Shaking Table Testing of Retrofitted 3-storey Building

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ABSTRACT: The paper investigates the seismic performance of an existing 3–storey structure, built in the 70's. Not complying with capacity design principles, the structure is prone to soft-storey collapse, calling for retrofit through addition of shear walls. Two alternatives are considered with respect to the foundation of the latter: (a) conventional design; and (b) rocking isolation. In the latter case, the foundation is intentionally "under-designed" to fully mobilize its capacity acting as a "fuse". A reduced-scale model of the soil–structure system is tested in the shaking table of the Laboratory of Soil Mechanics. At reduced-scale, it is practically impossible to maintaining similarity in terms of stiffness, and achieve the desired bending moment capacity of structural members at the same time. Therefore, each beam–column connection is modeled with artificial plastic hinges. It is shown that the rocking–isolated structure outperforms the conventional, when subjected to very strong seismic shaking.

1 INTRODUCTION

Modern seismic design principles aim at controlling seismic damage, rather than to avoid it. Ductility design aims at ensuring that critical structural members can sustain accidental loads (such as from earthquake) without collapsing, while capacity design is targeted at guiding failure to non-brittle mechanisms and to less important structural members [Park & Paulay, 1976]. Unfortunately, most existing structures do not comply with current seismic design codes. Their vulnerability has been manifested during devastating seismic events. Most importantly, the lack of adequate ductility and capacity design is bound to lead to brittle types of failure.

This paper presents an experimental study of the seismic performance of an existing structure, retrofitted through the addition of shear walls. Focusing on the performance of the foundation of the shear walls, two design alternatives are considered: (a) conventional design, as entrenched in current seismic codes; and (b) rocking isolation. According to the latter, the foundation is intentionally under-designed to fully mobilize its moment capacity, acting as a "fuse" [e.g., Mergos & Kawashima, 2005]. Recent research efforts have shown that such strongly nonlinear foundation response may be beneficial, limiting the inertia forces 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]. Guiding the "plastic hinge" at the foundation can effectively bound the seismic demand and act as an energy dissipation mechanism, leading to an appreciable increase of safety margins [e.g., Anastasopoulos et al., 2010; Gelagoti et al., 2012].

2 DESCRIPTION OF THE PROTOTYPE

An idealized structure is considered as conceptual prototype, inspired from the real-scale building that was tested during the SPEAR project [Fardis & Negro, 2006; Di Ludovico, 2007]. As schematically illustrated in Fig. 1, the under study structure is a typical 3–storey building of Southern Europe, designed and constructed during the 70's.

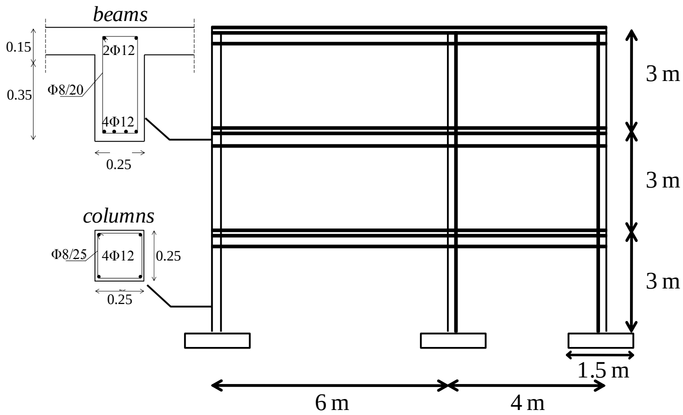

Figure 1. Prototype structure, inspired by the SPEAR project.

A representative "slice" is modeled, corresponding to one third of the entire structure. The reinforced concrete (RC) columns of the prototype are of square cross-section, having a width of 25 cm. The RC beams have a 25 cm x 50 cm (width x height) cross section, while the floor slabs have a thickness of 15 cm. 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 surface foundations of width B = 1.5 m, considered realistic for competent soil.

The structure is retrofitted through addition of RC shear walls, placed at the middle of the building. The latter were designed according to EAK [2000] and the provisions for rehabilitation and strengthening of existing structures. As shown in Fig. 2, having set as the retrofit target a design acceleration a = 0.24 g, the RC shear walls have a length of 1.5 m and a thickness of 0.3 m. The steel reinforcement is arranged following the logic of two "hidden" columns. The longitudinal reinforcement of each one of the two "hidden" columns is 6*Φ*18, while the transverse reinforcement is *Φ*10/10 cm. Additional reinforcement is required for the main core of the shear wall, as shown in the figure.

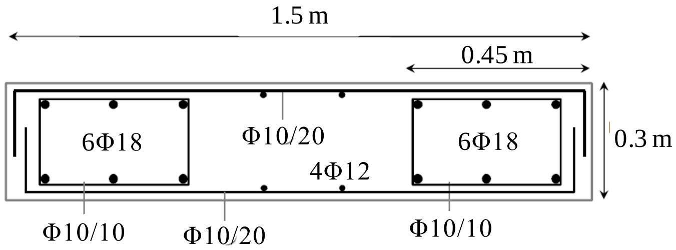

Figure 2. Plan view of the RC shear walls of the prototype.

3 PHYSICAL MODEL

A reduced-scale model of the soil–foundation–structure system was tested at the shaking table of the NTUA Laboratory of Soil Mechanics. Initially, the seismic performance of the original structure was investigated to confirming its seismic vulnerability. Then, the model of the retrofitted structure was subjected to a variety of seismic shaking scenarios, using real seismic records as base excitation.

Given the capacity of the shaking table, a scale factor N = 10 was selected. As illustrated in Fig. 3, the physical model consists of two identical frames, connected together through evenly distributed steel plates, representing the mass of each floor. Columns and beams are made of commercially available aluminium plates of appropriate thickness and width, so as to maintain similarity in terms of stiffness. At reduced-scale, it is practically impossible to maintain similarity in terms of stiffness, *and* achieve the *scaled* bending moment capacity of the structural members at the same time.

Figure 3. Photo of the reduced-scale (N =10) physical model.

For this purpose, each beam-column connection is modelled with custom-built artificial plastic hinges (Fig. 4). The ultimate bending moment M_{ult} of each plastic hinge was 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 push-over 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.

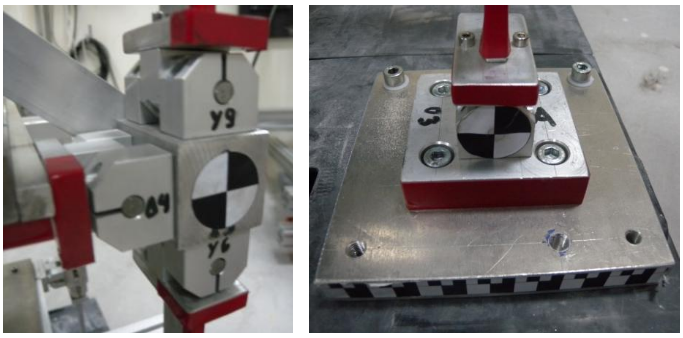

Figure 4. Close up of beam-column (left) and foundation-column connection (right), showing the artificial plastic hinges.

4 SETUP AND INSTRUMENTATION

The model was installed in a transparent soil container, filled with dry Longstone sand. The latter is an industrially-produced very fine quartz sand, having mean grain size d_{50} = 0.15 mm [Anastasopoulos et al., 2010b]. The sand was pluviated using an automated sand raining system, capable of achieving controllable (and repeatable) relative density D_r ranging from 10 to 93%. Three different soil densities were tested, ranging from D_r = 93 % (dense sand) to D_r = 45 % (loose sand).

The model was installed on top of the soil using four mechanical jacks. Substantial effort was made to achieve accurate positioning (using digital spirit levels) without disturbing the soil. As depicted in Fig. 5, floor accelerations were measured by three accelerometers, while additional sensors were placed inside the soil to measure ground accelerations. SpaceAge wire displacement transducers were utilized to measure floor displacements and inter-storey drifts, as well as rotations and sliding displacements of the footings. With the exception of the accelerometers that were placed inside the soil mass, the instrumentation was installed installing the building on top of the soil specimen.

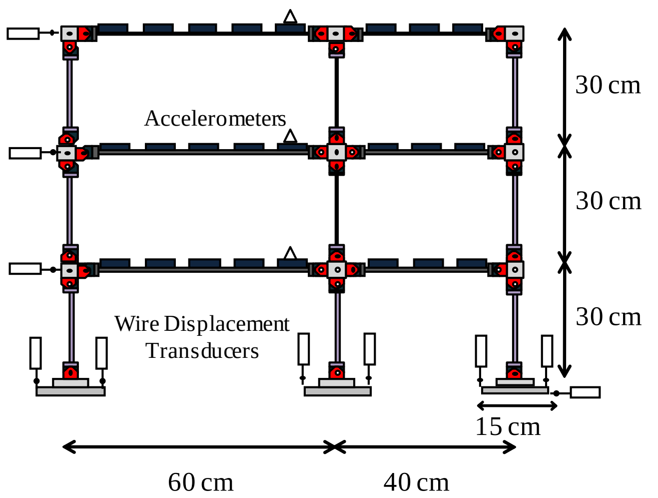

Figure 5. Instrumentation of the physical model.

Initially, the original building was tested to confirm its vulnerability. At a second step, the equivalent of a RC shear wall was added to simulate the retrofitted structure. The shear wall was modeled by a stiff aluminum plate, rigidly connected on each floor. An artificial plastic hinge was installed at its base. Aluminum plates were added at the two sides of the central footing to increase its width, modeling the foundation of the shear wall. With respect to the latter, its width was varied from B = 6 m (in prototype scale), corresponding to conventional design, to B = 2 m, representing rocking isolation.

A variety of real seismic records were used as seismic excitation. Records from Greece (Fig. 6a) were used to simulate moderate seismic shaking, and were mainly used to confirm the vulnerability of the original structure. The same records were used to verify the effectiveness of the retrofit for seismic motions not exceeding the design assumptions. In addition, strong (Sakarya, Kocaeli 1999) to very strong seismic motions (Northridge 1994–Rinaldi; Kobe 1995–JMA and Takatori) were also used to comparatively assess the performance of the two retrofit alternatives (Fig. 6b). The latter substantially exceed the design, and were investigated to explore the margins of safety. Each system was subjected to various sequences of seismic motions.

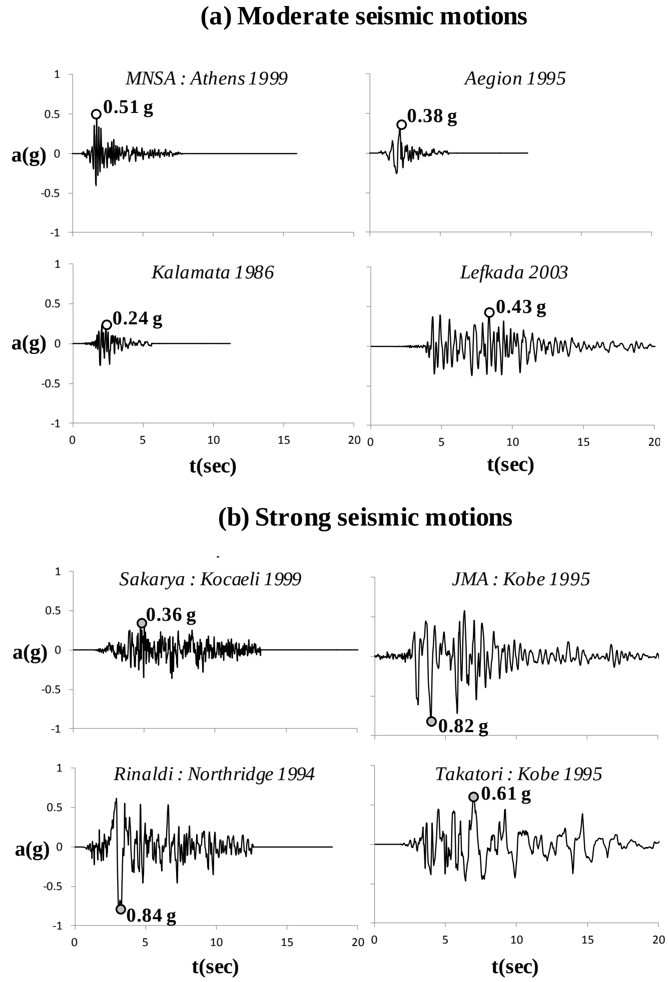

Figure 6. Seismic excitations used in the tests: (a) moderate; and (b) strong to very strong seismic shaking.

5 VULNERABILITY OF ORIGINAL BUILDING

The seismic vulnerability of the original structure was confirmed through seismic shaking with moderate intensity seismic excitations (Fig. 6a). The shaking sequence started with the MNSA record from the Athens 1999 earthquake, followed by Aegion 1995, Kalamata 1986, and Lefkada 2003. The original building survived the first three excitations, collapsing when subjected to the Lefkada 2003 record.

Fig. 7 summarizes the performance of the building for the Aegion 1995 and Lefkada 2003 seismic excitations. Despite being of moderate intensity, both seismic excitations exceed the capacity of the structure. The latter has a pseudo-static capacity of the order of 0.13 g, in accord with the structure that was tested in SPEAR. 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 roughly 1%. The drift ratios of upper floors are much lower, revealing that plastic deformation is localized in the first floor.

The building collapses when subjected to the Lefkada 2003 record. The mechanism is clearly that of soft-storey collapse, as revealed by the time histories of drift. The abrupt increase of the first story drift at $t \approx 5$ sec is due to the initiation of the collapse mechanism. The structure accumulates excessive amounts of drift at the first floor, and at $t \approx 6$ sec δ surpasses 20% (see snapshot of Fig. 8). Just a little later, the first storey collides on the stopper, and the drift ratio of the first floor cannot increase further. The two overlying floors keep moving, until finally colliding on the stopper as well. In all cases examined, the settlement and rotation of all footings was practically negligible, in accord with conventional capacity design.

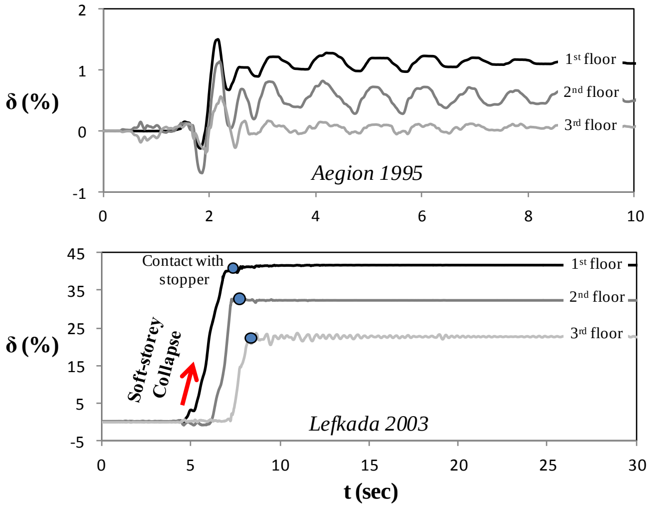

Figure 7. Time histories of drift ratio of the original building.

Based on the experimental results, it is concluded that the un-retrofitted structure is insufficient in terms of strength and ductility, not being able to survive even moderate seismic shaking. Such a conclusion is not only consistent with the results of the SPEAR project, but also serves as a confirmation of the equivalence of the developed reduced-scale model. In order to reduce its seismic vulnerability, retrofitting is considered necessary.

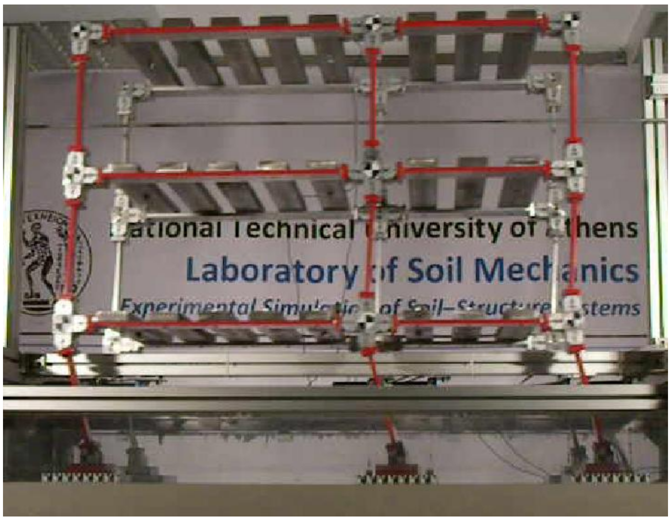

Figure 8. Snapshot of the original building subjected to the Lefkada 2003 record (t = 6 sec).

6 PERFORMANCE OF THE RETROFITTED STRUCTURE

The design of the retrofit was conducted following the provisions of the relevant Greek Regulation [KAN.EPE, 2009]. As previously mentioned, the RC wall is positioned in front of the middle column of the frame. A design coefficient A = 0.24 g is set as the retrofit target, yielding design acceleration Φ_d = 0.20 g assuming behavior factor q = 3. Besides from increasing strength and enhancing ductility, the shear walls will homogenize the drifts, leading to more uniform damage distribution and prohibiting soft-storey collapse.

6.1 *Effectiveness of the Retrofit*

The conventionally retrofitted structure (with B = 6 m foundation) was subjected to the sequence of moderate intensity seismic motions of Fig. 6a in order to verify the effectiveness of the retrofit. As illustrated in Fig. 9, the structure is forced to follow the deformation of the shear walls. In contrast to the original building, the drift is evenly distributed and almost no difference can be observed between the three floors.

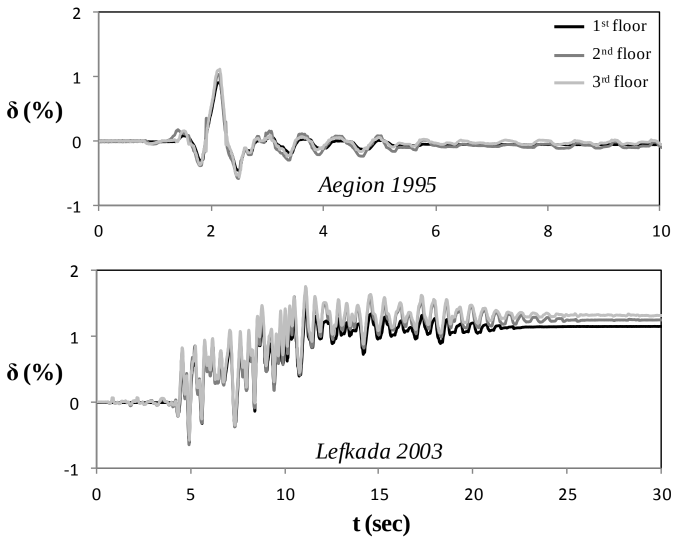

Figure 9. Performance of the conventionally retrofitted structure subjected to moderate seismic shaking. Time histories of inter-storey drift ratio δ for the Aegion 1995 and the Lefkada 2003 seismic excitations.

In the case of the Aegion 1995 seismic excitation, the maximum δ merely exceeds 1%, while the residual is almost 0%. Observe that due to the kinematic constraint that is provide by the shear walls, the drift is evenly distributed on all three storeys. The damage of the structure can only be characterized as negligible, confirming the effectiveness of the retrofit.

In contrast to the original structure, the retrofitted building sustains the Lefkada 2003 seismic record with minor damage: the residual drift ratio is of the order of 1%. At the end of the seismic excitation, the deformation of the structure is not easily observable (Fig. 10). The performance of the retrofitted structure is totally consistent with its design assumptions, confirming the effectiveness of the retrofit. The response of the conventionally-designed foundations is practically elastic, with negligible settlement and rotation.

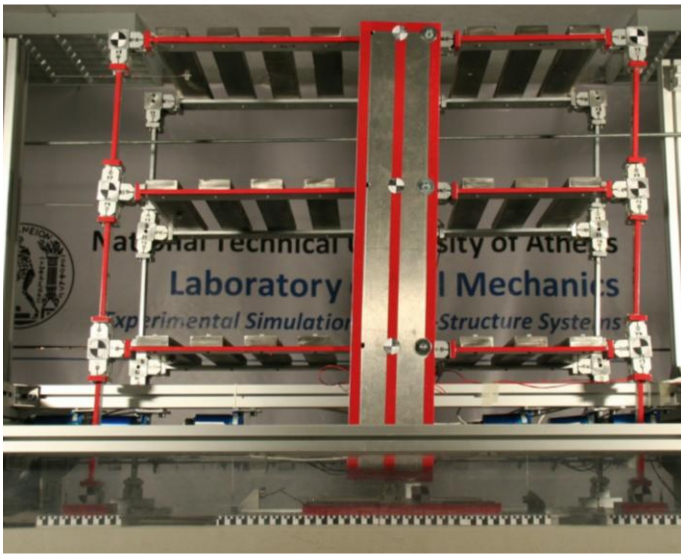

Figure 10. Snapshot of the conventionally retrofitted structure subjected to the Lefkada 2003 seismic excitation.

6.2 *Conventional design vs. rocking isolation*

The performance of the two design alternative subjected to very strong seismic shaking, substantially exceeding the design, is explored utilizing the seismic records of Fig. 6b. The conventionally–designed structure, equipped with a large B = 6 m foundation, is compared to the rocking–isolation alternative. Rocking isolation is achieved by under–designing the foundations of the shear walls, substantially reducing their width to B = 2 m foundation. The two systems are compared, using the Rinaldi record from the devastating 1994 Ms 6.7 Northridge earthquake as an illustrative example.

Fig. 11 compares the performance of the two design alternatives in terms of inter-storey drift ratio δ. The differences are quite pronounced. As revealed by the time histories of δ, the conventionally-designed system sustains severe damage, with the residual δ reaching almost 8%. Observe that the drift is almost purely due to flexural distortion of the shear wall, or to be more precise to the plastic rotation taking place within the artificial plastic hinge. As shown in Fig. 12, even under such strong seismic shaking, the rotation and settlement of the conventional B = 6 m foundation are almost negligible.

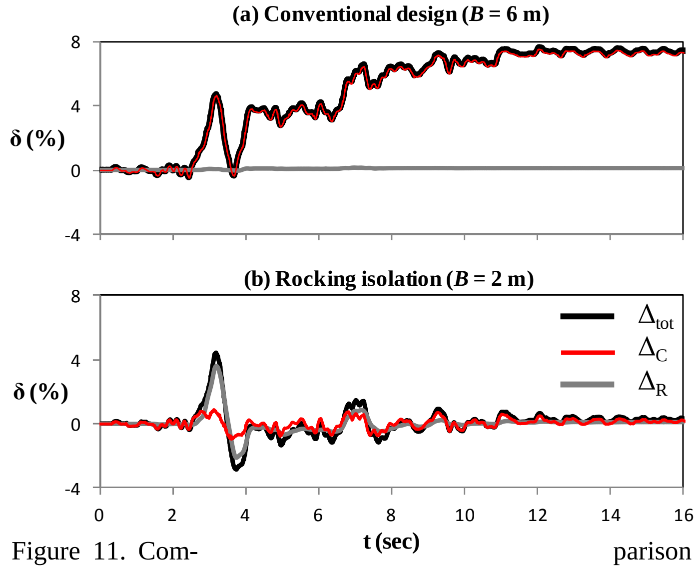

Figure 11. Comparison of conventional design with rocking isolation for strong seismic shaking (Rinaldi). Time histories of drift ratio δ (total δ_{tot}, rotational δ_R, and flexural δ_C) for: (a) conventional design; and (b) rocking isolation.

The performance of the rocking–isolated alternative is superior in terms of drift. The residual δ is practically equal to 0%, while the maximum does not exceed 4%. In contrast to the conventionally-designed structure, most of the drift is associated with foundation rotation, with the flexural distortion component being minor. The performance of the rocking-isolated structure is superior, with the slightly increased settlement being the only downside (Fig. 12). The under-designed B = 2 m foundation acts like 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.

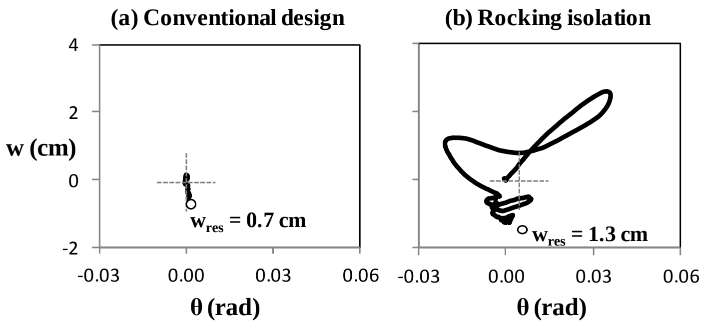

Figure 12. Comparison of conventional design with rocking isolation for strong seismic shaking (Rinaldi seismic excitation). Settlement-rotation response for: (a) conventional design (B = 6m foundation); and (b) rocking isolation (B = 2m).

7 SUMMARY AND CONCLUSIONS

This paper has presented an experimental study of the seismic performance of a 3-storey building, retrofitted with RC shear walls. Designed in the 70’s, in accordance with obsolete seismic codes, the original 3-storey building cannot sustain seismic motions even of moderate intensity. It is found capable of surviving the Kalamata and Aegion seismic motions, but collapses when subjected to the Lefkada 2003 seismic excitation. Besides from having inadequate strength and ductility, the lack of capacity design leads to the soft-storey collapse.

Retrofit through addition of a shear wall is quite effective, leading to substantial increase of strength and ductility, but also to homogenization of deformation on all floors (Fig. 13). The conventionally retrofitted structure (*B* = 6 m foundation) is capable of sustaining all moderate intensity seismic motions with minimal damage. Subjected to very strong seismic motions, substantially exceeding its design, it is bound to severe damage or collapse: the residual drift δ_{res} reaches 7% for the JMA record, increasing to 10% for Rinaldi, and to 14% for Takatori.

The rocking–isolated alternative (*B* = 2 m) is equally successful in moderate seismic shaking, revealing its superiority when subjected to strong seismic shaking. 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. Due to its inherent self-centering characteristics, rocking isolation allows the superstructure to return to its initial position even after such strong seismic excitations. The only price to pay is the increased settlement, which however, can be tolerable.

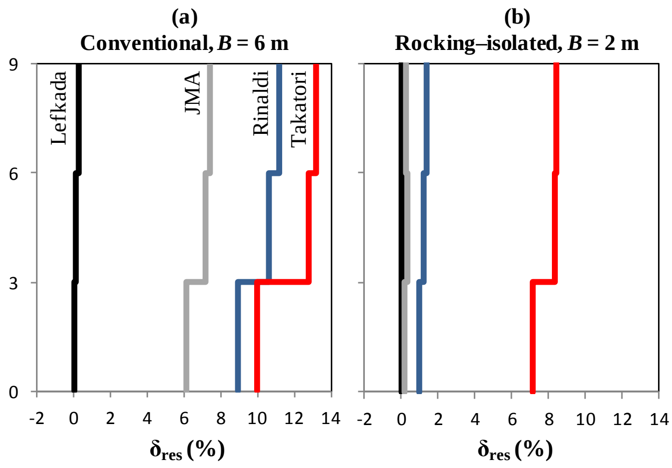

Figure 13. Summary of the performance of the two retrofit alternatives. Distribution with height of residual inter-storey drift ratio δ_{res} for: (a) conventional design; and (b) rocking isolation.

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9 REFERENCES

Anastasopoulos I., Gazetas G., Loli M., Apostolou M, Gerolymos N. 2010a. Soil Failure can be used for Earthquake Protection of Structures. *Bulletin of Earthquake Engineering* 8(2): 309-326.

Anastasopoulos I., Georgarakos T., Georgiannou V., Drosos V., Kourkoulis R. 2010b. Seismic Performance of Bar-Mat Reinforced-Soil Retaining Wall: Shaking Table Testing versus Numerical Analysis with Modified Kinematic Hardening Constitutive Model. *Soil Dynamics & Earthquake Engineering* 30(10): 1089-1105.

Apostolou M., Gazetas G., Garini E. 2007. Seismic response of slender rigid structures with foundation uplifting, *Soil Dynamics and Earthquake Engineering*, 27(7): 642–654.

Di Ludovico M. 2007. *Comparative Assessment of Seismic Rehabilitation Technique on the Full Scale SPEAR Structure*, Doctoral Thesis. Univ. di Napoli Federico II.

Fardis M.N., Negro P. 2006. SPEAR—seismic performance assessment and rehabilitation of existing buildings. *Proc. Int. Workshop on the SPEAR project*, 4-5 April, Ispra, Italy.

Gajan S., Kutter B.L., Phalen J.D., Hutchinson T.C., and Martin G.R. 2005. Centrifuge modeling of load-deformation behavior of rocking shallow foundations. *Soil Dynamics and Earthquake Engineering*, 25: 773–783.

Gajan S., Kutter B.L. 2008. Capacity, settlement, and energy dissipation of shallow footings subjected to rocking. *J. Geotech. and Geoenv. Eng.*, ASCE, 134:8, 1129-1141.

Gajan S., Kutter B.L. 2009. Effects of Moment-to-Shear Ratio on Combined Cyclic Load-Displacement Behavior of Shallow Foundations from Centrifuge Experiments. *J. Geotech. and Geoenv. Eng.*, ASCE, 135(8): 1044-1055.

Gelagoti F., Kourkoulis R., Anastasopoulos I., Gazetas G. 2012. Rocking-isolated frame structures: Margins of safety against toppling collapse and simplified design approach. *Soil Dynamics and Earthquake Engineering* 32(1): 87-102.

KAN.EPE. 2009. *Regulation for Strengthening and Rehabilitation*. OASP, Athens-Greece.

Kourkoulis R., Gelagoti F., Anastasopoulos I. (2012), Rocking Isolation of Frames on Isolated Footings: Design Insights and Limitations. *J. Earthq. Eng.* 16(3): 374-400.

Mergos P.E., Kawashima K. 2005. Rocking isolation of a typical bridge pier on spread foundation. *Journal of Earthquake Engineering*, 9:S2, 395-414.

Paolucci R. 1997. Simplified Evaluation of Earthquake Induced permanent Displacement of Shallow Foundations. *Journal of Earthquake Engineering*, 1:3, 563-579.

Paolucci R., Shirato M. and Yilmaz M.T. 2008. Seismic behaviour of shallow foundations: Shaking table experiments vs numerical modeling, *Earthquake Engineering and Structural Dynamics*, 37: 577–595.

Park R., and Paulay T. 1976. *Reinforced Concrete Structures*. John Wiley & Sons, New York.

Pecker A. 1998. Capacity Design Principles for Shallow Foundations in Seismic Areas, *Proc. 11th European Conf. on Earthquake Engineering*, A.A. Balkema Publishing.

Pecker A. 2003. A seismic foundation design process, lessons learned from two major projects: the Vasco de Gama and the Rion Antirion bridges”, ACI Int. Conf. on Seismic Bridge Design and Retrofit, UC San Diego, La Jolla, USA.

Pender M. 2007. Seismic design and performance of surface foundations, *4th Int. Conf. on Earthquake Geotechnical Engineering*, Thessaloniki, Greece (CD-ROM).

Shirato M., Kouno T., Asai R., Nakani N., Fukui J., and Paolucci R. 2008. Large-scale experiments on Nonlinear Behaviour of Shallow Foundations Subjected to Strong Earthquakes. *Soils & Foundations*, 48(5): 673–692.

Vassiliou M.F., Makris N. 2011. Analysis of the rocking response of rigid blocks standing free on a seismically isolated base. *Earthquake Engineering and Structural Dynamics*, 41:2, 177–196.