COMPARATIVE SEISMIC PERFORMANCE OF A BASE CONTROL SYSTEM BASED ON MEASURED AND CALCULATED RESPONSES

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ABSTRACT :

Finite element (FE) models of two similar buildings, one constructed with a traditional fixed-base foundation and the other provided with a Base Control System (BCS), were developed and calibrated. The two instrumented buildings, which are practically identical 3-story reinforced concrete structures, are located in Mendoza, Argentina. The accelerometers recorded the response due a 5.7 magnitude earthquake that shook the buildings on August 5, 2006. The BCS, developed by Gerb GmbH in Germany, consists of a combination of helicoidal springs and viscous dampers. The numerical models were used to carry out a comprehensive comparison of the seismic response of the two buildings and to evaluate the performance of the isolation system. Commercial finite element software was used to calculate the response due to the recorded seismic excitation in time domain, considering the non proportional damping due to the viscous dampers as a nonlinear load term. After the FE models were calibrated, the response time histories of both buildings were compared. The response examined were the acceleration at different levels, the base displacements, the axial forces and bending moments on different columns and the base shear. The results showed that the BCS was able to reduce the accelerations and internal forces in the isolated building to less than one third of the values corresponding to the rigid base structure.

KEYWORDS:

Base isolation, Base Control System, helical springs, instrumented structures, viscous dampers

1. INTRODUCTION

The Mendoza Campus of the Technical National University of Argentina commissioned in 2004 the construction of a set of buildings with the main purpose of providing housing facilities to students. The Mendoza province is located in Western part of Argentina, in the most seismically active region of that country. Two of the dormitory buildings constructed in the Mendoza Campus were identically designed and built. The only difference is that in one of the buildings, hereafter referred to as B3, was installed a Base Control System (BCS) whereas the other sister building, identified as B2, has a rigid traditional foundation [Stuardi, 2003]. The Base Control System installed in B3 provides flexibility and damping to the base of the building, both in the vertical and horizontal directions, by means of a combination of springs and viscous dampers [Nawrotzki, 2001; Nawrotzki, 2002]. The system was provided by Gerb GmbH [2007] in Germany. Both buildings were instrumented with accelerometers in 2005 and since then, a data acquisition system has been obtaining information about the seismic response.

The recorded response of base isolated buildings is very useful to verify the design and performance of structures fitted with these protective devices. This information, coupled with system identification techniques, can provide valuable information on the dynamic properties of the isolated structure and the isolation system. In addition, it can provide an assessment of the analysis techniques. The recorded response was used by researchers for different purposes as described next.

There are a limited number of instrumented base isolated buildings that were affected by strong earthquake motions. For instance, Papageorgiu and Lin [1989] analyzed the recorded response of a base isolated 4-story building to the 1985 Redlands, California earthquake (M_L 4.8) and used it to develop shear building models by means of system identification techniques. The recorded response of two buildings with elastomeric bearings to the 1994 (M_W 6.7) Northridge earthquake was used to evaluate their seismic performance and verify analytical modeling techniques by Nagarajaiah and Sun [2000; 2001]. Bozorgnia et al. [1998] studied the vertical response of the 8-story concrete building in Seal Beach, California to the same Northridge earthquake. Makris and Deoskar [1996] examined the response of a building isolated with the helical spring-viscous fluid damper system that experienced the effects of the 1994 Northridge earthquake. The base isolation system of this building is similar to the one considered in the present paper. The objective of the article by Makris and Deoskar [1996] was to validate a constitutive model developed earlier by Makris and Constantinou [1992] for the isolation system. The analysis of the combined base isolation-superstructure system was done in the frequency domain. In addition, a simplified method based on classical modal analysis to predict the response of the superstructure was presented. The studies of Makris and Deoskar were based on acceleration responses only. Further studies have shown that the level of internal stresses and base reaction forces could be reduced significantly by the Base Control System [Nawrotzki, 2004].

The effectiveness of the BCS is demonstrated in the present work by comparing different dynamic response quantities of the isolated building (B3) and its sister structure (B2). The area where the buildings are located was struck by a medium intensity seismic motion on August 5th, 2006 caused by a 5.7 magnitude earthquake with an epicenter located 30 km away from the City of Mendoza. The motion triggered the instruments and the acceleration of the two structures and the free field acceleration were registered. Finite element models of both buildings were created and then calibrated using the signals measured at the foundation and at the first floor and roof levels. The aim of this paper is to use the calibrated models to carry out a thorough comparison of the buildings' dynamic behavior and to evaluate the performance of the BCS.

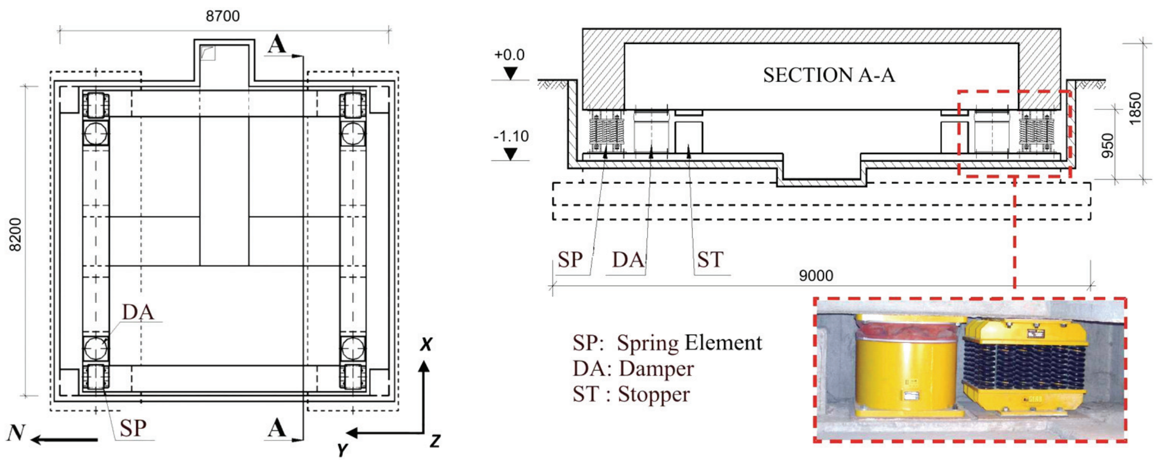

Figure 1 Plant view and section of base of building #3

2. BUILDING CHARACTERISTICS

The buildings under study were constructed with reinforced concrete frames and walls of reinforced masonry. Their dimensions are 8.2 by 8.7 m in plant, with a total height of 8.6 m and they have a total weight of 3200 kN. Building #3 is identical to building #2 with the only difference that the structure B3 has a peripheral beam located at the base whose function is transmitting the loads to the four sets of protection devices located under each corner of the building foundation. Each set of devices consists of a spring element and a viscous damper. A plant view and section of the isolated building is shown in Figure 1. The location of the protection devices at the corners of the building can also be observed together with the four added protection stoppers. The latter are simply concrete blocks located under the peripheral beam with a predefined gap between them and the superstructure. The stoppers can support the building in case of fire or extremely intense earthquakes. Figure 1

also shows a zoomed photograph of the installed spring elements and dampers.
The nominal average values of the stiffness and damping coefficients of the sets of protection devices are presented in Table 1. The vertical displacement due to the self weight is 23.4 mm.

Table 1 Stiffness and damping coefficients of the BCS components

Direction	Stiffness [kN/mm]	Damping coefficient [kN s/m]
Horizontal	4.57	360
Vertical	34.22	180

A twelve-channel data acquisition system was installed in the basement of building #3. The channel identification numbers, locations and recording axes of the accelerometers are summarized in Figure 2.

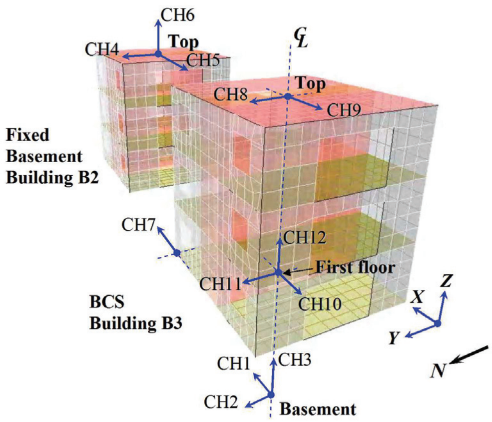

Figure 2 Locations of the accelerometers in the two buildings

3. MEASURED SIGNALS AND CALIBRATION

3.1. Measured ground accelerations

The three components of the ground acceleration measured at the basement of the building B3 along the three coordinate axes during the 2006 Mendoza earthquake are shown in Figure 3. The figure also shows the respective elastic ground response spectra for five damping ratios. The accelerometer that recorded the two horizontal components X and Y and the vertical component Z shown in Figure 3 is located at the geometrical center of the basement, as shown in Figure 2.

3.2 Computational Models

A finite element model of building #2 was first generated with the computer program SAP2000 [Computers and Structures, 2006]. In order to simplify the calculations and use standard commercial software, the stiffness and damping coefficients of the devices were considered to be constant in the B3 model. A slight dependency on velocity of the damping of the viscous dampers was neglected. Due to the presence of dampers at the base of the structure B3, the damping model of the structure-isolation system becomes non classical. The equations of motion were solved by numerical integration in the time domain over a reduced modal subspace and considering the damper forces as external forces.

3.3. Model Calibration

The natural frequencies of the model of the building with rigid base were first adjusted by using the signals measured by the accelerometers at the basement. The three components of the recorded accelerations were used as input and the corresponding components of the absolute accelerations at the roof were calculated. The natural frequencies were adjusted by successive comparisons between the computed accelerations and those measured by the triaxial accelerometer at the roof of structure B2. Both time histories and Frequency Response Functions

(FRF) were used for comparison and adjustment. Standard modal analysis procedures were used to achieve the best fit. Because the objective of this work was not a complete parametric modal calibration, it was sufficient to calibrate the first modes that were excited by the ground accelerations acting on the buildings. The parameters used to adjust the frequencies were the shear stiffness of the walls and the mass distribution in height and in plant. The shear stiffness values of the walls of the three floors were simultaneously adjusted at each step of the iteration process. Table 2a shows the first two natural frequencies obtained from the calibrated model of B2. The modal damping ratios were also adjusted by iterations so that the computed roof accelerations approximately match the measured response, both in the time and frequency domain. Following this procedure, the modal damping ratios were set to 3% for all modes.

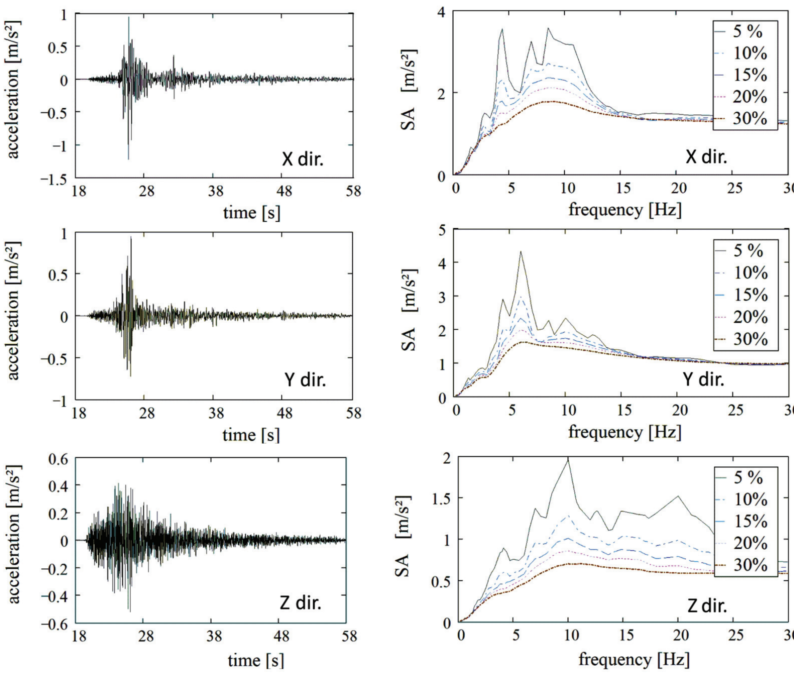

Figure 3 Measured ground accelerations and their response spectra

The rigid base model was then modified by replacing the base restraints by four sets of protection devices to obtain a first trial model of the isolated building. Stiffness of helical springs and damping coefficient of viscous dampers, both in horizontal and vertical directions were adjusted. Different functional dependences of the damping coefficient on the velocity were tried, but finally it was considered to be constant without appreciable error on the results. The first modes of the building with the BCS can be approximately defined using a rigid body -single mass- model representing the building upheld by flexible supports. Table 2b displays the seven lower natural frequencies of the BCS building. The first six vibration modes correspond to approximately rigid body motions of the building with high deformations of the protection devices, from the seventh mode and up, they are made up of a combination of elastic and rigid body modes.

Table 2 Modes and natural frequencies of the rigid base and BCS building

a

Nr.	Mode Rigid Building	Frequency [Hz]
1	YZ plane	4.44
2	XZ plane	5.78

b

N	Mode BCS building	Frequency [Hz]
1	Rocking motion in XZ plane	1.38
2	Rocking motion in YZ plane	1.43
3	Torsional	2.23
4	Vertical	3.61
5	Pendulum motion XZ plane	3.82
6	Pendulum motion YZ plane	4.13
7	Elastic building mode	10.75

3.4. Accuracy of the calibrated model

The degree of adjustment obtained after calibrating the model of building B2 was measured by comparing the accelerations registered at the top of the building with the corresponding accelerations calculated using the calibrated model. The Pearson product-moment correlation coefficient was used to ascertain the level of adjustment. The correlation coefficient between the measured and computed accelerograms was 87% and 93% for the X and Y directions, respectively. The adjustment achieved for the model of the B3 is demonstrated in the same way. The correlation coefficient between the measured and calculated signals was 0.942, 0.943 and 0.895 for the signals in the X, Y, and Z directions, respectively. Once the successful calibrations of both models are authenticated, they were used to calculate different response quantities that are presented in the following section.

4. COMPARISON OF THE SEISMIC RESPONSES

4.1. Floor accelerations

Figure 4 shows the absolute accelerations at the center of the roof slabs of the buildings with fixed base and with base isolation computed with the calibrated models. The accelerograms for the horizontal (X, Y) and vertical (Z) directions are shown for the time frame where they have the highest magnitudes. Note that the accelerations in the horizontal directions were reduced significantly in the building with the BCS. It can also be noticed from the figure that there was a small increase in the vertical accelerations compared to those from the fixed base structure. However, the vertical accelerations at the top of the building with the BCS were not amplified compared to the ground accelerations (which is usually the case in isolated structures) but slightly reduced.

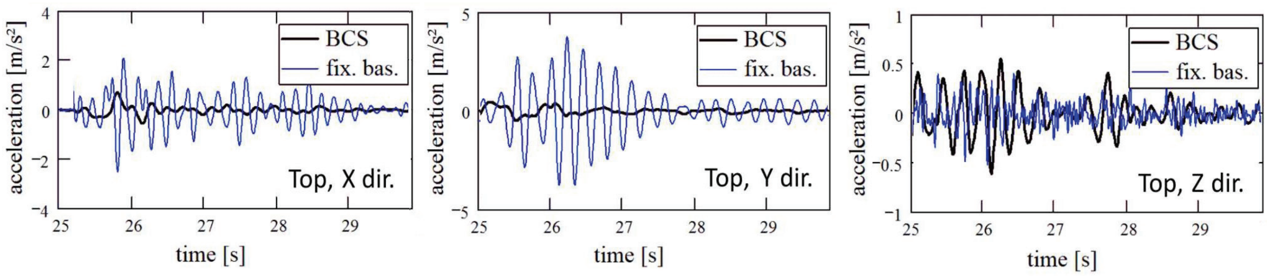

Figure 4 Comparison of accelerations at the top of the two buildings

4.2. Base Shear

The total base shear for the fixed base and isolated building is compared in Figure 5. The left graph shows the base shear time history in the X direction and the graph to the right displays the same quantity in the Y direction. The ratios between the peak shear values for the isolated and fixed base buildings are 0.25 and 0.20 for the X and Y directions, respectively.

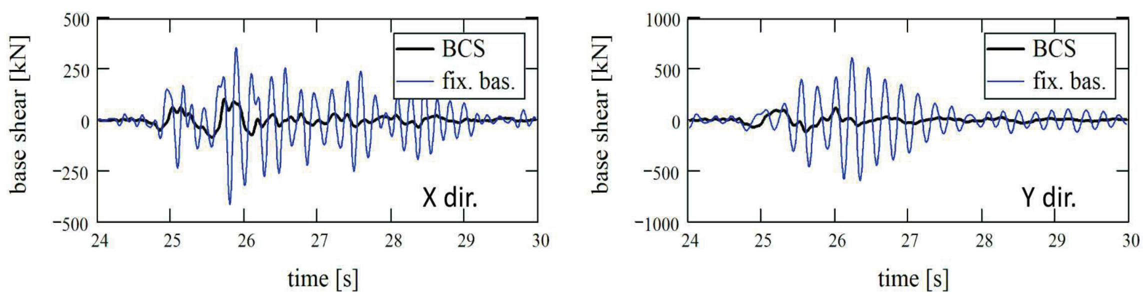

Figure 5 Comparison of the total base shear of the two buildings

4.3. Internal frame forces

The calibrated numerical models of the two buildings were used next to calculate the internal forces in the columns due to the three components of the recorded ground acceleration acting simultaneously. The results are presented in Figure 6. Two columns at the lowest floor were selected to present a comparison between the internal forces in the two buildings: a column located in the South-West corner (the lower-right column in Figure 1) and another center column at a lateral frame in the North side of the building. The top two graphs in Figure 6 show the time histories of the axial forces in the corner and center column. This is followed by two plots that display the shear forces in the two selected columns in the X direction. By glancing at the figures it becomes apparent the considerable reduction in the internal forces.

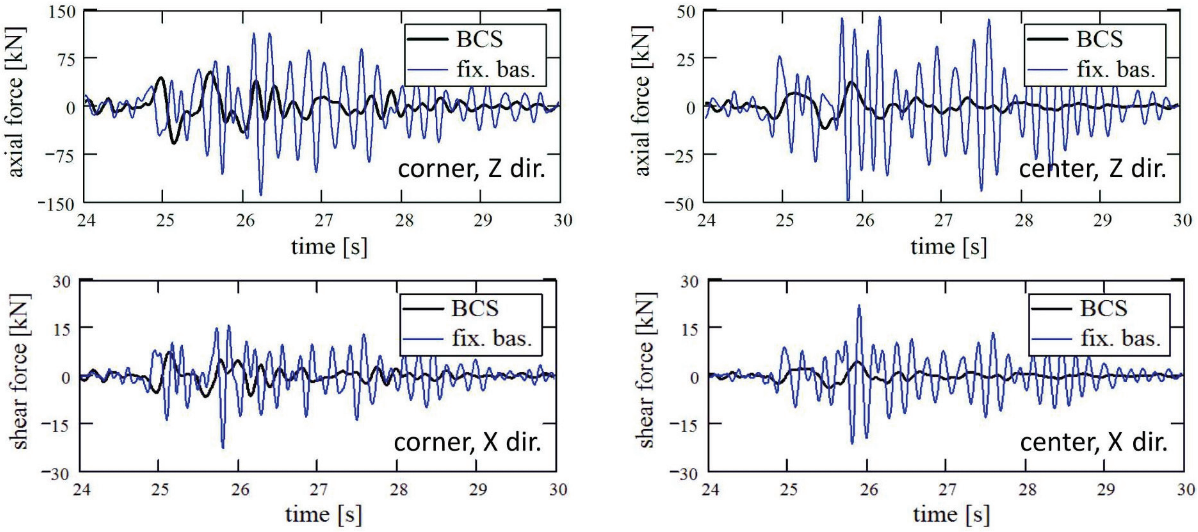

Figure 6 Axial and shear forces in the two buildings

4.4. Comparison of peak response values

To quantify the level of reduction, a summary of the peak values of the acceleration and internal forces of both buildings is presented in Figure 7 in terms of bar graphs. The first two plots display the absolute acceleration and the base shear in the X and Y direction. The following plots show the axial force and the bending moment at the selected corner and center columns of the lowest floor. The last two graphs display the shear force in the X and Y direction in the corner column and in the center column.

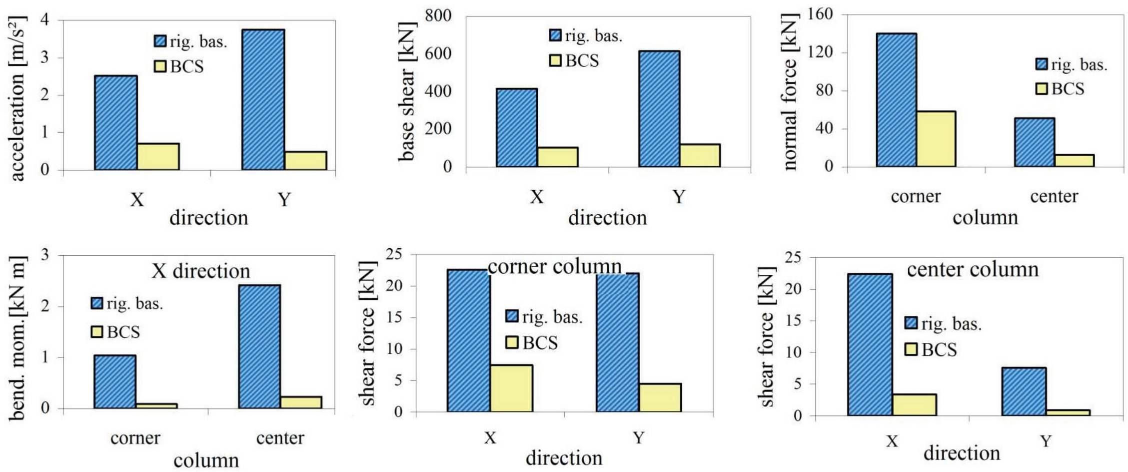

Figure 7 Summary of peak accelerations and internal forces

The left graph in Figure 8 displays the variation of the maximum and minimum absolute accelerations along the height of the building for points located in the southwest corner. The solid line corresponding to B3 indicates that the acceleration is almost constant along the height of the structure. The dashed line which corresponds to building B2 shows the usual increase with height in the values of the accelerations. The right graph in Figure 8 displays the relative displacements, calculated at the geometrical center of the slabs, with respect to the ground floor in the Y direction. The dashed lines with markers in the form of upward-pointing triangles are the displacements for the rigid base structure. The solid line represents the displacements of the isolated building. To facilitate the correct interpretation of these curves, the relative displacements caused by the deformation of the devices are also shown in the figure: they are the dashed lines with cross markers identified as the “rigid body motion” curve. For level 0, these displacements coincide with the horizontal deformation of the top of the devices. For levels 1 to 3, the “rigid body motion” displacements are the sum of the horizontal displacements caused by the rocking motion of the foundation plus the deformation of the devices. The interstory drift, which is a response quantity associated with structural damage, can be calculated from the differences between the BCS and the “rigid body motion” curves. The three components of the ground motion considered produced a peak horizontal deformation in the devices of less than 3mm whereas the maximum vertical displacement did not exceed 2mm.

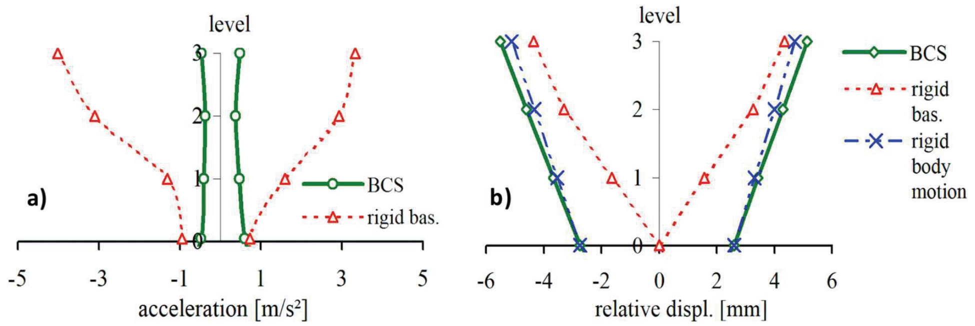

Figure 8 Peak accelerations at different building levels and peak relative displacements

5. CONCLUSIONS

The relative displacements, absolute accelerations, base shears and internal forces in the BCS and in the fixed base buildings were calculated and compared using two calibrated 3D computational models. Three components of recorded accelerations were used as input. The correlation coefficients between the measured and calculated signals were above 95% for all cases. This gives assurance that the calibrated models yielded realistic dynamic responses of the two buildings, and thus it permitted to carry out a comparison of different seismic response quantities and to draw conclusions about the performance of the isolated system.

The results show that the BCS is very effective in reducing the internal forces compared to those in a conventional rigid base building. The horizontal accelerations along the building height were effectively reduced: they decreased by more than 70% at the roof level. It is relevant to point out that the vertical accelerations at the top of the BCS building were not amplified with respect to the ground accelerations but rather slightly reduced. The axial forces were reduced by more than 60%, the shear forces more than 75%, and bending moments on different columns were decreased by about 90%.

The calculation of the interstory drift requires a special consideration when a building is seismically protected with a BCS. This is so because this building experiences small rocking motions due to the vertical flexibility of the BCS devices. Considering only the displacements caused by the distortion of the structure and not by the rigid body motion, drift is reduced by more than 80% compared to the values for the rigid base structure.

The earthquake ground motion considered in this study is the greatest in magnitude since the buildings were completed in July 2004 and, although it cannot be regarded as a strong event -having a PGA of 0.12 g-, it is useful to examine the performance of the protection system. A similar performance of the BCS is expected for earthquakes with larger magnitude because both the devices’ stiffness and damping remain approximately constant for greater demands.

The horizontal displacement demand at the devices’ level, i.e. the relative displacement between the cap and bottom plate, was less than 3 mm. The allowable displacements of the Base Control System are much higher than the measured value. According to the layout of the system, also a PGA up to 0.8 – 0.9 g would not lead to any damage of the devices. The peak structural displacement, that is the relative displacement of the top of the building with respect to the first floor slab, was also less than 3 mm. However, only 0.5 mm of the 3 mm displacement is due to the sum of the interstory drift along the height of the building. These values give an indication that the Base Control System is an effective seismic protection device that provides a good reduction of the accelerations and dynamic forces with low levels relative displacements of the building.

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