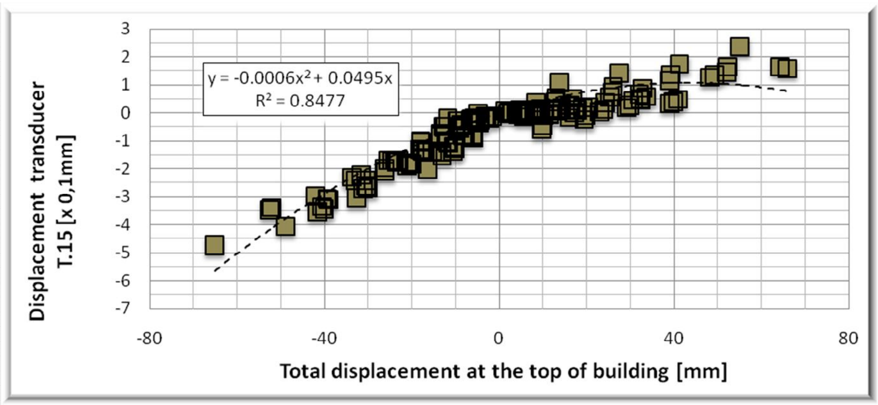


Fig. 10 – Second story joint; the transversal deformation of the dowel/matching crack *versus* Total Lateral Displacement.

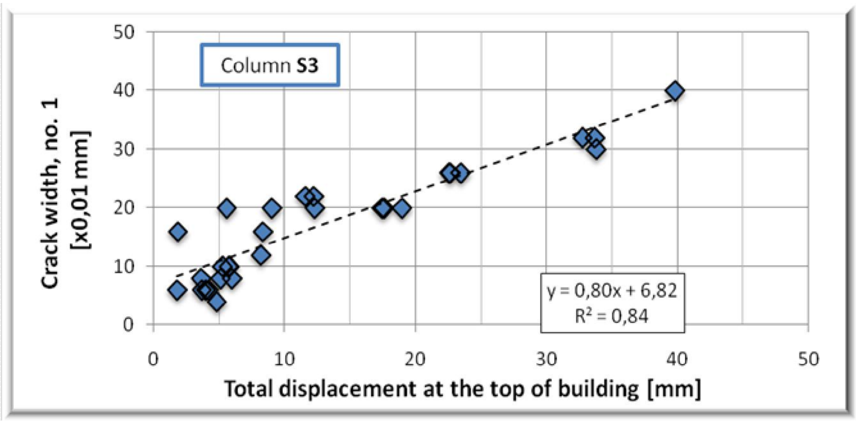


Fig. 11 – Evolution of crack no. 1 placed at base of column S3.

The graphic reveals a gradual opening of the crack, for small story drifts, below 1.40% specific to a serviceability limit state (SLS) and life safety (LS), the width is around 0.20 mm which is acceptable, while for higher drifts up to 3.50%, specific to SLU and collapse prevention (CP), the width increased up to 0.40 mm which again is acceptable in most cases. Also the behaviour is specific to a ductile reinforced concrete element.

### 3. Compliance of the Structure in Terms of ACI T1.1-01

According to this standard, the experiment on a specimen is considered to be relevant if the ratio between model and prototype is not less than 1:3. The maximum story drift imposed should be not less than 3.5% and at least one specimen of each type needs to be tested. The minimum extend of modules on either side of beam–column joint shall be the distance between the contraflexure points nearest that joint (ACI T1.1-01, 2001). The tested specimen fulfilled all the before mentioned requirements.

The behaviour of the tested modules to be considered accepted will need to satisfy a few conditions. The structure should have a similar response in both positive and negative direction of the lateral load applied, fact confirmed by the symmetry of displacements and forces applied during a cycle (Figs. 3 and 4). The attained lateral resistance of structure should be equal or greater than the total shear force considered in the design, consistent with the allowable story drift limitation of the International Building Code, (2011). The maximum total shear force considered in the design of the frame structure was 101kN, while during the experiment a resistance force of over 138 kN was recorded (Fig. 12). For cycling at a story drift of 3.50%, the characteristic of the third complete cycle

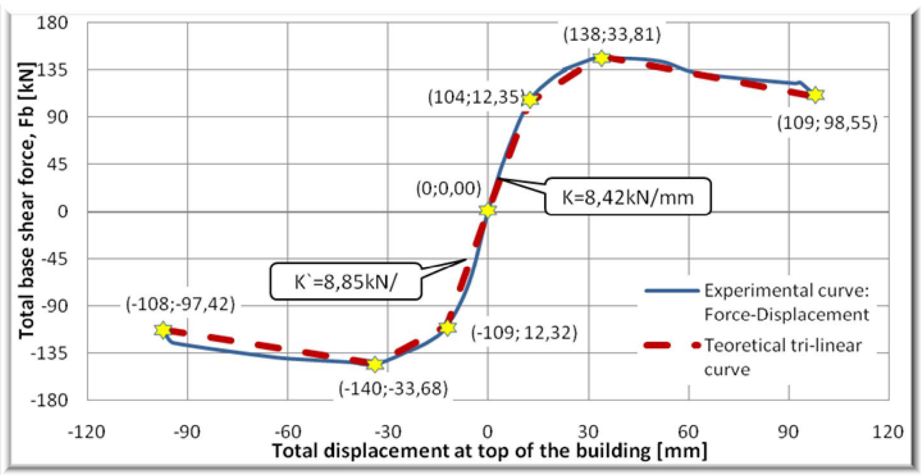


Fig. 12 – Ductility and rigidity on both directions („+” and „–”).

shall have satisfied the following: peak force for a given loading direction will not be less than 75% of the maximum force for the same direction. For positive direction, 109 kN > 75% × 138 kN, while for negative direction: 108 kN > 75% × × 140 kN. The tested specimen fulfilled all the before stated conditions (Fig. 12).

## 4. Conclusions

In the current design practice, the connection between precast reinforced concrete elements is pinned. Seldom, the joints are fully fixed, but such solutions are much more expensive from the economical point of view as well more meticulous in terms of execution. The beam–column connection used is innovative offering a fair speed in execution as well as small costs. The behaviour of such connection is specific to semi-rigid connections. The transition from a pinned beam–column connection to the one used increases significantly the rigidity of the structure to lateral forces and also keeps intact the high ductility of the structure, so very important for buildings placed in seismic regions.

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STUDIUL EXPERIMENTAL AL UNEI STRUCTURI PREFABRICATE ÎN CADRE SOLICITATĂ LA FORȚE LATERALE

(Rezumat)

Structurile în cadre din beton armat prefabricat supuse la acțiuni orizontale au o comportare diferită față de cea a unor structuri similare în variantă monolită, datorită tipurilor de îmbinare grindă–stâlp utilizate. Pentru o mai bună înțelegere a comportării îmbinărilor, a fost implementat un program experimental pe o structură reală. Lucrarea de față cuprinde o analiză asupra unei structuri P+1, scara 1:3, în variantă prefabricată şi testată la solicitări laterale. Astfel se încearcă simularea efectului unei acțiuni seismice prin aplicarea incrementală a unei forțe laterale, pe direcția longitudinală a clădirii, şi

urmând procedura de testare descrisă în standardul ACI T.1.1-01. Specimenul de structură testat a fost compus dintr-o tramă cu stâlpi şi grinzi prefabricate, cu noduri semirigide şi plăci monolite ce descarcă unidirecțional. Stâlpii au fost încastrați în fundații de tip pahar ; în timp conlucrarea plăcii cu grinzile s-a realizat prin intermediul conectorilor prevăzuți în grinda prefabricată. Structura a fost supusă la mai multe cicluri de încărcare–descărcare, impunând o deplasare laterală majorată gradual, experimentul încheindu-se cu atingerea driftului de nivel maxim impus.