The Role of Soil–Foundation–Structure Interaction on the Performance of an Existing 3-storey Building : Shaking Table Testing

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SUMMARY:
This paper investigates experimentally the seismic performance of an existing building, with emphasis on the effects of nonlinear soil–foundation–structure interaction (SFSI). An idealized 3-storey structure is considered, inspired from the large-scale tests of the SPEAR project. The seismic performance of the original structure is simulated in a first step, confirming its vulnerability. Then, the building is retrofitted with the equivalent of a RC shear wall, following the provisions of modern seismic codes. A reduced-scale physical model of the soil-structure system is tested in the shaking table of the Laboratory of Soil Mechanics of NTUA. It is shown that SFSI may substantially alter the collapse capacity of the structure. Moreover, it is concluded that mobilization of foundation bearing capacity may be beneficial for the performance of the rehabilitated structure, and should therefore be considered in design.

Keywords: soil–structure–interaction, seismic strengthening, shear wall, rocking isolation

1. INTRODUCTION

It has been more than 30 years since the realization that structural damage is inevitable under unexpectedly high levels of seismic attack, and that the increase of strength does not always result in enhanced safety. This recognition lead to the development of modern seismic design principles, which aim at controlling seismic damage rather than to avoid it: *ductility* and *capacity design*. While the first aims at ensuring that critical structural members may sustain loads that exceed their capacity without collapsing, the latter focuses on *guiding* failure to less important structural members (*beams instead of columns*) and to non-brittle mechanisms (*bending instead of shearing*) [Park & Paulay, 1976]. Moreover, understanding that structural damage is more directly related to deformation lead to the development of displacement-based and performance-based design [Bertero, 1996; Calvi, 1999; Priestley, 2000], and to a rather substantial improvement of seismic codes.

Unfortunately, however, most existing structures do not comply with current seismic design provisions. In Greece, for example, about 85% of the building stock dates before 1985, built in accordance with obsolete seismic codes. Their vulnerability has been manifested rather dramatically during devastating earthquakes. Most importantly, even relatively small magnitude earthquakes may cause substantial damage or failure of existing structures. For example, a M_s 5.9 earthquake near Athens (Greece, 1999) lead to 145 fatalities due to collapse of 100 buildings, and damage beyond repair to 13000 buildings [Papadopoulos et al., 2000]. Moreover, the lack of adequate ductility and capacity design is bound to lead to brittle types of failure.

This paper investigates experimentally the seismic performance of an existing building, with emphasis on the effects of nonlinear soil–foundation–structure interaction (SFSI). For this purpose, an idealized 3-storey structure is considered, inspired from the large-scale tests of the SPEAR project [Fardis, 2002; Fardis & Negro, 2006; Di Ludovico, 2007]. A reduced-scale model of the soil-structure system is tested in the shaking table of the Laboratory of Soil Mechanics of NTUA. The seismic performance of the original structure is simulated in a first step, confirming its vulnerability. Then, the building is retrofitted with the equivalent of a RC shear wall, following the provisions of modern seismic codes. In both cases, a variety of real seismic records is used as base excitation.

Two alternatives are considered with respect to the foundation of the shear wall: (a) conventional design, following the provisions of current seismic codes; and (b) rocking isolation. In the latter case, the foundation is intentionally *under-designed* to promote uplifting and fully mobilize its moment capacity, thus acting as "rocking isolation" [Mergos & Kawashima, 2005]. Recent studies have shown that such exploitation of strongly nonlinear foundation response may be beneficial, limiting the inertia transmitted onto the superstructure [Paolucci, 1997; Pecker, 1998; 2003; Gazetas et al., 2003; Gajan et al., 2005; Apostolou et al., 2007; Pender, 2007; Paolucci et al., 2008; Gajan & Kutter, 2008; 2009; Shirato et al. 2008; Vassiliou & Makris, 2011; Panagiotidou et al., 2012]. Allowing such "plastic hinging" at the foundation level may act as an energy dissipation mechanism that bounds the seismic demand, thus providing adequately large safety margins, even for seismic motions that substantially exceed the design limits [Anastasopoulos et al., 2010a; Gelagoti et al., 2012; Kourkoulis et al., 2012].

2. PROBLEM DEFINITION AND EXPERIMENTAL SETUP

The under study structure is a typical 3–storey building of Southern Europe, designed and constructed during the 70's (Figure 1). A representative "slice" of the building is modelled, corresponding to 1/3 of the whole structure. The square columns of the prototype are 25 cm in width, while the beams have a 25 cm x 50 cm (width x height) cross section. Designed and constructed in the 70's, the structure does not comply with capacity design principles and is prone to soft-storey collapse. The foundation consists of square surface foundations of width B = 1.5, considered realistic for competent soil.

Taking account of the capacity of the shaking table, a scale factor N = 10 was selected. The physical model (Figure 2a) consists of two identical frames, connected together through evenly distributed steel plates, also representing the mass of each story (including dead and live loads). The structural members (columns and beams) are made of commercially available aluminium plates of appropriate thickness and width, so as to maintain similarity in terms of stiffness [Gibson, 1997]. At reduced-scale, it is practically impossible to model stiffness correctly (maintaining similarity) *and* achieve the desired (*scaled*) bending moment capacity of the structural members at the same time. For this purpose, each beam-column connection is modelled with custom-built artificial plastic hinges (Figures 2b and 2c). The ultimate bending moment M_{ult} of each plastic hinge is calibrated through adjustment of the applied torque. The calibration of each assembly was performed through static and slow-cyclic pushover testing, utilizing a screw-jack pushover apparatus. Multiple tests were conducted for each artificial plastic hinge, in order to verify that their moment capacity is not altered after multiple loading cycles.

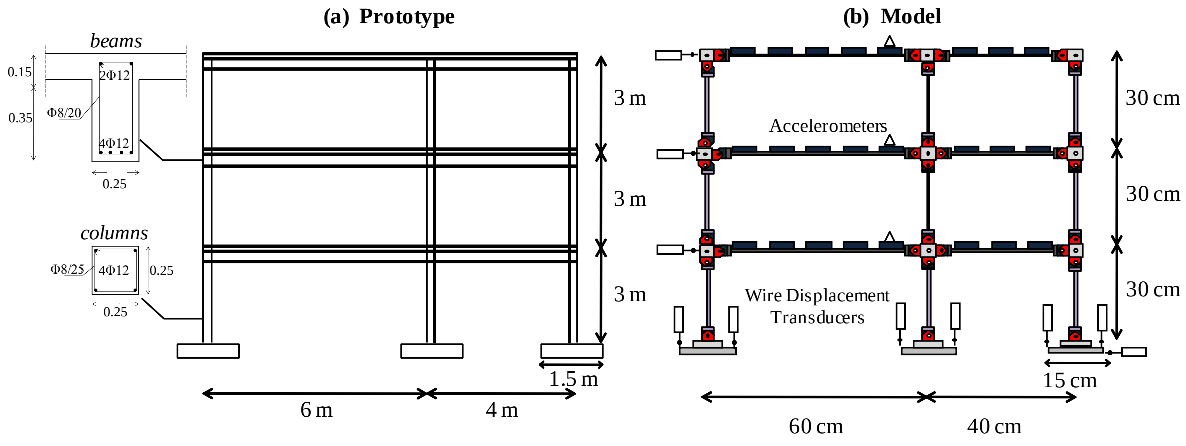

Figure 1. The under-study 3-storey building, inspired by the SPEAR building: (a) prototype, and (b) model.

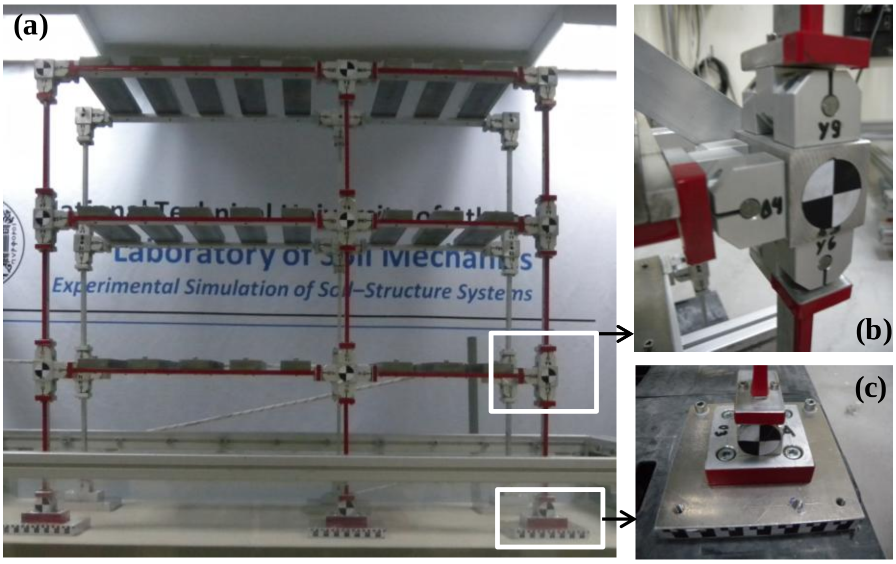

Figure 2. (a) Photo of the physical model of the original building; (b) assembly of first floor artificial plastic hinges; and (c) artificial plastic hinge at the base of the first-floor column.

The physical models of the building were installed inside a transparent soil container. The soil consists of dry “Longstone” sand, a very fine industrially-produced uniform quartz sand having a mean grain size d_{50} = 0.15 mm [Anastasopoulos et al., 2010b]. The sand specimens were prepared through dry pluviation using an in-house custom-built automated sand raining system. The density of the sand depends on the raining height and velocity, and the aperture of the soil hopper. The system is capable of achieving relative densities D_r ranging from 10 to 93%, ensuring repeatability. In the tests reported herein, three different densities were investigated: (a) D_r = 93 % (dense); (b) D_r = 65 % (medium-dense); and D_r = 45 % (loose).

The physical model of the building was installed on the soil by means of four mechanical jacks. Special care was taken during installation so as to achieve accurate positioning without disturbing the soil surface. Electronic spirit-levels were used to ensure that the building was placed horizontally on the soil surface without initial inclination. With the exception of accelerometers placed inside the soil mass, the instrumentation was installed afterwards (Figure 1b). Floor accelerations were measured by three accelerometers, one on each story. Wire displacement transducers were used to measure inter-storey drifts, and rotations and sliding displacements of the footings.

After testing the original structure, the equivalent of a RC shear wall was added to the model to simulate the performance of the retrofitted structure. The shear wall was modeled by a stiff aluminum plate, rigidly connected on each floor, and equipped with an artificial plastic hinge at its base. The original footing of central column was increased in width by rigidly connecting additional aluminum plates at both of its edges. With respect to the width of the shear wall footing different alternatives were tested, ranging (in prototype scale) from B = 6 m, corresponding to conventional design, to B = 2 m, for the rocking-isolated alternative.

The investigated soil–foundation–structure systems were subjected to a variety of seismic motions, including real records and artificial (sinusoidal motions) motions. Moderate intensity seismic records from Greece were utilized for the original (un-retrofitted) structure (Figure 3a). As discussed in the sequel, the original building was found incapable of surviving stronger seismic motions, sustaining soft-storey collapse when subjected to the Lefkada 2003 record. The retrofitted structure was also subjected to these records, but also to strong (Sakarya, Kocaeli 1999) and very strong seismic motions: Northridge 1994–Rinaldi; Kobe 1995–JMA and Takatori (Figure 3b). The latter exceed substantially the design limits of the retrofit, and were investigated to explore the margins of safety of different foundation design alternatives. Each system was subjected to various sequences of seismic motions.

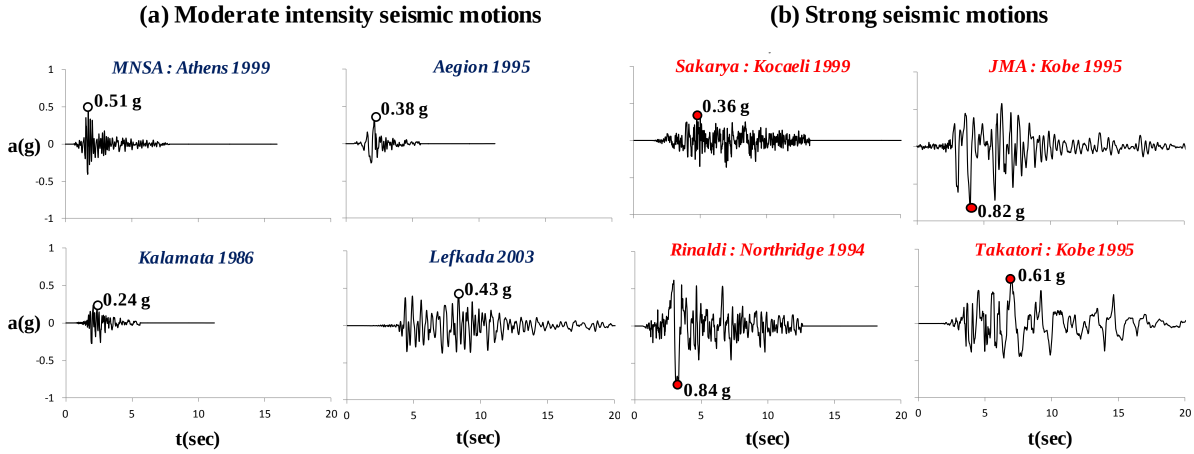

Figure 3. Real seismic records used as seismic excitation in the tests: (a) moderate intensity seismic motions recorded in Greece; and (b) strong to very strong seismic motions.

3. PERFORMANCE OF THE ORIGINAL BUILDING

The original structure was initially tested to confirm its seismic vulnerability. Unless otherwise stated, the results are presented in prototype scale. The shaking sequence started with the high-frequency MNSA record from the Athens 1999 earthquake, followed by Aegion 1995, Kalamata 1986, and Lefkada 2003. The un-retrofitted structure survived the first three seismic excitations, collapsing during the fourth one.

Figure 4a summarizes the performance of the building for the Aegion 1995 and Lefkada 2003 seismic excitations. Both are of moderate intensity, exceeding however, the capacity of the structure. The latter has a pseudo-static capacity of the order of 0.13 g, in accord with the SPEAR building [Di Ludovico, 2007]. When subjected to the Aegion 1995 seismic excitation, the maximum inter-storey drift ratio Δ reaches 1.5% in the first floor, while the residual is very close to roughly 1%. The corresponding drift ratios of the second and third floor are substantially lower, revealing that plastic deformation is localized in the first floor columns.

The un-retrofitted structure finally collapses when subjected to the Lefkada 2003 record. As revealed by the time history of inter-storey drift ratio Δ, the mechanism is clearly that of soft-storey collapse. Observe the abrupt increase of the first story Δ at $t \approx 5$ sec. With initiation of the collapse mechanism, the structure moves laterally accumulating huge amounts of Δ at the first (soft) story. At $t \approx 6$ sec, the first story drift ratio has reached $\Delta > 20\%$ and the collapse mechanism is quite evident in the snapshot of Figure 4b. A little later, the first storey collides on the stopper (installed to avoid model and instrument damage). After this point, the drift ratio of the first floor cannot increase further. Due to their inertia, the two overlying storeys keep moving and accumulating drift, until finally colliding on the stopper as well. In all cases examined, the response of the footings was practically elastic, in accord with conventional capacity design principles (the foundation has to be stronger than the column). As a result, the settlement and rotation of all footings was practically negligible.

In accord with the SPEAR project, it is concluded that the original (un-retrofitted) structure is insufficient in terms of strength and ductility, being unable to survive even seismic motions of (relatively) moderate intensity. This conclusion is not only consistent with the SPEAR test results, confirming the equivalence of the reduced-scale model tested herein, but also compares well with reality: many such buildings sustained major damage or collapsed during the aforementioned (M ≈ 6) earthquakes in Greece. Retrofitting is therefore considered necessary, in order to increase its seismic resistance and increase the safety margins against collapse.

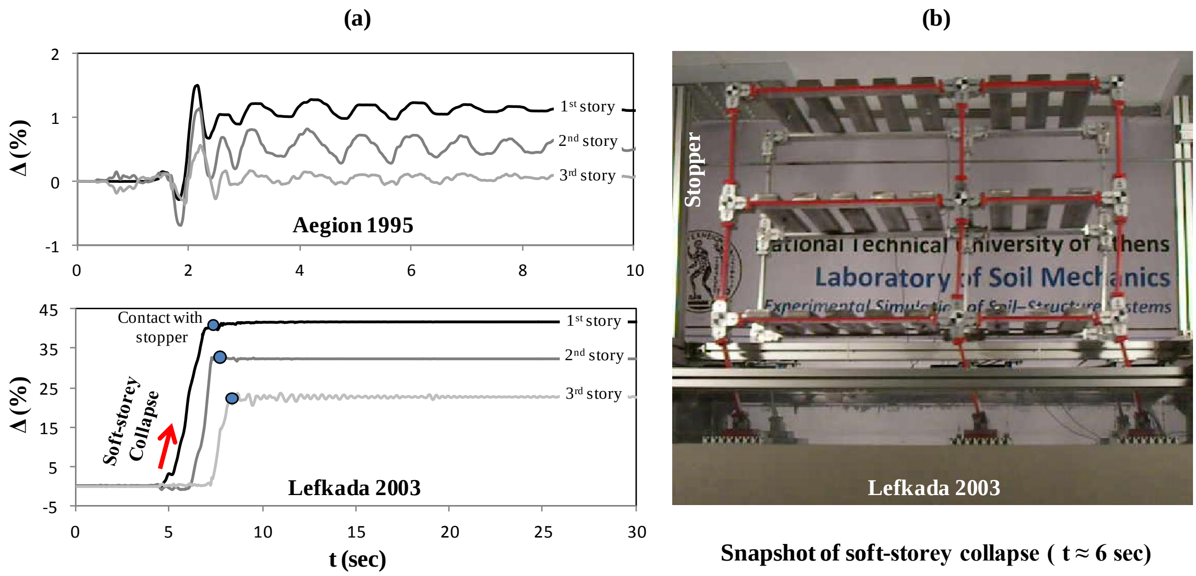

Figure 4. Performance of the original building subjected to moderate intensity seismic records from Greece: (a) time histories of inter-storey drift Δ for the Aegion 1995 and the Lefkada 2003 seismic excitations; and (b) snapshot of soft-storey collapse during the Lefkada 2003 seismic excitation.

4. PERFORMANCE OF THE RETROFITTED STRUCTURE

As previously mentioned, the building is retrofitted through addition of a RC shear wall. The design of the retrofit was conducted following the provisions of the relevant Greek Regulation [KAN.EPE, 2009]. The RC wall is positioned in front of the middle column of the frame, having an eccentricity towards the 6 m span (for practical purposes only). A design coefficient A = 0.24 g is assumed as the retrofit target, yielding design acceleration Φ_d = 0.20 g assuming a behavior factor q = 3. Besides from the increase in strength and ductility, the addition of the shear wall will homogenize the lateral deformation of the structure (acting as a kinematic constraint), leading to a more uniform damage distribution in all three storeys and prohibiting the development of a soft-storey collapse mechanism.

4.1. Confirmation of retrofit effectiveness

To confirm the effectiveness of the retrofit, the rehabilitated structure with conventionally *over-designed* B = 6 m foundation is subjected to the sequence of moderate intensity seismic motions of Figure 3a. As expected, the performance of the retrofitted structure is improved substantially. As depicted in Figure 5a, the deformation of the structure is forced to follow that of the shear wall. As a result, the drift is evenly distributed between the three stories and almost no difference can be observed in the time histories of inter-storey drift Δ.

When subjected to the Aegion 1995 seismic excitation, the maximum inter-storey drift ratio Δ merely exceeds 1% (evenly distributed on all three storeys), while the residual is practically equal to 0%. This implies that the building sustained negligible damage during this seismic excitation, confirming the effectiveness of the retrofit. It is reminded that the original structure sustained substantial permanent deformation during this seismic excitation (see Figure 4a).

In contrast to the original structure which collapsed when subjected to the Lefkada 2003 record, the retrofitted building survives almost unscathed: the residual drift ratio is of the order of 1%. At the end of the Lefkada 2003 seismic excitation, the deformation of the structure is not easily observable (Figure 5b). As with the original structure, the response of the conventionally-designed foundations is practically elastic, with minimal settlement and rotation. It is concluded that the performance of the retrofitted structure is totally consistent with its design, confirming the effectiveness of the retrofit.

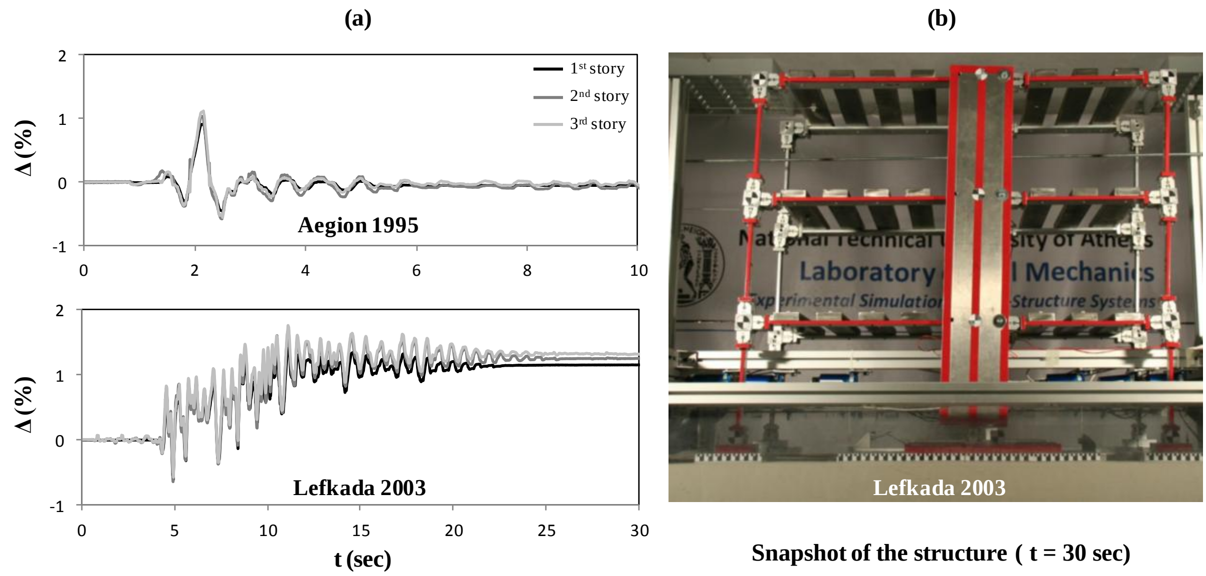

Figure 5. Performance of the retrofitted building subjected to moderate intensity seismic records from Greece: (a) time histories of inter-storey drift Δ for the Aegion 1995 and the Lefkada 2003 seismic excitations; and (b) snapshot of the structure at the end of the Lefkada 2003 seismic excitation.

4.2. Performance in seismic motions exceeding the design: conventional vs. rocking–isolation

The performance of the retrofitted structure when subjected to very strong seismic shaking, substantially exceeding the design, is explored utilizing the strong seismic motions of Figure 3b. The performance of the conventionally–designed structure with a large *over–designed* B = 6 m foundation is compared to a rocking–isolation alternative, having a substantially *under–designed* B = 2 m foundation. The Rinaldi record from the devastating 1994 M_s 6.7 Northridge earthquake is utilized herein as an illustrative example of such very strong seismic shaking, substantially exceeding the design limits.

As illustrated in Figure 6, the differences in the performance of the two design alternatives are quite pronounced. As revealed by the time history of inter-storey drift ratio Δ, the conventionally-designed system sustains major damage (Figure 6a): the residual Δ reaches almost 8%. Observe that, in accord to its design principles, the drift is purely due to the flexural distortion of the shear wall, or to be more precise to the plastic rotation taking place within the artificial plastic hinge at the base of the shear wall. Even under such strong seismic shaking, the rotation and settlement of the *over–designed* B = 6 m foundation are practically negligible: the residual settlement does not exceed 0.7 cm.

As observed in the time history of inter-storey drift ratio Δ (Figure 6b), the performance of the rocking–isolated alternative is markedly different: the residual Δ is practically equal to 0%, while the maximum observed does not exceed 4%. In contrast to the conventionally-designed structure, most of the drift is now due to foundation rotation, with the flexural distortion (i.e., the plastic rotation of the artificial plastic hinge at the base of the shear wall) being only minor. Evidently, the performance of the rocking-isolated structure is superior, with the slightly increased settlement being the only price to pay: w_{res} = 1.3 cm instead of 0.7 cm of the conventional system. It is apparent that the rocking foundation acts as a fuse, preventing damage on the shear wall. In addition, thanks to its self-centering attributes (driven purely by gravity), the residual drift is also minimized.

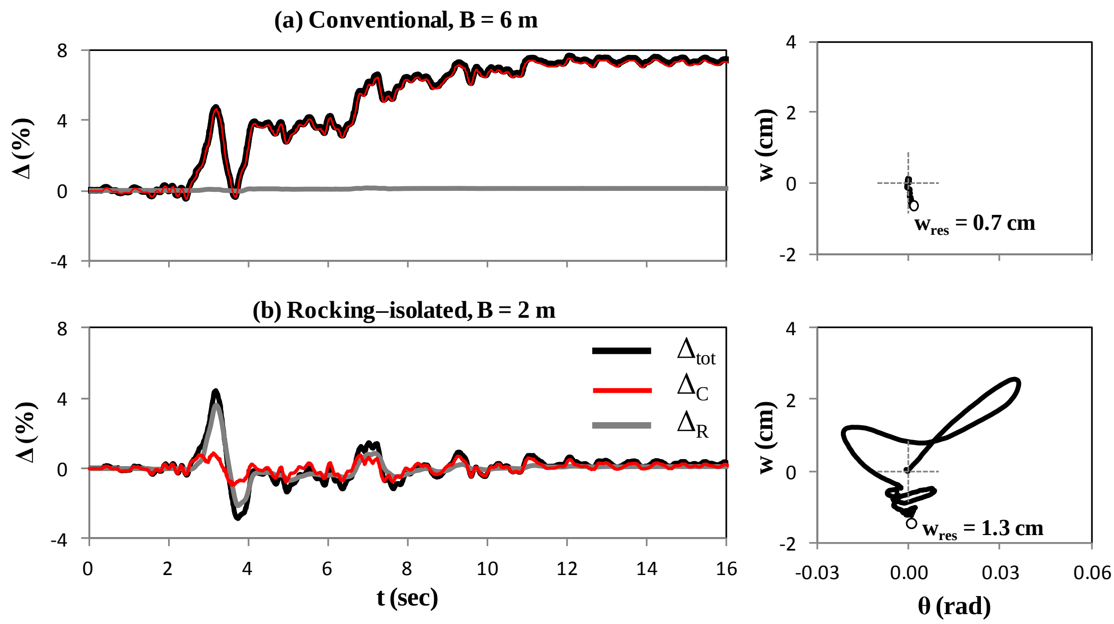

Figure 6. Performance of the retrofitted structure subjected to a seismic motion substantially exceeding the design (Rinaldi). Time histories of inter-storey drift ratio Δ measured on the shear wall (total Δ_{tot} , due to rotation Δ_R, and due to flexural distortion Δ_C) and settlement–rotation (w–θ) response for: (a) conventionally–designed B = 6m foundation, and (b) rocking-isolated structure with B = 2m foundation.

5. CONCLUSIONS

Figure 7 summarizes the experimental results in terms of distribution with height of the residual inter-storey drift ratio Δ_{res}. The main conclusions of the study presented herein can be summarized as follows:

- Designed in the 70's, in accordance with obsolete seismic codes, the original 3-storey building cannot withstand seismic motions even of moderate intensity (Figure 7a). It is found capable of surviving the Kalamata and Aegion seismic motions, but collapses when subjected to the Lefkada 2003 record. Besides from having inadequate strength and ductility, the lack of capacity design (*weak columns–strong beams*) leads to the development of a soft-storey mechanism and collapse.

- Retrofitting by addition of a shear wall is proven quite effective, leading to a substantial increase of strength and ductility, but also to homogenization of deformation and evenly distributed inter-storey drifts on all floors (Figure 7b). The retrofitted structure with conventionally *over-designed* B = 6 m foundation is proven capable of withstanding all moderate intensity seismic motions with minimal damage. When subjected to very strong seismic motions substantially exceeding its design limits, it is bound to severe damage or collapse: the residual drift Δ_{res} reaches about 7% for Kobe JMA, increasing to 10% for Rinaldi, and to roughly 14% for Takatori.

- The rocking–isolated retrofit alternative with (substantially) *under-designed* B = 2 m foundation is equally successful in moderate intensity seismic motions (Figure 7c). Its advantageous performance is revealed when subjected to very strong seismic motions substantially exceeding the design limits. In stark contrast to the conventionally retrofitted structure, it is found capable of surviving with minimal to severe damage. The residual drift Δ_{res} is almost 0% for Kobe JMA, increasing to about 2% for Rinaldi, and to roughly 8% for the devastating Takatori record. Evidently, due to its inherent self-centering characteristics, rocking isolation allows the superstructure to return to its initial position even after such strong seismic excitations. As previously discussed, increased settlement is the price to pay, which however, can be tolerable.

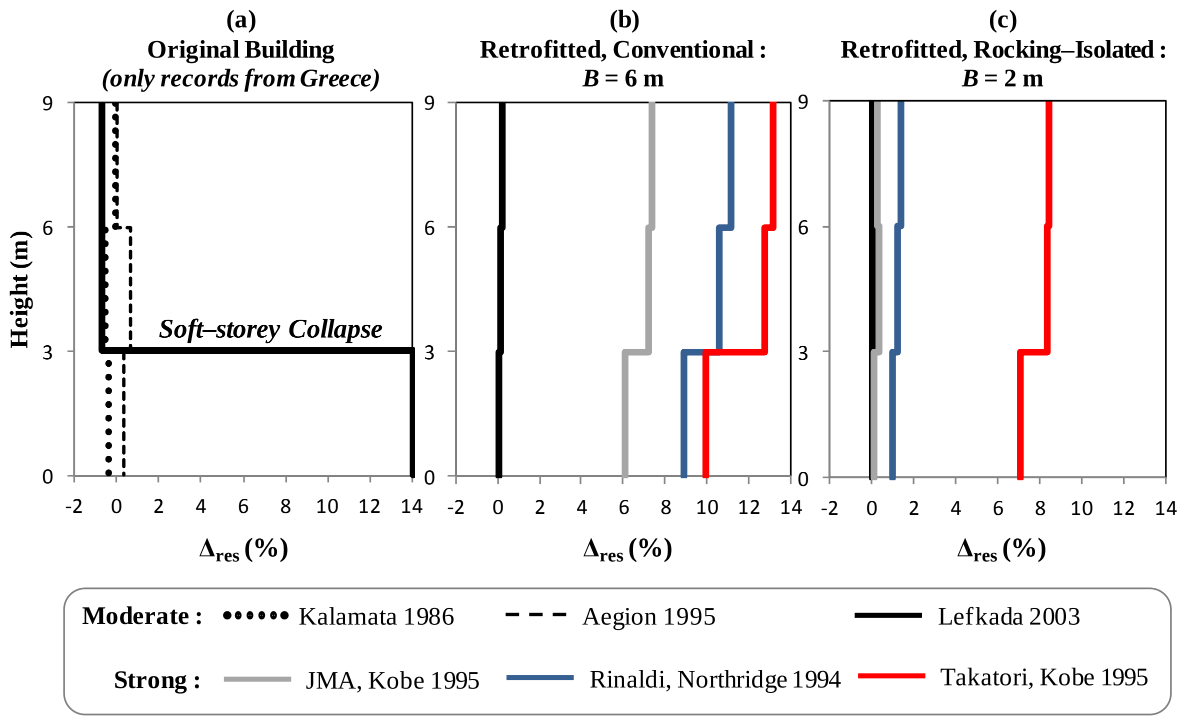

Figure 7. Synopsis of experimental results. Distribution with height of the residual inter-storey drift ratio Δ_{res} for: (a) the original building, (b) the retrofitted structure with conventionally–designed B = 6 m foundation, and (c) the rocking-isolated retrofitted structure with B = 2 m foundation.

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