SEISMIC ASSESSMENT OF AN IRREGULAR BASE ISOLATED HOSPITAL

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ABSTRACT :

The present paper discusses the results of extensive dynamic analyses carried out on a large irregular base isolated hospital building, under construction in Naples, Italy. The framed structure of the multi-storey building is assumed as a sample system to assess the reliability of linear and non linear dynamic analyses implemented in modern codes of practice world-wide and they conservatism, if any. In so doing, a suite of spectrum compatible natural records are utilized to perform linear and non linear response analyses. The results of such analyses are then compared with simplified methods based on single-degree-of-freedom approximations. It is found that the selection of adequate natural records from existing earthquake catalogues is not straightforward, especially for base isolated structures which are affected chiefly by long distance and high magnitude events. A non linear model to simulate the dynamic response of rubber bearings is also presented and issues related to the calibration of such model are also highlighted; these calibrations are significantly affected by the fundamental frequency of vibration of the sample structural system. It is concluded the linear analyses may underestimate the displacements of the devices in base isolated structures when compared with non linear response analyses. To enhance the reliability of the analysis results, it is of paramount importance to calibrate the dynamic response of the rubber isolators on the fundamental frequency of vibration of the base isolated framed system and to select long distance and high magnitude pairs ground motions from existing catalogues of earthquakes, if available.

KEYWORDS: Base isolation, seismic assessment, structural analysis, modeling, rubber isolators, seismic codes

1. INTRODUCTION

Base isolation is a viable strategy for the design of critical facilities, in particular hospital buildings. A large number of base isolated health care facilities have been built world-wide in the last twenty years (Naeim and Kelly, 1999); some of them have already experienced high magnitude earthquakes without structural damage. There are, however, still several critical issues relative to the assessment of the seismic performance of such structural systems, especially when non linear behaviour of the isolation devices is accounted for. Recent code provisions, e.g. Eurocode 8 (2004), FEMA 450 (2004) and DM (2008) include comprehensive sections dealing with base isolated structures. Nevertheless, the reliability of the analysis type, e.g. linear and non linear, and the selection of the appropriate earthquake ground motion should be further investigated.
This paper presents the study of a large base isolated building under construction in the suburb of Naples, Italy. Three different types of dynamic analyses will be performed to estimate the differences, if any, for the sample structural system. Response spectrum, linear and non linear response-history analysis are employed to evaluate the seismic performance of the base-isolated structure. Structural response is expressed chiefly in terms of deformation quantities.

2. CASE STUDY

2.1. General description

The case study used to assess the reliability of different types of dynamic analyses is the structural system of a large base isolated hospital under construction in the suburbs of Naples, Italy. Further details of the sample structure can be found in Di Sarno *et al*. (2008). The “Ospedale del Mare” is an health care facility that is being built in Naples, in South of Italy. The whole building block comprises a number of building with different destinations. The plan layout of building is about 150x150m and the total height is about 32m. The structural system utilized for the super-structure is a reinforced concrete (RC) multi-storey framed system. The structural system exhibits a large mass eccentricity because of the different height (3 and 8 storeys, respectively) of the two L-shape blocks of the super-structure. The interstorey height of the building is 3,95m for the ground floor, 3,90 for the first, second and third floors and 3,60 for all the other storeys. The thickness of the slabs is equal to 40cm for all but the ground floor; for the latter a RC 50cm thick solid slab was adopted. The longitudinal cross-section of the structure and an aerial view are shown Figure 1.

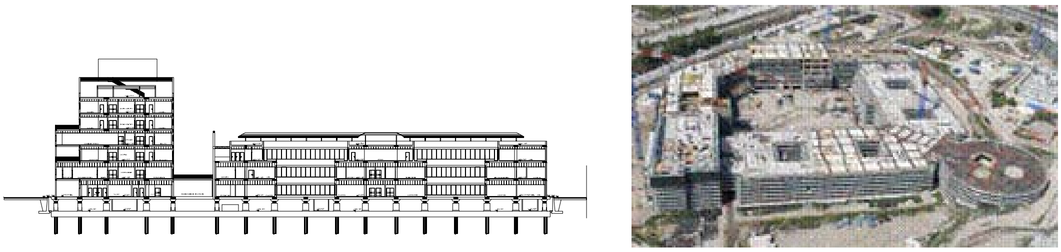

Figure 1 Longitudinal cross-section and aerial view of the structure.

The adopted foundation system includes the following structural parts (Figure 1):
Foundation sub-structure (inferior slab);
Foundation (superior slab);
Seismic protection system (rubber isolators).
The design of the above foundation components was carried out in compliance with the provisions included in the recent seismic national regulations adopted in Italy (OPCM 3431, 2005). The RC foundation mat rests on piles of medium diameter (800mm) with a length of 15.00 m. The thickness of the RC mat is 1200 mm while the superior slab includes a RC solid slab with a thickness of 500 mm.
To design the base isolation system, a complete model of the fixed base structure was considered. The maximum axial loads were computed at the base of the framed system both at ultimate limit state and serviceability. Such loads were used as design parameters to select the diameters of the rubber isolators. Three types of high damping rubber bearings (HDRBs) were selected; the total number of devices is 327; thus the

sample structure is the largest base isolated hospital in Europe. Table 1 provides the geometrical and mechanical properties of the devices adopted for the assessed structure. The ratio of the vertical (k_v) to the horizontal (k_h) stiffness of the devices is higher than 800 (minimum ratio k_v/k_h = 808) and hence the effects of the vertical flexibility of the devices are not significant.

Table 1 Geometrical and mechanical properties of the isolators

Isolator diameter (mm)	Number of Devices	Horizontal Stiffness K_h (kN/mm)	Vertical Stiffness K_v (kN/mm)	Stiffness Ratio K_v/K_h
600	122	1.51	1802	1195
650	108	2.98	2472	830
800	97	4.89	3949	808

2.2. Structural modeling

The framed system of the hospital building was modeled with finite elements (FEs) as implemented in the computer program SAP2000 (CSI, 2003), which may be used to perform static and dynamic analysis of structures. Beam and columns are modeled with linear frame elements, while shell elements are used for the floor slabs. Diaphragm constrains were not utilized because of the large openings in the slabs, particularly where roof gardens are located. Different FE models were used to simulate the structural response of the isolator devices. In the linear analyses the isolators are modelled as linear rubber isolator link which are defined by two parameters, i.e. secant stiffness and damping constant. The stiffness of the devices, summarized in Tables 1, are constant values and were based on experimental tests on sample devices carried out by the manufacturer. Damping coefficients are assumed equal to zero because damping ratio is accounted for through the setting of the analyses parameters: equivalent viscous damping equal to 10% for first three modes of vibration. Non linear analyses were carried out assuming linear elastic behaviour for frame and shell elements of the super-structure; the nonlinearity is concentrated at the base isolation system. It is thus of paramount importance to simulate reliably the nonlinear response under earthquake ground motions of the rubber isolators. To perform nonlinear dynamic analyses, it is assumed, in the analysis settings, that damping coefficients are zero because the dissipated energy is accommodated by the nonlinear isolator behaviour.
To assess the equivalent damping coefficient ξ_{eq} of each isolator the following relationship (Chopra, 2002) may be used (1) and it is computed at resonance, i.e. with $\omega = \omega_n$:

$$\zeta_{eq} = \frac{1}{4\pi} \cdot \frac{1}{\omega/\omega_n} \cdot \frac{E_D}{E_{so}} \tag{1}$$

where:

ζ_{eq} = equivalent damping coefficient;

E_D = hysteretic dissipated energy;

E_{so} = strain elastic energy;

ω = circular frequency of the system;

ω_n = circular frequency of the external force.

The equivalent viscous damping ratio is estimated from the results of the experimental tests carried out by the manufacturer. The initial elastic stiffness is known from the experiments, thus the rubber isolators can be simulated with a non linear elastic-plastic model with hardening. It is assumed that the dissipated energy gives the same value of equivalent damping ratio (energy equivalence).Figure 2 provides the typical experimental response of 600 mm rubber devices. The numerical models used to perform linear and nonlinear dynamic analyses are compared with the experimental data. Table 2 summarizes the fundamental parameters used to model the rubber isolators of the sample structure. It is worth noting that the equivalent elastic stiffness does not correspond to the same value of the force corresponding to the test and assumed in the bilinear model:

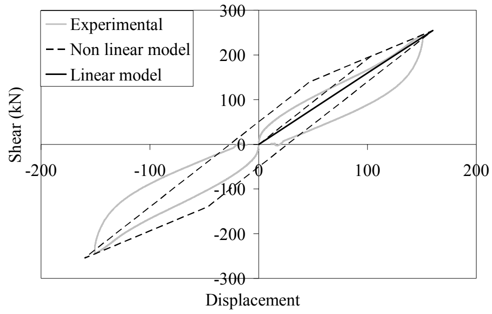

Figure 2 Seismic isolator model

Table 2 Model parameters of seismic isolator devices

Isolator diameter (mm)	Linear stiffness (kN/mm)	Initial stiffness K_1 (kN/mm)	Post-yeld stiffness K_2 (kN/mm)	Hardening ratio K_2/K_1	Yielding force Fy (kN)	Equivalent damping ratio (%)
600	1.51	1.90	1.01	0.53	197.66	8.10
650	2.98	4.45	2.15	0.48	459.27	9.22
800	4.89	6.68	3.32	0.50	545.75	9.56

2.3 Dynamic properties

The dynamic properties of the fixed and base isolated models of the sample building are summarised in Table 3. Periods of vibration and participating mass ratios are outlined in the table.

Table 3 – The dynamic properties of the fixed and base isolated models.

Fixed Base					
		Participating Mass Ratios			
Mode (-)	Period (sec)	X (%)	Y (%)	ΣX (%)	ΣY (%)
1	0.78	24.0	20.0	24.0	20.0
2	0.76	24.0	29.0	48.0	49.0
3	0.69	5.8	3.8	54.0	53.0

Isolated Base					
		Participating Mass Ratios			
Mode (-)	Period (sec)	X (%)	Y (%)	ΣX (%)	ΣY (%)
1.00	2.06	0.1	64.0	0.1	64.0
2.00	1.99	99.0	0.0	99.0	64.0
3.00	1.87	0.0	35.0	99.0	99.0

There is a number of observations stemming from the computed values of the modal masses. Firstly, the efficacy of the base isolation system is demonstrated by the high values of participating masses of the first three modes of vibrations of the structural system, i.e. those corresponding to the modes of the base isolation system. The dynamic response of the fixed base structure is rather complex and higher modes affect significantly the global response. Conversely, the participating mass ratios for the isolated structures are characterized by the onset of the 99% of the total mass in the first three modes.
In order to compare the analysis results, design acceleration spectra and time-histories are applied along the X-direction, so that the response of the multi-degree of freedom system is equivalent to the single-degree of freedom system (SDOF).

3. SEISMIC INPUT

3.1. Acceleration response spectra

Four structural limit states (LSs) are provided in the recent national seismic code of practice (NTC, 2008); these LSs include two elastic (namely Operational and Damageability LSs, OLS and DLS, respectively) and two inelastic (Life Safety and Collapse Prevention LSs, LSLS and CPLS, respectively). Four response acceleration response spectra are provided for seismic analysis and design. Such spectra are defined as a function of coordinates, i.e. latitude and longitude, of the site location, geotechnical characteristics of the soil and type of use of the building structure. The fixed shape of the acceleration response spectra is given as a function of the bedrock acceleration a_g, the amplification factor F_O and the corner period T_C corresponding to the constant velocity branch in the response spectrum. Table 4 provides the parameters used to evaluate the response spectra of the horizontal components of earthquake ground motion for the site of the construction of the sample structure assessed herein, i.e. the South-East suburb of Naples, in the South of Italy. The four spectra are plotted in Figure 3 for the four LSs. For base isolated structures the acceleration response spectra are scaled for periods greater than 0.8 T_{is}, where T_{is} is the fundamental period of the base isolated structure.

Table 4 Parameters used for the acceleration response spectra

Limit States		F_O	T^*_C	S_S	S_T	C_C	P_{vr}
		(-)	(sec)	(-)	(-)	(-)	(%)
Elastic	OLS	2.33	0.33	1.20	1.00	1.37	81
	DLS	2.30	0.30	1.20	1.00	1.37	63
Inelastic	LSLS	2.54	0.35	1.13	1.00	1.36	10
	CPLS	2.58	0.35	1.11	1.00	1.36	5

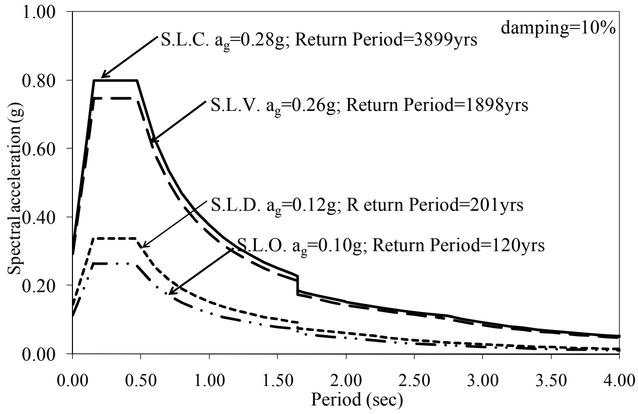

Figure 3 Horizontal acceleration response spectra

3.2. Accelerograms

Natural accelerograms were utilized to perform elastic and inelastic response history analyses of the sample base isolated structure. The requirements for the selection of the natural earthquake records are as follows (EC8, 2006; NCT, 2008):

- The choice of time-histories shall be representative of seismic hazard of the site;
- Time-histories shall match design spectra in adequate period ranges.

These requirements are reported also in EC8 where there is the additional condition that the values of recorded accelerograms shall be scaled to the values of $a_g \cdot S$ for the zone under consideration.

The above provisions are clearly rather general and hence the rules for the selection of artificial time-histories are employed (as suggested in EC8). For such records the seismic code of practice states that:

- The spectrum of the ensemble of the ground motions shall be evaluated by taking the average value of the spectra of the individual earthquake of the period step;
- The ensemble spectrum shall be scaled so that it is not lower than 0.9 times the 5% damped elastic response spectrum in the period range between 0.15 and $1.2T_{IB}$ where T_{IB} is the fundamental period of the isolated base structure.

In the present study a group of seven pairs of code-compliant earthquake time-histories were utilized. This use of seven strong motion records is sufficient to estimate the average of the structural response quantities. The set of natural records were derived from a selection proposed by Iervolino *et al.* (2007); these records can be downloaded from European Strong Motion Database (ESD) (http://www.isesd.cv.ic.ac.uk). The selected time histories are summarized in Table 5.

Table 5 Set of earthquake natural records

Record information for the set					
Site/Zone	Code	Event Name	Country	Date	Station Name
B - 2	197	Montenegro	Yugoslavia	15/04/1979	Ulcini-Hotel Olimpic
	199	Montenegro	Yugoslavia	15/04/1979	Bar-Skupstina Opstine
	228	Montenegro (aftershock)	Yugoslavia	24/04/1979	Bar-Skupstina Opstine
	231	Montenegro (aftershock)	Yugoslavia	24/04/1979	Tivat-Aerodrom
	4673	South Iceland	Iceland	17/06/2000	Hella
	6263	South Iceland	Iceland	17/06/2000	Kaldarholt
	6334	South Iceland (aftershock)	Iceland	21/06/2000	Solheimar

The fundamental period of vibration of the base isolated structure is 2.06 sec; the acceleration response spectrum compatibility has to be verified for values of periods ranging between 0.15 and 2.47 sec. Scale factors equal to 1.50 and 0.96 were adopted for the x- and y-directions of the plan layout of the building structure, respectively.

Fourier spectra were also evaluated for the sample suite of earthquake records. Figure 4 (left) shows the envelope Fourier spectrum of all time-histories and shows that the fundamental frequency of the structures is lower than predominant frequencies of the natural records. In the Figure 4 (right) there is also the average of Fourier spectra.

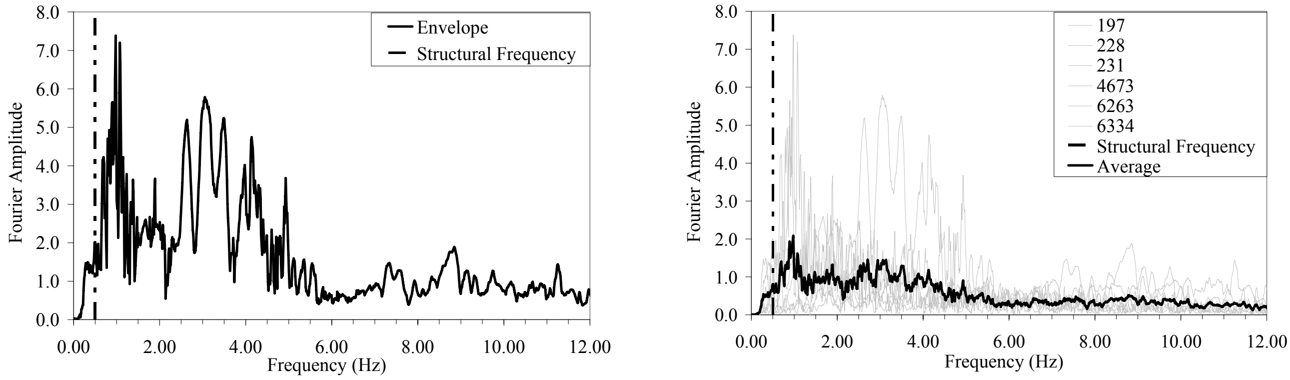

Figure 4 Envelope (*left*) and medium (*right*) Fourier spectra in X-direction of the sample frame

3.3. Sine-wave motion

To investigate the reliability of the adopted non linear isolator model and to further assess the response of the devices under dynamic loading regime, simplified sinusoidal time-histories with different periods and different amplitudes which produce the same maximum displacement on the linear elastic model were assumed. Linear and nonlinear response history analyses were carried out along the X-direction of the plan layout of the structure because it activates nearly 100% of the total seismic mass along this direction (see Table 3).The results are expressed through the coefficient C_1 equal to the ratio between maximum (or minimum) displacement of non linear and linear models. Target displacements (TDs), i.e. the value of maximum displacement obtained by linear model subjected to elastic spectrum at SLV (see Table 5), is initially equal to 12.90 cm . These results, provided pictorially in Figure 5, are compliant with eqn. (1). The coefficient C_1 is 1.0 only when sinusoidal period is equal to first natural period of the structure in X-direction. As a result, for this value of sinusoidal time-histories linear and non linear model are equivalent. Additionally, a TD equal to 4 cm, which is a value lower than the yielding displacement of non linear links (9 – 10 cm), was assumed. The nonlinear system corresponds to a linear system with stiffness equal to K_1 and without damping: C_1 is variable because two systems have different dynamic behaviour. Conversly, whether the linear system is characterized by K_1 stiffness, C_1 is always higher than unity, as displayed in Figure 5.

A further case considered herein is TD=30 cm. Under such displacement the equivalent stiffness of the nonlinear system is lower than the stiffness of the linear counterpart but the dissipated energy is almost the same as in the elastic model (see Figure 7). Consequently, C_1 is higher than unity for all the period of sine-wave input motion (Figure 6).

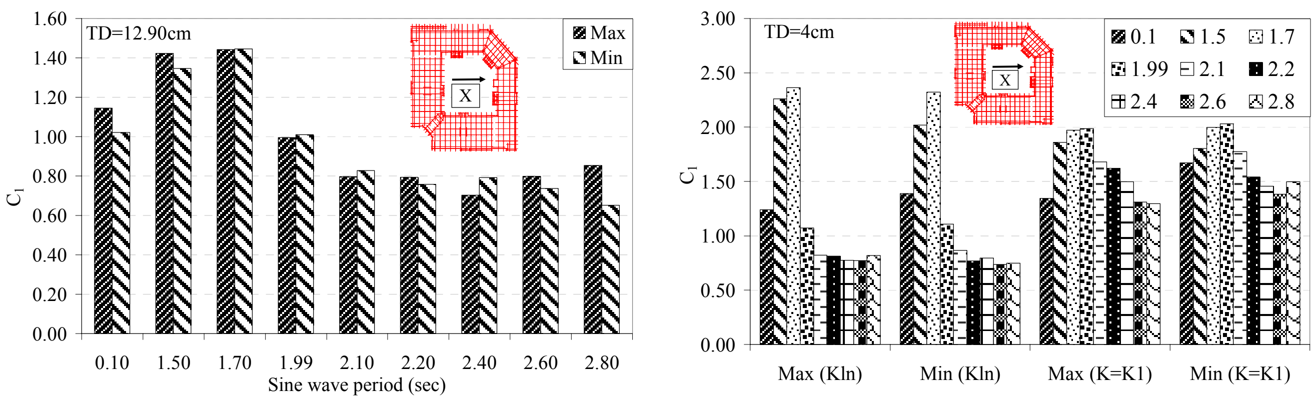

Figure 5 Coefficient C1 for sinusoidal time-histories: TD=12.90 cm (*left*) and TD=4cm (*right*)

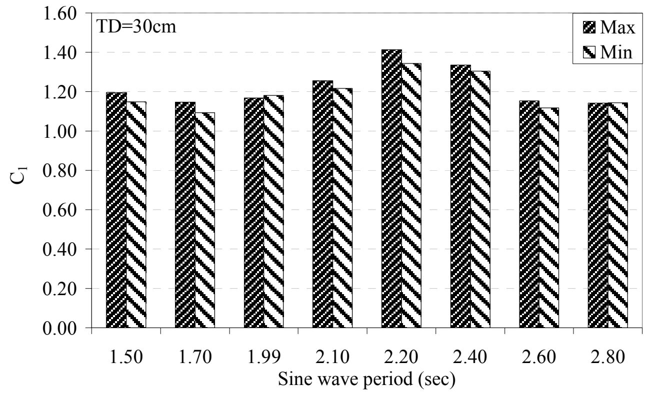

Figure 6 Coefficient C1 for sinusoidal time-histories: TD=30 cm

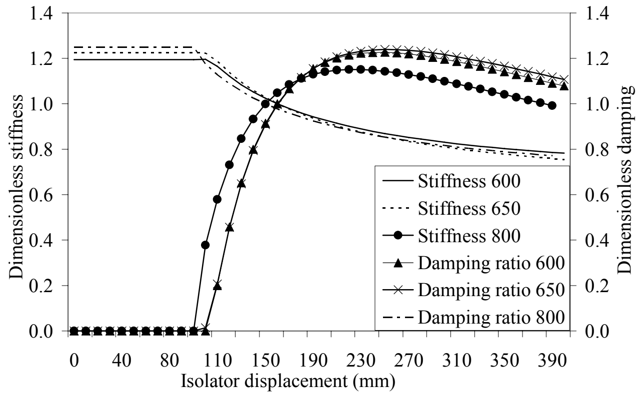

Figure 7 Equivalent stiffness and damping ratio for each device

For the sake of completeness, Figure 7 provides, for each non linear link, the equivalent stiffness and damping ratio respectively divided by linear stiffness and equivalent damping ratio reported in Table 2 versus the maximum device displacements.

4. ANALYSIS RESULTS

4.1. Displacement response analyses

The maximum displacements computed with the different dynamic analyses are listed in Table 6. These results are compared in Table 7.

Response spectrum analysis gives lower displacements that spectrum previsions, as expected. Linear time-history analysis provides lower values of displacements than the spectrum method. By contrast, the ratio between non linear and linear time-history analyses should be lower than unity. This is, however, significantly affected by the record characteristic and the expression used to estimate the equivalent damping coefficient. In fact for the formula (1), the non linear model assumed in this study is equivalent to the linear counterpart merely under a sinusoidal time-history with frequency equal to 0.50 Hz. Figure 4 shows that all selected time-histories have higher frequencies. Thus, ratios higher than unity are expected.

Table 6 – The dynamic properties of the fixed and base isolated models.

Medium displacements for LSLS (X direction)					
Diameter isolator (mm)	SDOF prediction from design spectrum (cm)	Response spectrum analysis (cm)	SDOF prediction from medium time-histories spectrum (cm)	Linear time-history analyses (cm)	Non linear time-history analyses (cm)
600	14.24	12.98	14.12	12.12	14.07
650	14.24	12.90	14.12	12.04	14.03
800	14.24	12.81	14.12	11.96	13.98
Average	14.24	12.90	14.12	12.04	14.03

5. CONCLUSIONS

This preliminary study shows that, to perform reliable dynamic analyses of structural systems, it is of paramount importance to select earthquake ground motions that are compliant with the fundamental period of vibration of the system, especially for base isolation systems. However, such structures exhibit periods greater than 2.0 seconds and it is not straightforward to select adequate strong motions in the available catalogues. Distant and high-magnitude earthquakes are effective, for example, for the sample base isolated structure, but such earthquakes are scarce in the world-wide seismic database. The selection of suites of natural accelerograms based merely on the spectrum compatibility may, however, result misleading and gives rise underestimations of the deformation quantities derived by simplified code-based dynamic analyses. Additionally, the calibration of the linear and nonlinear models adopted to simulate the dynamic response of the isolators should be based on the actual fundamental frequency of the structural system. This assumption may be effective to reduce the underestimation of the earthquake effects evaluated through linear dynamic analyses, which ranges between 15 and 20%.

Table 7 – Displacements analyses ratios

Displacements ratio for LSLS (X direction)			
Diameter isolator (mm)	SDOF prediction vs Spectrum analysis	Linear time-history-analyses vs Spectrum analysis	Non linear time-history-analyses vs Linear time-history-analyses
600	1.10	0.93	1.16
650	1.10	0.93	1.17
800	1.11	0.93	1.17
Average	1.10	0.93	1.17

In Figure 8 are shown typical displacements time-histories of two different isolator under accelerogram labelled as 199.

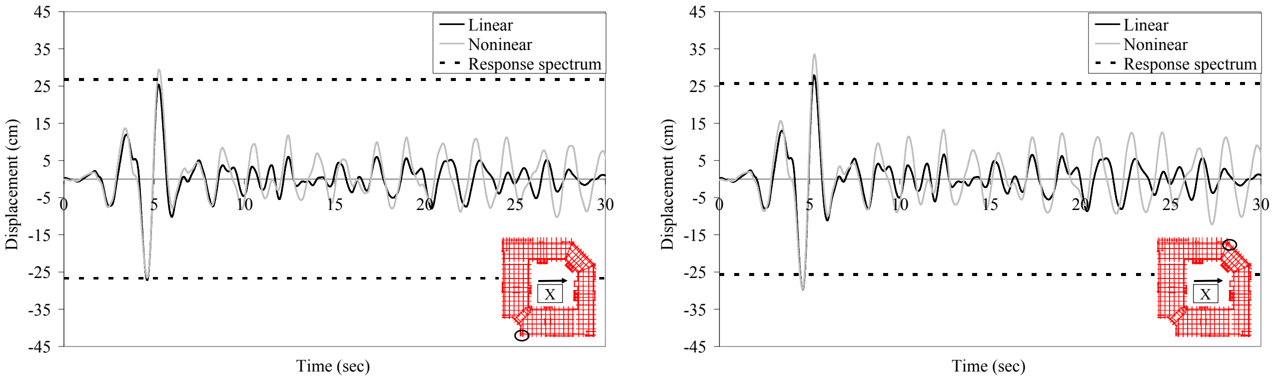

Figure 8 Typical displacement time-histories

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