

PRELIMINARY RESULTS FROM FULL-SCALE FREE VIBRATION TESTS ON A FOUR STORY REINFORCED CONCRETE BUILDING AFTER SEISMIC REHABILITATION BY BASE ISOLATION

Giuseppe OLIVETO, Professor, Department of Civil and Environmental Engineering, University of Catania, Viale Andrea Doria 6, 95125 Catania, ITALY

Email: goliveto@dica.unict.it

Mario GRANATA, Chief Engineer, S.A.P. Studio Engineering, Via Morosoli 15/b, 95124 Catania, ITALY

Email: granata@sapstudio.com

Giuseppe BUDA, Research Assistant, Department of Civil and Environmental Engineering, University of Catania, Viale Andrea Doria 6, 95125 Catania, ITALY

Email: gbuda@dica.unict.it

Piero SCIACCA, Laboratory Engineer, Department of Civil and Environmental Engineering, University of Catania, Viale Andrea Doria 6, 95125 Catania, ITALY

Email: psciacca@dica.unict.it

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Summary

The work describes static and dynamic tests performed on a reinforced concrete building retrofitted against seismic action by base isolation. After a brief description of the dual base isolation system used for seismic rehabilitation, the testing apparatus is presented including an original sudden release device, the measuring equipment and data acquisition system. The results of the tests are shown in terms of system stiffness, periods of vibration, equivalent damping ratios, building mass, static friction force. Expected design properties are shown for comparison. It is also shown that data contain information for evaluation of dynamic friction forces.



Figure 1. One of the Solarino buildings.



Figure 2. Partial view of foundation.

1. Introduction

Two four story reinforced concrete buildings were planned in the small town of Solarino in the late seventies by the Institute for Council Houses (IACP) of the Syracuse province in Eastern Sicily, Fig. 1. The contractor abandoned the building site after the concrete skeleton and the perimeter and partition infilling walls were completed. No finishing and services were ever implemented. In 2001 IACP of Syracuse decided to complete the buildings. However the original design did not consider seismic action because at the time the area was considered of insignificant seismic hazard. Meanwhile in 1981 the Italian seismic code classified the area as of medium seismic hazard with an anchoring acceleration for the elastic design spectrum of 0.25 g. Therefore the completion of the buildings required their seismic rehabilitation.

On-site inspection showed immediately favorable conditions for seismic rehabilitation by base isolation. In fact the existing foundation rests on limestone emerging from the Climiti Mountain formation and the structural system already appeared ideally suited to be easily supported for the operation of column cutting and insertion of the isolation bearings, Fig 2.

Site investigations were carried out to ascertain the mechanical characteristics of the existing materials and to check the correspondence of the built structure to the original design specifications. The characteristic compression strength of concrete was found to be 13 N/mm², nearly 50% lower than the design strength (25 N/mm²), while the characteristic yield strength of the ribbed reinforced steel bars was found to be 375 N/mm². Linear dynamic analyses showed the following periods and modes of vibration: T₁= 0.94 s – translational in the longitudinal direction; T₂= 0.86 s – torsional; T₃= 0.71 s – translational in the transverse direction. Those periods are by far too long for the building type.

Seismic vulnerability analyses showed a defective resistance in both the longitudinal and transverse directions. The resistance¹⁾ in the longitudinal direction was 92% of that required by the code while that in the transverse direction was only 60%. More important the maximum inter-story drift, 3.76·10⁻³, exceeded by a large amount tolerable values of 2.00·10⁻³. Not even rehabilitation by base isolation would have succeeded in ensuring sufficient resistance and deformation limits.

Therefore it was considered necessary to strengthen the superstructure. The present paper describes free vibration tests conducted on one of the buildings to verify the effectiveness of the rehabilitation design.

2. Strengthening of the superstructure and foundation enlargement

Strengthening of the superstructure was required to lower its fundamental period, to enhance its seismic resistance and to limit inter-story drifts. As is shown in Fig. 3 the first floor slab was largely rebuilt to ensure a safe transmission of stress from the superstructure to the foundation through the isolation bearings.

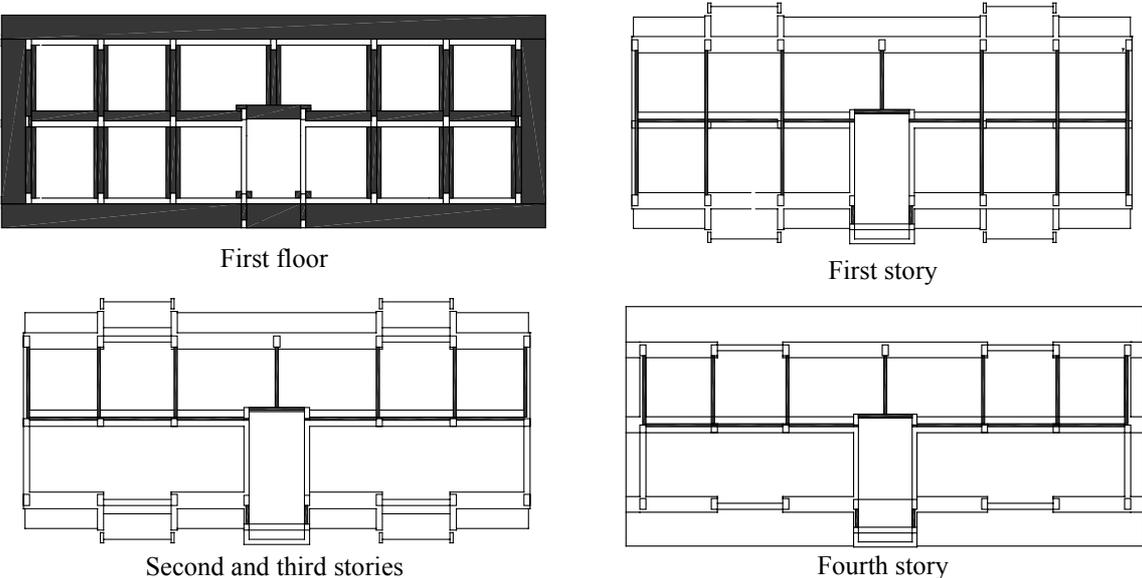


Figure 3. Strengthening of the superstructure.

In particular, the transverse beams were demolished and reconstructed, the cantilever slab surrounding the building was demolished and rebuilt and a portion of slab parallel to the longitudinal central beam was demolished to allocate the base for the central concrete wall. Thin concrete walls (15 cm thick) were built as shown in Fig. 3 for the complete height with

the intention of improving the seismic resistance of the building and to limit the inter-story drifts. The original foundation was locally enlarged as shown in Fig. 4 to accommodate the isolation bearings and to allow a better stress transmission to the foundation.

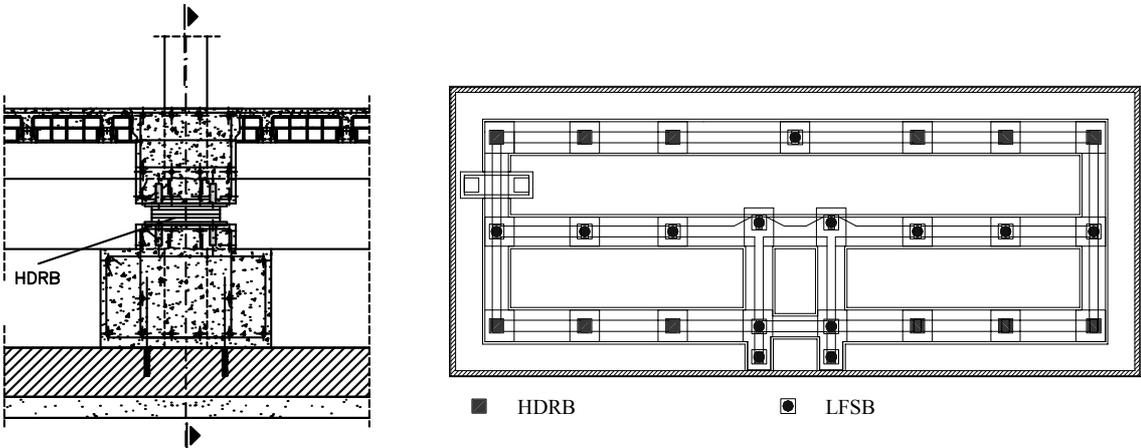


Figure 4. Foundation enlargement. Figure 5. Layout of the dual seismic isolation system.

3. The dual base isolation system

A dual system of 12 High Damping Rubber Bearings (HDRB) and 13 Low Friction Sliding Bearings (LFSB) was used for base isolation as shown in Fig. 5.

The HDRB isolators, Fig. 6, were built to design specification by FIP Industriale S.p.A. with the following characteristics: total height 169 mm, diameter 500 mm, working height 129 mm, rubber height 8×12 mm = 96 mm, steel height 3×11 mm=33 mm, end plates 2×20 mm = 40 mm, design stiffness 770 N/mm, test stiffness 790 N/mm, nominal damping ratio 10%.

The multidirectional VASOFLON bearing from FIP Industriale S.p.A was selected for the LFSB isolators, Fig. 7. The characteristics of the bearings used are specified by the manufacturer as follows: maximum load of 1500 kN for the 9 central bearings and maximum load of 1000 kN for the 4 lateral bearings; in both cases the maximum permissible displacement is 200 mm. Several tests performed by the manufacturer seem to indicate for the friction coefficient a static upper limit of about 2 % and a dynamic upper limit of about 1%.



Figure 6. High Damping Rubber Bearing.



Figure 7. Low Friction Sliding Bearing.

4. Mechanical properties of the base isolated building

The estimated mass of the building is 1567 kNs²/m while the design stiffness of the base isolation system is 9240 kN/m. Based on a test performed on two HDRB the global nominal stiffness has been evaluated to be 9480 kN/m. Therefore the undamped fundamental period of the building turns out to be 2.59 s in the design condition and 2.55 s in the test condition. The design displacement for a peak ground acceleration (PGA) a_g=0.25g was 13.3 cm, evaluated according to Eurocode 8²⁾ and National Guidelines³⁾ elastic design spectrum for a damping ratio of 10%.

4.1 Mechanical properties in the field test conditions

Several free vibration tests were performed on one of the Solarino buildings. However the tests were performed when the building was not yet finished. In particular pavements and internal wall plaster were still missing. Obviously all kind of live load such as furniture and people were also missing. The mass in the field test condition has been estimated as 1300 kNs²/m. Assuming a maximum error of $\pm 5\%$ on this estimate the variability range for the real mass of the building in the test condition is 1235 kNs²/m \div 1365 kNs²/m. The expected undamped period for the test is therefore in the range 2.27 s \div 2.38 s with a mean value of 2.32 s. The above values have been evaluated using the nominal stiffness derived from the laboratory tests on the HDRB.

5. Testing apparatus

The testing apparatus, Fig 8, consisted of three main items: the loading device, the measurement equipment and the data acquisition system.

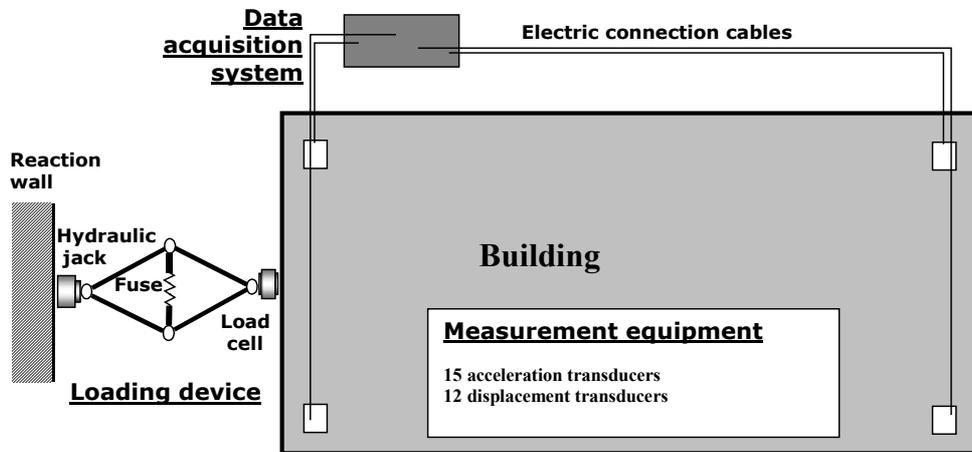


Figure 8. Testing apparatus.

5.1 Loading device

The loading device probably is the most original part of the experiment. It consists of a reaction wall, a hydraulic jack, a sudden release device and a load cell. The sudden release device was conceived as a quadrilateral linkage with a diagonal spring preventing deformation. By dosing the rupture strength of this brittle spring, N_r , the release load F_r may be evaluated as $F_r = m \cdot N_r$, Fig 9.

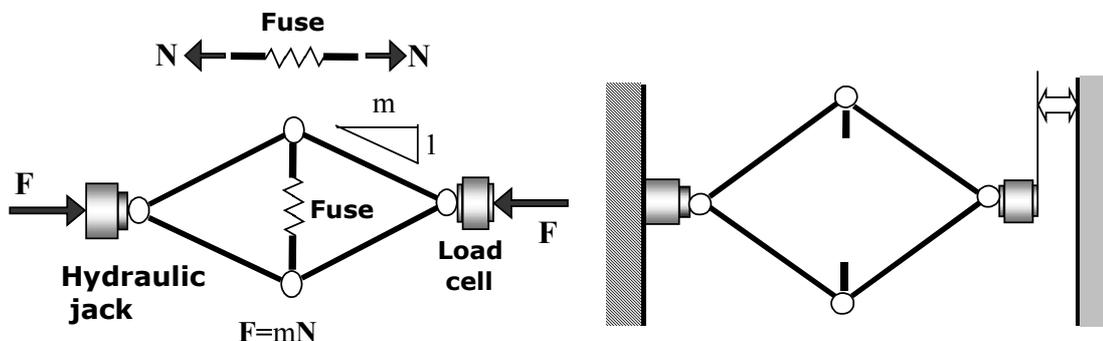


Figure 9. Sudden release device. Basic idea; F : applied load, N : traction force in the fuse.

The translation of this idea into practice resulted in the device shown in Fig. 10. The brittle spring has been constructed by means of a high strength steel bar calibrated to the right thickness so as to break under a specified load. This part of the loading device is shown in Fig. 11 and has been described as *the fuse*.

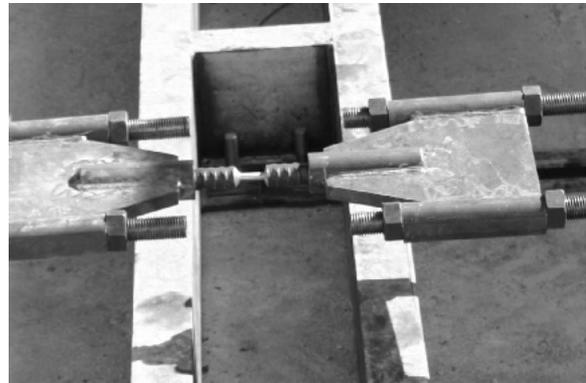


Figure 10. Sudden release device – Prototype. Figure 11. Fuse: calibrated HS steel rod.

5.2 Measurement equipment

Three types of measurements have been performed: force, displacement and acceleration. The applied force has been measured by a NOVATECH mod. F205-CFR0K0 load cell applied between the building and the head of the loading device, as shown in the photographs of Fig. 10.

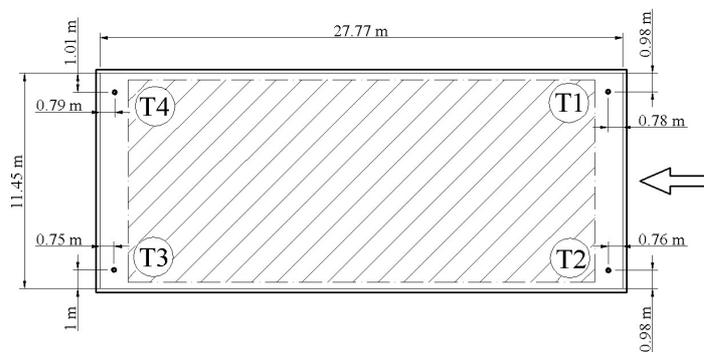


Figure 12. Location of the measuring stations.

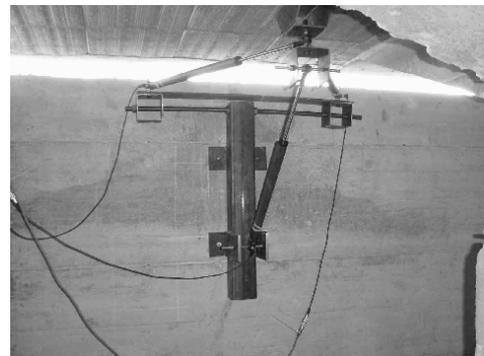


Figure 13. 3 LDS (SLS320/400).

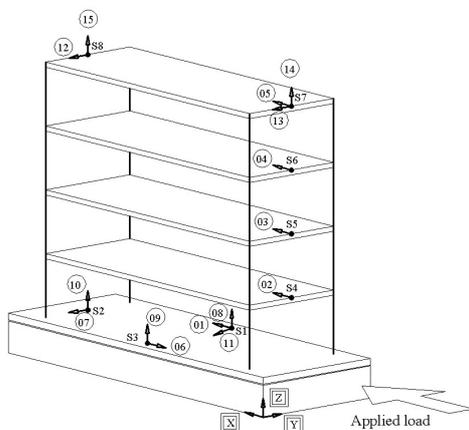


Figure 14. Schematic accelerometers layout.



Figure 15. 3 PCB model 393B31 sensors.

Displacements have been measured in 4 stations at the 4 corners of the building just above the base isolation level. The measuring stations were positioned as shown in Fig. 12. Each measuring station had 3 Linear Displacement Sensors (LDS), Penny & Giles model SLS320/400 shown in the photograph of Fig. 13.

Accelerations were measured by 15 seismic accelerometers, PCB PIEZOTRONICS model 393B31. The accelerometers were located as shown schematically in Fig. 14. The layout was designed so as to measure the 6 rigid body degrees of freedom and the deformation of the building along the loading direction. Three seismic accelerometers at station S7 are shown in Fig. 15.

5.3 Data acquisition system

Two data acquisition systems were used; one for the acquisition of displacement and force, the AT-MIO16X analogical-digital converter, the other for acquisition of acceleration and force, the DAQCard-AI-16E-4, both manufactured by NATIONAL INSTRUMENTS. Data was stored in ASCII format on a PC. The applied force was acquired and stored in both systems to ensure signal synchronization.

6. Description of tests and results

Three tests were performed on 8 July 2004. The first was a static test where the load was gradually increased to a maximum value of 1085 kN and a corresponding displacement of 12.80 cm. The main results of this test were the evaluation of the static friction force, the load-displacement relationship during the loading and unloading phases, the residual displacement and the dissipated energy corresponding to the loading-unloading process. Two trial dynamic tests were performed on the same day mainly to calibrate the sudden release device. Three dynamic tests were performed on 9 July with different load and displacement amplitudes. Displacements and accelerations were acquired during these tests and are still being processed to recover useful information. Damped periods of vibration at various displacement amplitudes were evaluated from displacement records. Also the building mass in testing conditions was estimated from the initial acceleration. Research is underway to try to establish the HDRB displacement dependent damping coefficient and the dynamic friction force. In the following the available results will be reported and commented upon.

6.1. Static test

The load-displacement curve for the loading and unloading phases in the static test is shown in Fig. 16. The curve derives from the average displacement of measuring stations T3 and T4 in Fig. 12.

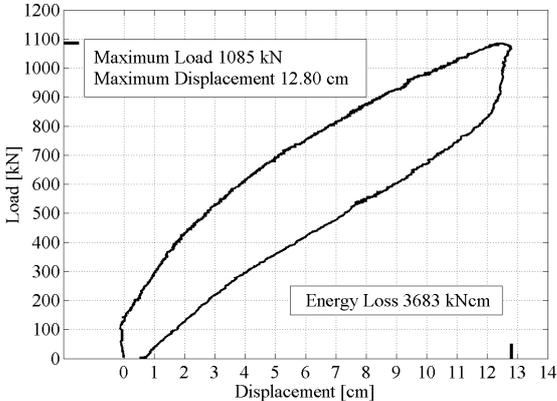


Figure 16. Load-Displacement curve (L-D).

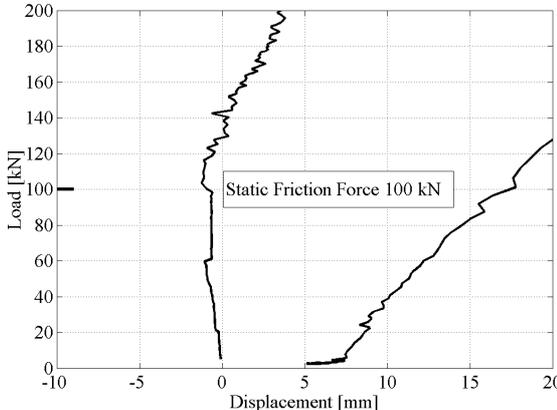


Figure 17. Zoom on L-D curve.

From this curve the static friction force and the displacement dependant stiffness of the HDRB system may be evaluated. A closer view of the initial part of the loading curve and final part of the unloading curve is shown in Fig. 17. From this figure the value of the static friction force is evaluated at about 100 kN and the residual displacement upon unloading comes out at about 7 mm. The additional recovery to 5 mm at zero external load is probably due to viscous deformation of the floor slab.

The load displacement curve shows a decreasing slope (tangent stiffness) with increasing displacement amplitude. The tangent stiffness is initially infinite, due to the friction force and then varies from a maximum of 15102 kN/m when the friction force is overcome to a minimum of 4787 kN/m at maximum displacement. In the unloading phase two branches may be identified; in the first one the average stiffness is 6468 kN/m and in the second one becomes 8833 kN/m. The tangent stiffness at the end of the unloading phase (8833 kN/m) is slight lower than the design value (9240 kN/m). The low value of the tangent stiffness at the maximum displacement (4787 kN/m) may be due to the loading history where 7 long steps at constant load have been used. The average energy loss is evaluated at 3683 kNcm and accounts for viscous behavior in the rubber and for the work of the friction force.

6.2 Dynamic tests

The time-displacement curves in three dynamic tests performed on 9 July 2004 are shown in Fig. 18. These curves were obtained from the average measurements at the four stations shown in Fig. 12. In the first test a maximum displacement of 11.48 cm was applied under a load of 1027 kN. The building completed two oscillations before coming to rest.

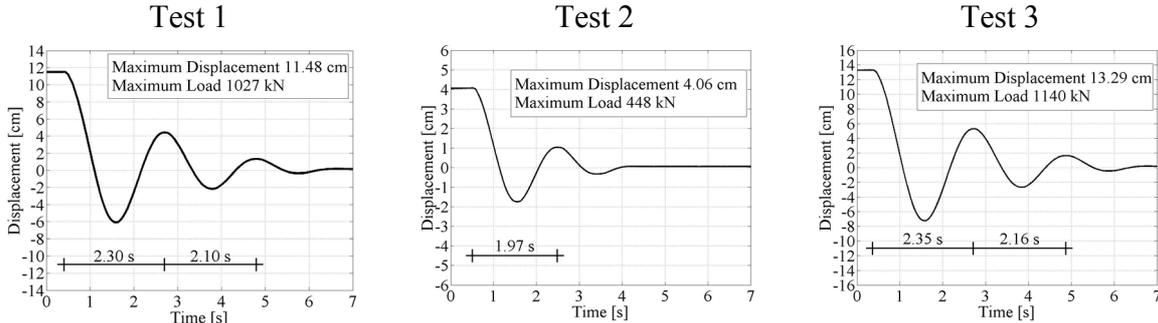


Figure 18. Displacement curves in three dynamic tests performed on 9 July 2004.

In the second test a maximum displacement of 4.06 cm was applied under a load of 448 kN. The building came to a rest after completing a cycle of oscillation. In the third test the displacement of 13.29 cm was applied under a load of 1140 kN. The building completed two oscillations before coming to rest. The residual displacement was always of the same sign and equal to 1.66 mm in the first test, 0.66 mm in the second and 1.82 mm in the third test. This is to be added to the 5mm in the static test leading to a maximum residual displacement of 8.08 mm. Unfortunately no record was kept of the trial dynamic tests performed on 8 July.

Fig. 19 shows the load displacement curves for the static phases of the dynamic tests performed on 9 July 2004. The residual friction force that must be overcome to start displacing the building can be directly measured from those curves. Other relevant information is provided by the tangent stiffness in the initial stage, middle stage and final stage of the static loading phases. The Estimated Residual Displacement (ERD) has been determined from the friction force evaluated in the static test and the initial stiffness evaluated in the dynamic tests. These results are summarized in Tab. 1.

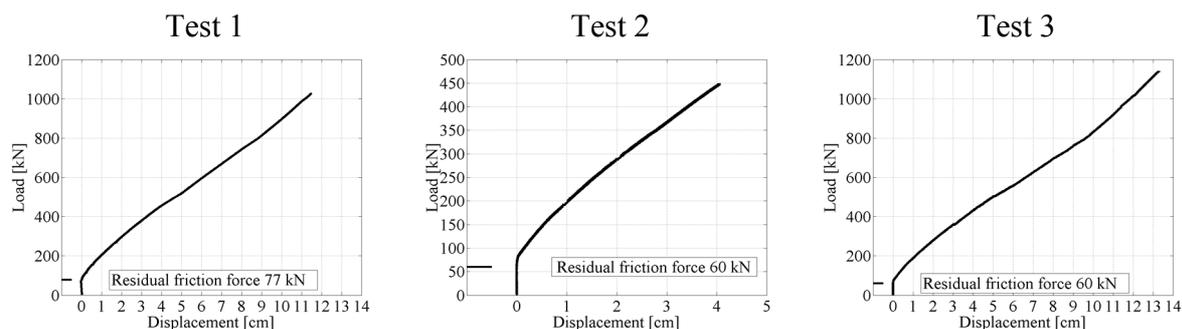


Figure 19. Load-displacement curves for the static phases of dynamic tests.

Table 1. Tangent stiffness in the loading phases of static and dynamic tests.

Test	Load	Displacement	Friction Force	ERD	Stiffness [kN/m]		
	[kN]	[cm]	[kN]	[mm]	Initial	Medium	Final
Static	1085	12.80	100	6.62	15102	–	4787
Dynamic 1	1027	11.48	77	8.33	12000	7150	8218
Dynamic 2	448	4.06	60	8.80	11367	7730	–
Dynamic 3	1140	13.29	60	9.05	11048	6518	9392

The results in Table 1 deserve a few comments. First the measured residual friction force is always smaller in later tests than in the first test. This is due to the residual displacement and to the trapped elastic force. The initial stiffness of the system is considerably larger than the design stiffness, but it appears to diminish with successive tests. The intermediate stiffness is always smaller than the design one. The final stiffness was considerably less than the design one in the static test, but increased considerably in the dynamic tests approaching the design value from below, showing hardening of the rubber. It should be mentioned that the residual displacements have not been accounted for in the above.

Periods of vibration have been evaluated directly from the oscillation curves of Fig. 18. A list of the evaluated periods of vibration is shown in Tab. 2 together with the maximum displacement at the beginning of the corresponding cycle. In the same table the equivalent viscous damping evaluated by the logarithmic decrement method is also shown. As expected the damped period of vibration increases with the displacement amplitude, denoting a decreasing effective stiffness with increasing displacement amplitude. Also the equivalent viscous damping decreases with displacement amplitude showing that HDRB are more effective in energy dissipation at low displacement amplitudes.

Table 2. Periods of vibration and equivalent viscous damping ratio.

Maximum Displacement [cm]	Period [s]	Equivalent damping ratio [%]	Comments
4.06	1.97	21.7	From test 2, first cycle.
4.20	2.10	19.0	From test 1, second cycle.
5.10	2.16	18.7	From test 3, second cycle
11.48	2.30	15.2	From test 1, first cycle
13.29	2.35	14.7	From test 3, first cycle

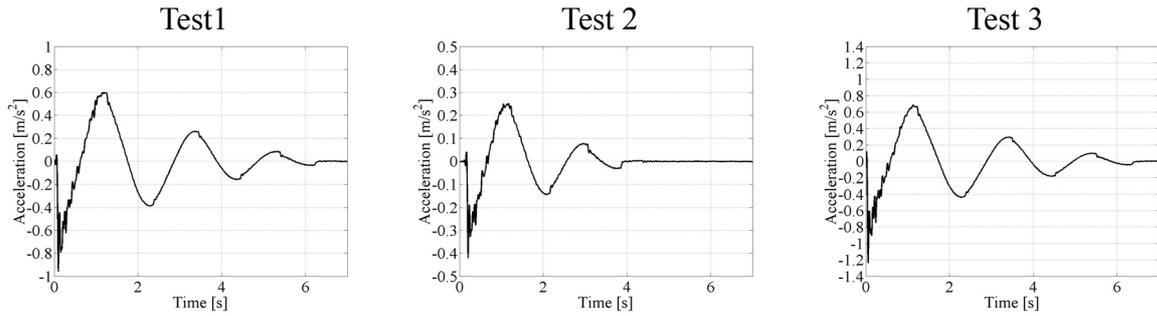


Figure 20. Accelerations measured at the height of the center of mass (Station S5 – Fig.14).

Table 3. Low frequency component of peak acceleration and estimated building mass.

Test number	Applied Load [kN]	Peak acceleration [m/s^2]	Estimated mass [kNs^2/m]	Expected range
1	1027	0.80	1284	1235 ÷ 1365 kNs^2/m .
2	448	0.33	1358	
3	1140	0.90	1267	

The horizontal accelerations recorded at the level of the center of mass of the building are shown in Fig. 20. The acceleration jumps due to the sign change in the dynamic friction force are clearly visible in the graphs of this figure. These jumps may be used to evaluate the dynamic friction force in the system. The results will be presented elsewhere. The peak acceleration occurs at the sudden load release transforming the pushing force into acceleration of the center of mass of the building. This allows for an estimate of the mass of the building. The peak accelerations obtained after removing the high frequency components are shown in Tab. 3 together with the estimated mass and the expected range evaluated from material weights. Individual values are within the expected range and their average (1303 kNs^2/m) compares well with the central value.



Figure 21. View of the tested Solarino building.

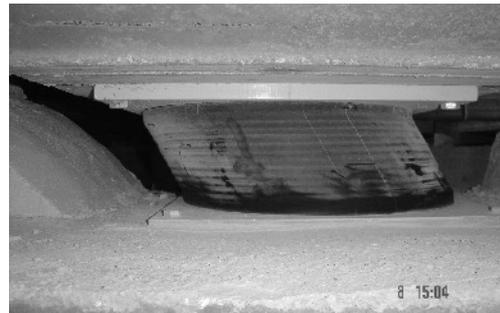


Figure 22. HDRB under dynamic test.

The companion building to the one in Fig. 1 is shown in Fig. 21 in the test conditions. One HDRB fitted in the building of Fig. 21 is shown in Fig. 22 during one of the tests.

7. Conclusion

The preliminary results of free vibration tests on a reinforced concrete building retrofitted against seismic action by base isolation have been presented. The retrofitting system and the reasons for its adoption have been briefly described. The mechanical properties of the base isolated building have been evaluated during the tests and compared with the design values. Periods of vibration for different displacement amplitudes and corresponding equivalent damping ratios have been derived from the building response. The original device for load

application and sudden release has also been illustrated. A multitude of results have been obtained from the tests though only a few have been presented. The tests have shown that most of the theoretical basis for the design is valid, but field results seem to show aspects not easily predictable from simple design theory. Most important from a practical point of view appears to be the evaluation of the static friction force and the residual displacement. The mass of the building in the test conditions has also been evaluated from the tests to a considerable degree of accuracy. Finally it has been shown that the dynamic data acquired should also allow the evaluation of the dynamic friction force.

8. References

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9. Acknowledgments

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