Pseudodynamic Tests and Numerical Simulations on a Full-Scale Prototype of a Multi-Storey Precast Structure

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SUMMARY:
This paper summarizes the results of a numerical investigation carried out on a full-scale prototype of a multi-storey precast structure with different types of connections. This investigation is developed within the European research project SAFECAST, which is the last of a series of co-normative researches that supported the standardisation of precast structures within Eurocode 8. Pseudo-dynamic tests have been performed on different structural schemes, and linear and nonlinear dynamic analyses have been carried out in order to highlight the importance of the higher vibration modes and the accuracy of the response spectrum method in the prediction of design parameters such as displacement and storey forces. For the cases studied it is shown that the response spectrum modal analysis allows a reliable estimation of the storey displacements, whereas it may lead to a significant underestimation of the internal forces. The numerical model used in the investigation is finally calibrated by comparison with the results of the pseudo-dynamic tests by taking into account the influence of connections on the overall seismic behaviour.

Keywords: Multi-storey precast buildings, Pseudo-dynamic tests, Connections, Modal analysis

1. INTRODUCTION

SAFECAST project (European Programme FP7-SME-2007-2) is the last of a series of co-normative researches made to support European standardisation for what concerns the seismic design of precast structures, which aim is to investigate seismic behaviour of the connections of this kind of structural systems (Biondini and Toniolo, 2011). The experimental qualification of these connections has been based on the main parameters of seismic response, namely strength, ductility, dissipation, deformation, decay and damage. A large number of cyclic and dynamic tests on sub-assemblies of structural elements connected at their joints has been performed at the laboratories of Lisbon, Milan, Ljubljana, Athens and Istanbul. The present building codes, such as Eurocode 8 (CEN-EN 1998-1, 2004), give the same importance to different systems of connection, both hinged and emulative. This statement is justified after the results of past experimental and numerical investigations, carried out within the European research projects ECOLEADER and GROWTH (Biondini and Toniolo, 2009). These studies have been focused on the evaluation of the global ductility related to single-storey precast industrial buildings with hinged connections, comparing it with the ductility related to cast-in-place structures.

The most relevant series of tests has been performed at the European Laboratory for Structural Assessment (ELSA) of the Joint Research Centre of the European Commission at Ispra, Italy (Donea et al., 1996). The prototype represents a typical precast building. For the design of the structure and the sequence of pseudo-dynamic tests preliminary numerical analyses have been performed to estimate storey displacements and forces. The main results of these numerical analyses are presented in this paper to show that, even if storey displacements are comparable, storey forces evaluated through the dynamic modal analysis with response spectrum could be significantly lower than those predicted by nonlinear dynamic analysis. In this way internal forces tend to be underestimated, leading to a not suitable design of structural members. This indicates that the dynamic modal analysis with response spectrum, that is based on a linear elastic analysis where the attenuation effects of the energy dissipation is taken into account through a force-reducing behaviour factor q, is a conventional method that is not able to predict in a reliable way the actual seismic behaviour of the structure. This approach is however generally safe when the structural elements have adequate ductility.

The numerical model used in the investigation is finally validated by comparison with the experimental results of the pseudo-dynamic tests. Since the connections play an important role in the global response, an attempt has been made to propose a modelling of their nonlinear behaviour. The results of the nonlinear time history analyses coming from the improved model have been therefore compared with the experimental results to prove the effectiveness of the proposed modelling.

2. DESCRIPTION OF THE PROTOTYPE

The aim of the large size experimentation within the SAFECAST project is to provide proper reliable evidences about the seismic behaviour of a common type of precast multi-storey buildings widely used for commercial and industrial purposes (Biondini et al., 2010). In particular, the role of the beam-to-column connections, hinged or moment-resisting, has to be investigated with respect to the inter-storey drift control as regulated by the code requirements. This also involves the verification of the accuracy of the ordinary methods of analysis. The choice of the prototype has been addressed to a three-storey building with a number of spans and bays sufficient to represent the behaviour of this type of buildings, the dimensions in plan being the maximum compatible with the capacity of the testing plant of ELSA laboratory. On a mesh of 7.5 by 7.5 m, a 2 by 2 spans/bays had been planned (14.0 by 14.0 m at the column axis), filling up the dimensions of the anchoring base slab of the testing plant.

A zone of medium-high seismicity with peak ground acceleration PGA=0.25g has been considered for design of the prototype. This leads, with a soil factor S=1.2 (ground type B), to a design ground acceleration a_g=0.30g. Ordinary live loads for handicraft activities have been assumed in the calculations for the proportioning of the structure, following first a simplified static linear analysis and a subsequent verification with a dynamic modal analysis with response spectrum, assuming a behaviour factor q=3. This latter analysis pointed out the relevant effects of the higher vibration modes on this type of flexible structure, but did not change the overall proportioning made with reference to the internal forces obtained at the base of the structure through the static analysis. In particular, the floor forces obtained from the dynamic modal analysis were lower than the maximum capacities of the testing equipment (2 jacks of 1000 kN at each floor).

The prototype has 2 by 2 bays as shown in Fig 1. The length of each bay is equal to 7 m. The structure presents three stories, with heights equal respectively to 3.5, 3.2 and 3.2 m. Columns have the same cross-section along the height of the structure, with a size of 50x50 cm, and longitudinal reinforcement equal to 8Φ20. Along the main direction there are hollow core beams, with a maximum width of 2.25 m and a minimum one equal to 1.85 m. The two ribs are 30 cm thick. In the orthogonal direction, prestressed floor elements are used. The first storey has box elements, connected by welded steel angles. In the second storey there are TT elements, connected in the same way of first floor elements. Finally, the third storey has spaced box elements. Materials used are concrete C45/55 and steel B450C, with characteristic values respectively equal to f_{ck} = 45 MPa and f_{yk} = 450 MPa. Fig. 2 shows a view of the prototype under construction.

The same prototype has been analyzed through different structural systems, Fig. 3, changing the type of beam to column connections. In particular, four schemes have been considered:

- Model 1: Three storey frame with shear walls;
- Model 2: Three storey frame with hinged connections at all stories;
- Model 3: Three storey frame with moment-resisting connections on the top floor and hinged connections at the first and second story;
- Model 4: Three storey frame with moment-resisting connections at all stories.

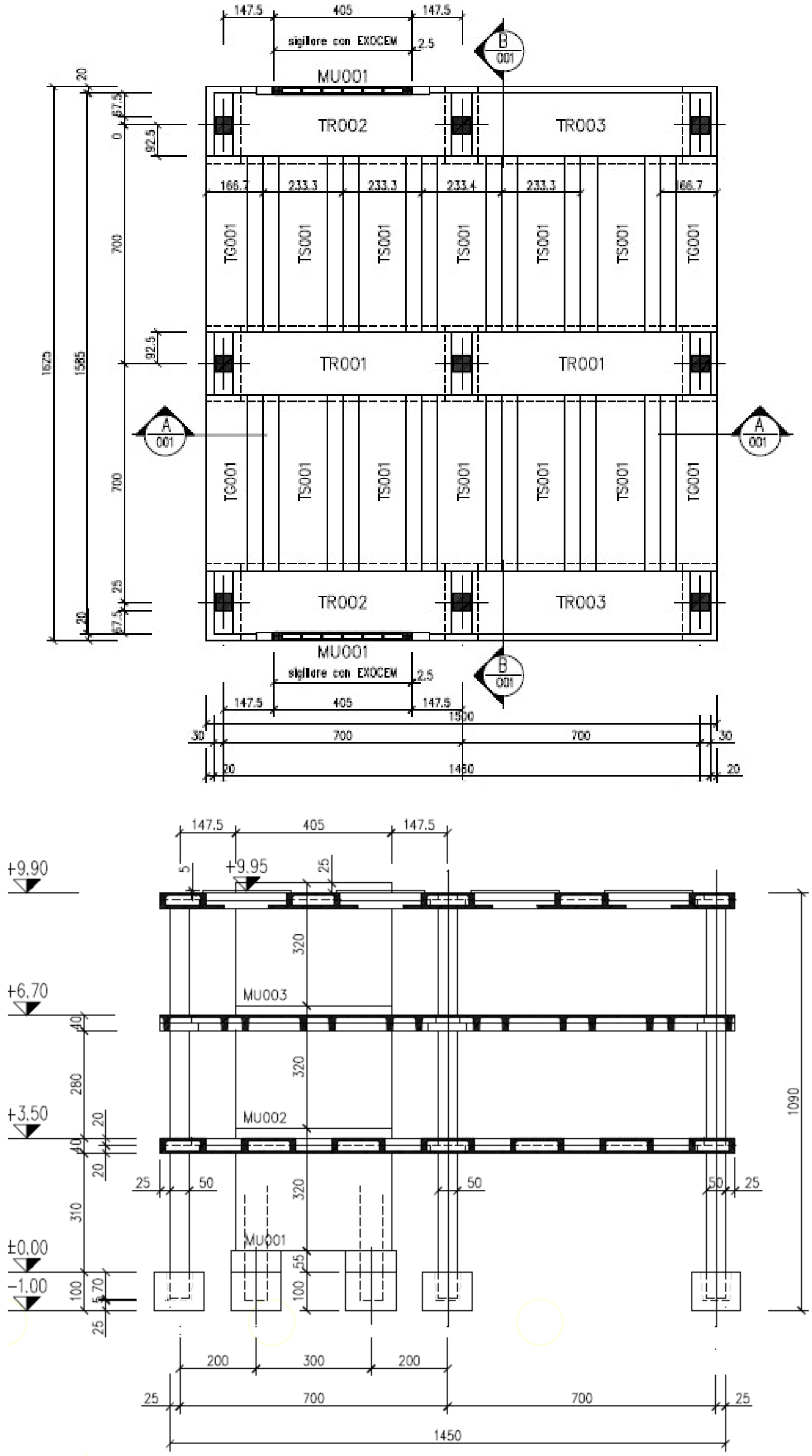

Figure 1. Plan and section views of the prototype.

Figure 2. The structural prototype under construction.

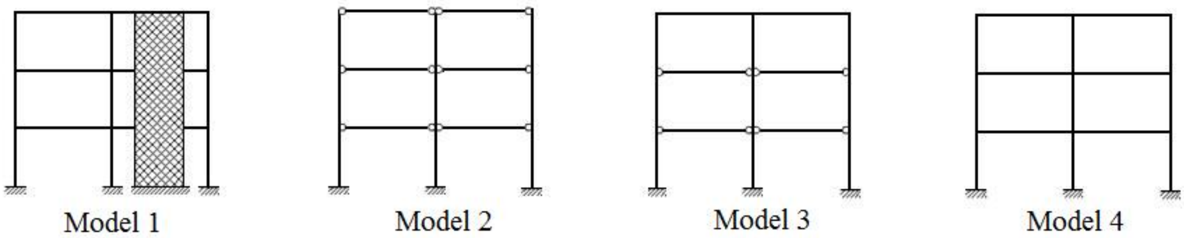

Figure 3. Schemes of the structural prototype.

3. ACCURACY OF THE RESPONSE SPECTRUM MODAL ANALYSIS

Preliminary nonlinear dynamic analyses have been performed before starting the experimental campaign, in order to verify and to check the accuracy of the results obtained through the response spectrum modal analysis, as well as to have an information in advance on the global seismic behaviour of the prototype considering different connections, with particular reference to frame systems (indicated as Model 2, Model 3 and Model 4 in Fig. 3). Seismic input is represented by the East-West component of the modified Tolmezzo accelerogram, Fig. 4a, with a total duration of 12 s. The corresponding spectrum matches very well the EC8 response spectrum for a soil class B, Fig. 4b. The selection of the accelerogram is based on an intensity index that measures the capacity of the earthquake to induce structural damage in the range of intermediate periods (Fajfar et al., 1990). The maximum amplitude of the accelerogram is scaled to 1.2x0.25=0.3g.

A concentrated plasticity model has been adopted, where nonlinearity is concentrated at both ends of columns and beams. In particular, plastic hinges are modelled according to the Takeda hysteretic law shown in Fig. 5, where parameters α e β are set respectively to 0.5 and 0.0 for columns and 0.0 and 0.6 for beams (Lu and Silva, 2006). The model proposed by Mander et al. (1988) is used for concrete to take into account the increase in strength and ductility due to confinement in the columns. This effect is however considered negligible for the beams. For steel, an elasto-plastic behaviour with a hardening branch is adopted. The envelope of the hysteretic loops is calibrated through the moment-curvature diagrams of the critical sections, considering for each column the axial load coming from the static analysis. A bilinear approximation is therefore assumed, computing the yield strength in a way that the area under the curves is equal. The analyses have been performed considering second-order effects and neglecting viscous damping, supposing that all the dissipation is included in the hysteretic behaviour, while modal analysis is based on a 5% damped response spectrum.

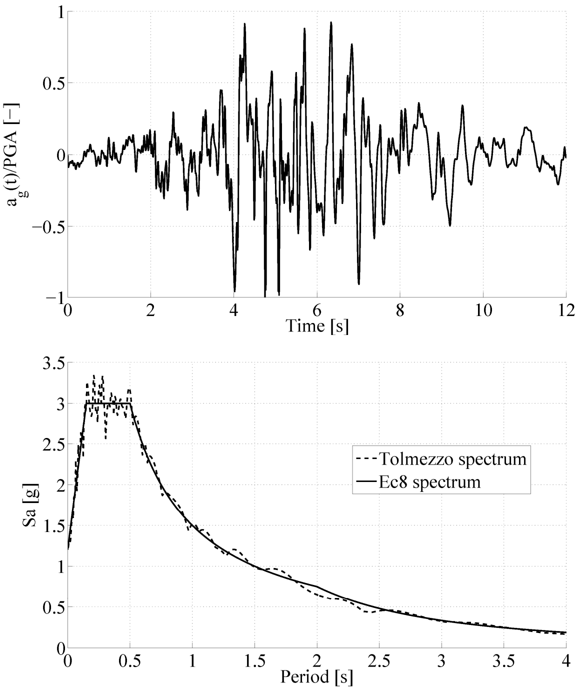

Figure 4. (a) Modified Tolmezzo accelerogram and (b) response spectrum compared with EC8 spectrum for soil type B.

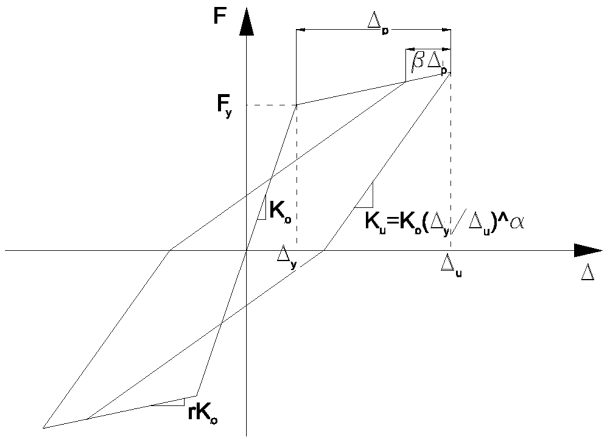

Figure 5. Takeda hysteretic law adopted to simulate the nonlinear behavior of structural members.

Table 1 shows the comparison between storey forces and displacements, for the three frame systems previously indicated (Model 2, Model 3 and Model 4), obtained by using response spectrum modal analysis (in the following indicated as MOD) and nonlinear time history analysis (NLI). The modal contributions are combined by means of the rule SRSS (square root of the sum of the squares), since the periods of adjacent modes are sufficiently different.

Figs. 6, 7 and 8 show maximum displacements and storey forces for each model. For Model 2 and 3 the displacements resulting from modal analysis are slightly greater than those coming from nonlinear analysis, except for the first floor. Model 4 shows an opposite trend, but the difference is limited, not exceeding 2 cm. Moment resisting connections allow to reduce the displacements, with values at the top floor decreasing from 17 to 6 cm.
These results are however indicative, because in real design the columns of the hinged solution would have a section increased. The good agreement found in terms of displacements is lost when storey forces are considered. These differences are therefore reflected in the estimation of the maximum bending moment at the base of the columns (Biondini et al., 2011).

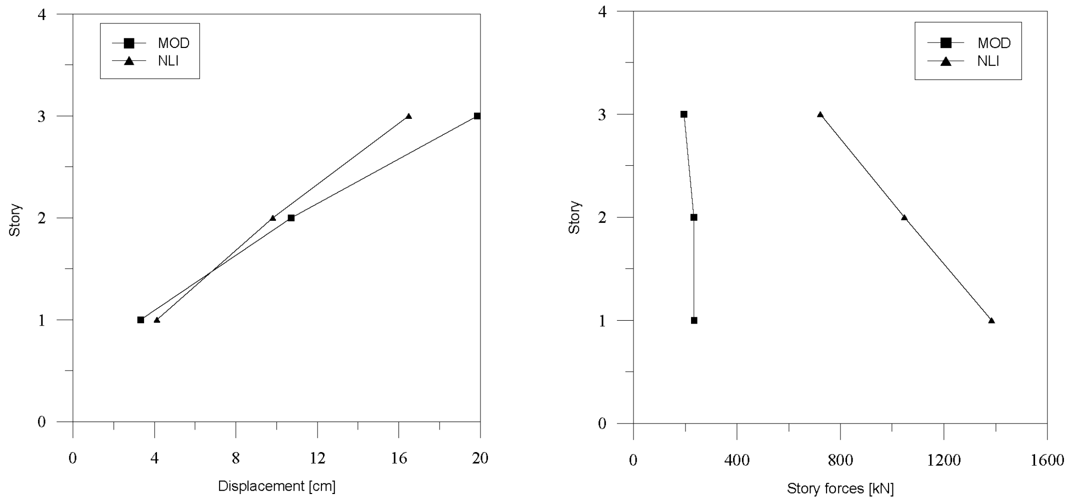

Figure 6. Comparison of the results of modal and nonlinear dynamic analyses for Model 2: (a) displacements, and (b) storey forces.

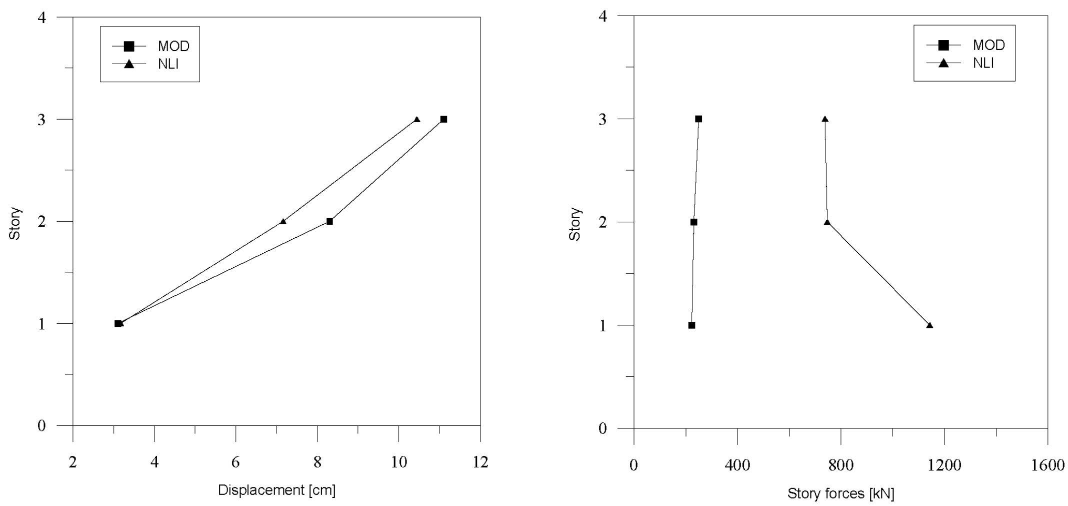

Figure 7. Comparison of the results of modal and nonlinear dynamic analyses for Model 3: (a) displacements, and (b) storey forces.

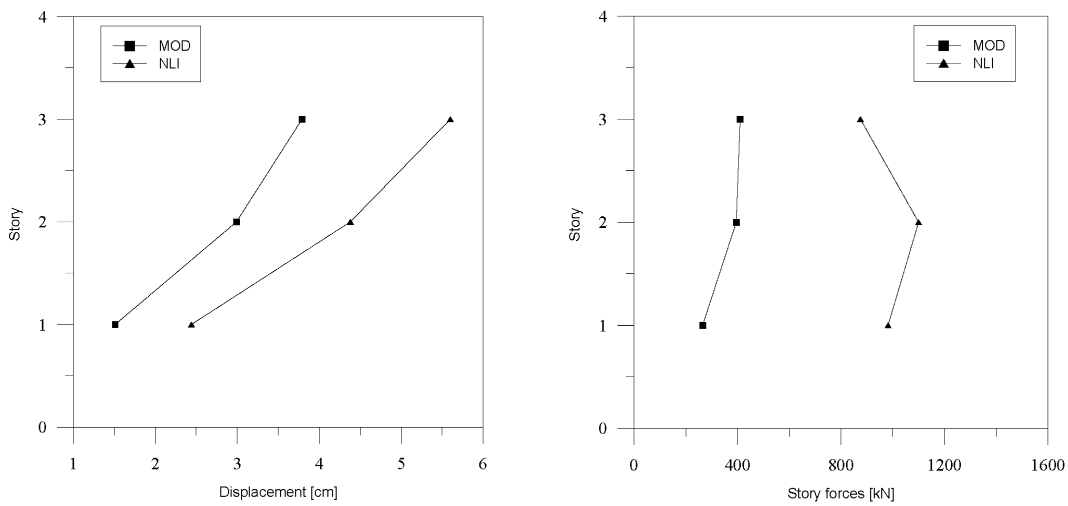

Figure 8. Comparison of the results of modal and nonlinear dynamic analyses for Model 4: (a) displacements, and (b) storey forces.

Table 1. Storey forces and displacements evaluated through modal analysis and nonlinear time history analysis for frame systems studied within SAFECAST project

		Displacement [cm]		Forces [kN]	
	Storey	MOD	NLI	MOD	NLI
Model 2	III	19.86	16.48	195	722
	II	10.72	9.81	233	1047
	I	3.32	4.12	234	1384
Model 3	III	11.10	10.44	250	738
	II	8.34	7.16	231	747
	I	3.14	3.17	223	1143
Model 4	III	3.79	5.60	410	875
	II	2.99	4.38	395	1100
	I	1.51	2.44	265	982

4. NUMERICAL SIMULATION OF THE PSEUDO-DYNAMIC TESTS

The prototype has been subjected to pseudo-dynamic tests at the ELSA Laboratory (Donea et al., 1996). A series of tests has been scheduled on the multi-storey prototype shown in Fig. 2, changing subsequently the arrangement of the connection system. First, a sequence of tests on the dual wall-frame system with the structure connected to the two lateral bracing walls has been performed. Then the bracing walls have been uncoupled and the structure reduced to a pure frame system. A second sequence of tests has been performed on this frame system with all hinged beam-to-column connections. A third sequence of tests has been carried out after restraining the top floor joints turned into moment-resisting connections. The final sequence of tests has been performed with the joints of all the floors turned into moment-resisting connections.

A key aspect within SAFECAST project was the numerical simulation of the series of pseudo-dynamic tests, in order to better understand the seismic behaviour of multi-storey precast structures. To this aim, a fibre modelling of structural members has been adopted, since such approach usually gives more accurate results. However, other formulation can successfully be used to capture the seismic behaviour at collapse.(Ibarra et al., 2005, Fischinger et al., 2008, Liel et al., 2009, Titi, 2012) if collapse simulation is required.

One of the most important features emerged is the significant influence of higher modes, especially for frame system with hinged connections (Model 2 in Fig. 3). Such characteristic is clearly highlighted in Fig. 9, which shows the comparison between experimental and numerical results in terms of shear forces and drift at the first storey of the frame. Moreover, even if numerical model matches very well experimental data up to 8 seconds, a time shift subsequently occurred, but amplitudes are almost the same, Fig. 10. This can be probably explained through the actual behaviour of connections, which are not perfect hinges but allow to transfer a little amount of moment. In particular, until they do not reach a certain deformation, the behaviour is similar to a perfect hinge, so moment transferred is very low. Subsequently, a nonlinear behaviour starts and moment increases, affecting the global response of the structure.

4.1. Numerical modelling improvement of the hinged frame

Since the hypothesis of perfect connections is not well appropriate for the frame studied, an attempt has been done in order to derive, on the base of experimental data, a preliminary nonlinear model for the connections. In particular, the hysteretic material included in OpenSees library (Mazzoni et al. 2005) has been applied. A first branch characterized by a small stiffness is used, since at the beginning of the simulation the connection behaves approximately as a perfect hinge. However, when a certain rotation is reached, moment can be transferred through the connection, and the stiffness of the connection increases. The results of the improved model can be appreciated from Fig. 11, where the comparison between experimental test and numerical simulation is shown for all the three stories. In this case, the time shift emerged in Fig. 10 (model with perfect hinges) disappears.

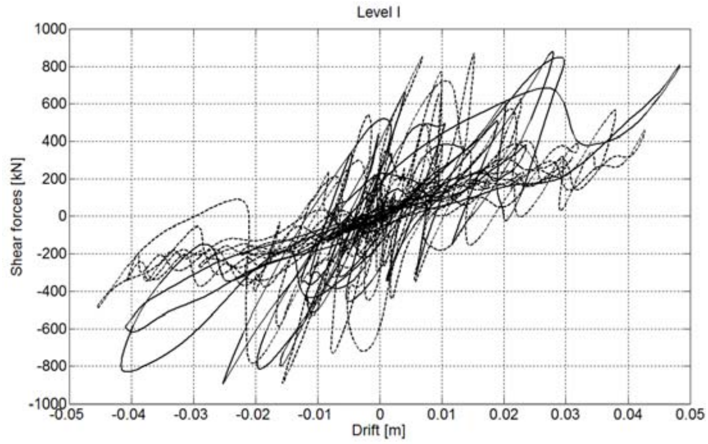

Figure 9. Comparison between numerical simulation and experimental results in terms of shear force versus drift diagrams for Model 2, PGA=0.30g.

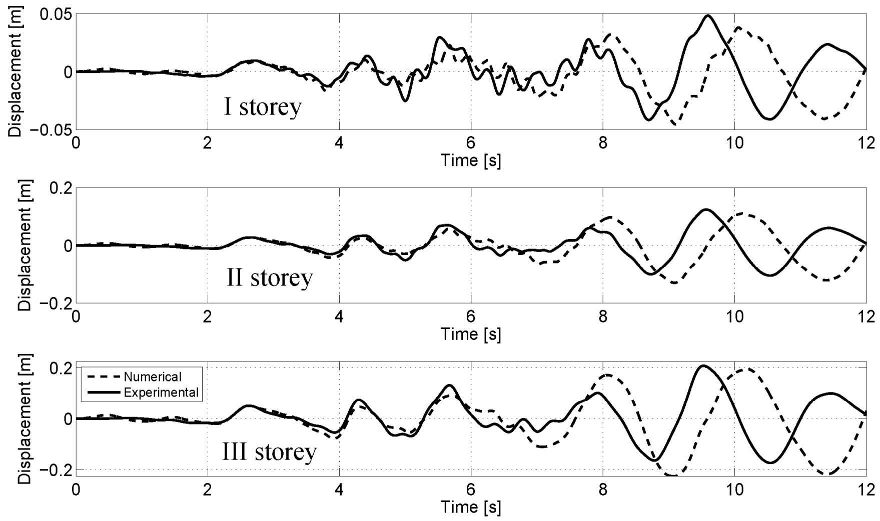

Figure 10. Time history of the storey displacements for Model 2 with perfect hinges (PGA=0.30g): comparison between numerical and experimental results.

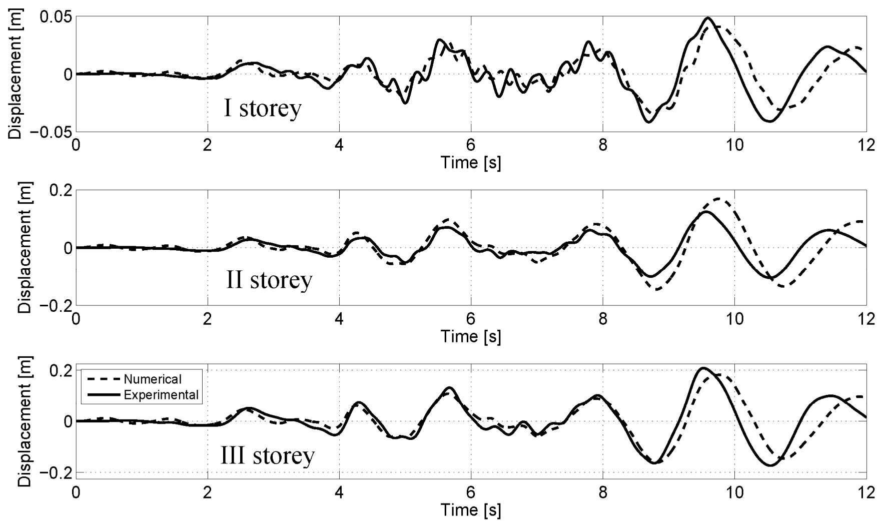

Figure 11. Time history of the storey displacements for Model 2 with nonlinear connections (PGA=0.30g): comparison between numerical and experimental results.

5. CONCLUSIONS

The results of a numerical investigation carried out on a full-scale prototype of a multi-storey precast structure with different types of connections have been presented. Numerical nonlinear time history dynamic analyses have been performed in order to check the accuracy of the design process based on linear analysis methods. The results highlighted that the response spectrum modal analysis allows a reliable estimation of the storey displacements, but it may lead to a significant underestimation of the floor forces and, consequently, of the internal stress resultants with respect to the results provided by more accurate methods of nonlinear analysis. This is mainly due to the role of the higher vibration modes, since the opposing vibrations of the floors bring to higher storey forces, modifying their distribution along the height of the structure with respect to the force distribution assumed in the linear analysis methods. The numerical model used in this investigation has been finally validated by comparison with the results of pseudo-dynamic tests on the full-scale multi-storey prototype. Since the connections play a crucial role in the global seismic response, an attempt has been also made to propose a modelling of their nonlinear behaviour. The good accuracy of the numerical results obtained from nonlinear time history dynamic analyses based on the improved model demonstrated the effectiveness of the proposed approach. This highlights the importance of a proper modelling of the connections for a reliable assessment of the seismic performance of multi-storey frame precast structures.

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