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Numerical simulation of the seismic behavior of passively controlled precast concrete buildings

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Summary

The poor performance of some precast structures have limited their use in seismic zones due to their low level of structural damping, P-Δ effects and low ductility of de structural joints. These characteristics allow proposing the use of passive dissipating devices for improving their behavior. The seismic response of two precast buildings is studied in this work. The response of the structures equipped with energy dissipators is compared with the non-controlled case. The first structure is a low damped industrial precast concrete building with low ductility connecting joints. The second one is a 3D frame typically built in urban areas. The structures are simulated using the Simo's formulation for beams. Each beam section is meshed in a secondary grid of fibers along the beam axis. The materials of each fiber can be composed of several components having appropriated constitutive laws. The simple mixing theory is used to treat the resulting composite. A special kind of element is developed for modeling the dissipating devices. The results obtained in this work allow validating the use of passive control for improving the seismic performance of precast structures.

KEYWORDS: seismic analysis, beam model, numerical methods.

1. INTRODUCTION

The use of precast concrete structures in seismic areas have been frequently limited, by the lack of confidence about their performance in seismic regions as well as by the absence of seismic design provisions or specific codes for analysis and design of critical zones in the structure, i.e. the connecting joints. Due to these reasons, the recognized advantages of precast concrete construction over cast-in-place methods, which commonly are mainly referred only to construction aspects (quality control, velocity of erection), have captured the most of the attention by the researchers, while its structural efficiency is overlooked. The poor performance of several precast parking structures in the 1994 Northridge Earthquake due to

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incorrect design detailing has probably increased the lack of confidence on such structural systems, contributing to further restrictions on precast usage in seismic zones. In the last years some alternative concepts for the analysis and design of precast concrete structures in seismic zones have been investigated as opposed to the frequently accepted criteria of "emulation of cast-in-place concrete" [Pampanin, 2003].

Among the most frequently noted disadvantages associated with the traditional precast concrete structures [Mata, Barbat et.al. 2004] are:

- Low *global structural damping coefficient*. On the one hand, the 5% of the global structural damping frequently associated to conventional cast in place concrete structures is not necessarily the most appropriated value for precast ones, which could have considerably smaller values, of the order of 2%, with the consequent amplifications in the dynamic response.
- Important *PΔ effects* for the case of some flexible structures. The precast industry tends to generate more flexible elements mainly designed for permanent death load and, therefore, PΔ effects could be increased for lateral loading paths.
- *Non ductile connecting joints*. The conventional seismic design is not directly applicable to the case of precast concrete structures, because the connecting joints are not monolithic. Furthermore, the joints are points where the ductility demand is important and, therefore, they are critical points of the structure where it is expected that damage concentrate. Additionally, a zone where damage is concentrated presents softens mechanical behavior and it can be identified with a plastic hinge in the structure. As it is well known, only a limited number of plastic hinges can be developed in the structure before a structural behavior corresponding to a mechanism is obtained.

By other hand, the effectiveness of passive control techniques is well recognized for reducing the dynamic response of structures subjected to seismic actions. It is possible to improve the seismic behavior of precast concrete structures by using energy dissipation devices to absorb a part of the energy induced by earthquakes and to concentrate the damage in specific zones.

One choice to perform realistic analysis of structures equipped with energy dissipating devices for seismic loading is by means of employing material and geometrical nonlinear time history analysis assuming appropriated constitutive descriptions for the materials and applying acceleration records to the base of the structure. The numerical model should be able to simulate the changes of configuration of the structure during the earthquake, especially for the case of flexible structures.

In this work the latter is achieved by mean of employing the Simo-Vu Quoc formulation for beams, which is capable of undergoing large strains and

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displacements [Simo et.al. 1985]. Each beam section is meshed into a grid of fibers directed along the beam axis. Two kinds of materials are employed: concrete and steel. For describing the mechanical behavior of concrete, a local damage constitutive model based on Kachanov´s theory is used. Reinforcing steel bars are treated using a fiber plastic model. The material associated with a fiber is treated by means the simple mixing theory [Oller et.al. 1997]. The incorporation of energy dissipating devices is obtained developing a special rod element.

In this paper the numerical simulation of the dynamic response of two typical precast concrete building subjected to earthquake loads, using passive energy dissipating devices is carried out. Both structures present the deficiencies previously described. The first one corresponds to a plane frame employed to build industrial buildings. The second one is a three-dimensional concrete frame corresponding a building constructed in urban areas. Nonlinear behavior for both frame structures and energy dissipating devices are considered in the computational simulations.

2. NUMERICAL TOOL

A specific software package, *PLCDYN Plastic Concrete Dynamic*, has been developed to simulate the nonlinear behavior of civil engineering structures including those based on beam elements. The developed code allows solving problems in many different areas of the mechanic of solids: static, dynamic, with material and geometric non-linearity, thermally coupled problems and composite based structures [Car et.al. 2000].

For the case of beam like structures the geometrically exact formulation due to Simo–Vu Quoc is implemented. The kinematical assumptions of the model allow simulating finite strains and large displacements and rotations during the dynamic action [Ibrahimbegovic, 1995; Simo et.al. 1985, 1986, 1988; Mata et.al. 2005]. For dynamic analysis a Newmark scheme, which update consistently all the dynamic variables associated with finite rotations, has been implemented [Simo et.al. 1989]. Each beam section of the elements is meshed into a secondary grid of quadrilaterals for including a non-homogeneous distribution of materials. Each quadrilateral corresponds to a fiber oriented along the centroid axis of the beam. See Figure 1. The material of each fiber is composed by several components, having each of them its own constitutive law. By this way, it is possible to consider the steel reinforcement as one of the components of the composite located in the quadrilaterals on the boundary of the section. The resulting composite is treated according to the *simple mixing theory* [Car et.al. 2000], which impose the same strain field for all components in a material point. The stress field is recovered for each component according to its constitutive law, the total stress is determined supposing that each component contributes to the total stress according to its volumetric participation in the mixture.

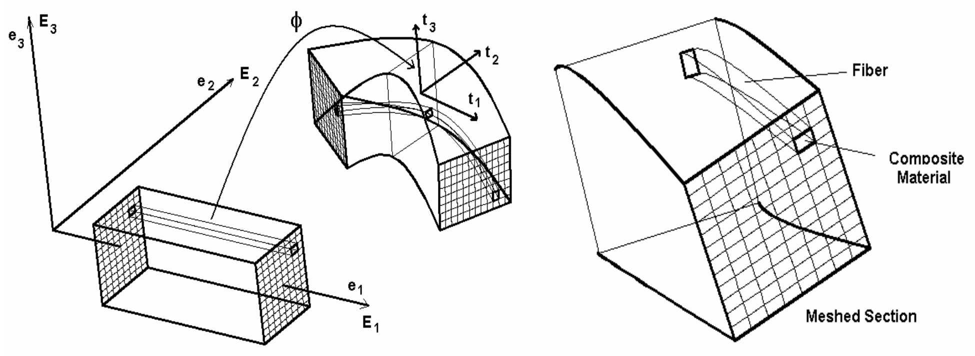

Figure 1. Kinematics of beams and grid of quadrilaterals in each section

The sectional forces and moments are obtained integrating the stresses over the whole section in each integration point. This kind of approximation avoids the development or employment of constitutive laws based on force–displacement for the element, which is the most common way to model the nonlinear beam behavior, but this kind of laws are valid only for a certain geometry of sections or mechanical behaviors of the beams [Barbat et.al. 1997]. Sectional forces and moments are then used to check global equilibrium of the dynamical system. The iterative process is repeated until convergence is obtained.

2.1. Constitutive laws for materials

The failure of concrete for different strain or stress conditions is simulated employing an *isotropic damage model* based on fundamental thermo dynamical principles [Barbat et.al. 1997]. This model is able to simulate in a simple and efficient fashion one of the basic features of the concrete behavior: *degradation*, strain softening under tension–compression stress states. Figure 2 shows the shape of the damage criterion in the principal stress space employed for the concrete. In this figure it is possible to see that the model takes into account different properties for tension or compression states. The resulting integration algorithm for this model is simple and suitable for large-scale computations. In this category, nonlinear behavior is monitored through a single internal scalar variable, called *damage* or *degradation* [Hanganu et.al. 2002].

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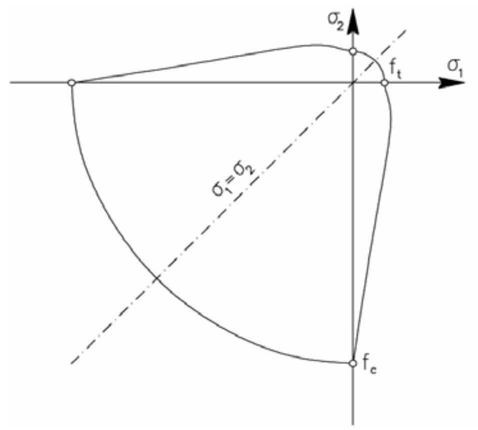

Figure 2. Kinematics of beams and grid of quadrilaterals in each section

Steel bar reinforcements and stirrups are modeled by mean of the *plastic fiber behavior* model. The model consists in an orthotropic material with a steel elastic modulus in the direction of the reinforcement and concrete properties in the other two directions. Plastic flow is oriented along the fiber once the yielding criterion is reached [Car et.al. 2000].

2.2. Strain localization

Strain localization is expected to occur when large incursion in the nonlinear range are attained by the structure [Hanganu et.al. 2002]. The objectivity of the response is obtained by means of carrying out a regularization the dissipated energy in each integration point considering the characteristic length of the finite element where the is strain localization have place. In this way the maximum dissipated energy by a material is limited by its fracture energy. This corrective procedure became the global structural response objective but the length of the zone where strains are localized is still mesh dependent.

2.3. Dissipating device element

The energy dissipating devices are modeled by mean of a bi–pinned rod element with only one integration point in the middle of the rod span. The bi–pinned condition of the ending nodes allows obtaining displacements in the direction of the axes element and, therefore, only axial strains have place. Specific one-dimensional constitutive laws have to be provided for the element. In this work, only devices with plasticity as constitutive law will be employed but the developed element can be employed with any other kind of constitutive relation, e.g. [Mata et.al. 2006].

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3. NUMERICAL SIMULATIONS

3.1. Precast concrete industrial frame

The nonlinear seismic response of a typical plane precast industrial building, Figure 3, is studied. The building has a bay width of 24 m and 12 m of inter–axes length. The story high is 12 m. The concrete of the structure is H–35, (35 Mpa, ultimate compression), with an elastic modulus of 290.000 Mpa. It has been assumed a Poison coefficient of 0.2. The steel reinforcement of the sections considered in the study corresponds to the 10% of the sectional area and the quadrilateral discretization of the sections is presented in the same figure for each element. The ultimate tensile stress for the steel is 510 Mpa. The dimensions of the columns are 60x60 cm^2. The beam has a variable section with an initial high of 60 cm on the supports and 160 cm in the middle of the span.

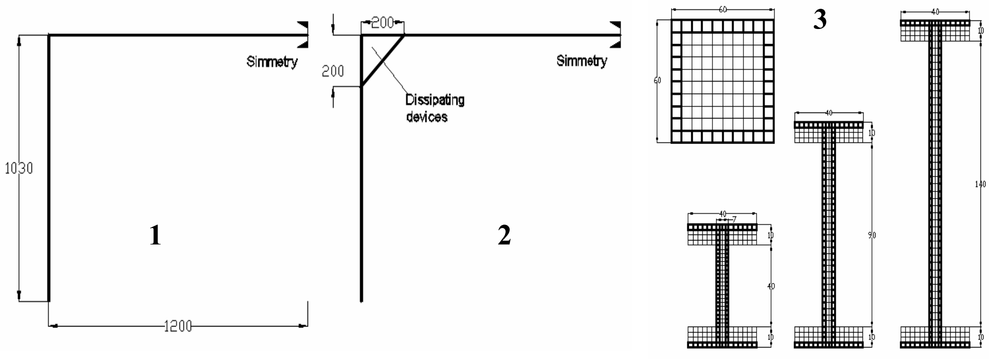

Figure 3. Half part of 2D precast industrial frame. 1: Normal frame 2: Energy dissipating devices incorporated (diagonal elements). 3: Numerical model of Column and beam sections

The permanent loads considered are 1050 N/m^2 and the weight of upper half of the closing walls (432,000 N). The employed acceleration record is the N–S component of the El Centro earthquake, 1940.

The energy dissipating devices were simulated by means of employing the previously described model to obtain only axial force in each element. The properties of the dissipating devices were designed for yielding with an axial force of 150.000 N and for a relative displacement between the two ending nodes of 1.5 mm. The length of the devices was of 2,00 m.

A nonlinear static analysis has been performed on the structure with and with out energy dissipating devices. A sequence of imposed displacements with sinusoidal

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form is applied on the upper corner of the structure. The results are summarized in figure 4 where base shear is draw as function of the top displacement. In this figure it is possible to see that the controlled structure increments its stiffness and resistance when compared with the non-controlled case. Additionally, more energy dissipating capacity is obtained for the controlled structure as it can be evidenced from the greater hysteretic cycles obtained.

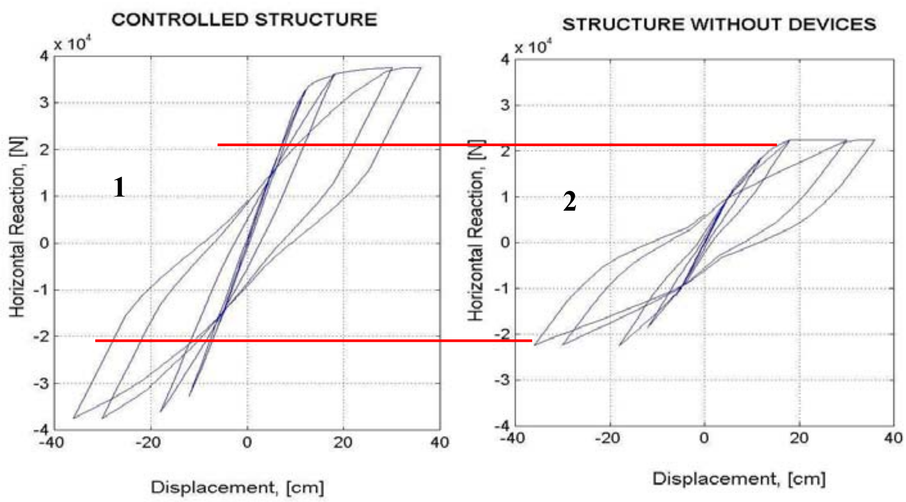

Figure 4. Hysteretic cycles: 1. Structure; 2. Energy dissipating devices

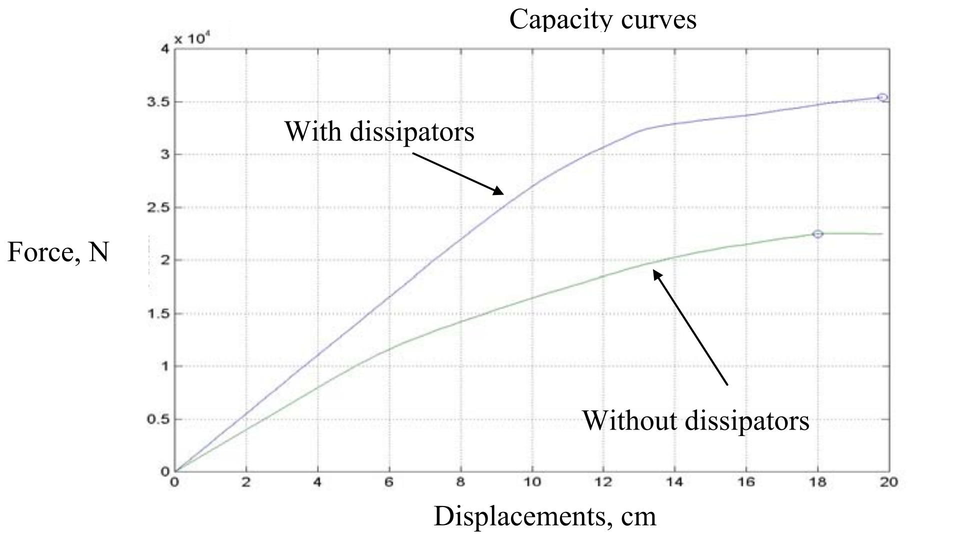

Figure 5. Capacity curves

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By other hand, ductility of the structure is increased when control devices are employed as it can be seen in Figure 5.

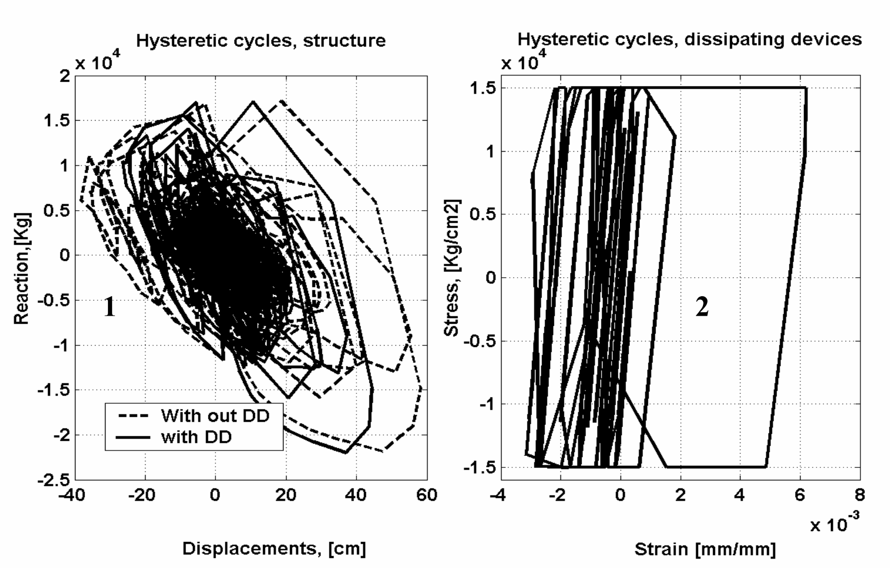

Figure 6. Hysteretic cycles: 1. Structure; 2. Energy dissipating devices

The results of the numerical simulations allow seeing that the employment of plastic dissipating devices contributes to improve the seismic behavior of the structure for the case of the employed seismic record. Figure 6 shows the hysteretic cycles obtained for the structure with and with out devices. For the case of the whole structure the cycles are obtained from the lateral displacement of the upper beam–column joint and the horizontal reaction (base shear) in the columns. It is possible to appreciate that the non-controlled structure (bare frame) presents greater lateral displacements and a more structural damage is observed, (greater hysteretic area than for the controlled case). In the case of the structure equipped with dissipator a stiffer response is obtained and a part of the dissipated energy is concentrated in the controlling devices as expected.

Figure 7 shows the time history responses of the upper beam–column joint. A reduction of 12 % is obtained for the maximum lateral displacement compared with the bare frame. Acceleration and velocity are controlled in the same way, but only 4 and 5 % percent of reduction is obtained. A possible explanation for the low effectiveness of the dissipators is that the devices only contribute to increase the ductility of the beam – column joint with out alleviating the base shear demand on the columns due to their dimensions and location in the structure. By other hand, joints are critical points in precast structures and therefore, the employment of dissipators combined with a careful design of the columns can help to improve the seismic their behavior.

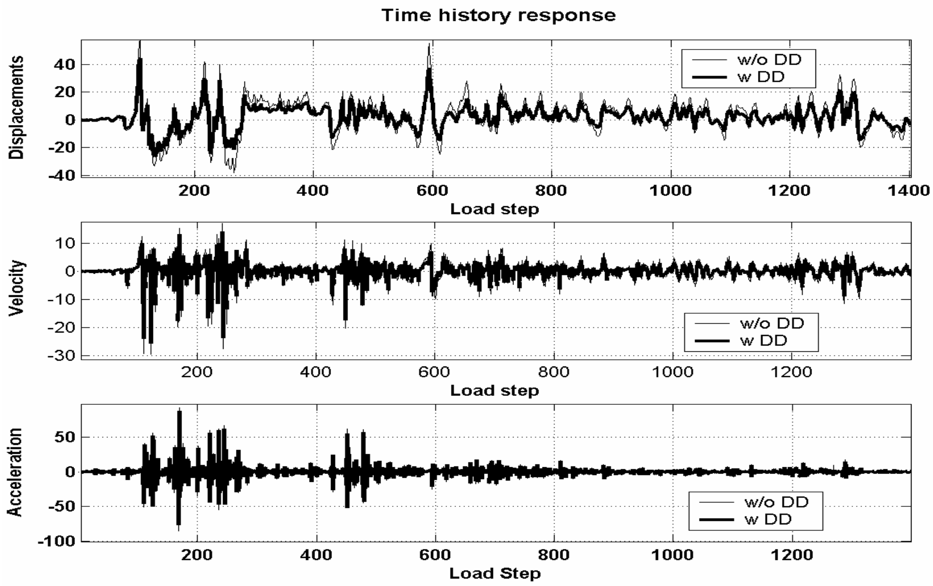

Figure 7. Time history response of the structure with and with out dissipating devices

3.2. Precast concrete building

The nonlinear seismic response of a precast building constructed in urban areas is studied. See Figure 8. The building has one bay and two stories of 6 and 3 m width, respectively. The concrete of the structure is H–25, (25 Mpa, ultimate compression), with an elastic modulus of 25.000 Pa. It has been assumed a Poison coefficient of 0.2. The steel bar reinforcements considered in the study are those corresponding to the 8% of the sectional area for elements near to the joint (25 % of the column or beam length), and 4% for elements in the middle part of the span. The ultimate tensile stress for the steel is 510 Mpa. The dimensions of the columns are 30x30 cm^2. The beams have a section of 15x30 cm^2.

The permanent loads considered are the weight of the concrete floors, a live load of 2500 N/m^2 and the weight of the closing walls, (432,000 N). The employed acceleration record is the same as before in the direction X and the same record scaled by 0.3 in the orthogonal direction.

No accidental or structural eccentricities were considered in this work, but it is possible to do it modifying the mass density of the beams or adding another structural element in the same of the planar frames.

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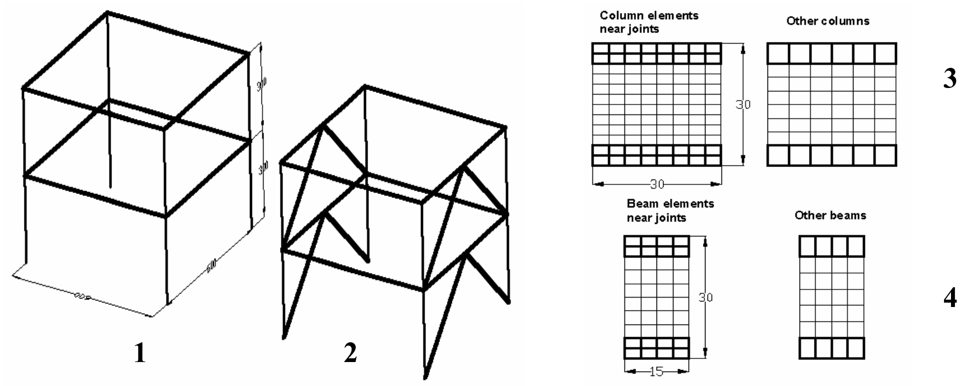

Figure 8. 1. 3D Frame; 2. Dissipating devices incorporated; 3, 4. Column and beams sections.

Several numerical simulations were carried out to obtain an optimized combination of the characteristic of the energy dissipation devices for knowing what is the yielding level, F_y, stiffness, K, and yielding displacement, D_y, which give a biggest protection level to the structure. The properties of the employed energy dissipating devices are summarized in Table 1.

Table 1. Parameters of the energy dissipation devices

Device Characteristics		Device Characteristics		Device Characteristics		Device Characteristics		Device Characteristics	
F_y, (N) D_y, (mm)	1000 1.250	F_y, (N) D_y, (mm)	2000 1.250	F_y, (N) D_y, (mm)	3000 1.250	F_y, (N) D_y, (mm)	4000 1.250	F_y, (N) D_y, (mm)	5000 1.250
F_y, (N) D_y, (mm)	1000 2.500	F_y, (N) D_y, (mm)	2000 2.500	F_y, (N) D_y, (mm)	3000 2.500	F_y, (N) D_y, (mm)	4000 2.500	Fy, (N) Dy, (mm)	5000 2.500
F_y, (N) D_y, (mm)	1000 5.000	F_y, (N) D_y, (mm)	2000 5.000	F_y, (N) D_y, (mm)	3000 5.000	F_y, (N) D_y, (mm)	4000 5.000	F_y, (N) D_y, (mm)	5000 5.000

The results of the simulation are expressed in terms of maximum top and middle floor displacements; base shear and over-tuning moment are presented simultaneously as function of the type of employed device in figure 9. From this figure it is possible to see that even when the biggest benefits in terms of the selected global variables are attained for different device characteristics, the more advantageous characteristics are related with flexible devices (K=8000 N/mm) with a medium yielding displacement (approx. 2.5 mm) and yielding force around the 4000 N.

Therefore, the selected properties of the dissipators were: Plastic yielding for a axial force of 4000 N, relative yielding displacement between the two ending nodes

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of 2.50 mm. The length of the dissipating devices is of 6.7 m. The dissipating devices only were incorporated in the direction where the strongest ground acceleration record is applied.

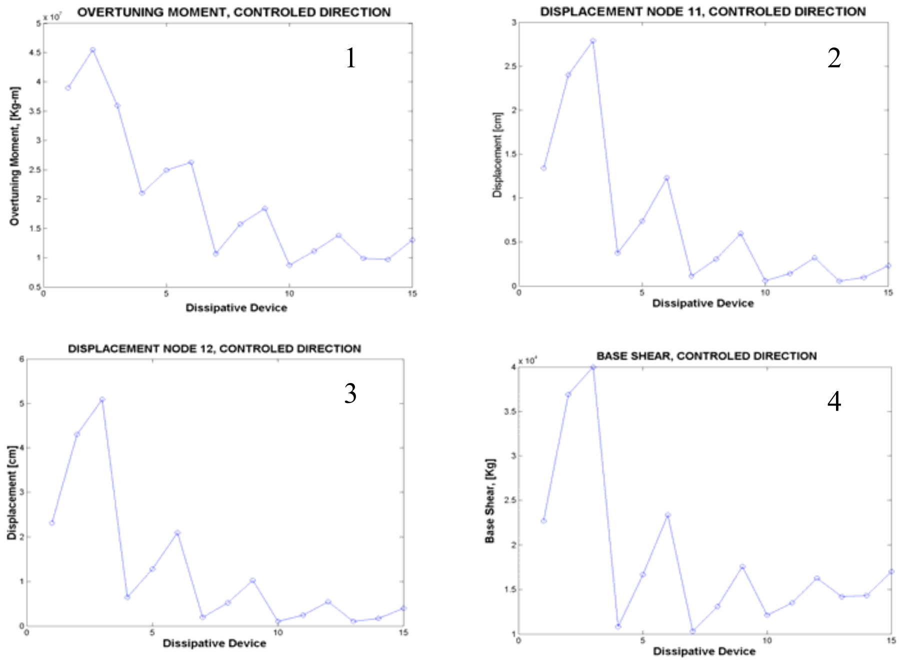

Figure 9. Maximum response for each energy dissipating device: 1. Over–tuning Moment; 2. Top floor displacement; 3. Middle floor displacement; 4. Base shear.

4. CONCLUSIONS

The seismic behavior of two typical precast concrete structures is studied employing a numerical code, which incorporate a geometrically exact finite strain formulation for rods using appropriated constitutive laws for materials. The simple mixing rule is employed to treat the resulting composites. The fiber beam model presented in this work provides a useful tool to simulate the earthquake effects on structures. A specific plastic energy dissipating device element is employed in the simulations. The advantages of employing dissipating devices to protect and improve the seismic behavior of flexible and low damped precast structures with non ductile connecting joints is studied for the 2D and 3D cases presented here. From the results it is possible to see that several numerical simulation are required to validate the best choice for selecting the mechanical characteristics of the control devices to ensure the biggest improvements in the seismic response of the controlled structure.

Acknowledgments

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