Seismic Performance of Mechanical Connections in the SAFECAST Precast Building

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SUMMARY:
A full-scale three-storey precast building was tested under seismic conditions at the ELSA Laboratory in the framework of the SAFECAST project. The unique research opportunity of testing a complete structural system was exploited to the maximum extent by subjecting the structure to a series of PsD tests and by using four different structural layouts of the same mock-up, while approximately 160 sensors were used to monitor the response of each layout. The main investigated parameter among all four structural configurations was the behavior of the mechanical connections between various precast elements. Dry mechanical connections were adopted to realize the joints between: floor-to-floor, floor-to-beam, wall-to-structure; column (and wall)-to-foundation and beam-to-column. The results demonstrate that the new beam-to-column connection system is a viable solution toward enhancing the response of precast RC frames subjected to seismic loads, in particular when the system is applied to all joints and strict quality measures are enforced in the execution of the joints.

Keywords: Precast Concrete Structures ; Beam-column joints; Dry connections; Mechanical connections.

1. INTRODUCTION

The research on the seismic behaviour of precast concrete structures is very limited if compared to traditional cast-in-situ frame reinforced concrete (RC) structures. In fact, in spite of the overgrowing diffusion of this kind of structures, their peculiar characteristics and, in particular, their response to seismic excitation, have not been so thoroughly investigated and univocally determined at present. From a general point of view, there are two alternatives to design precast structures. One choice is the use of precast concrete elements interconnected predominantly by hinged connections, whereas the other alternative is the emulation of monolithic RC construction. The emulation of the behavior of monolithic RC constructions can be obtained using either "wet" or "strong" (dry or partially dry) connections. A "wet" connection between precast members uses cast-in-place concrete or grout to fill the splicing closure. Precast structural systems with wet connections must then comply with all requirements applicable to monolithic RC construction. A "strong" connection is a connection, not necessarily realized using cast-in-situ concrete, that remains elastic while designated portions of structural members undergo inelastic deformations under the design ground motion.

The state-of-the-art today on the seismic design of precast concrete building structures comprises a limited number of scientific reports. The ATC-8 action– "Design of prefabricated concrete buildings for earthquake loads", in the proceedings of its workshop contain eighteen state-of-practice and research papers and six summary papers in particular related to the precast systems in New Zealand, Japan, USA and Europe. Simeonov et all. (1988) addressed the seismic behaviour of specific joints used in large panel precast systems of the Balkan region. Another major project, called PRESSS (PREcast Seismic Structural Systems), was made in the 1990s. Specific structural systems with ductile dissipative connections using unbonded PT tendons were addressed by the US and Japanese researchers (Priestley 1996, Nakaki et al. 1999, Shiohara and Watanabe 2000). A relatively recent state-of-art report was published by the fib-Task group 7.3 (fib 2003) reporting on (at that time) latest developments on the seismic design of precast concrete building structures in New Zealand, Mexico, Indonesia, Chile, USA, Slovenia, Japan and Italy. In other related documents (Shiohara and Watanabe 2000, Sheppard 1981, Restrepo et al. 1993) special attention is given to the seismic behaviour and analytical modeling of the connections. However, although these are the most comprehensive existing documents, they cover only some specific precast structural systems and connections. The Balkan project was strongly oriented to large panel systems, which were extensively used in Eastern Europe but are nowadays outdated. Most other works are limited to moment resisting precast frames based on the emulation of the monolithic structural systems

This research was focused on the categories of dry connections, consisting of mechanical devices, which are the most common type in modern precast buildings in Europe. The advantages of dry connections, in terms of quick erection, maintenance, re-use, make them even more appealing in an environmentally friendly, life-cycle performance oriented perspective. Figure 1 illustrates each category of connection between the different structural elements creating the structural body of a precast building. The first category of connections is that between adjacent floor or roof elements. These connections are those affecting the diaphragm action of the roofing of precast structures. The second category refers to connections between floor or roof panels and supporting beams. These connections enforce and guarantee the perimetral restraints of the diaphragm made of the panels in its in-plane behaviour. The third category refers to connections between columns and beams. The beam-to-column joints shall ensure the required degree of restraint in the frame system. The fourth category of connections used to join columns and foundations is typically realized by positioning the precast columns into pocket foundations. Finally, the fifth category comprises connections between wall (or cladding panels) and slab elements.

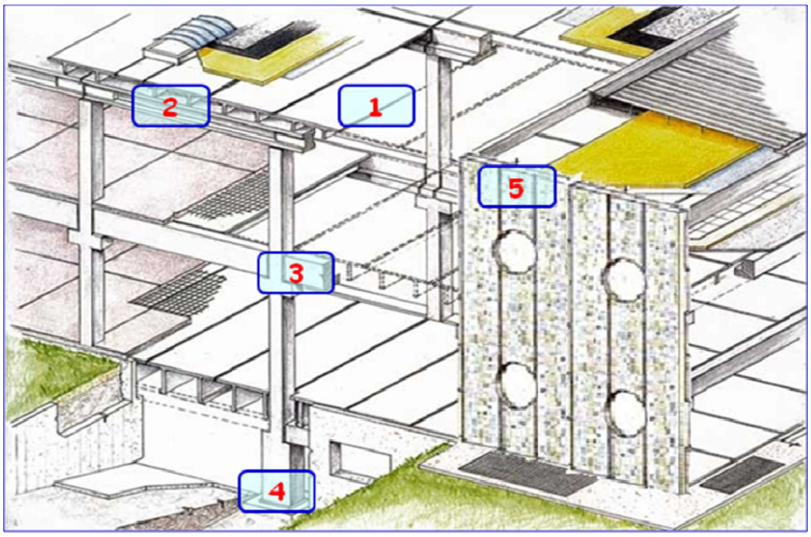


Figure 1. Categories of connections between the different structural elements of a precast concrete building.

The seismic behaviour of the first four categories of connections was investigated in the frameworks of the SAFECAST project. This paper investigates the seismic behaviour of mechanical beam-column connections through reference pseudodynamic tests on a full-scale 3-storey precast concrete building, carried out at the European Laboratory for Structural Assessment (ELSA) of the European Commission in Ispra.

2. TEST STRUCTURES AND INVESTIGATED PARAMETERS

The test structure was a three-storey full-scale precast residential building, with two 7m bays in each horizontal direction as shown in Fig. 2. The structure was 15 × 16.25 m in plan and had a height of 10.9 m (9.9 m above the foundation level) with floor-to-floor heights equal to 3.5 m, 3.2 m and 3.2 m for the 1st, 2nd and 3rd floor, respectively. The columns cross-section was constant along the height of the structure, equal to 50x50 cm, with 1% longitudinal reinforcement (8Φ20). Along the main direction there were beams, with a maximum and minimum width of 2.25 m and 1.85 m, respectively. In the orthogonal direction there were slab elements. Detailed description about the geometry and reinforcing details of all structural members used, namely precast concrete columns, beams and walls, is given in the companion paper (Negro et al. 2012). This paper is focused on the seismic response of the mechanical connections used between precast concrete members.

The SAFECAST specimen was specially constructed with an innovative structural layout which allowed four different structural precast systems to be tested. The behaviour of two types of mechanical beam-column connections was investigated. Firstly, the seismic behavior of "traditional" for the European countries pinned beam-column connections was assessed experimentally for the first time in a multi-storey building. In this case, the columns are expected to work principally as cantilevers. Then a second type of beam-column connection with innovative mechanical devices which allow for the realization of dry fixed connections was applied and experimentally validated.
The first specimen (prototype 1) comprised a dual frame-wall precast system, where the two precast shear wall units were connected to the mock-up. In this structural configuration, the effectiveness of the three floor systems in transmitting the in-plane seismic storey forces from to the vertical elements of the lateral resisting system was investigated. In the second specimen (prototype 2), the building was PsD tested in its most typical configuration, namely with hinged beam-column connections by means of dowel bars. The possibility of achieving emulative moment resisting frames by means of a new connection system with dry connections was investigated in the third and fourth structural layouts. In particular, in the third layout (prototype 3) the beam-column connections were restrained only at the third floor, whereas in the last fourth layout, the connection system was activated in all beam-column joints (prototype 4).

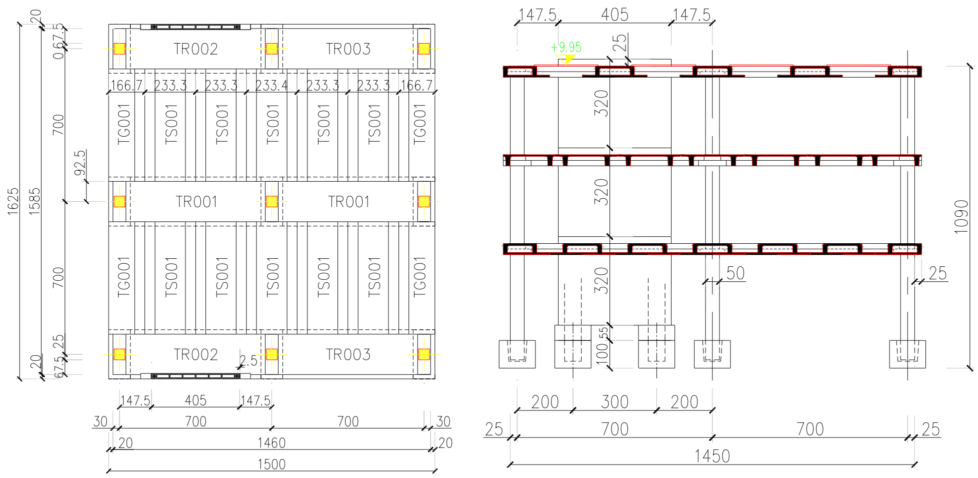
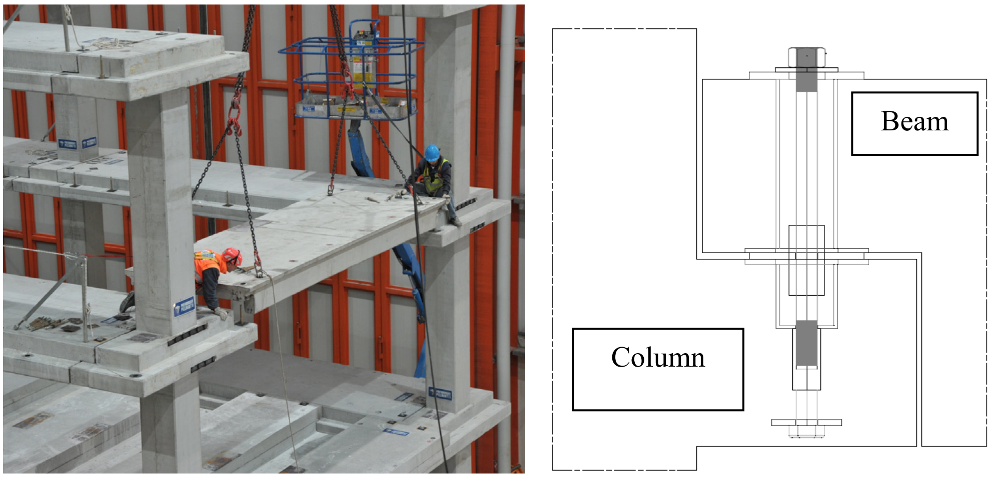


Figure 2. Plan and section views of the mock-up.

2. DESCRIPTION OF THE MECHANICAL CONNECTIONS

Two different types of beam-to-column connection were used in the test structures. The first comprised hinged beam-column connections by means of dowel bars (shear connectors). This type of connection is able to transfer shear and axial forces both for the gravity and seismic forces and possible uplifting forces due to overturning. By definition, they cannot transfer moment and torsion, although in reality they do transfer a small amount of moment. The horizontal connection between the beam and the column was established by means of two vertical steel dowels which were protruding from the column into the special beam sleeves. This pinned beam-column connections were constructed by seating the beams on the column capitals and by holding the beam ends in place by the use of the two vertical steel dowels, as shown in Fig. 3a. The dowels were anchored into the capital. The sleeves were filled with a fine non-shrinking grout, while a steel pad 1.0 cm thick was placed

between the column and the beam in order to enable the relative rotations between the elements.
The large storey forces which were calculated through non-linear dynamic analyses for the hinged three-storey structure (due to the higher modes effect-Olgiati et al. 2010, Fischinger et al. 2010), resulted also in large actions on the connections. This shear force demand in the connections increased further when relative capacity design rules were applied. Thus, it turned out that the required diameters for the dowels were quite large for each storey. In order to have such big diameter at the critical sections, a new dowel was specially developed and used within SAFECAST. This special device has a co-axial tube that increases the resisting area (the diameter) in the critical section, namely in the vicinity of the beam-column shear interface (Fig. 3b). The same dowels with increased diameter at the critical section were also used for the connection between slab and beam elements. Each slab element seating on beam capitals' was connected through four dowels, namely two on each edge of the slab. Identically, two dowels provided the necessary shear reinforcement area in beam-column connection. Table 1 summarizes the diameters and the mechanical properties of the steel dowels (Fe430B) used in all pinned beam-column and slab-beam connections.



(a) (b)

Figure 3. (a) Seating of a secondary beam on the column capital. (b) Detail of a pinned beam-column joint connection and a dowel with increased diameter at the critical section.

Table 2.1. Mechanical properties and diameters of steel dowel and emulative connectors

	Connection Type					
	Hinged (Beam-Column & Slab-Beam)			Emulative (Beam-Column)		
Floor	First	Second	Third	First	Second	Third
Dowel diameter (at the critical section) (mm)	24.4 (40)	24.4 (40)	24.4 (52)	--	--	--
Rebar diameter (mm) (no. of rebars in the joint)	--	--	--	Φ25 (4)	Φ16 (8)	Φ20 (4)
Yield stress, f_y (MPa)	265	265	265	417	528	422
Tensile strength, f_u (MPa)	410	410	410	620	634	622
Ultimate strain, ε_u (%)	20.0	20.0	20.0	N/A	10.5	25.2

The second connection type, which emulates fixed beam-column joints by means of dry mechanical connections, was investigated in the third and fourth structural configurations (prototypes 3 and 4) with the aim of achieving emulative moment resisting frames. Thus, in order to provide continuity to the longitudinal reinforcement crossing the joint, an innovative ductile connection system, embedded in the precast elements, was activated. This connection system comprises four steel rebars slightly enlarged at their ends, two thick steel plates and a bolt that connects the two steel plates, as shown in Fig. 4a. Regarding the realization of this connection system into the mock-up, the bolts that were initially loosen into the joint in prototypes 1 and 2 were properly screwed (Fig 4a) and activated (Fig 4b) to connect the steel devices in the columns and beams. Then, the small (approximately 10-15 mm) gaps between beams and columns were filled by placing a special mortar. Table 1 illustrates the results of these (bare) connection systems used in the joints of each floor for creating a moment resisting beam-column connection.

(a) (b)

Figure 4. (a) Loosen and activated connection system in its bare configuration. (b) Activation of the loosen bolts to provide continuity to the longitudinal bars crossing the beam-column joint.

3. TESTING PROGRAMME

The prototypes were subjected to a series of pseudo-dynamic (PsD) tests. The seismic action was simulated by a real accelerogram modified to be compatible with the EC8 response spectrum for soil type B. Two PsD tests at peak ground accelerations (PGAs) of 0.15g (Prot1_0.15g) and 0.30g (Prot1_0.30g) were initially conducted on prototype 1. The same test sequence was repeated (when the walls were disconnected) for prototype 2 (Prot2_0.15g and Prot2_0.30g). Prototype 3 was subjected only to the higher intensity earthquake of 0.30g (Prot3_0.30g), whereas prototype 4 was tested pseudodynamically at the PGAs of 0.30g (Prot4_0.30g) and 0.45g (Prot4_0.45g). Finally, a sequence of cyclic tests was performed, controlling the top displacement of the structure and constraining the floor forces to an inverted triangular distribution, in order to approach the ultimate capacity of the structure. The lateral displacements were applied on the mid axis of the two bays by two hydraulic actuators. Steel beams were placed along the two actuator axes to connect all the floor elements and distribute the applied forces. An instrumentation network of 175 channels was used to measure: 1) The horizontal displacements of the three frames of the structure (two externals and one central) at the level of each storey. 2) Absolute rotations within the plane of testing of all ground storey columns, 300 mm above their bottom. 3) Absolute rotations within the plane of testing for the beams and columns in the vicinity of all beam-column joints of the central frame and one of the external frames. 4) The beam-to-column joint shear displacement measured in selected beam-to-column joints. The PsD method used, the test set-up adopted as well as the selected input motion are described in detail in the companion paper (Negro et al. 2012).

4. GLOBAL RESPONSE

The global response of all prototypes tested under the PGA of 0.30g is summarized in Fig. 5 in the form of base shear force versus roof displacement hysteresis loops. Key results about prototypes' general behaviour in every test are also summarized in Table 2. They include: (a) The maximum base shear in the two directions of loading. (b) The peak roof displacement. (c) The maximum storey forces recorded in each floor. (d) The maximum rotation measured with inclinometers 300 mm above the base of the ground floor columns. (e) The curvature ductility factor, which is defined as $\mu_\phi = \phi_{max} / \phi_y$, where ϕ_y and ϕ_{max} are the mean curvatures of the column at yield (calculated with cross-section analysis), and the maximum curvature measured during the tests, respectively. The experimental curvature was derived from the relative rotation measured over the lower 300 mm of the column above the base, including the rotation of the column section at the face of the footing and the effect of bar pull-out from the base.

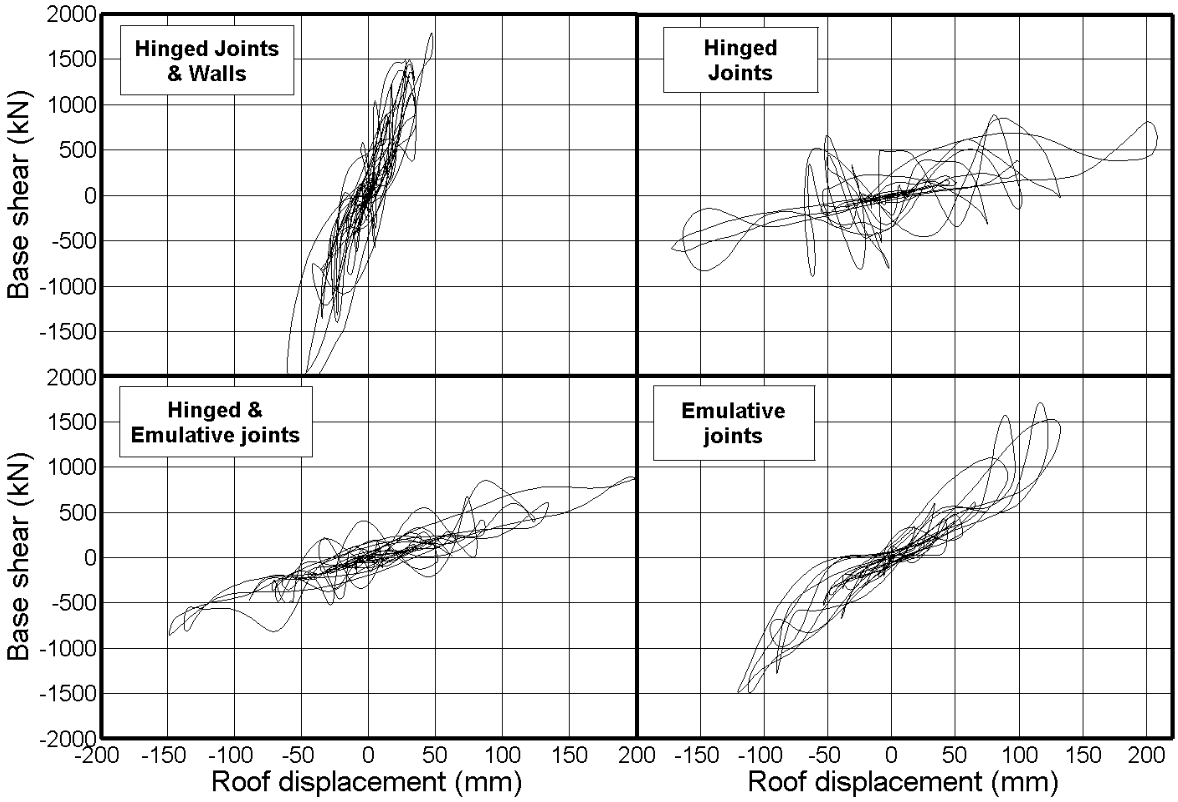


Figure 5. Base shear versus roof displacement response of the four structural systems at PGA of 0.30g.

Table 4.1. Summary of test results

Specimen notation	Maximum base shear (kN)		Peak roof displacement (mm)		Maximum storey forces (kN)						Maximum rotation at the column base, θ_{max}, (%)	Curvature ductility factor
					Pull			Push				
	Pull	Push	Pull	Push	1st	2nd	3rd	1st	2nd	3rd		
Prot1_0.15g	1340	-1457	21.9	-16.8	491	595	688	-475	-577	-581	0.08	0.29
Prot1_0.30g	1780	-2146	48.2	-60.3	722	788	1027	-848	-974	-1166	0.18	0.73
Prot2_0.15g	500	-442	97.4	-86.6	345	336	325	-303	-284	-261	0.28	1.44
Prot2_0.30g	882	-895	208.2	-172.9	795	649	577	-769	-676	-599	0.66	2.86
Prot3_0.30g	889	-859	198.7	-148.4	651	561	540	-691	-453	-471	0.85	3.71
Prot4_0.30g	1715	-1454	132.5	-121.2	921	828	777	-629	-686	-600	0.95	3.85
Prot4_0.45g	1846	-1902	189.3	-206.5	924	794	1133	-848	-855	-772	1.89	7.33
Cyclic Test	2237	-2031	388.1	-415.6	754	1494	974*	-677	-1357	-934*	6.11	22.3

5. RESPONSE OF THE BEAM-COLUMN JOINTS

5.1. Hysteretic behaviour of the joint

In Figure 6, the diagrams of the joint shear force versus the joint slip (horizontal displacement) loops are presented for an external beam-column joint of the third floor, subjected to the 0.30g (prototypes 2, 3 and 4) and 0.45g seismic excitations (prototype 4), respectively. The horizontal opening of the joint (joint slip) was as expected higher in the case of prototype 2 (Fig. 6a) with pinned connections. At the 0.30g test, the average joint slip among the beam-column joints of the third floor that were monitored, was 7.1 mm, 4.7 mm and 1.99 mm, for prototypes 2, 3 and 4, respectively. Consequently, for the same seismic input motion of 0.30g, the joint slip was reduced dramatically in the case of moment resisting joints, that is 3.5 times lower than its counterpart with hinged beam-to-columns joints. A similar trend was observed for the joint axial elongation which from 1.91 mm in Prot. 2, was reduced to 0.97 mm in Prot. 3 and 0.78 mm, when the mechanical connection system (Fig. 8) was activated in all joints. The joint axial elongation that is (essentially) attributed to the relative beam-column rotation can be approximately considered equal the elongation of a dowel well anchored to its ends.

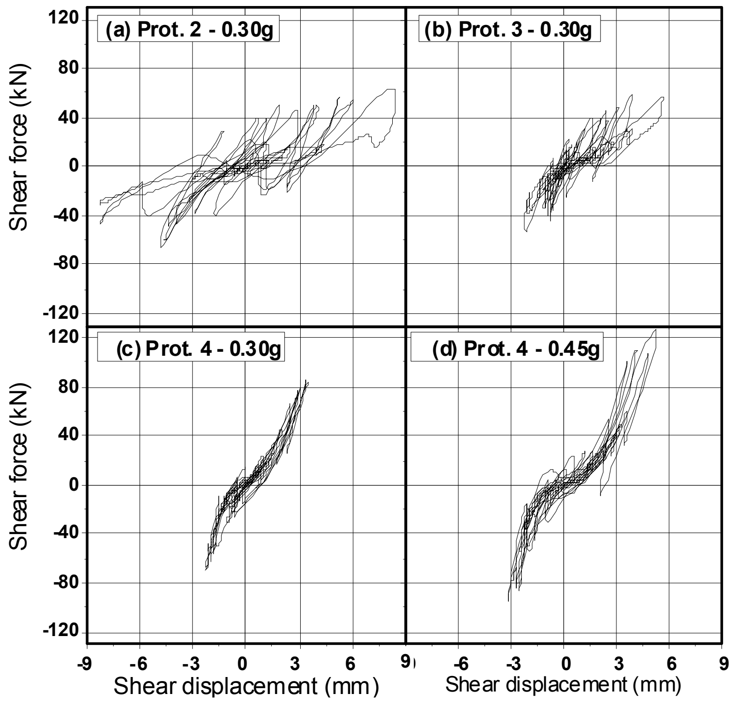


Figure 6. Joint shear force versus the joint slip loops for a beam-column joint of the third floor of: (a) Prototype 2 at PGA 0.30g. (b) Prototype 3 at PGA 0.30g. (c) Prototype 4 at PGA 0.30g. (d) Prototype 4 at PGA 0.45g.

5.2. Column and beam rotation

In a perfectly hinged beam-to-column joint there is no moment transfer to the beam and consequently the last does not rotate. On the contrary, in a monolithic-moment resisting-connection the beam is fixed to the column and ideally rotates as much as the last does. The rotations measured experimentally in many joints of the three floors, though, did neither confirm the first nor the second hypothesis concerning fully hinged or fixed joints.

Figure 7 presents the evolution of the column and beam rotation in a typical (external) joint of the first floor, for all structural configurations subjected to the 0.30g seismic excitation. Once more two main aspects can be observed: 1) higher participation of the beams in the frame behaviour of prototype 4; and 2) the beam-column joint response in prototype 4 is quite different from an emulative joint. It should be pointed out that the execution of this mechanical connection has no quality control or certification for the time being. The state of the mortar filling in the gaps between columns and beams was not identical in all joints and in some cases the penetration of mortar in the gaps was poor. Figure 14 illustrates both cases of a well executed and a non satisfactory-filled joint, as revealed during the demolition phase of the mock-up. This resulted in a semi-rigid beam-column joints with asymmetric (in the two directions of loading) and unequal (between beams and columns) rotations in the beam-column joints, as shown in Fig. 7.

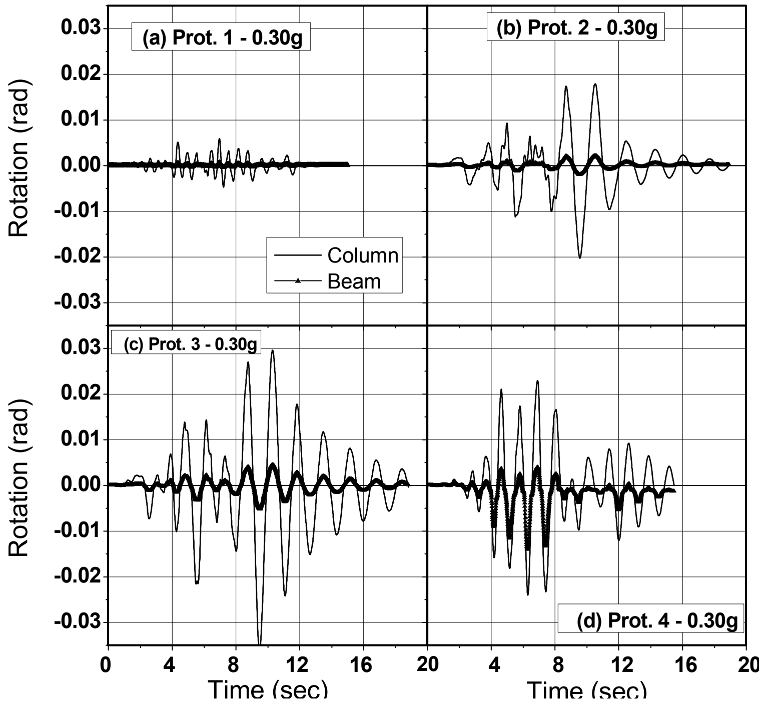


Figure 7. Evolution of column and beam rotation in first floor's joint at PGA of 0.30g for: (a) Prototype 1. (b) Prototype 2. (c) Prototype 3. (d) Prototype 4.

5.3. Energy dissipation

To further evaluate the effectiveness and the seismic response of both types of beam-column connections, the cumulative dissipated energies - computed by summing up the area enclosed within the load versus displacement curve - were recorded for each prototype subjected to the 0.30g PGA seismic excitation and plotted in Fig. 8. Overall, the PsD tests demonstrated that the energy dissipation of the mock-up with pinned connections is smaller than the case of "emulative" connections. Due to this, the hysteretic loops are slimmer (i.e. Fig. 5 and Fig. 6) in prototype 2 in comparison with prototype 4. The energy dissipated by prototype 4 (Fig. 8b) with moment resisting joints during the 0.30g earthquake was about 50% higher that the corresponding energy dissipated by its counterpart with hinged beam-to-columns joints (prototype 2-Fig. 8c) for the same seismic input motion. The restraining of the top beam-column joints (only) realized in Prot. 3 had practically no improvement in the energy dissipation capacity of the structure with hinged beam-to column connections. As the intensity of the seismic input motion increased from 0.30g to 0.45g PGA, the energy dissipated by in prototype 4 was almost doubled. Finally, the energy dissipated during the "funeral" cyclic test was nearly five times higher than that dissipated by prototype 4 in the 0.30g earthquake.

Figure 8 decomposes also the total energy to the energy dissipated by the three individual floors. With the exception of prototype 1, all other layouts displayed considerably higher energy dissipation in the first floor compared to the second and third one. This is attributed to the flexural cracking and yielding which was mainly concentrated at the base of the ground floor columns for the prototypes 2, 3 and 4, as it is explained in the companion paper. The energy dissipation in the third floor was identical for all specimens. In prototype 4, the energy dissipated in the second and first floor was respectively 53% and 72% higher than the energy dissipated by prototype 2 in the corresponding floors. Beyond the flexural cracking and yielding at the base of the ground floor columns, the enhanced energy dissipation in the first floor of prototype 4, is also ascribed to the higher activation of the beams (Fig. 7) and their considerable flexural cracking achieved at the first floor.

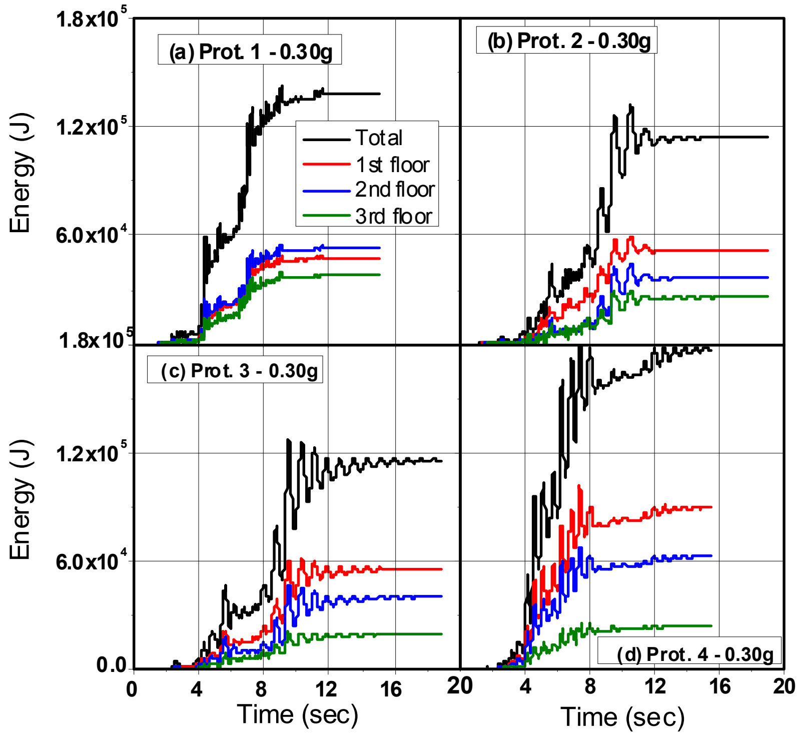


Figure 8. Total and cumulative energies dissipated in each floor at PGA of 0.30g, for: (a) Prototype 1. (b) Prototype 2. (c) Prototype 3. (d) Prototype 4.

6. CONCLUSIONS

A full-scale three-storey precast building was subjected to a series of pseudodynamic tests in the European Laboratory for Structural Assessment. The mock-up was constructed in such a way that four different structural configurations were investigated experimentally. Therefore, the behaviour of two types of beam-column connections of the three-storey precast building was investigated. Firstly, the most common connection system in the construction practice in the European countries comprising pinned beam-column joints was assessed. Afterwards, the possibility of achieving emulative moment resisting frames by means of a new connection system with dry connections was investigated. The main conclusions of the beam-column connections local seismic response are summarized as follows.

It has been shown that there seems to be no upper limit for the storey forces when the structure enters into the nonlinear regime, as one would expect as a consequence of capacity design. This results into large (i.e., much larger than those divided by the q factor) forces in the connections.

The beam-column joint slip was reduced dramatically in the case of moment resisting joints, that is 3.5 times lower than its counterpart with hinged beam-to-columns joints.

Higher was the participation of the beams in the frame behaviour of prototype 4, however; the beam-column joint response in prototype 4 was quite different from an emulative joint. The quality of execution this mechanical connection has no quality control or certification for the time being. This resulted into a semi-rigid beam-column joints with asymmetric (in the two directions of loading) and unequal (between beams and columns) rotations in the beam-column joints.

Finally, the energy dissipation of the mock-up with pinned connections was smaller than in the case of emulative connections. The energy dissipated by prototype 4 with moment resisting joints during the 0.30g earthquake was about 50% higher that the corresponding energy dissipated by its counterpart with hinged beam-to-columns joints (prototype 2) for the same seismic input motion.

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