THE BASE ISOLATED MASS OF THE DYNAMIC LABORATORY AT UNIVERSITY OF NAPLES FEDERICO II

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ABSTRACT :

A base isolated system for the reaction mass of two 3x3m movable shaking tables was designed and built to perform experimental tests on large scale structures, especially subjected to asynchronous earthquake ground motion. Such reaction mass, built in the laboratory of the Department of Structural Engineering at University of Naples Federico II in Italy, is part of major works in progress to upgrade the testing facility and allow for full use of system of shake tables. The adopted isolation system consists of about 100 air springs located at the base of about 1,400 tons ring-shaped mass and eight groups of eight dampers located at the edges of the above mass. To optimize the performance of the system, a reduction of the lateral displacement of the mass along with the accelerations transmitted to the foundation, was considered. The selection of the allowable lateral displacements of the large reaction mass was driven by the need to be compatible with the adjacent existing structure. In fact, the shaking tables were mounted on an existing strong floor in the laboratory of the university department.
Several retrofitting interventions on the existing structures, e.g. the cutting of the existing columns and the use of special concrete for casting, were employed to achieve adequate structural performance under the working conditions of the shaking tables.

KEYWORDS: Dynamic analysis, shaking tables, isolation, damping, experimental texts.

1. INTRODUCTION

In the framework of nationals and internationals research projects, the laboratory of the Department of Structural Engineering at University of Naples Federico II is involved in numerous experimental activities. Such activities concern both materials and structural members. The laboratory is equipped with three strong floors of dimensions 10.3 m x 19.7 m (strong floor A), 5.1 m x 12.5 m (strong floor B) and 13.2 m x 15.2 m (strong floor C) as pictorially shown in Figure 1. In particular, the strong floors A and C are mainly addressed to dynamic tests on shaking tables. The installed shaking table system consists of two square tables, with side of 3 meters; each table is characterized by two degrees of freedom (DoFs) in the two horizontal directions and can be moved on the strong floor in order to reproduce the seismic asynchronous effect on structures with high spans. The maximum payload is 200 kN for each table with a frequency range of 0-50 Hz, velocity peak of 1 m/sec and total displacement of ±250 mm; the hydraulic system has 6 motor pumps groups (each one consisting of two motor pumps) with a maximum total capacity of 2500 l/min.
The shaking tables were first placed on the strong floor A (Figure 1, left), with the mechanical (pumping) system located at the basement level, i.e. below the strong floor.
The construction works described in this paper were necessary to achieve the full development of a shaking

tables system. Such works include also the movement of the shake tables from the strong floor A to C (Figure 1, right). Additionally, to optimize the use of the testing system, it was essential to create a seismically isolated reaction mass.

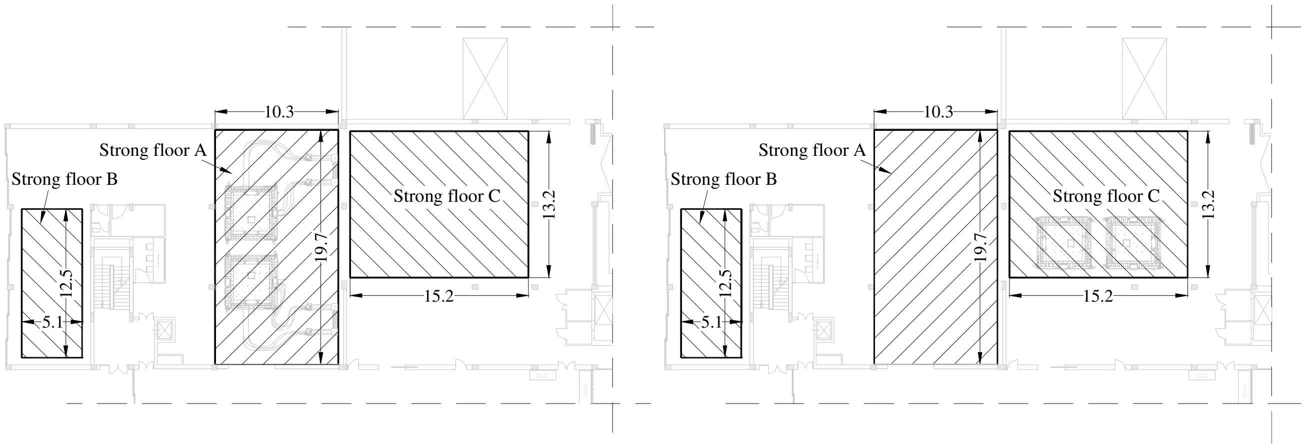

Figure 1 General plan layout of the three strong floors: in original (left) and present (right) configurations (dimensions are in metres).

2. EXISTING FACILITY

In Figure 2, original plan layout and cross-sections of the strong floor C are displayed. It was characterised by 1.2 m thickness of reinforced concrete (RC) and about 200 m^2 of total area. The loads at foundation levels were carried by 20 columns with cross-sections of 40x40cm.

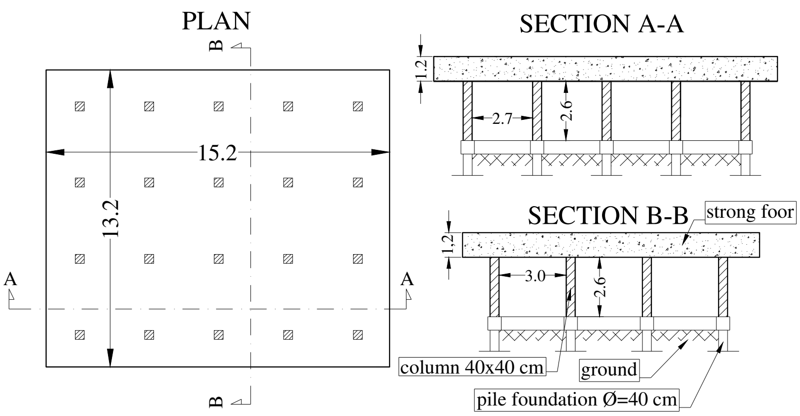

Figure 2 Plan layout and cross-sections of strong floor C (dimensions in metres)

The net distance between the columns was of 2.7 m and 3.0 m in the two orthogonal directions. The columns were connected to 60 cm thick foundation beams. Horizontal and vertical loads were transmitted to the ground by 20 piles (one for each column) with a diameter of 40 cm and length of 10 m.
The structure of the strong floor was entirely independent from the adjacent structural system of the laboratory. This is also the case for the backbone of strong floor A, where the tables were initially located.
Due to the technical features of the shaking tables system of University of Naples Federico II (range of frequency 0-50 Hz, accelerations up to 1 g with a specimen of 20 tons), propagation of seismic waves to the surrounding buildings may become a serious threat to deal with.
To investigate the spreading of such waves, a number of experimental tests were carried out by using the shaking tables in the original location (Figure 1, left). It was found that such experiments caused discomfort to the building occupants.
The accelerations imposed on the building by the movement of the table platform were recorded at different

levels stations height wise. The shaking was obtained with a sinusoidal signal and under conditions of payload equal to zero. The monitoring of the building was carried out with the installation of 4 accelerometers located to the second and the third floor of the building, at the level of strong floor and foundation.

Table 1 Acceleration records

SINE SIGNAL INPUT				**RECORDS (max acceleration)**	
f [Hz]	*A [mm]*	*V_{max} [cm/sec]*	*A_{max} [m/s^2]*	*Accelerometer position*	*m/s^2*
8	5	25	12.6	3rd floor	0.04
				2nd floor	0.06
				Foundation level	0.20
				Strong floor level	0.65
22	1	14	19.1	3rd floor	0.08
				2nd floor	0.07
				Foundation level	0.30
				Strong floor level	0.75

The results provided in Table 1 show for two different values of frequency (f) and amplitude (A) of input sinusoidal signal that the accelerations are higher at the base of the building (around 1/20 of the max acceleration of the input signal (A_{max})) and smaller at 2nd and 3rd level, (equal to about 1/100 of the max acceleration of the input signal). These values proved that there could exist, through the ground, the propagation to the building of the accelerations caused by the movement of the tables.
Furthermore, considering that the results were obtained without fully exploiting the power of the facility (i.e. movement of only the superior platform of one table, absence of payload, frequency of the signal of only 22 Hz, amplitude of the signal of few mm), these values assume a particular importance.

3. UPGRADE OF THE LABORATORY

To avoid that the vibrations caused by the tables during operation were propagated to the strong floor and to the structure of the laboratory, two different design solutions, both referred to the strong floor C (Figure 1), were assessed as discussed hereafter.

3.1. Solution n. 1- Single block of concrete

A first solution was based on filling with concrete the whole volume under the strong floor C in order to realize a single large mass of reaction able to mitigate the vibrations produced by the tables in movement.
As a result, it could be possible to obtain a reaction seismic mass of 2,050 tons; the maximum mass in movement (obtained considering, for both the tables, the weight of the two platform in movement and the max payload) is equal to 111 tons. The ratio between reaction mass and the mass in movement was equal to 18, which may result highly efficient to minimize the transmission of seismic vibrations.
The structural modelling was performed by utilizing bi-dimensional finite elements (FE), in particular shell elements, subsequently extruded (CSI, 1995).
In a FE model the piles of foundations were also included. The constraint conditions of the system were simulated assigning below the block of concrete springs with a vertical stiffness distributed and to the piles springs with horizontal stiffness (with a step of 2 meters). A very strong system with fundamental frequency of 80 Hz was obtained. This value, not lying in the frequency range of the tables (0-50 Hz), could prevent problems of resonance between the movement of the tables and the vibrations of the reaction mass.
Subsequently, the FE model was improved taking into account the dynamic contribution of the soil below the foundation for a thickness of 1.5 meters. The estimation of thickness of ground interacting with the motion of the reaction mass was evaluated through a back-analysis, performed on the strong floor A (Figure 1). Through the records of the accelerometers positioned on the strong floor and on the foundation of the same, the fundamental frequency of the system was estimated; subsequently, the thickness of ground to be considered cooperating to the motion in order to obtain, with a FE model, the same frequency of vibration was estimated.
In the new model, for the soil, a specific weight equal to 13 kN/m^3 was assumed. To simulate the vertical

stiffness of a ground "overloaded" by a weight of about 2,500 tons of concrete, fixed supports were assigned below the soil; this is an acceptable approximation. Using such structural modelling the fundamental frequency of the system was found to be equal to 15 Hz, values within the range of frequencies of the system (0-50 Hz).
As a result, the solution “single block” was discarded. In fact, even if the solution gives rise to a ratio between reaction mass and the mass in movement equal to 18, which may result highly efficient to minimize the transmission of seismic vibrations, it is also necessary to take in account the effect of the dynamic amplification of the actions in resonance condition. In other words, when the tables simulate signals with a fundamental frequency around 20 Hz, the dynamic amplification could bring acceleration values until 10 times the input. Under these conditions, such actions, even if damped by the great reaction mass, would propagate with a reasonable intensity, to the foundations, to the ground and therefore to the surrounding structures.

3.2. Solution n. 2- Isolated reaction mass

An alternative to the above solution was a reaction mass "isolated" from the foundations through the insertion of "air springs" (Figure 3, CF Gomma, model Torpress 29). This solution provides opposite dynamic conditions with respect to the previous, because it is able to obtain large displacements, but at the same time to transfer lower accelerations to the ground (i.e. very flexible system).

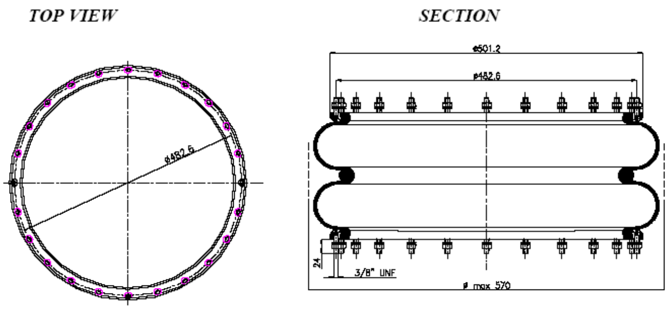

Figure 3 Plan and section of an air spring (CF Gomma, model Torpress 29)

To further increase the volume of the reaction mass, two different technological hypotheses, both obtained with use of reinforced concrete, were considered (Figures 4 and 5).

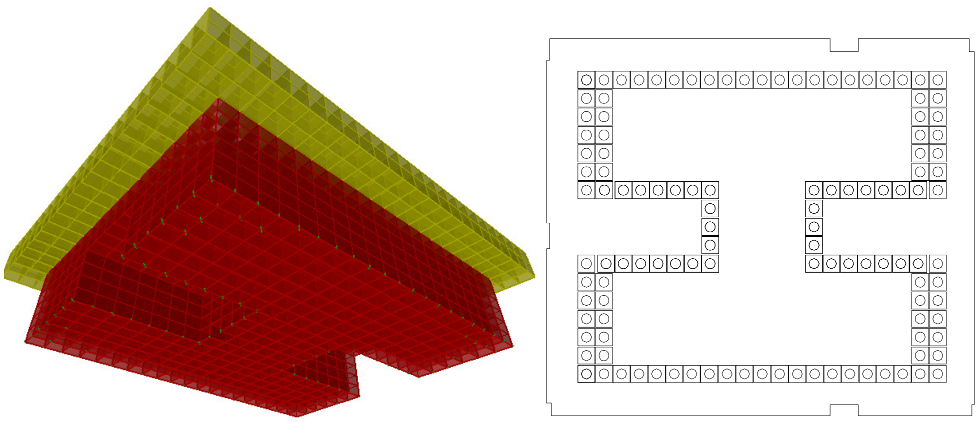

Figure 4 FE modelling (left) and placing of air springs (right)

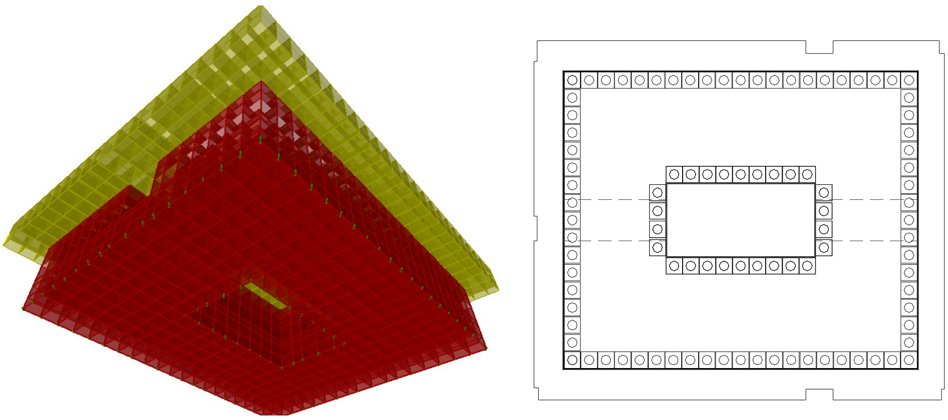

Figure 5 FE modelling (left) and placing of air springs (right)

In the solution of Figure 4 (reaction mass with "H" shape) the total reaction mass is 1,308 tons (assuming a specific weight for reinforced concrete of 25 kN/m^3) sustained by 108 air springs with a pressure of 7 bar. The ratio between inertial mass and mass in movement is approximately equal to 12.

In the solution of Figure 5 (reaction mass with "ring" shape), the total reaction mass is 1,385 tons sustained by 100 air springs with an operating pressure equal to 8 bar. The ratio between inertial mass and mass in movement is equal to 12.6.

From the technological standpoint the two solutions are similar, i.e. for both is possible to realize the casting in the desired shape, subsequently, after concrete curing to allocate the air springs and cut the columns.

For a dynamic evaluation, considering for the air springs, a vertical and horizontal stiffness of 654 kN/m and 165 kN/m respectively, a FE model was developed. The two shake tables are supposed located on two opposite corners of the reaction mass with the maximum payload of 20 tons (≈200 kN). As shown in table 2, the two solutions are similar, assuring extremely low frequencies for the six vibration modes of rigid body.

Table 2 Frequencies of first 8 modes of vibration

	Solution "H"	**Solution "ring"**
mode	*f (Hz)*	*f (Hz)*
1	0.93	0.52
2	0.97	0.52
3	1.21	0.61
4	1.40	1.10
5	1.76	1.22
6	1.80	1.23
7	31.0	33.1
8	48.0	52.6

The last two frequencies refer to local modes of the system, and therefore are characterised by a participant mass practically irrelevant.

4. ADOPTED SOLUTION

Although from a dynamic point of view the two solutions presented in the previous paragraph can be considered equivalent, the "ring" shape was selected (Figure 5). This solution allows a rational distribution in plane of the air springs and at the same time a good distribution of the loads on the foundation. Furthermore, with the "ring" is possible to avoid the double line of air springs (Figure 4) and the number of devices to install is lower. As far as vibration modes are concerned, the first and second are translational along the two horizontal directions, the third is rotational around the vertical axis; the fourth is translational along the vertical direction, while fifth and sixth modes of vibration are rotational, around the two horizontal axes.

Beginning from dynamic results and considering the tables in the condition of maximum performances, the

values of maximum displacements were obtained.
It is important to underline that, for a deformable system as the one of an isolated foundation, the problem does not concern the values of the acceleration transferred to the ground but the evaluation of the compatibility among the displacements and those allowed by the boundary conditions (equal to 10 cm for all the sides of the reaction mass).
To evaluate the maximum displacement in dynamic conditions, an amplification factor of N=3.33, was considered. This value takes into account a damping ratio equal to 15% of the critical value. Considering the maximum performance of the system at the fundamental frequency of the first and second mode (0.52 Hz), a maximum horizontal dynamic displacement of 60 mm and a maximum lateral acceleration of 0.63 m/s^2 were obtained (Table 3).

Table 3 Maximum displacement and acceleration considering maximum performance of the system

	Displacements		**Accelerations**	
Modes of vibration	*Horizontal [mm]*	*Vertical [mm]*	*Horizontal [m/s^2]*	*Vertical [m/s^2]*
1st: f=0.52 Hz	60	-	0.63	-
2nd: f=0.52 Hz	60	-	0.63	-
3rd: f=0.61 Hz	46	-	0.13	-
4th: f=1.10 Hz	-	-	-	-
5th: f=1.22 Hz	-	48	-	1.90
6th: f=1.23 Hz	-	43	-	1.80

Considering the maximum performance of the system at the frequency (f=1.22 Hz) of the fifth mode of vibration (rotation around horizontal axis) a maximum vertical displacement equal to 48 mm was obtained.
No evaluation of displacements considering the maximum performance of the table at the fundamental frequency of the fourth mode of vibration was performed (there is not a vertical degree of freedom for the shaking tables).
The additional damping is obtained placing at the four corner of the seismic reaction mass, 8 groups of devices, constituted by 8 dampers (MSA, 2008) for each group.

5. CONSTRUCTION STAGES OF THE REACTION MASS

The peculiarity of the solution adopted does not consist in the complexity of the dynamic problem, but rather in problems of technological nature. The technological difficulties of the design concern the problem that the reaction mass is partially already existing (Figure 2). The design solution deals with different issues: adding about 800 tons of concrete to 500 tons of existing mass; upgrading of the foundation in order to withstand the new increased load; cutting of the existing twenty columns.
The technological steps described above were recently completed at the Department of Structural Engineering of the University of Naples Federico II in Italy. In this paragraph some of construction stages are described.
First of all, the strengthening of the foundation was realized changing from the condition of foundation on beams to foundation on plate. For the design, a FE model was created and two extreme load conditions were considered. In particular, a first condition that considers a vertical load of 20 tons for each air spring (evaluated to the Ultimate Limit State according to Eurocode 2 and 8 (CEN, 2004; CEN, 2003) and a second condition simulating that the load does not lean on the air springs but directly on the foundation (due to an improper functioning of the air springs). For each condition, the soil was considered with two different values of vertical stiffness: a low value (k_v=20 N/cm^3) and a high one (k_v=200 N/cm^3). The obtained steel reinforcement is a two layers square grid of rebars of 20 mm @ 20 cm. In the zones below the air springs, only at the upper edge, the reinforcement spacing is reduced to 10 cm (Figure 6, left).

Figure 6 Grid foundation plate and casting of the concrete (left); spatial grid of the reaction mass (centre); details of the welding (right)

To enhance the ratio between the reaction mass and the mass in movement, the increase of thickness of the existing strong floor was necessary. The existing 1.2 m thick reinforced concrete (RC) strong floor was assumed a height of about 3.6 m (Figure 7). To create a monolithic block, special systems of connection were studied. The reinforcement rebars of the new block were adequately welded to the lower bars of the existing strong floor (Figure 6, right).

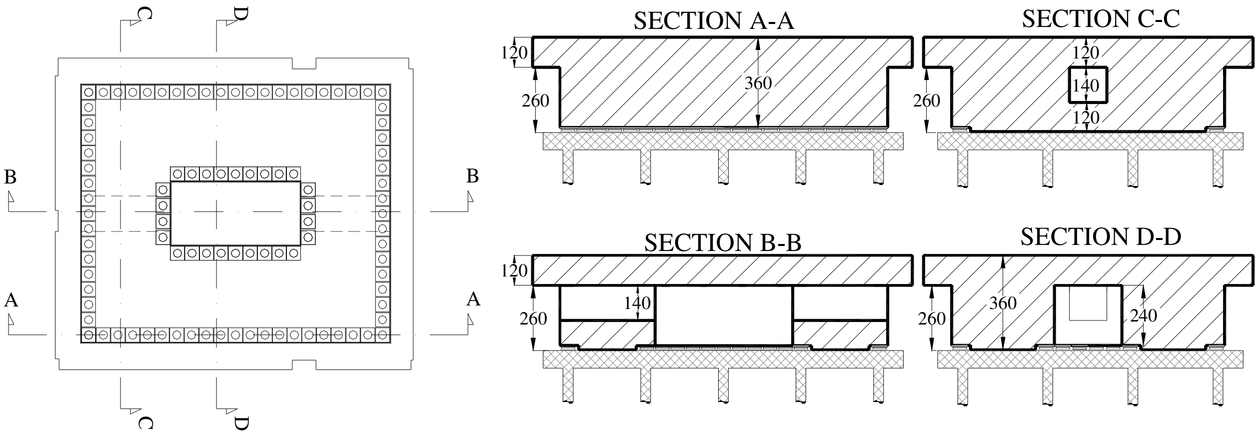

Figure 7 Plan and sections of the isolated reaction mass (distance in centimetres)

To prevent the effects of high temperatures, the casting was performed in three different layers. SCC concrete (self compacting concrete) was used to allow a greater fluidity and therefore a capacity of self compacting. The inability of the vibration is due to the presence of a very complicated spatial grid (the spacing of the longitudinal rebars in the three orthogonal directions is equal to 40 cm (Figure 6, centre); this grid is so dense to assure the monolithic behaviour between the mass of the existing strong floor and the one in addition below it. In total, about 450 m^3 of concrete SCC was used. After the curing of the concrete, the cutting of the existing 20 columns and the placing of the 100 air springs were carried out. Finally, through an appropriate system of piping, the lift of the reaction mass was obtained. In particular, with the purpose to assure the best lift, three different lines of air piping were designed and realized. The lift system is completed by a blower with a maximum pressure of 12 bar ($\approx$1.2 N/mm^2), 9 accumulators (necessary to allow an immediately air compensation) and a control panel with a software program. The software program is able to check the vertical position of the reaction mass (three sensors (LVDT) were installed on three sides) and if necessary switch on/off the blower and/or close one or more air piping lines. In this way, the lift system is able to assure in "real time" the horizontality of the strong floor.

6. CONCLUSIONS

In this paper, the design of an isolation system that is effective in limiting the seismic waves propagation caused by the shaking tables is summarized. In particular, the different construction stages used to transform the existing 1.2 m thick reinforced concrete (RC) strong floor and the about 2.7 m high basement located below the

strong floor into a unique RC reaction mass suspended on 100 air springs are illustrated in a detailed fashion.
The designed structural system is a typical example where the dynamic evaluation is crucial in order to obtain the desired performance.
A complex spatial grid (the spacing of the longitudinal rebars in the three orthogonal directions is equal to 40 cm) necessary to assure the monolithic behaviour between the mass of the existing strong floor and the one in addition below was realized. Special systems of connection were performed and the reinforcement rebars of the lower block were opportunely welded to the lower reinforcement of the existing strong floor.
To avoid problems of high temperatures, the casting was performed in three different layers. SCC concrete (self compacting concrete) was used to allow a greater fluidity and therefore a capacity of self compacting.
After the curing of the concrete, the cutting of the existing 20 columns and the placing of the 100 air spring were realized. Finally, through an appropriate system of piping, the lift of the reaction mass was obtained. In particular, with the purpose to assure the best lift, three different air lines of piping were designed and realized. The lift system is controlled by a software program able to check the vertical position of the reaction mass (three sensors (LVDT) were installed on three sides) and if necessary switch on/off the blower and/or close one or more piping lines. In this way, the lift system is able to assure in "real time" the horizontality of the strong floor.

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REFERENCES

Calvi G.M., Pavese A., Ceresa P., Dacarro T., Lai C.G., Beltrami C. (2005). Design of a large-scale dynamic and pseudo-dynamic testing facility, Iuss Press. Pavia. Italy
CEN. (2004). EUROCODE 2, Design of concrete structures – Part 1-1: General rules and rules for buildings, EN 1992-1-1.
CEN. (2003). EUROCODE 8, Design for structures for earthquakes resistance – Part 1 – General rules, seismic actions and rules for buildings, Final Draft – prEN 1998-1.
Cf_Gomma (2007). Attuatori Pneumatici, Technical Flyer (in Italian), Milan, Italy,
CSI. (1995). Computer and Structures, Inc. SAP Nonlinear, 8.1.2 Structural Analysis Program, Berkeley, California, US.
Fabbrocino G. and Cosenza E. (2003). Shaking Tables for Seismic Simulation overview of the CRdC-AMRA Project, Proceedings of the Workshop on Multidisciplinary Approach to Seismic Risk Problem. Sant'Angelo dei Lombardi, September 22.
MSA Spa. (2008). Mediterr Shock Absorbers - Gimon Dampers, Asti, Italy.
Pavese A., Lai C.G., Calvi G.M., Ceresa P. and Beltrami C. (2005). Reaction mass and foundation of the high-performance, one-degree-of-freedom EUCENTRE shaking table, in Pavia, Italy, Proceedings of The First International conference on Advances in Experimental Structural Engineering, Nagoya, Japan,
Petrini L., Pinho R. e Calvi G.M., Criteri di Progettazione Antisismica degli Edifici (2004). RELUIS, Collana di manuali di progettazione antisismica, Manuale 1, IUSS Press, Pavia. (in Italian).