Seismic Testing of the SAFECAST Three-Storey Precast Building

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SUMMARY:
In the framework of the SAFECAST Project (Performance of Innovative Mechanical Connections in Precast Building Structures Under Seismic Conditions), a full-scale three-storey precast building was subjected to a series of pseudodynamic tests in the European Laboratory for Structural Assessment (ELSA). The mock-up was constructed in such a way that four different structural configurations could be investigated experimentally. Therefore, the behaviour of various parameters like the types of mechanical connections (traditional as well as innovative) and the presence or absence of shear walls along with the framed structure were investigated.

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

1. INTRODUCTION

A collaborative three-year research project called SAFECAST was undertaken by European national associations of precast concrete producers, along with universities and research centres, to study the behavior of precast concrete structures under earthquake loading. The main objective of the project was to fill the gap in the knowledge of seismic behaviour of precast concrete structures, with emphasis on the connections between precast members. A major part of the experimental phase of this programme, consisted of pseudodynamic tests on a full-scale 3-storey precast concrete building, carried out at the European Laboratory for Structural Assessment (ELSA), Joint Research Centre (JRC) of the European Commission in Ispra.

2. THE MOCK-UP

The specimen structure was a three-storey full-scale precast residential building, with two 7m bays in each horizontal direction as shown in Fig. 1. 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 level, respectively. The floor systems, which were of high interest in this research, were carefully selected to gather the largest possible useful information. To accomplish this, three different pretopped floor systems were adopted. As shown in Fig. 1, the 1st floor at 3.5 m was constructed with box-type elements put side by side and welded to each other; similarly the 2nd floor at 6.7 m comprised double-tee elements put side by side welded to each other; and finally the 3rd floor at 9.9 m was realized with the same box slab elements of the 1st floor, but spaced to simulate diaphragms with openings.

The precast three-storey columns had a cross-section of 0.5 x 0.5 m, which was kept constant along their height and were embedded by 0.75 m into 1-m-deep, 1.3 × 1.3 m in plan, pocket foundations. All columns were constructed with wide capitals at the level of each floor, with widths of 0.90 m and 2.25 m in the loading and transverse direction, respectively, in order to allow the mechanical beam-to-column connection, the details of which are presented in a companion paper. The capitals of the columns were designed as cantilevers, fixed at the axis of the columns with bending and shear reinforcement. The longitudinal beams connected to the columns' capitals were precast box-type hollow core elements, with a cross-section 0.4 x 2.25 m and a length of 6.38 m.

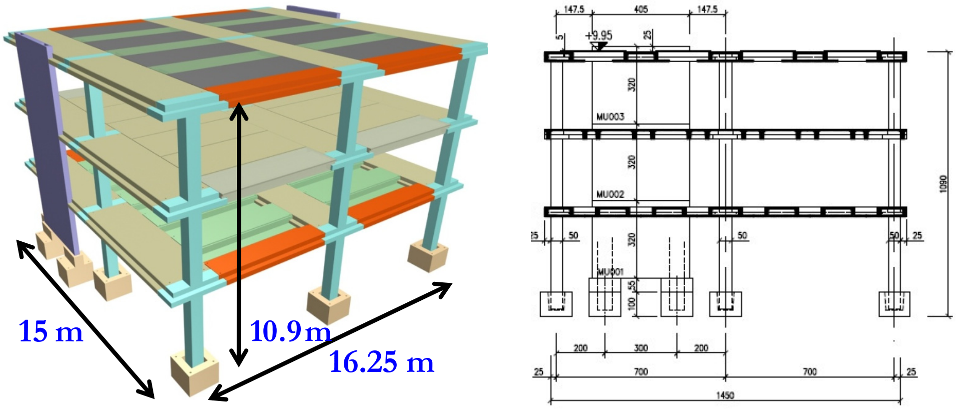


Figure 1. The test structure.

As it can be seen in Fig. 1, two 4.05-m-long x 9.6-m-tall x 0.25-m-thick (4.05 m x 9.6 m x 250 mm) precast concrete walls were connected to the mock-up in order to compose with the columns a dual frame-wall precast system. Each wall comprised 3 wall hollow-core precast elements 3.2-m-tall (Fig. 2), which were joined among themselves by means of vertical reinforcement crossing their gaps at the level of each floor. Concrete was cast only at the two edge cores of the section, where the wall vertical reinforcement was concentrated in "boundary elements". The confinement of the concrete was also there foreseen (Fig. 2). The longitudinal reinforcement was lap-spliced at the mid-height of the second floor. Similarly to the columns, the walls-to-foundation connection was realized through two pocket foundations in which the walls' longitudinal reinforcement was anchored (Fig. 2).

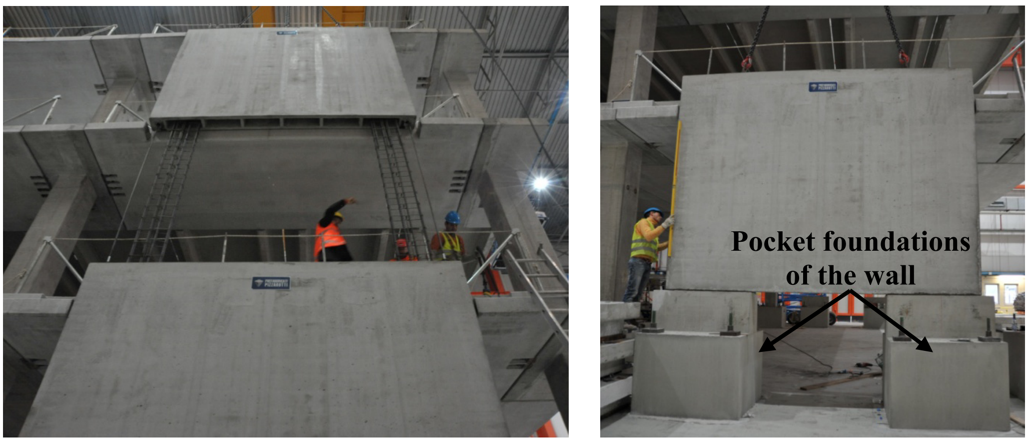


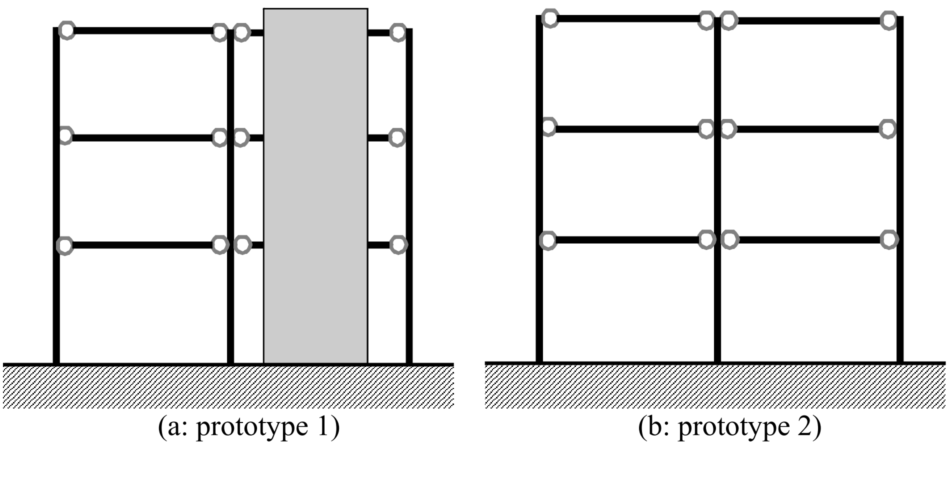
Figure 2. (a) Assembling phase of the precast wall elements. (b) Pocket foundations used for the precast walls.

All precast elements (columns, walls, beams, slabs) were cast using the same concrete class, namely C45/55, which turned out to have a 28-day strength, measured on 150 x150 mm cubes, equal to 64.5 MPa. The steel reinforcement cast into the members had a yield stress of 527 MPa, a tensile strength of 673 MPa, and an ultimate strain equal to 10 %. Table 1 summarizes the dimensions and percentages of steel reinforcement of all prefabricated structural bearing elements used for the construction of the mock-up.

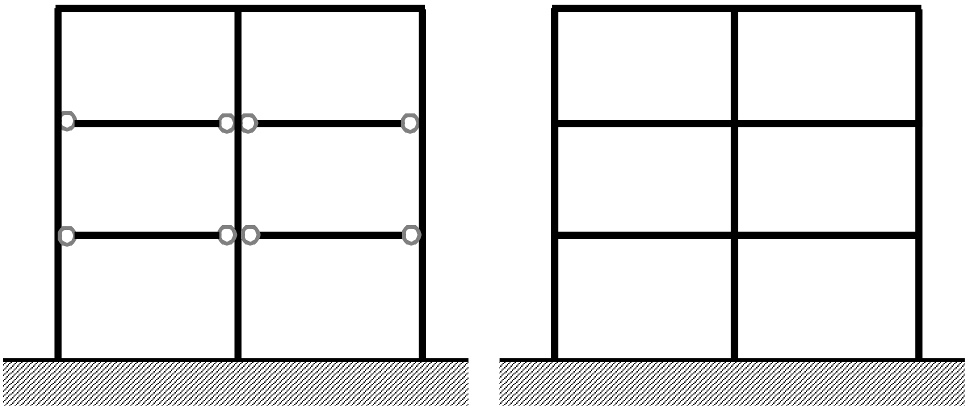
The mock-up was constructed in such a way that the effectiveness of four different structural precast systems could be investigated experimentally. Therefore, the behaviour of a series of parameters, including several types of mechanical connections (traditional as well as innovative) and the presence or absence of shear walls along with the framed structure, could be assesed. The first layout (prototype 1) comprised a dual frame-wall precast system, where the two precast shear wall units were connected to the mock-up (Fig. 3a). In this structural configuration, the effectiveness of precast shear walls in terms of increasing the stiffness of a relatively flexible three-storey precast building with hinged beam-to column joints was examined. In the second layout (prototype 2-Fig. 3b), the shear walls were disconnected from the structure, to test the building in its most typical configuration, namely with hinged beam-column connections by means of dowel bars (shear connectors). This configuration, which represents the most common connection system in the construction practice in the European countries, had been investigated only for industrial typically single-storey precast structures (Negro et al. 2003). Thus, the second layout investigates for the first time experimentally, the seismic behavior of a flexible multi (three)-storey precast building with hinged beam-to-column connections, where the columns are expected to work principally as cantilevers. Afterwards, 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 configurations. With the target of providing continuity to the longitudinal reinforcement crossing the joint, an innovative connection system, embedded in the precast elements, was then activated by means of bolts connecting the steel devices in the columns and beams. A special mortar was placed to fill the small gaps between beams and columns. In particular, the first solution examined was oriented to reduce the flexibility of such structures with hinged beam-to-column joints by restraining just the last floor of multi-storey buildings; and thus, in the third layout (prototype 3 - Fig. 3c), the connectors were restrained only at the third floor. Finally, in the last fourth layout, the connection system was activated in all beam-column joints (prototype 4 - Fig. 3d).

Table 2.1. Dimensions and percentages of steel reinforcement of all prefabricated bearing elements

Precast element	Concrete strength f_c, MPa	Type of the cross-section	Dimensions of the cross-section (m)	Amount of longitudinal reinforcement (mm^2)	Geometrical ratio of longitudinal reinforcement ρ_s, %
Column	64.5	Solid	0.5 x 0.5	2513	1.00
Beams of the 1st floor	64.5	Hollow-core	0.4 x 2.25	1810	0.35
Beams of the 2nd floor	64.5	Hollow-core	0.4 x 2.25	1609	0.32
Beams of the 3rd floor	64.5	Hollow-core	0.4 x 2.25	1473	0.29
Wall	64.5	Hollow-core	2.4 x 0.25	8952	1.30
Wall end	55	Solid	0.8 x 0.25	2767	1.38
Wall web	64.5	Hollow-core	2.45 x .25	3418	1.20



(a: prototype 1) (b: prototype 2)



(c: prototype 3) (d: prototype 4)

Figure 3. Structural layouts of the four Prototypes.

3. TESTING PROGRAMME

The continuous PsD method developed at the ELSA laboratory of the European Commission JRC (Pegon et al. 2008) was used for testing the mock-up. The PsD method couples the properties of the structure into a physical quasistatic model tested in the laboratory and a computer model representing inertia.

Translational masses of 186 857 kg at the first floor, 168 404 kg at the second floor and 132 316 kg at the top floor, were numerically represented in the PsD test of prototype 1. The corresponding masses for the layouts without shear walls (prototype 2, 3 and 4) were considered equal to 170 948 kg, 157 978 kg and 127 013 kg, for the first, second and third floor, respectively. The above simulated masses in the PsD tests are larger than the actual masses of the specimen in order to reproduce the effect of additional loads beyond self-weights.

An overview of the experimental set up adopted is shown in Fig. 4. The lateral displacements were applied equal on the mid axis of the two transversal bays by two hydraulic actuators with a capacity of 1000 kN at the 2nd and 3rd floor levels, while at the 1^{st} floor level (due to the availabilities of these devices in the laboratory), four actuators with capacity of 500 kN were used (two of which controlled in force). Steel beams were placed along the two actuator axes to connect all the floor elements and distribute the applied forces.

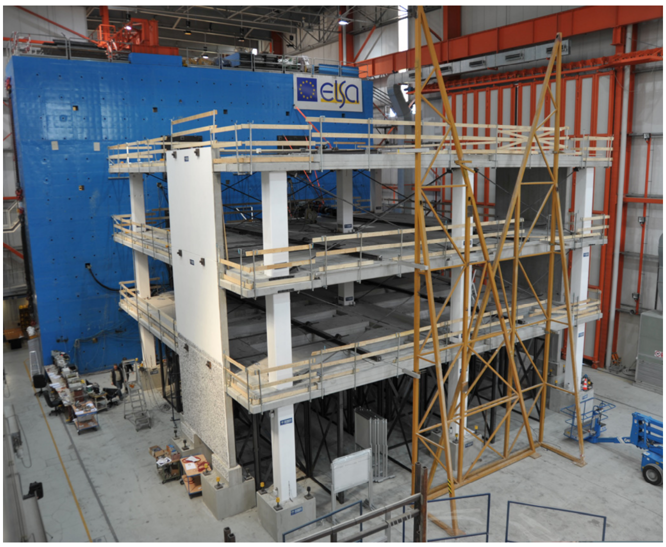


Figure 4. General view of the experimental set-up.

The reference input motion used in the PsD tests is a unidirectional 12 s-long time history, shown in Fig. 5a for a PGA of 1.0g. The selected seismic action was represented by a real accelerogram (Tolmezzo 1976) modified to fit the Eurocode 8 (EC8-EN 1998-3) response spectrum type B all over the considered frequency interval. Figure 5b illustrates the spectra of the modified EW component of Tolmezzo recording. The accelerogram was scaled to the chosen peak ground accelerations of 0.15g for the serviceability limit state, and 0.30g for the no-collapse limit state. Two pseudodynamic tests at a PGAs of 0.15g (Prot1_0.15g) and 0.30g (Prot1_0.30g) were initially conducted on prototype 1, namely the dual frame-wall precast system. After the walls were disconnected from the structure, the same excitation sequence was repeated for prototype 2 (Prot2_0.15g and Prot2_0.30g), which had hinged beam-column connections in all joints. Prototype 3, which had emulative beam-column connections only at the top floor was subjected only to the higher intensity earthquake of 0.30g (Prot3_0.30g), whereas prototype 4, which had emulative connections in all beam-column joints, was tested pseudo-dynamically at the PGAs of 0.30g (Prot4_0.30g) and 0.45g (Prot4_0.45g). A zero-acceleration signal was added after the end of the record, to allow for a free vibration of the test structures, giving total durations ranging between 15s to 19 s for the applied record. To approach the ultimate capacity of the structure, a final "funeral" sequence of cyclic tests was performed, controlling the top displacement of the structure and constraining the floor forces to an inverted triangular distribution, which is consistent with the assumptions of most seismic codes including EC8.

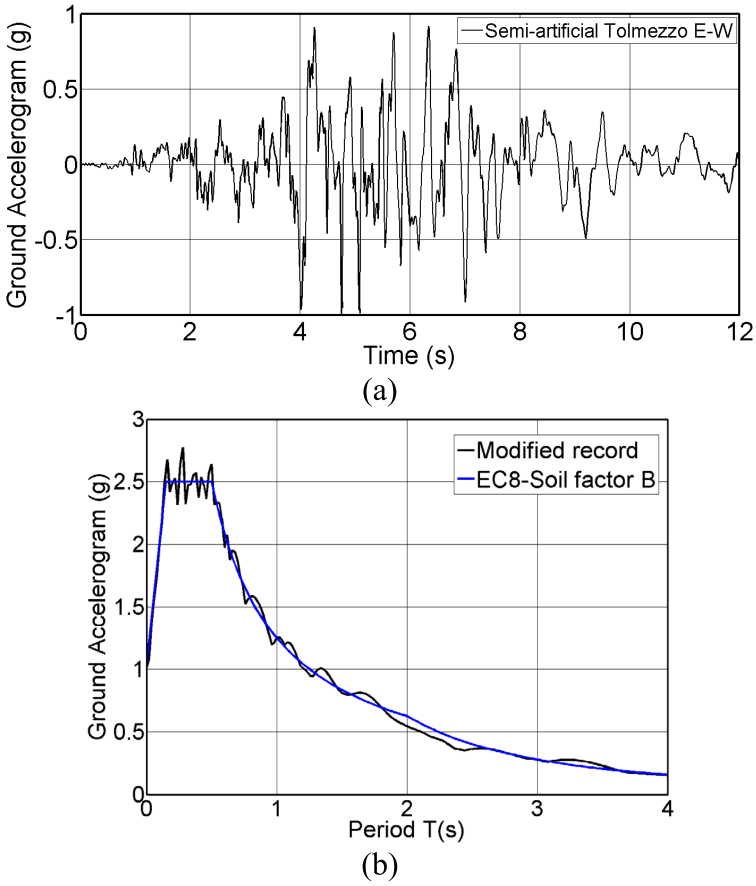


Figure 5. (a) Input motion, scaled to PGA of 1g; (b) Spectra of the modified EW component of Tolmezzo recording.

4. RESULTS

4.1. Prototype 1

Prototype 1 (Fig. 5a) was tested under two input motions scaled to a PGA of 0.15g and 0.30g. The time histories of floor displacements and restoring forces measured in these two PsD tests are shown in Fig. 6. This dual wall-frame precast system was as expected stiff with an (experimental) fundamental natural vibration period of T= 0.30 sec for the 0.15g PGA. At the higher intensity earthquake, namely 0.30g PGA, the response curves were characterised by lower frequencies (natural vibration period shifted to T= 0.46 sec). These period estimations, as for all the tests, were obtained from the measured response by means of identification of equivalent linear models (Molina 2011). The global base shear force versus roof (3rd floor) horizontal displacement hysteretic response is plotted in Fig. 7 for 0.15g and 0.30g tests. At the 0.15g PsD test, corresponding to the serviceability limit state (SLS) earthquake, the response remained practically within the elastic range as it is illustrated in Fig. 7a. This PsD 0.15g test deformed the building to a maximum roof displacement equal to 21.9 mm, while the maximum base shear force was 1457 kN. The maximum interstorey drift ratio was recorded at the third floor equal to 0.31%. Figure 7b plots the base shear versus roof displacement hysteretic curves for the 0.30g test. At this higher intensity earthquake corresponding to the ultimate design limit state (ULS), the response of the dual wall-frame system was characterized by some non linear effects with noticeably wider force-displacement loops. The peak roof displacement and maximum base shear force measured in this test were 60.3 mm and 2146 kN, respectively. The maximum interstorey drift ratios recorded at the first, second and third floor kept low, namely equal to 0.42%, 0.71% and 0.72%, respectively.

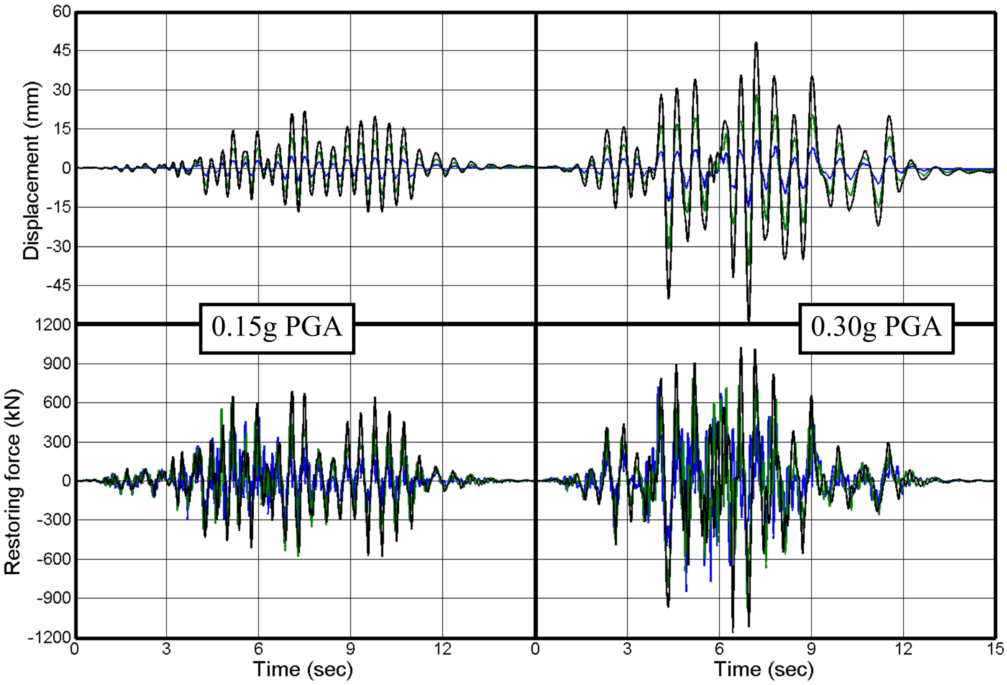


Figure 6. Time histories of floor displacements and restoring forces of prototype 1 at the PsD tests.

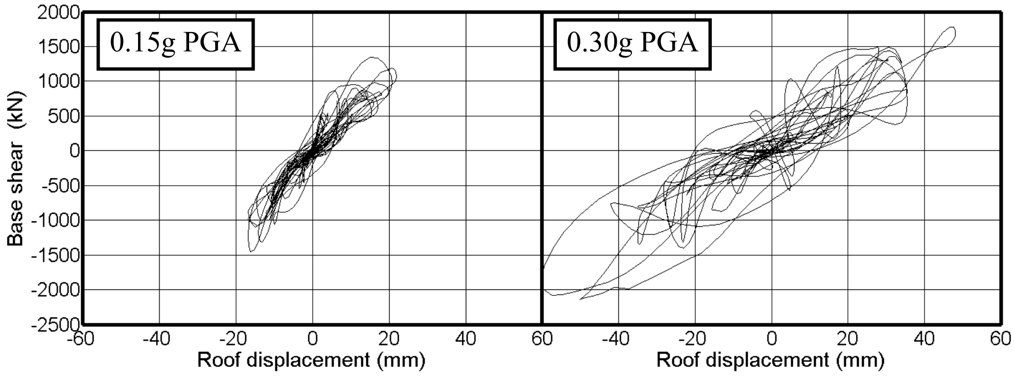


Figure 7. Base shear versus roof displacement response of prototype 1 at the 0.15 and 0.30 PGAs.

4.2. Prototype 2

Prototype 2 (Fig. 5b) was subjected to the 0.15g and 0.30g earthquakes. The time histories of floor displacements and restoring forces measured in these two PsD tests are shown in Fig. 8. The fundamental vibration period of this flexible structural system was T= 1.10 sec for the 0.15g PGA, whereas at the 0.30g PGA, the response curves were characterised by lower frequencies (natural vibration period shifted to T= 1.40 sec). The seismic response of prototype 2 was much influenced by the effect of higher modes. As can be observed in Fig. 8, the floor displacements and restoring forces are out of phase for both earthquake intensities at some moments. In addition, from the shape of the base shear force - top displacement curves, illustrated in Fig. 9, it is evident that the higher modes significantly influenced the values of shear forces, for both 0.15g and 0.30g earthquakes. Moreover, there seems to be no upper limit for the storey forces when the structure enters into the nonlinear regime, an effect which was anticipated in the preliminary nonlinear calculations (Olgiati et al. 2010). This effect, which is a direct consequence of the large higher modes contribution, results into large force demands in the connections (Bournas and Negro 2012).

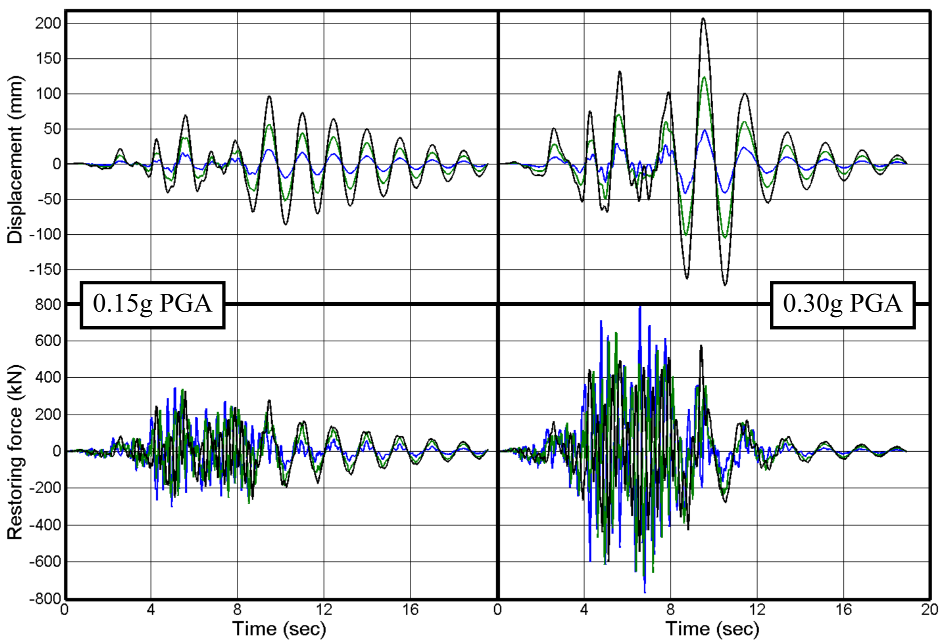


Figure 8. Time histories of floor displacements and restoring forces of prototype 2 at the PsD tests.

At the 0.15g PsD test, corresponding to the frequent (SLS) seismic action, the overall response of prototype 2 remained practically within the elastic range as it is illustrated in Fig. 9a. However, the EC8 damage limitation requirement for buildings, which is simply expressed by an upper limit on the interstorey drift ratio, equal to 1% for the serviceability limit state, was exceeded in the second and third floor. In particular, the maximum interstorey drifts were equal to 0.57%, 1.06%, and 1.18%, at the first, second and third storey, respectively. Figure 9b plots the base shear versus roof displacement hysteretic curves for the 0.30g test. At this higher intensity earthquake corresponding to ULS, the response of this precast system with hinged beam-to-column joints was characterized by excessive deformability. The peak roof displacement and maximum base shear force measured in this test were 208 mm and 895 kN, respectively. Maximum interstorey drift ratios recorded at the first, second and third floor were 1.29%, 2.18% and 2.37%, respectively. Moreover, the st

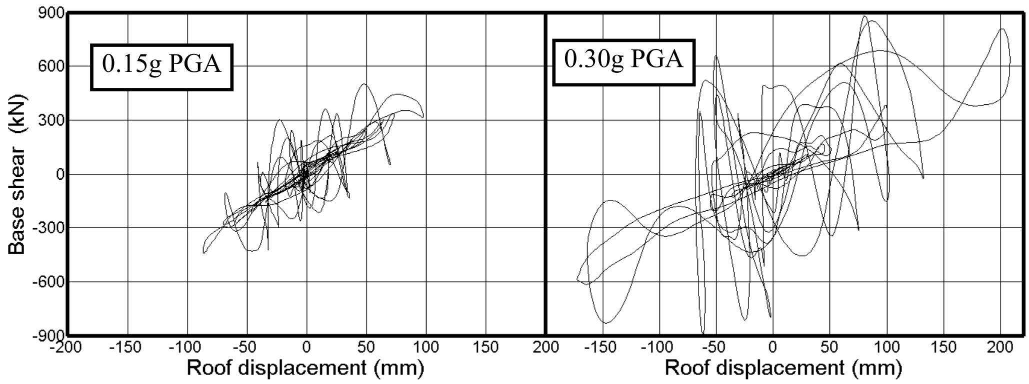


Figure 9. Base shear versus roof displacement response of prototype 2 at the 0.15 and 0.30 PGAs.

4.3. Prototype 3

In the 3rd structural configuration, the mechanical connection system embedded in the beam-column joints was activated to create moment-resisting connections only at the top floor and then prototype 3 (Fig. 5c) was subjected to the earthquake of 0.30g. The time histories of floor displacements and restoring forces measured in this PsD test are shown in Fig. 10a. The fundamental vibration period of the structural system was reduced by 23% in comparison with its counterpart with hinged beam-column connections (T= 1.08 sec, as opposed to T= 1.40 sec in prototype 2). However, it turned out that the concept of emulative beam-column joints at the top floor only was not quite effective as a means of reducing interstorey drifts and the overall displacements of the structure. In fact, as illustrated in Fig. 10a, the floor displacements and restoring forces are still out of phase for the design level earthquake, a fact which, as in the case of prototype 2, corresponds to a seismic response considerably influenced by the effects of higher modes.

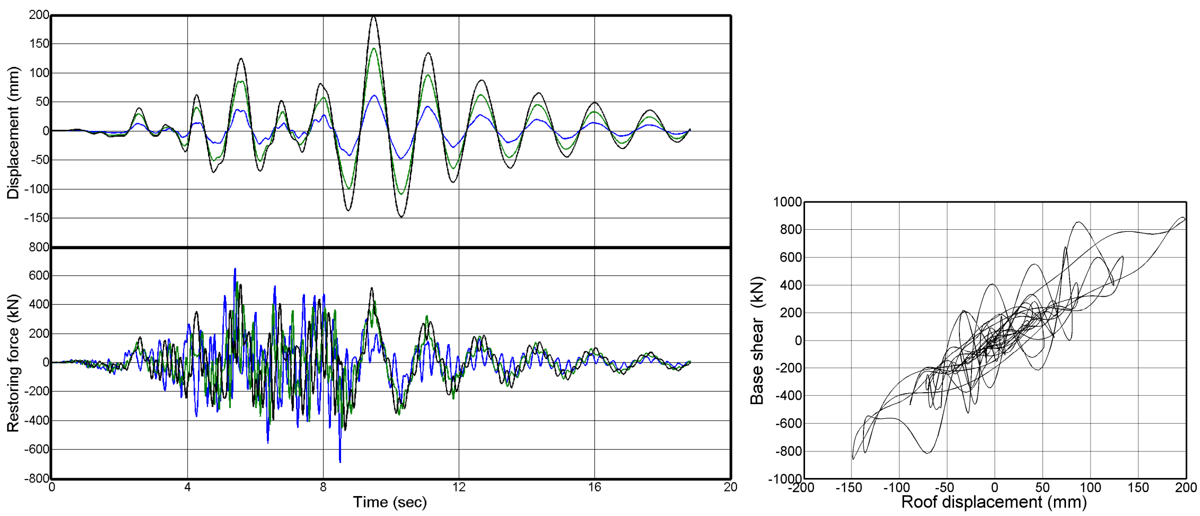


Figure 10. (a) Time histories of floor displacements and restoring forces of prototype 3 at the 0.30g PsD test. (b) Base shear versus roof displacement response of prototype 3 at the 0.30 PGA.

Figure 10b plots the base shear versus roof displacement hysteretic curves for the 0.30g test. The peak roof displacement and maximum base shear force measured in this test were 199 mm and 889 kN, respectively. The maximum interstorey drifts were equal to 1.56%, 2.23%, and 1.50%, at the first, second and third storey, respectively. Comparing the above values to the corresponding maximum interstorey drifts marked in prot2_0.30g, (namely 1.29%, 2.18% and 2.37%), it is evident that by constraining the beam-column joints of the top floor, the problem of large interstorey drifts was moved from the third to the second and first floors.

4.4. Prototype 4

In the fourth and last layout, the mechanical connection system was activated in all beam-column joints with the aim of fully creating emulative moment-resisting frames. Prototype 4 (Fig. 5d) was tested under two input motions scaled to 0.30g and 0.45g. Figure 11 illustrates the time histories of floor displacements and restoring forces measured during these two PsD test. The natural vibration period of Prototype 4 was 0.66 sec, approximately half than the period measured in its counterpart with hinged beam-column joints (prot2_0.30g). This stiffer precast system led as a consequence to higher inertia forces and lower storey drifts. Moreover, its vibration for 0.30g PGA was not affected by the higher modes to the same extent, as was the case in the prototypes with hinged beam-to-column joints. This can be clearly noticed in Fig. 11, where the restoring forces at the three floors are in phase with the applied horizontal floor displacements. At the maximum considered earthquake, namely 0.45g PGA, though, the response curves were characterised by much lower frequencies (natural vibration period shifted to T= 1.25 sec) due to the crack opening of the beam-column joints, caused by the previous 0.30g test.

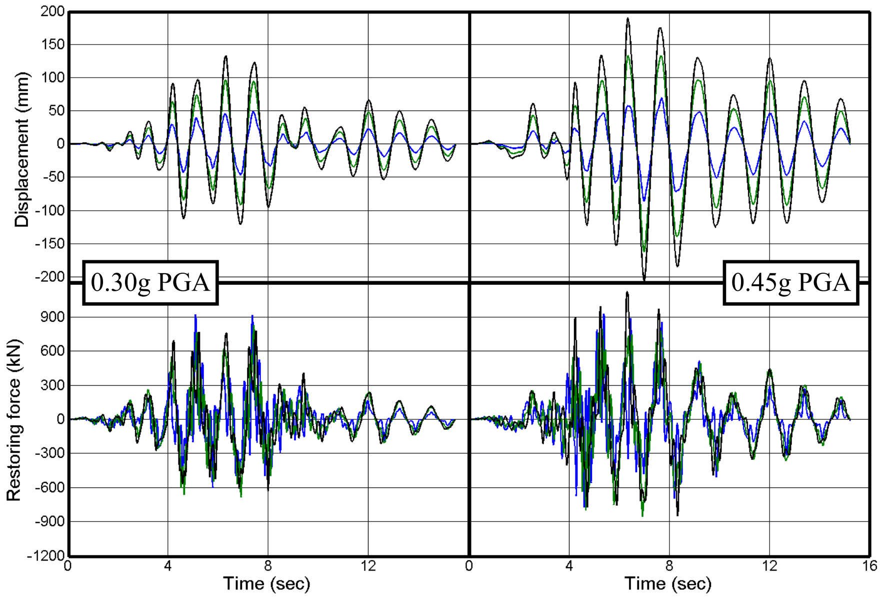


Figure 11. Time histories of floor displacements and restoring forces of prototype 4 at the PsD tests.

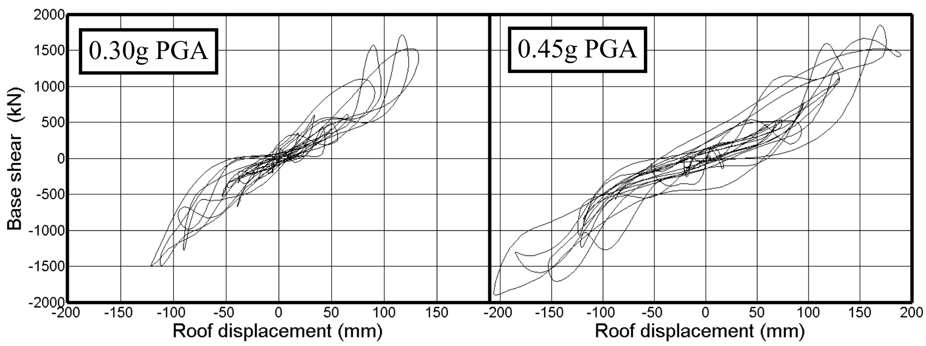


Figure 12. Base shear versus roof displacement response of prototype 4 at PGAs of: 0.30g and 0.45g.

Figure 12 presents the base shear versus roof displacement loops of prototype 4 for both 0.30g and 0.45g earthquakes. In the ULS seismic excitation of 0.30g, the response of prototype 4 underwent reduced non linear effects (Fig. 12a). The 0.30g PsD test deformed the building to a maximum roof displacement of 132.5 mm, while the maximum base shear force was 1454 kN. The maximum interstorey drifts were significantly lower than the corresponding ones in prototypes 2 and 3, namely they were equal to 1.35%, 1.51%, and 1.05%, at the first, second and third storey, respectively. A visual inspection made at the end of the 0.30g at prototype 3 did not reveal any new damages to the specimen. Since prototype 4 survived the design level earthquake (0.30g PGA) without significant damages, it was decided to proceed with the more intense seismic excitation of 0.45g, which might be regarded as representative of the maximum considered earthquake. The peak roof displacement and maximum base shear force measured in this test were 206.5 mm and 1902 kN, respectively. Under the 0.45g excitation, the maximum interstorey drifts were increased to 2.21%, 2.37%, and 1.61%, at the first, second and third storey, respectively. A visual inspection of the structure after the 0.45g PsD test revealed dense flexural cracking at the base of the ground floor columns, but without considerable damage.

6. CONCLUSIONS

A full-scale three-storey precast building was subjected to a series of PsD 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 various parameters like the type of mechanical connections (traditional as well as innovative) and the presence or absence of shear walls along with the framed structure were investigated. The dual wall-frame precast system (prototype 1) was stiff and quite effective in limiting the maximum interstorey drift ratios for both the serviceability and ultimate limit states. The seismic response of prototype 2 was highly influenced by the effect of higher modes. There seems to be no upper limit for the storey forces when the structure enters into the nonlinear regime. This effect, which is a direct consequence of the large higher modes contribution, results into large force demands in the connections. The 1% drift limitation imposed by EC8 for the SLS was exceeded and at the higher intensity 0.30g earthquake corresponding to ULS, the response of this precast system with hinged beam-to-column joints was characterized by excessive deformability. However, no significant permanent damage was observed after this earthquake. After the seismic test results of prototype 3, it turned out that the concept of emulative beam-column joints at the top floor only was not quite effective as a means of reducing interstorey drifts and the overall displacements of the structure. Finally, the PsD test results showed that, when activated at all the floors (prototype 4), the proposed connection system is quite effective as a means of implementing dry precast (quasi) emulative moment-resisting frames especially when all beam-column joints are made rigid.

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