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SHAKE TABLE TESTING FOR SEISMIC PERFORMANCE EVALUATION OF NON-STRUCTURAL ELEMENTS

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Abstract

In the last years an increasing interest has been addressed to the assessment of the expected mean annual losses of single buildings as well as of building portfolio(s). Field observation of damage/failure in the aftermath of past earthquakes demonstrates that the losses related to the non-structural elements could significantly exceed the structural losses. At the same time, it is worth noting that the damage related to the non-structural elements could affect the functionality of the buildings and critical facilities. Based on these considerations, a detailed assessment of the expected annual losses requires data both on the structural and non-structural elements. In comparison to structural elements and systems, however, there is much less information and experimental data to undertake the assessment and design of non-structural elements for multiple-performance levels. Shake table testing could thus be very useful to assess the seismic performance of non-structural elements and to achieve the required information for loss estimation studies. In this paper, the results of shake table testing of acceleration-sensitive non-structural elements are presented. Equivalent single degree of freedom numerical models of the analyzed acceleration-sensitive non-structural element was developed using the results of the tests and a performance evaluation was carried out.

Keywords: Non-structural elements, shake table testing, loss estimation, fragility analysis, experimental testing.

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1 INTRODUCTION

In the last years many efforts have been spent to develop advanced or simplified methodologies in order to evaluate the earthquake related losses and to ensure a desired building performance for a given intensity of seismic excitation [1-2]. Figure 1 illustrates the four steps required to perform the probabilistic seismic assessment according to the Performance-Based Earthquake Engineering (PBEE) framework [3].

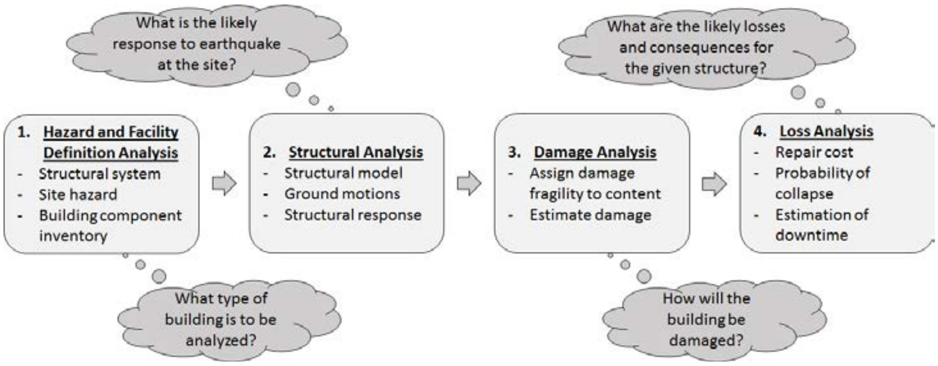


Figure 1: Overview of the four stages of PEER PBEE framework, after [3].

Within the PBEE framework the non-structural elements (NSEs) are of paramount importance, in particular in the damage (step 3) and loss analyses (step 4). During the damage analysis, the probability that a certain element (structural or non-structural) in the building will exceed a certain damage state for a given intensity level is established. At this stage, the availability of fragility functions for both structural and non-structural elements is necessary. In the literature, few experimental investigations are available for non-structural elements [4-7]. For this reason, many fragility functions are based on expert judgments. In loss estimation frameworks, the influence of non-structural elements is of paramount importance because they represent most of the total investments in typical buildings as well as because the non-structural elements show damage for low seismic intensities with respect to the structural ones [8-9]. The influence of non-structural elements has been demonstrated both looking at the damage reported during past earthquakes [10-11] as well as from loss estimation studies on buildings both at single and regional scale [12-14].

The lower seismic performance of non-structural elements is often related to the fact that, in comparison to structural elements, there is much less information and specific guidance available on the seismic design of NSEs for multiple-performance levels. The seismic qualification of NSEs by means of shake table tests could thus be a very useful tool to characterize the seismic response of the NSEs and to evaluate the performance parameters required for their seismic design. In this context, the ASCE 7-16 [15] standard in the US requires that certain critical non-structural elements must be seismically qualified to demonstrate their functionality after being subjected to design earthquake motions. The implementation of such seismic qualification procedure in the US resulted in significant benefits in terms of seismic performance of NSEs (i.e. definition of damage state, improved safety, reduced losses, etc.). According to ASCE 7-16 [15], the seismic qualification testing can be accomplished by either shake table testing, analysis or experience data.

This paper focuses on the definition of a simple procedure to evaluate the seismic performance of non-structural elements through the results of shake table tests. Shake tables tests were used both to qualify the case study non-structural element and to develop a simplified numerical model for the non-structural element of interest such that the outcomes of the testing could be reproduced and complemented with numerical simulations that in turn propagate the effects of uncertainties/variability in the seismic input (and eventually in the non-structural element and the building). For this specific application, the developed numerical model allowed the authors to verify the performance of the tested non-structural element, when installed in a case study building. The simplified modeling approach, based on the results of shake table tests, could significantly help in the assessment of the seismic performance of non-structural elements and could be applicable to both fragility analysis as well as loss estimation frameworks.

2 CASE STUDY NON-STRUCTURAL ELEMENT

The seismic performance of a coaling machine was investigated in this study (see Figure 2). Shake table seismic qualification tests according to AC156 [16] and ISO13033 [17] were performed at the EUCENTRE laboratories using a multi-axial shaking table developed to test non-structural elements. The cooling machine is characterized by the following geometrical properties: length = 1000 mm, width = 890 mm, height = 1980 mm, operating weight = 4.51 kN. The specimen was connected to the shake table using custom interface plates designed by the staff of EUCENTRE. Every plate was connected to the shake table using four M30 bolts; threaded holes were manufactured at the Laboratory to fit the standard base connecting bolts of the tested unit (4 M10 bolts).

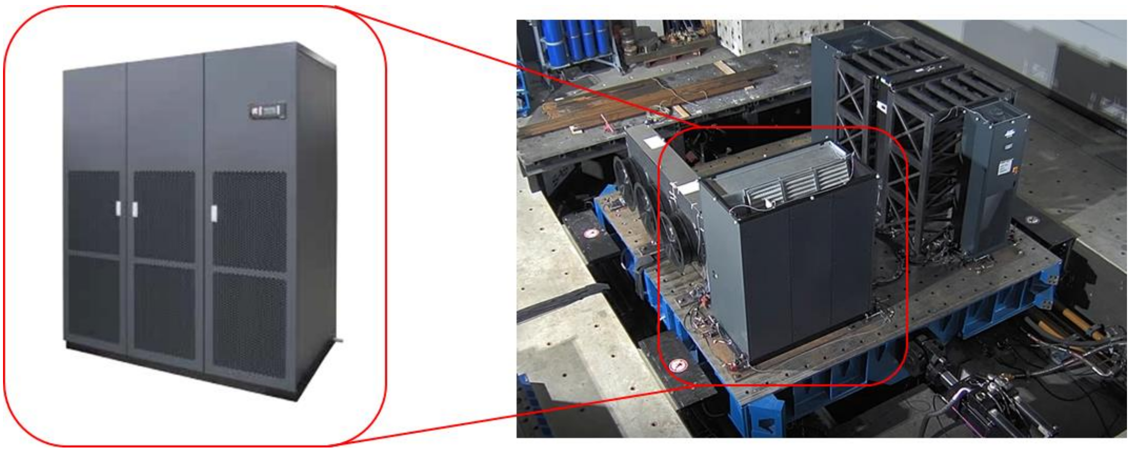


Figure 2: Tested Unit.

3 SEISMIC QUALIFICATION BY SHAKE TABLE TESTING

In this section, the main results of the experimental tests are reported. Only the results related to the shake table test performed according to ISO13033 [17] are reported. The ISO13033 document is the first document in Europe to provide recommendations about the procedures to be adopted for the verification of NSEs capacity and to ensure that those capacities exceed the seismic demands. In this study the original ISO13033 was modified and contextualized to the force-based formulation included in Eurocode 8 [18] to verify/design the non-structural elements.

3.1 Modal identification

Besides the seismic qualification tests, resonance search tests were also carried out, as specified by both AC156 [16] and ISO 13033 [17] test protocols. The resonance search was performed using white noise as input signal. The white noise consisted of a low-level acceleration (1.0 m/s^2 peak nominal) flat spectrum with frequency content from 0.25 Hz to 50 Hz, with approximately 60 seconds duration. Each axis (longitudinal and vertical) was tested separately. The white noise signal was generated using the MTS Systems Corporation Seismic Testing

Execution (STEX) software. Figure 3 shows an example of the acceleration time-histories used for the resonant frequencies search along the longitudinal direction.

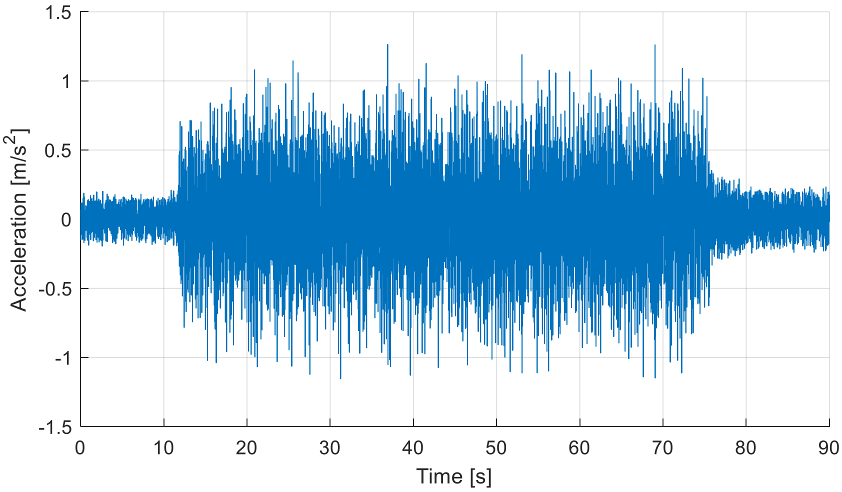


Figure 3: Time history of the white noise input for the longitudinal dof.

Data was analyzed using a transfer function algorithm with a frequency resolution of 0.0625 Hz. Based on the obtained results, the identified fundamental frequencies in the two main directions are 10.2 Hz (X direction) and 10.7 Hz (Y direction), respectively. The damping ratios were also estimated; they resulted equal to 7.1% and 6.3% respectively in the X and Y direction.

3.2 Shake table testing

The seismic qualification tests were performed according to a modified ISO13033 protocol [17] evaluating the Required Response Spectrum (RRS) for different seismic zones in Italy. Tests were conducted simultaneously in the longitudinal and vertical directions, with each direction excited by an independently (uncorrelated) generated table acceleration profile. The bi-directional testing protocol was conducted twice, in two horizontal testing directions rotated 90° from one another.

According to ISO 13033, the RRS can be evaluated for generic or specific buildings. The generic RRS can be evaluated according to the following equations:

$$A_{fleible} = k_{I(u\ or\ s)} \cdot k_{H,i} \cdot k_{R,p,flexible} \tag{1}$$

$$A_{rigid} = k_{I(u\ or\ s)} \cdot k_{H,i} \cdot k_{R,p,rigid} \tag{2}$$

where:

- $A_{flexible}$ is the design horizontal acceleration at the center of mass of flexible NSEs;
- A_{rigid} represents the design horizontal acceleration at the center of mass of rigid NSEs;
- $k_{I(u\ or\ s)}$ is the ground motion intensity factor to be provided by regional and national standards. The subscript u and s are referred to the PGA, modified to account for the soil conditions, at ultimate and serviceability limit state, respectively;
- $k_{H,i}$ is the floor response amplification factor. It is equal to:

$$k_{H,i} = \left[1 + \alpha\left(\frac{z_i}{H}\right)\right] \tag{3}$$

in which α is a parameter function of the type of seismic force-resisting system ($\alpha \leq 2.5$), z_i is the elevation of level i relative to the grade elevation, H is the average roof elevation of the structure relative to the grade elevation;

- $k_{R,p,flexible}$ is the NSEs amplification factor for flexible systems. This parameter is related to the NSEs element period, damping ratio, geometry, method of attachment, and inelastic behaviour. According to ISO 13033, $k_{R,p,flexible}$ can vary between 1 and 2.5 (or more). See ANNEX D of ISO 13033 for more information about the allowed values;

- $k_{R,p,rigid}$ is the NSEs amplification factor for rigid systems. It is assumed equal to 1.0.

The approach proposed by ISO13033 to define the RRS has been modified in order to make it compatible with the design horizontal equivalent static force evaluated according to Eurocode 8 [18]. In Eurocode 8, the spectral acceleration, S_a, used to calculate the horizontal equivalent static force to be applied to the NSE is given by:

$$S_a = \alpha \cdot S \cdot \left[3 \cdot \frac{\left(1+\frac{z_i}{H}\right)}{\left(1+\left(1-\frac{T_a}{T_1}\right)^2\right)} - 0.5 \right] \tag{4}$$

The parameter α in Eq. 4 represents the ratio of the design ground acceleration on soil type A to the acceleration of gravity g, while S is the soil amplification factor. The term in square brackets in Eq. 4 is used to replace the product $k_{H,i} \cdot k_{R,p}$ in Equations 1 and 2. This term accounts for the amplification of the peak floor acceleration due to the location of the NSEs in the building and for the spectral amplification as a function of the ratio of the fundamental period of the NSEs (T_a) to the fundamental period of the building (T_1). In order to define the spectral acceleration for flexible and rigid NSEs in the RRS, the ratio T_a/T_1 is assumed equal to 1 for flexible NSEs and equal to 0 for rigid NSEs. Based on this assumption, it is possible to reproduce the RRS by replacing Equations 1 and 2 with the following two equations:

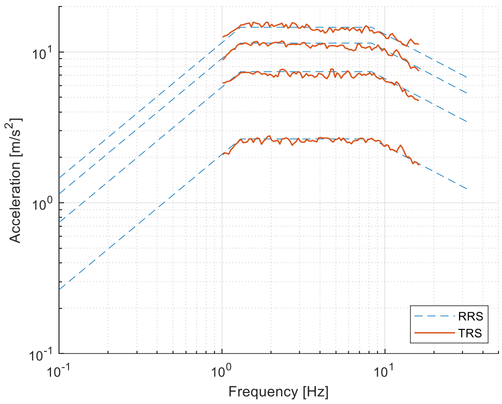
$$A_{flexible} = k_{i(u\ or\ s)} \cdot \left[3 \cdot \left(1 + \frac{z_i}{H}\right) - 0.5 \right] \tag{5}$$

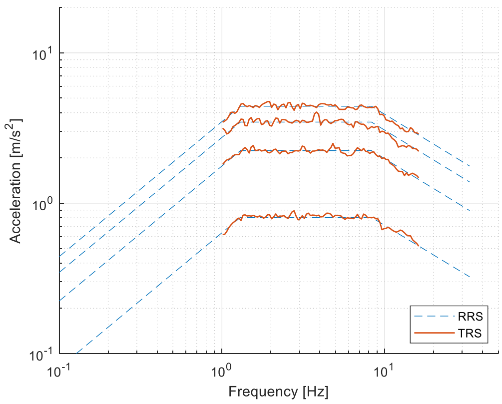
$$A_{rigid} = k_{i(u\ or\ s)} \cdot \left[3 \cdot \frac{\left(1+\frac{z_i}{H}\right)}{2} - 0.5 \right] \tag{6}$$

The vertical RRS is assumed to be 2/3 of the horizontal RRS with $z_i/H=0$, as suggested by AC156 [16]. The corner frequencies are also defined according to AC156: f_0=1.3 Hz, f_1=8.3 Hz and f_3=33.3 Hz.

Four seismic intensity levels were considered for the seismic qualification tests. The selected seismic intensities are representative of the Italian context. The probabilistic seismic hazard assessment proposed by Stucchi et al. [19], that provides the horizontal peak ground acceleration (PGA) with a 10% probability of exceedance in 50 years, was used to select the seismic intensities. In particular, seismic sites characterized by the following PGA on firm soil were taken into account: 0.05g (low intensity), 0.14g (medium intensity), 0.21g (medium-high intensity) and 0.27g (high intensity).

Figure 4 shows the comparison between the RRS and TRS for the vertical and horizontal testing directions. A good match between the RRS and TRS spectra is observed both for the horizontal and the vertical directions.





a) Horizontal component

b) Vertical component

Figure 4: Comparison between RRS according to modified ISO13033 and TRS.

After each test, a visual inspection was undertaken to verify the structural integrity of the cooling machine as well as the presence of damage in the anchoring system. No evident sign of damage was observed after the tests for all considered intensities. Figure 5 shows photographs of the cooling machine after the test in the Y direction for the highest seismic intensity. Only a small distortion of the covering panels was observed, but the structural integrity was not compromised. A functionality test was also performed after the seismic qualification and the machine functioned perfectly.

a) Deformation panels

b) Deformation panels

Figure 5: Visual inspection after the seismic qualification tests.

4 SEISMIC PERFORMANCE EVALUATION OF THE CASE STUDY NON-STRUCTURAL ELEMENT

The results of the shake table seismic qualification tests were used to derive an easy-to-use modeling idealization for the investigated cooling machine and to carry out seismic performance evaluation of it, when integrated in a case study structure. In what follows, account is given of the main characteristics of the studied building archetype along with a description of the ground motion sets and seismic intensity levels assumed for the assessment. Capacity modeling was then treated, leading to seismic performance evaluation of the selected non-structural element.

4.1 Case study structure

A four-story RC moment-resisting frame was designed according to Eurocode 8 [18] assuming a ductility class B (q = 3.75) and a firm soil condition. The frame was supposed to be located in construction site near Cassino, Italy characterized by a design peak ground acceleration of 0.21g (return period of 475 years). The seismic weight of each floor was defined assuming a tributary width of 5 m. The strength of the concrete (f'c) was taken equal to 32 MPa, while the yield strength of the steel was taken equal to 375 MPa. The dimensions of the frame are shown in Figure 6.

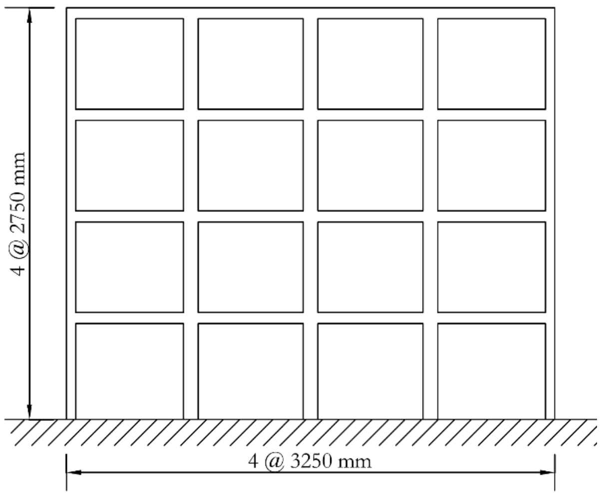


Figure 6: Case study RC frame

The model of the frame was developed using the Opensees software [20]. The numerical models were developed assuming the fiber force-based approach [21]. Inelastic beam-column fiber elements were used to model the frame members, explicitly including geometric and material nonlinearities. A distributed plasticity approach was thus adopted to simulate the spreading of inelasticity over the member length and cross section. The uniaxial confinement model proposed by Chang and Mander [22] was considered to simulate the cyclic behavior of concrete, the hysteretic rules of which were established based on statistical regression analysis on the experimental data from cyclic compression tests conducted by a number of researchers. A simple bilinear constitutive model with isotropic strain hardening was assigned to the steel rebars.

4.2 Ground motion selection

A set of 20 horizontal accelerograms at the site of the case-study buildings were selected from the PEER NGA-West database [23]. Based on a preliminary estimation of the Italian and European seismicity, the selection of the site was carried out in order to represent a medium-to-high seismicity in Italy. The selected intensity corresponds with the medium-high intensity used in the seismic qualification tests.

Hazard-consistent record selection was based on spectral compatibility (matching of the geometric mean) with a conditional mean spectrum according to the methodology proposed by Jayaram et al. [24]. This approach considers the conditional variance given a return period of spectral acceleration at the selected period. The conditional period, T*, to be used for the nonlinear time history analyses (NLTHAs) of RC frame was selected based on the results of eigenvalue analyses, and it is equal to 0.5 sec. Five return period of the seismic intensity were selected to perform the NLTHAs (50, 200, 475, 975 and 2475 years). Figure 7 shows the mean response spectrum and all individual record response spectra for the 20 considered ground motions for a return period equal to 475 years.

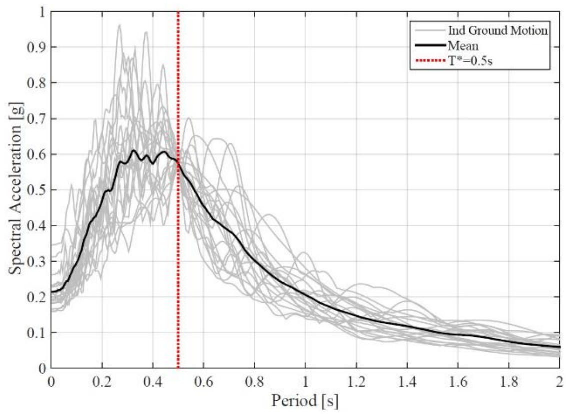


Figure 7: Mean response spectrum and acceleration response spectra of all considered ground motions for a conditional period, T*, equals to 0.5 sec

In order to perform the time history analyses tangent stiffness proportional Rayleigh damping was introduced to the 1st and 3rd modes of the structure. The inherent damping ratio was assumed to be 5.0% of the critical.

4.3 Numerical modeling of the case study cooling machine

For the purposes of this study, the non-structural element of interest was modeled as a simple bilinear idealization, with the initial elastic stiffness being computed according to test data from modal identification. More in detail, the initial stiffness of the bilinear single degree of freedom (SDoF) system was calculated as reported by Equation 7:

$$k = m \cdot (2\pi \cdot f)^2 \tag{7}$$

where m is the mass of the cooling machine and f is its fundamental frequency in the most critical of the two main directions. By taking Y-direction as reference, f equals 10.7 Hz and, hence, k equals 2078 kN/m.

Furthermore, the peak capacity of the bilinear model was assumed to correspond to the shear resistance of the 4 M10 bolts used to anchor the cooling machine to the shake table for the tests. Equation 8 reports how the total shear yield/ultimate capacity of the bolts ($V_{y/u}$) was computed:

$$V_{y/u} = 0.6 \cdot A_s \cdot n \cdot f_{y/u} = 0.6 \cdot A_s \cdot 4 \cdot f_{y/u} \tag{8}$$

where A_s is the cross-section area of the bolt, n is the number of bolts, and $f_{y/u}$ is the yield/ultimate stress of steel. The class of the bolts is 4.6, meaning that f_y is equal to 240 MPa. The bilinear model is presented in the S_a-S_d format in order to easily undertake performance evaluation and compare the results with the absolute acceleration – relative displacement floor response spectra obtained by the NLTHAs. In order to convert $V_{y/u}$ in the acceleration format, the maximum shear capacity is simply divided by the seismic mass of the cooling machine.

A simple parametric study was also performed to evaluate the bilinear model varying the bolts' diameter. To this end, three diameters were considered: 6, 8 and 10 mm. Figure 8 shows the S_a-S_d relationship of the calculated bilinear models.

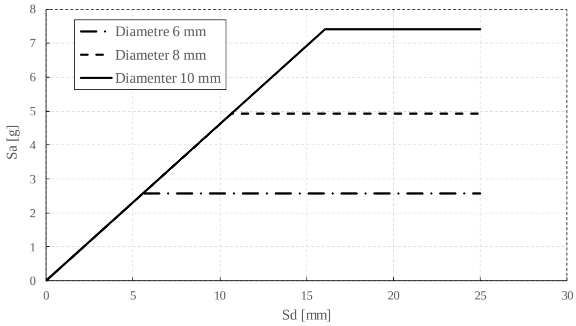
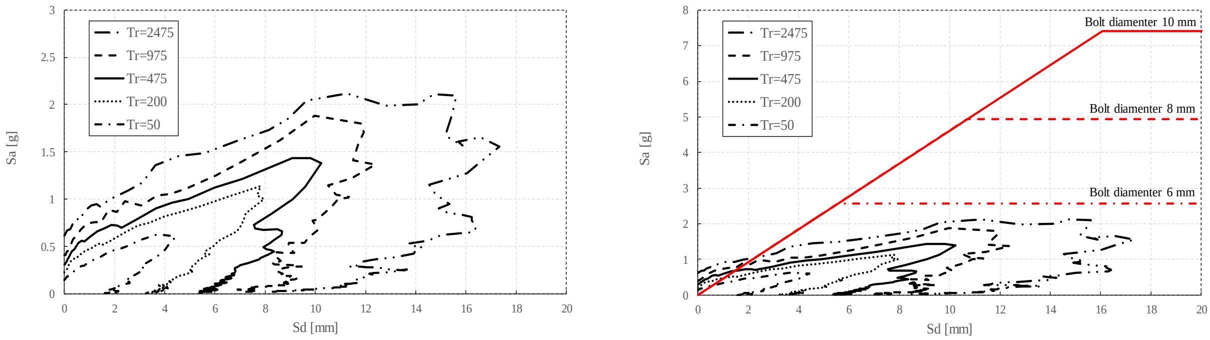


Figure 8: Bilinear model tested cooling machine

4.4 Seismic demand and assessment of the non-structural element

In order to carry out the performance evaluation, the absolute acceleration – relative displacement floor response spectra for the analyzed case study building were first calculated, for different return periods of the seismic intensities. Note that the cooling machine is supposed to be installed at the third floor of the building, and hence Figure 9a reports the median S_a-S_d response spectra corresponding to this floor. As reported in Figure 9b, all considered bolt's diameters satisfy the performance evaluation, for all considered return periods. It is worth to be noted that high return periods, such as 975 and 2475 years, are generally not considered for non-structural elements performance evaluation but are reported here as illustrative example.



a) Floor response spectra b) Performance evaluation

Figure 9: Floor response spectra and performance evaluation

As a closing remark, it is worthy of mention that the adopted assessment methodology could be also used and extended to perform fragility analysis and develop fragility functions in which probabilities of exceeding given damage state conditions are expressed in terms of spectral accelerations at the fundamental period of the non-structural elements $S_a(T_1)$. The assumption of $S_a(T_1)$ as engineering demand parameter could significantly improve the loss estimation evaluation for acceleration-sensitive non-structural elements, with respect to the peak floor accelerations, because it takes into account the dynamic amplification due to the non-structural element stiffness.

5 CONCLUSIONS

The damage observed during past earthquakes demonstrated the importance of the non-structural elements in the performance-based seismic design of buildings. However, few experimental data are available to characterize the performance on non-structural elements and to carry out seismic performance assessment. Within this context, this paper describes the results of shake table seismic qualification tests on a cooling machine. The shake table tests were first presented and then used to define a simple SDoF bilinear model of the investigated cooling machine. The proposed mechanics-based idealization was used to undertake the performance evaluation of the case study cooling machine, when installed in a typical four-storey RC frame. The results of the assessment demonstrated the effectiveness of the proposed approach, which could be extended and applicable to fragility analysis assuming a new, and more consistent, engineering demand parameter.

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