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PSEUDO-DYNAMIC TESTS OF A BASE-ISOLATED STEEL FRAME

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ABSTRACT

The need for more experimental information on the behavior of base-isolated structures motivated the application of the pseudo-dynamic method to isolated specimens carried out in the tests described in this paper. The tests were performed on a full-scale 3-story steel frame provided with high-damping rubber bearings. Special attention was paid to the quantification and compensation of the strain-rate-effect errors which are introduced by the slow testing speed of the pseudo-dynamic method. The application of the same technique for the isolated as well as for the non-isolated structure and for different historical accelerograms was also used to check the capacity of these tests to assess the performance of the adopted isolation design, which in general proved to be high.

KEYWORDS

Pseudo-dynamic test; base isolation; strain-rate effect; high-damping rubber bearings; full-scale test; seismic protection; multistory steel frame; pseudo-dynamic substructuring; seismic test; snap-back test.

1 INTRODUCTION

In theoretical studies as well as in existing real structures, base isolation has proved to be a very effective approach for seismic protection (Kelly, 1993). However, widespread diffusion of this technique has not yet been achieved, mainly due to the lack of extensive experimental information which would allow the adoption of less overconservative design codes. Shaking table tests of base-isolated specimens (Kelly, 1991) have been successfully used for this purpose but only for small-scale models presenting some limitations for the extrapolation of the results to the real scale. On the other hand, full scale on-site experiments (Bettinali *et al.*, 1991) represent a good alternative but are limited to small amplitude forced vibrations or snap-back tests and so the experimental response to a specified accelerogram is not directly rendered. As a promising third option, the pseudo-dynamic (PsD) test method (Donea *et al.*, 1990) seems to be well suited for obtaining the seismic response of large-size isolated specimens thanks to the use of quasistatic-loading equipment, but with the possible disadvantage of strain-rate-effect (SRE) errors which may be introduced by the rubber bearings.

With the intention of evaluating the possibilities of the European Laboratory for Structural Assessment (ELSA) of the Joint Research Centre (JRC) of the European Commission to perform PsD experiments on a large isolated structure, a test campaign, which is described in this paper, was developed.

2 SPECIMEN ISOLATION DEVICES AND STRAIN-RATE EFFECT COMPENSATION

The isolator devices were rubber bearings with a diameter of 250 mm and made of 11 rubber layers with a thickness of 6 mm (66 mm of total rubber height), 10 steel layers of 1.5 mm (alternating between the rubber layers) and two end steel plates of 10 mm which included threaded holes for fixation. Consequently, the total height of each isolator was of 101 mm. They were designed for a working shear strain of 100% (66 mm of horizontal displacement) and a nominal vertical load of 400 kN. Four of these isolators, made of a high-damping rubber mixture called EN60, were made available to the ELSA for this test campaign.

Usually, for common building materials, the SRE errors introduced by a PsD test may be disregarded since they are less important than the existing variability from specimen to specimen (Gutierrez *et al.*, 1993a). However, for elastomer bearings, a decrease of testing speed of two or three orders of magnitude --as is usual for a PsD test-- may introduce considerable changes in the stress-strain behavior, especially for filled rubber (Kelly, 1993). These changes may be described as a loss of stiffness (Gutierrez and Verzeletti, 1993b) or, alternatively, they may be considered as a proportional reduction of the force. To compensate this effect, at every step of the PsD integration, the measured forces may be corrected so as to account for an increase of a specified percentage on the stress at the isolators. From the results of specific characterization tests on the isolators and snap-back tests on the isolated frame (Molina, 1996), a 20% of bearing shear force correction was adopted for the tests described here.

Each of the available rubber bearings was in fact an exact one-half scale model of a prototype of double height and diameter and with a vertical load capacity four times larger. For converting the behavior of the available bearings to that of the prototype, it was assumed that, for a common transversal strain, the shear strain is independent of the scale of the devices. On the other hand, for said common stress-strain state, the total shear load would be proportional to the total transversal area and the displacement would be proportional to the height of the elements. This substructured scale modeling was considered in the seismic PsD tests by modifying the measured forces --to account for the ratio of area between model and prototype-- and the imposed displacements --to account for the ratio of the bearing height.

3 TEST SPECIMEN AND TESTING PROCEDURE

As mentioned above, the purpose of the tests herein described was the assessment of the feasibility of PsD tests on a structure isolated with rubber bearings. With this aim in mind, a 3-story steel building which was available in the laboratory was elected as specimen. It is made of two single-bay moment-resistant frames with a total height of 10.40 m, 5.00 m wide and 5.00 m deep (Fig. 1) which had been previously used for several test campaigns (Kakaliagos, 1994) --without isolation-- and repaired afterwards. Each floor of the structure is made of a concrete slab to which two hydraulic actuators were attached (only the front one is seen in the figure). Since the bases of the columns were no longer resting on a rigid foundation, each frame was closed by a stiff beam with the same profile (HEB 400) as the columns and these beams constituted the base floor which was fixed onto the four isolators positioned at the corners. This new base --which will be called here floor 0-- was also provided with an additional concrete block of about 250 kN of weight to which a single actuator was attached. The main function of this block was to increase the total weight of the specimen, which was finally of about 750 kN. Although this load was only about one half of the design vertical load of the isolator set, it was

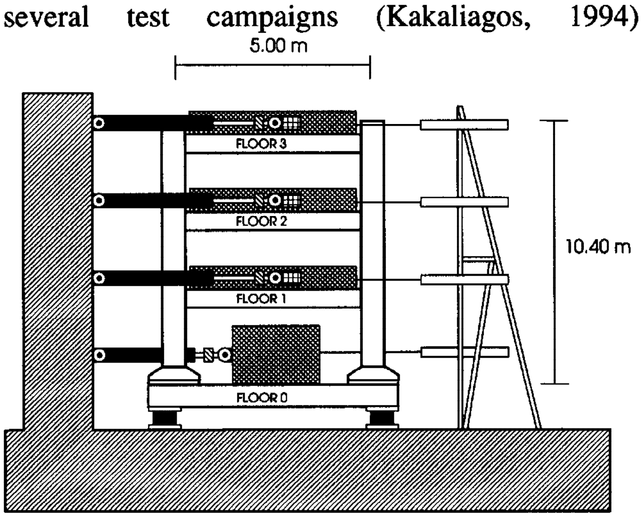

Fig. 1. Schematic representation of the test apparatus for the steel frame.

considered sufficient for the purpose of these tests since the addition of more heavy blocks would have considerably increased the cost of the set up.

As shown in Fig. 1, the bottom of the base isolators was fixed to the strong floor of the ELSA, while the seven actuators were connected to the reaction wall. Each actuator, with had a capacity of 500 kN, was equipped with a load cell and displacement controlled by means of a digital optical displacement transducer fixed to a common reference frame (on the right in the figure). The digital control loop of each actuator also included an additional feedback signal from an accelerometer which allowed relatively high testing speeds without instability or loss of accuracy. Every controller was communicated with the main processor which was in charge of the PsD integration computations (Magonette, 1991).

4 SEISMIC TESTS

The isolated steel frame specimen was pseudodynamically submitted to four different historical accelerograms which were applied for several amplification factors. Furthermore, in order to study the efficiency of the adopted base isolation for every accelerogram, PsD tests were also conducted on the same steel frame specimen but without isolation. Said non-isolated condition was simulated without changing the experimental setup, by means of blocking the base (floor 0 in Fig. 1) --by using a zero target at the corresponding cylinder-- and reducing the PsD model to the three upper floors. As was shown by the results of some stiffness tests (not included here), the obtained non-isolated condition was not completely equivalent to the rigid condition generated by clamping the bases of the columns directly onto the strong floor, as was done in previous non-isolated test campaigns on the steel frame. Nevertheless, said obtained condition still represented a realistic case of a non-isolated structure with a non-perfectly-rigid foundation. The accelerograms used for the test campaign were the following: Calitri (Campano Lucano 1980, Calitri station), Armenia (December 7, 1988, Spitak station, NE component), Mexico (September 19, 1985, SCT station, N90W component) and Kalamata (September 13, 1986).

4.1 Pseudo-Dynamic Integration and Prototype Definition

For coherence with previous test campaigns (Kakaliagos, 1994), the specimen frame was considered as only a part of a larger building prototype so that the values 34.7, 34.7, 34.7 and 32.9 ton were adopted as theoretical masses for these seismic tests. The adopted time increment for the PsD integration was 0.010 sec. For all the earthquakes, the integration was performed assuming that the isolators of the prototype had a height double that of the specimen ones by imposing to the specimen a base drift of only one half of the computed one. On the other hand, for the first three earthquakes, the total transversal area of the prototype isolators was considered as equal to that of the specimen (one prototype bearing of double diameter instead of four specimen bearings), so that the base shear was only corrected to account for the SRE.

4.2 Transmissibility Function from Seismic Test Results

Before commenting on the general results and in order to obtain a mean to understand the behavior of the isolated and the non-isolated structures, a plot of their respective transmissibility functions is shown in Fig. 2. Both curves were obtained as the quotient between the absolute acceleration at the third floor of the frame and the ground acceleration as obtained from the test response to the Calitri earthquake for an amplification factor of 0.4. As a matter of fact, these frequency response functions show in a simple way what the degree of isolation is for every frequency and what the resonance frequencies (see estimated values in the first two rows of Table 1) of both structures are, even though the non-linearity of the structure may slightly limit the generality of these data for other amplification factors and earthquakes. In particular, Fig. 2 shows that the structure accelerations are lower in the isolated case, almost for every frequency over 0.9 Hz and that the efficiency of the adopted isolation is very high for most of the earthquakes.

Table 1. Estimated eigen frequencies and damping ratios from the seismic response for the non-isolated frame and for the isolated one with a 20% or a 176% of shear load correction at the isolators.

	Mode 1		Mode 2		Mode 3		Mode 4	
	Hz	%Damp	Hz	%Damp	Hz	%Damp	Hz	%Damp
non-isolated	2.1	~0.5	7.2	~0.5	14.7	~0.5		
isolated 20% corr.	0.58	11	3.7	4	8.4	~1.6	15.0	~0.6
isolated 176% corr.	0.82	9	3.8	4	8.4	~1.6	15.0	~0.6

4.3 Hysteretic Loops at the Isolators.

As an example of the hysteretic behavior of the high-damping rubber bearings used for the tests, Fig. 3 gives a force displacement diagram for the specimen isolator set during the response to the Calitri earthquake with an amplification factor of 1.5. During this test, a maximum shear displacement of 74 mm (112% of deformation) was attained which would correspond to 148 mm of displacement for the prototype with double-height bearings. As seen in the figure, said strain level generated a certain amount of hardening in the rubber at the extreme displacement.

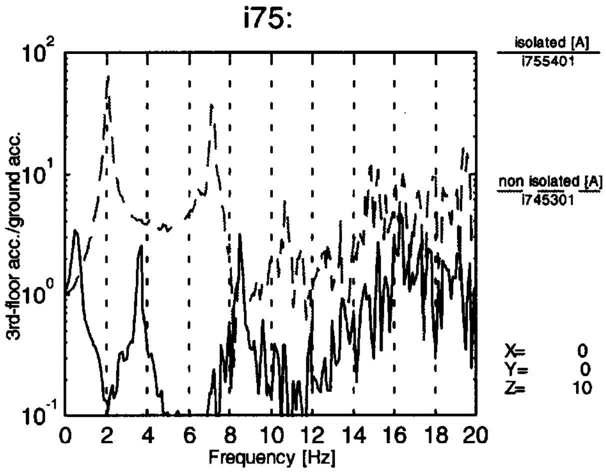

Fig. 2. Acceleration transmissibility function as obtained from the seismic response for the isolated (solid line) and non-isolated (dashed line) structures.

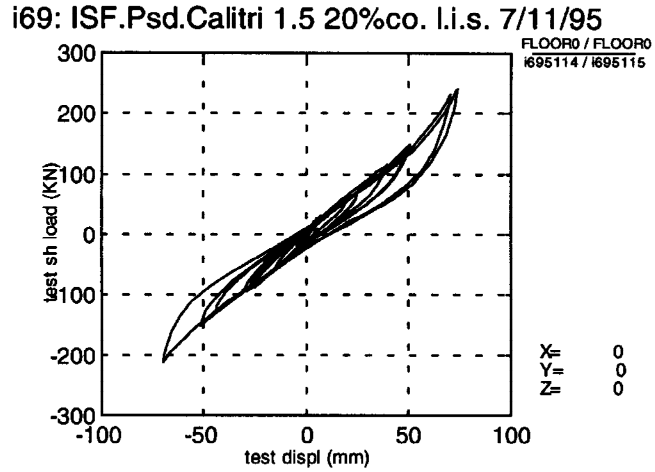

Fig. 3. Hysteretic behavior of the specimen isolators during the earthquake.

4.4 Results for the Calitri Accelerogram

The accelerogram of Calitri contained a maximum acceleration of 1.55 m/s^2 and its response spectrum (Fig. 4) showed important excitation content for several frequencies between 0.8 and 6 Hz. Figure 5 shows certain response maximum magnitudes for every floor and for the different amplification factors which were tested for the isolated and non-isolated configurations. The floor absolute accelerations may be used to assess the effects of the earthquake on the contents of the building, while the inter-story drifts and shear loads refer to the integrity of the structure. The drifts at floor 0 for the isolated structure represent the transversal displacement at the isolators. It can be seen how, for every common amplification factor, the response of the non-isolated structure is higher than for the isolated one with a ratio of between 3 and 4 for stresses and deformations. However, the tests performed on the non-isolated structure were extended only to an amplification factor of 0.7 so as not to damage the frame. Said damage, to a certain extent, would have probably contained the level of stresses, but was not desirable for the repeatability of the tests. Even though the Calitri spectrum presents an important peak at 0.8 Hz (Fig. 4) --which could have been dangerous for the isolated building-- the response of the first mode of the non-isolated one is even more ill-conditioned due to a casual tuning of its frequency of about 2 Hz (Table 1 or Fig. 2) with another important peak of the spectrum (Fig. 4).

4.5 Results for the Armenia Accelerogram

This accelerogram contained a maximum acceleration of 2.39 m/s^2 and its response spectrum (Fig. 4) showed several important peaks between 1.2 and 12 Hz. As can be seen in Fig. 5, in this case, the efficiency of isolation is even better than for the Calitri earthquake. This is due to the fact that, in the Armenia earthquake, the first peak of the spectrum appears at a frequency which is 0.4 Hz higher. The ratio between the non-isolated and the isolated structure for stresses and deformations is about 4, within the tested elastic cases.

4.6 Results for the Mexico Accelerogram

The accelerogram of Mexico, with a maximum acceleration of 1.71 m/s^2 and a spectrum which is almost monochromatic at 0.50 Hz (Fig. 6), was clearly an earthquake for which the elected isolated system could improbably have ever been designed since its rigid body mode has a frequency of 0.58 Hz (Table 1). As a matter of fact, the transmissibility function (Fig. 2) shows that, for excitation frequencies around 0.5 Hz, the response for the isolated frame is considerably larger than for the non-isolated one. The test was made for a single amplification factor of 0.5 and the results (Fig. 7) confirmed this prevision, that is to say, the isolated structure exhibited a greater response than the non-isolated one due to the resonance of the aforementioned rigid body mode. It is important to emphasize that, for the chosen isolated structure, this particular accelerogram was not to be expected since it is associated with a particular kind of soil for which the isolation system would have been designed with a lower frequency of resonance. Nevertheless, the execution of the test showed how the PsD technique is objective also for hypothetical cases in which the isolation is inefficient.

4.7 Results for the Kalamata Accelerogram

The definition of the prototype isolator set for the test series of this accelerogram was different from that for the previous accelerograms. Although the same theoretical masses were still used and the height of the rubber bearings was also maintained the same as in previous cases, their transversal section was considered as 2.30 times the section in the specimen. This section ratio was considered simultaneously with the SRE shear-load correction factor in a joint percentage of correction of $(1.20 \cdot 2.30 - 1) \cdot 100 = 176\%$. With this new definition of the isolation, the new estimated frequencies were those specified in the third row of Table 1. This accelerogram, for a wide range of amplifying factors, had been the object excitation on the same specimen in an extensive previous test campaign (Kakaliagos, 1994) for which the column bases were clamped onto the strong floor of the laboratory; so, these available "perfectly-rigid-foundation" data have been taken for comparison with the results for the isolated case instead of performing new non-isolated tests. The Kalamata accelerogram is characterized by a maximum acceleration of 2.62 m/s^2 and a spectrum (Fig. 6) with a predominant peak at 3.2 Hz which was far enough from the resonant frequency of 0.82 Hz (Table 1) of the isolated system so as to guaranty a high efficiency of the isolation. This is corroborated by the results shown in Fig. 7, in which one can see how, even though the damage originated in the non-isolated structure limits the growth of the stresses for the highest amplification factors, the isolated stresses are around three times smaller. One must also notice that, due to the rigid foundation of the non-isolated structure, its stiffness was considerably higher than the internal stiffness of the isolated superstructure, which explains the relatively small value of the inter-story drifts observed in the non-isolated case. However, the non-isolated drifts were found to be still twice the magnitude of the isolated ones.

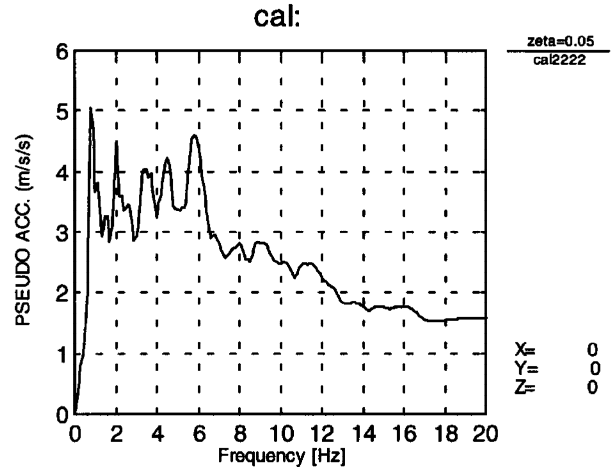

Fig. 4. Pseudo-acceleration response spectra of the Calitri (left) and Armenia (right) accelerograms for a damping ratio of 5%.

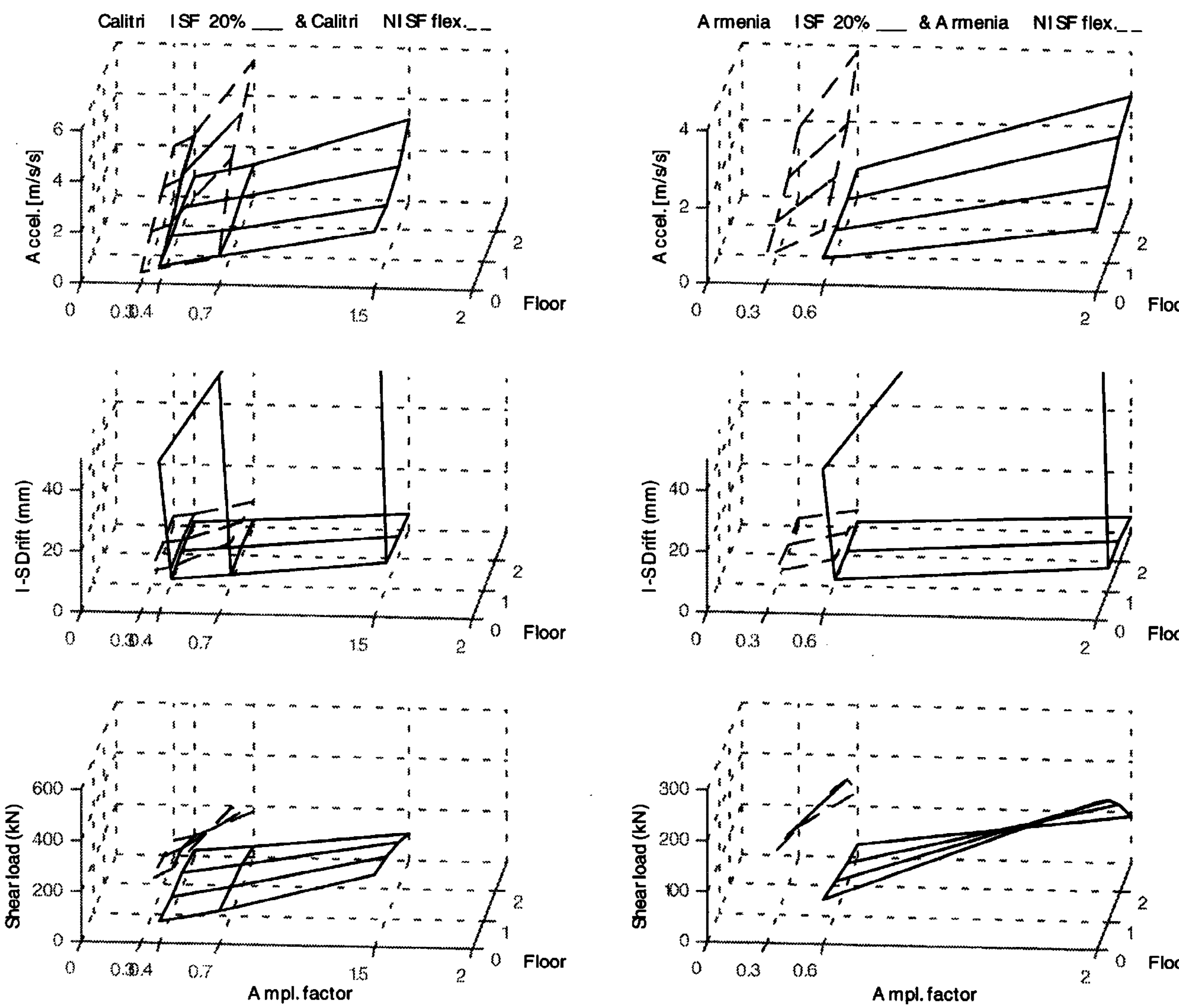

Fig. 5. Comparison of isolated (solid line) and non isolated (dashed line) response to the Calitri (left) and Armenia (right) earthquakes.

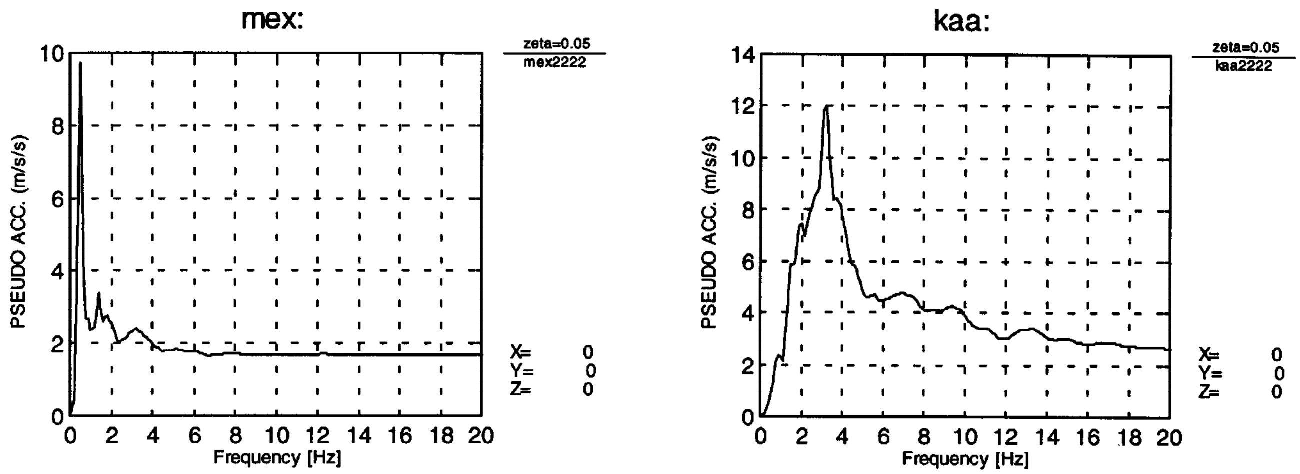

Fig. 6. Pseudo-acceleration response spectra of the Mexico (left) and Kalamata (right) accelerograms for a damping ratio of 5%.

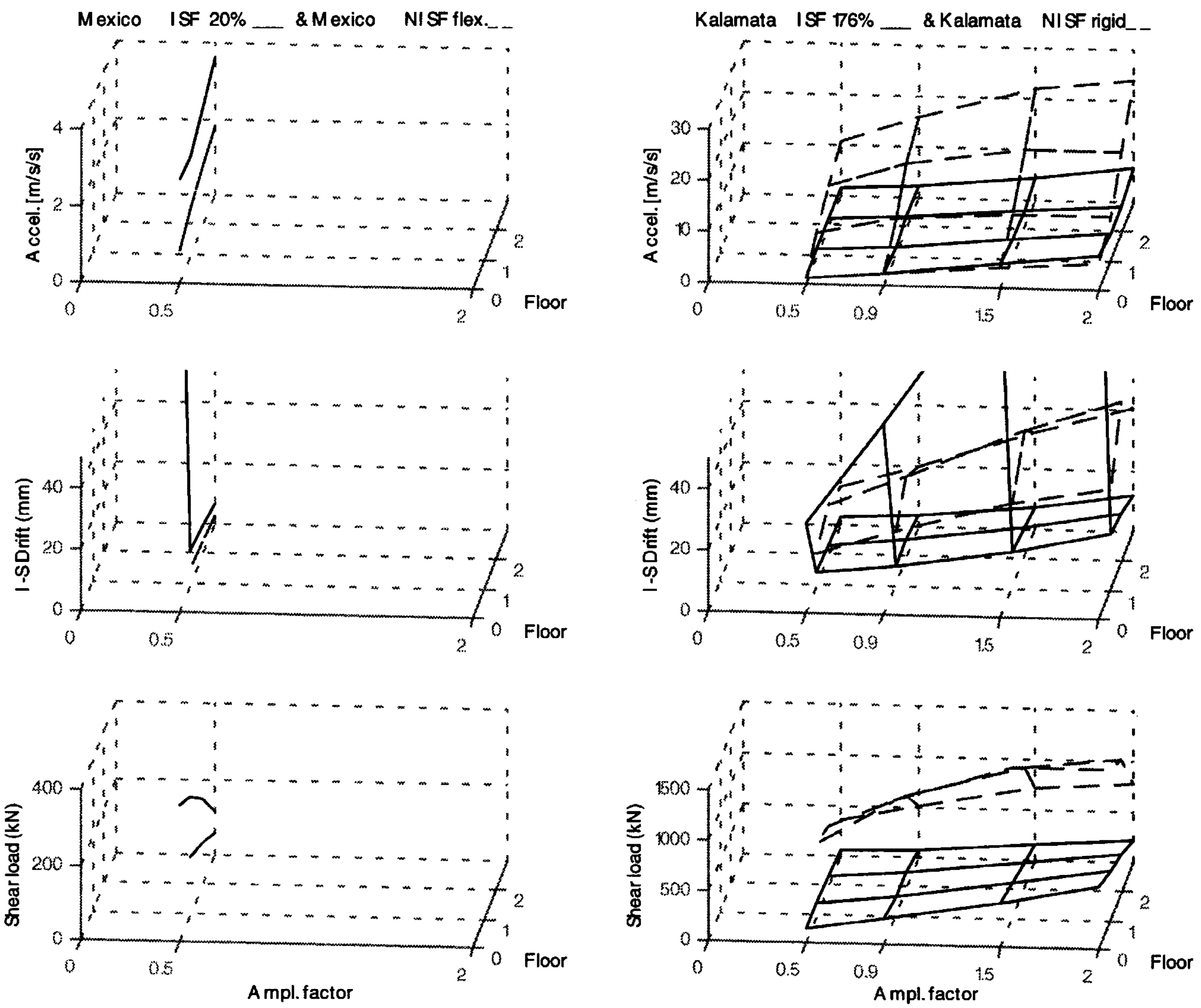

Fig. 7. Comparison of isolated (solid line) and non isolated (dashed line) response to the Mexico (left) and Kalamata (right) earthquakes.

5 CONCLUSIONS

The experimental results presented in this paper show the adequacy of the PsD technique for the testing of large-size isolated structures. For the high-damping rubber bearings used in this test campaign, the time scaling introduced by this technique may give rise to SRE discrepancies of the order of 20% for the stress at the isolators, but using a simple correcting technique, these effects can be largely compensated. The transmissibility functions --in this case experimentally obtained-- for the non-isolated and the isolated structure, although conceptually based on a linear system, turned out to be a very effective tool for the prediction of the efficiency of the adopted base isolation at the different tested earthquake spectra. The response of the isolated structure was in general very good, showing inter-story drifts and shear loads much smaller than the non-isolated ones and floor accelerations which remained very low up to the top of the building. The accelerogram of Mexico, with a predominant frequency of 0.50 Hz, which almost exactly coincided with the resonance frequency of the isolated system, was useful in showing that the objectivity of the PsD method also extends to hypothetical cases of inefficient isolation design.

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