



A FRAMEWORK FOR QUANTIFYING SEISMIC PERFORMANCE FACTORS FOR NON-STRUCTURAL ELEMENTS

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Abstract

The evaluation of the seismic performance of non-structural elements (NSEs) has gained special relevance in the last decades from the earthquake engineering community as a result of several aspects. The losses estimated following the seismic events that have occurred in urban regions in the last two decades, repeatedly showed that the NSE losses often exceed that of the structural components. This issue is partly due to the fact that the investment associated to the NSEs is on average higher than the cost of the structure, and partly due to the higher vulnerability of NSEs at lower seismic intensities if compared to their structural counterpart. Additionally, in the performance-based seismic design and assessment framework, the harmonization between the performance of both non-structural and structural elements plays a fundamental role, since a non-acceptable NSE performance level can completely compromise the global performance and functionality of a facility. Several methodologies have been developed to seismically design and analyze NSEs in the last decades. However, a complete comprehensive methodology to quantify the performance of NSEs is still not available due to the multitude of NSE typologies and to the difficulties to consider all the involved parameters, such as the prediction of a seismic demand representative of the different possibilities of building seismic-force-resisting systems (SFRSs) containing the NSEs. For building structures, the FEMA P695 methodology has been developed. It provides a standardized and objective methodology that defines how to calibrate seismic performance factors (SPFs), namely the response modification factor (R), the system overstrength factor (Ω_0), and the deflection amplification factor (C_d), for new SFRSs proposed for inclusion in model building codes in the United States. Nowadays, a proper equivalent methodology for NSEs is not yet available. In this paper, a standardized framework to evaluate performance and quantify SPFs for new and existing NSEs systems is developed. The new information required to implement the proposed standardized framework is highlighted. An illustrative example of the proposed framework is provided in which the quantification of the behavior factor (q_a) for suspended piping restraint installations designed according to Eurocode 8 is performed to meet various performance objectives. It is believed that the development of the proposed framework would provide higher uniformity in performance evaluation of NSEs, thereby providing a path in which designers, researchers and stakeholders can effectively evaluate, compare and even propose new NSEs components and systems and seismic design procedures for inclusion in model building codes.

Keywords: Non-structural elements; performance; seismic performance factors; methodology



1. Introduction

In the past several years, the occurrence of several seismic events in urban regions has shown the large influence of non-structural elements (NSEs) in various facilities' global performance and associated losses. There are two main issues that justify such a large non-structural influence: 1) the higher vulnerability of NSEs with respect to structural systems; and 2) the higher investment associated to NSEs in comparison with the total cost of the structure [1]. Additionally, several loss estimation studies have shown that losses related to NSEs are often higher to those of the structural elements, especially at lower seismic intensities [2, 3]. As a result, the harmonization of the performance of both non-structural and structural elements became a key factor in the performance-based seismic design and assessment framework.

The importance of NSEs in the performance of modern code compliant facilities has been recognized by the scientific community in the last years, as demonstrated by several studies dealing with the improvement of design and analysis methodologies for NSEs [4]. However, one of the main issues in the application of performance-based seismic design to NSEs concern the absence of a standardized methodology to quantify their seismic performance factors (SPFs) adequately. Assessing performance of NSEs entails a series of non-trivial difficulties that are absent in the performance evaluation of structural systems. When dealing with NSEs, some of the main challenges that must be dealt with are the determination of a seismic demand, the definition of archetypes models according to different typologies of NSEs, the selection of performance objectives and the calibration of SPFs (e.g. response modification factor and overstrength, among others).

The FEMA P695 document [5] provides a standardized methodology to quantify SPFs for new seismic force-resisting systems (SFRSs) to be included in building codes in the United States (US). The SPFs consists of the response modification factor (R), the overstrength factor (Ω_0) and the deflection amplification factor (C_d). These parameters can be used for the seismic design of structural systems in the US. The SPFs can be also used to assess the performance of new SFRSs that developers/manufacturers would like to see included in seismic design codes provisions. A similar approach would be of particular interest in the current NSEs context. Although increasing amount of research studies addressing NSEs has been conducted in the last years, and several new non-structural systems have been proposed by manufacturers, a standardized procedures to quantify NSEs SPFs, which would allow to perform their performance-based seismic design, is still missing.

To this aim, the main objective of this paper consists in establishing a general framework to quantify SPFs for NSEs. The FEMA P695 methodology was used as a reference guide in the development of the proposed framework. Although FEMA P-695 is intended for building SFRSs, the general organization of the methodology can be adapted for NSEs evaluation. The application of the proposed framework has been demonstrated through an illustrative example, which deals with the evaluation of the behavior factor (q_a) of suspended piping restraint installations to be designed according to the Eurocode 8 [6].

2. General overview of the framework

Analogous to the FEMA P695 methodology, the framework proposed herein is based on a threefold approach: 1) conduct seismic design of archetype NSEs according to the code design provisions; 2) conduct seismic analyses using advanced nonlinear modelling techniques under representative floor motions as an input; and 3) conduct performance assessment using risk-based techniques.

The flowchart of the proposed framework is presented in Fig. 1. The various phases of the framework phases coincide with those of the FEMA P695 methodology, with the exception of an additional phase entitled "Establish Seismic Demands". This additional phase is required because the determination of the seismic demand on NSEs is far less trivial if compared to the one of a building SFRS. The dynamic interaction between



the building's SFRS, the ground motion and the NSEs leads to the seismic demand on the NSEs being dependent on the dynamic response of the structural system.

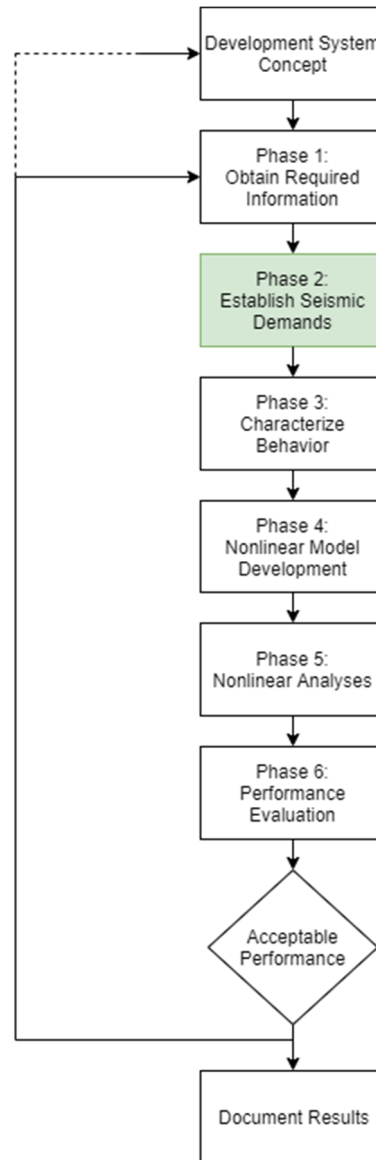


Fig. 1 – Flowchart of the proposed general framework for the quantification of seismic performance factor for non-structural elements

The implementation of the framework starts with the initial idealization of a non-structural system (Development System Concept). At this initial stage, it is crucial to determine the main characteristics of the non-structural system, as for example the materials to be used, recommended configurations, plastic energy dissipation mechanism, the range of application of the system and the typology of NSEs for which it is intended. As in structural systems, depending on the level of innovation that the system has, the required documentation may vary considerably if compared to more traditional non-structural systems.

The quantity of information required to conduct performance evaluation can be large depending on the system under study, which is treated in Phase 1 of the framework (Obtain Required Information). At this stage, it is important to highlight two important aspects: 1) the quantification of SPFs involves the use of advanced non-linear analysis techniques, which requires the definition and calibration of several parameters, added to



the fact that dealing with non-deterministic loads (such as earthquake loads) increases the complexity of the assessment techniques to be used; and 2) there is far less information available nowadays regarding the design and analysis of NSEs if compared to that of building SFRSs. These issues lead to code seismic design provisions for NSEs that are commonly based on engineering judgement rather than on technical data [7]. The required information needed to quantify SPFs should include both design requirements and experimental data for the non-structural system being proposed, so that its key behavioral characteristics can be successfully simulated. The recent experimental study conducted by Perrone *et al.* [8] on suspended piping restraint installations is a good example of the quantity of information required to satisfy Phase 1 of the proposed framework, as discussed later in the illustrative example.

One of the key features of the framework consists in the evaluation of the seismic demand at which the NSEs will be subjected (Phase 2 – Establish Seismic Demand). The evaluation of the seismic demand is a complicated issue since it is influenced by the dynamic filtering of the particular supporting structure in which the NSEs are installed. The direct analysis, in which the structure and the NSEs are modelled in the same numerical model, produces several numerical issues that are not easily dealt with in an explicit manner even with current state-of-the-art modelling approaches. For this reason, the cascading analysis methods are usually the most suitable approaches to analyze NSEs under seismic loading. These methods consist in uncoupling the responses of the building's SFRS and the NSEs, so that the NSE's response can be estimated by using as an input the response of the building's SFRS at the given NSE location. The framework herein presented proposes the use of the cascading analysis approach in order to determine the seismic demand for most non-structural systems.

Performance evaluation of a particular system (structural or non-structural) requires the definition of a representative sample of its possible design outcomes. This is the intent of the Phase 3 entitled "Characterize Behavior". Given that it is not feasible to consider all possible configurations that can be obtained from a given set of design requirements or intended range of a system's applications, it is required to define representative archetypes of the system. These archetypes are used in order to represent typical behavioral characteristics that can be obtained within the bounds of application of the proposed system. To perform the design of archetypes, trial values of the SPFs are used. These values will be assessed in the performance evaluation phase. It should be noted that the definition of the archetypes depends both on the typology of non-structural systems and NSEs under consideration. Also, NSEs archetypes possess a higher variability in comparison to structural systems, due to the large number of intended uses that they can have. For example, partition walls and suspended piping restraints differ significantly in terms of typical geometric configurations, response mechanisms and usual locations within building structures, leading to completely distinct archetype layouts. This makes the archetype definition process for NSEs more challenging than for SFRSs.

The quantification of performance factors requires the use of advanced nonlinear analysis techniques. Nonlinear numerical analyses are required to predict the damage mechanisms that could lead to the loss of functionality or collapse of the analyzed systems. This aspect is addressed in Phase 4 (Nonlinear Model Development) and Phase 5 (Nonlinear Analyses) of the framework. Non-structural archetypes' design is carried out in Phase 4 using the design requirements indicated in Phase 1. Once the archetypes are designed, it is possible to develop the numerical models using the nonlinear material and component properties defined and/or calibrated in Phase 1. As input of the nonlinear analyses (Phase 5), the floor motions generated in Phase 2 are used. The output of this process constitutes the basis for the performance evaluation to be carried out in the final phase of the framework.

The performance assessment of the system under evaluation corresponds to the last phase of the framework (Phase 6 - Performance evaluation). A statistical fragility analysis of the output generated during the analyses is carried out. At this stage, it is crucial to establish the performance objectives (acceptance criteria) that the non-structural system under evaluation is aiming to achieve. Depending on such performance objectives, the correctness of the SPFs assumed at the beginning of the process is evaluated. If the assumed SPFs does not comply with the established performance objectives, then the nonstructural system must be redefined by either redesigning the archetypes assuming different SPFs and/or performing changes in the



design requirements and/or the limits of the range of permissibility of the system until the acceptance criteria are met.

In the next section, an illustrative example of application of the proposed framework for the quantification of the behavior factor for suspended piping restraint installations designed according to Eurocode 8 is presented to demonstrate the application of the various phases of the procedure.

3. Illustrative example of the framework

With the purpose of demonstrating the implementation of the framework presented above, an illustrative example is presented in this section. The example deals with the quantification of the behavior factor (q_a) for suspended piping restraint installations to be designed according to Eurocode 8. The behavior factor can be thought as the equivalent parameter within Eurocode 8 to the response modification factor (R_p) in US standards. For more information regarding the behavior factor, the reader can refer to [9].

The development of the system concept will not be presented in this illustrative example. This phase is required when a new non-structural system is initially conceptualized for a given application. Since the example deals with a system that is commonly used nowadays, it is assumed that all the relevant characteristics of the system are well identified.

3.1. Phase 1: Obtain required information

The information required for the quantification of SPFs should include both design requirements and experimental data in order to accurately predict the response of the system under evaluation.

The design requirements provided in Chapter 4 of Eurocode 8 for non-structural elements [6] were followed in this illustrative example. According to Eurocode 8, the horizontal design seismic force to be applied at the centre of mass of NSEs, F_a , can be calculated as:

$$F_a = \frac{S_a W_a \gamma_a}{q_a} \quad (1)$$

where W_a is the operating weight of the element, γ_a is the importance factor of the element and S_a is the seismic coefficient applicable to non-structural elements, calculated as:

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

In Equation 2, α is the ratio of the design ground acceleration on type A ground to the acceleration of gravity, S the soil factor, T_a is the fundamental period of the NSE, T_1 is the fundamental period of the supporting structure, z is the elevation at the location of the NSE above the level of application of the seismic action, and H is the building height measured from the foundation or from the top of a rigid basement.

The value of the behavior factor (q_a) associated with suspended piping restraints installations in Eurocode 8 is equal to 2.

In regards to the experimental data, two sources were considered. For characterizing the response of the suspended piping restraints, the experimental program carried out by Perrone *et al.* [8] was used. In this experimental research, several suspended piping trapeze restraint assemblies were tested under monotonic and reverse cyclic loading in order to characterize their seismic behavior. Among the data reported by the authors,



the maximum load capacities, initial stiffness values, effective yield displacements and ultimate displacements will be used to establish performance objectives for the system, as discussed in Section 3.6. Additionally, in order to evaluate the seismic demand at which the NSEs are subjected, the population of 100 reinforced concrete frame buildings proposed by Perrone *et al.* [10] was chosen as a representative sample of supporting structures in which suspended piping trapeze restraint assemblies can be installed.

3.2. Phase 2: Establish seismic demands

Accurate prediction of seismic demands in NSEs needs to account for the dynamic response of the building SFRSs in which the NSEs are installed. Therefore, it is clear that depending on the type of supporting structure the seismic demand in a particular NSE can differ significantly. With the sole purpose of demonstrating the implementation of the framework, the reinforced concrete frame portfolio used in [10] was selected. It consists of 100 reinforced concrete frames that were generated using a Monte Carlo simulated design. The geometry, and a summary of the approach followed for the consideration of the random variables in the Monte Carlo simulation, are shown in Fig. 2. Further implications of the choice of this building population for the application of the proposed framework will be discussed in Section 4.

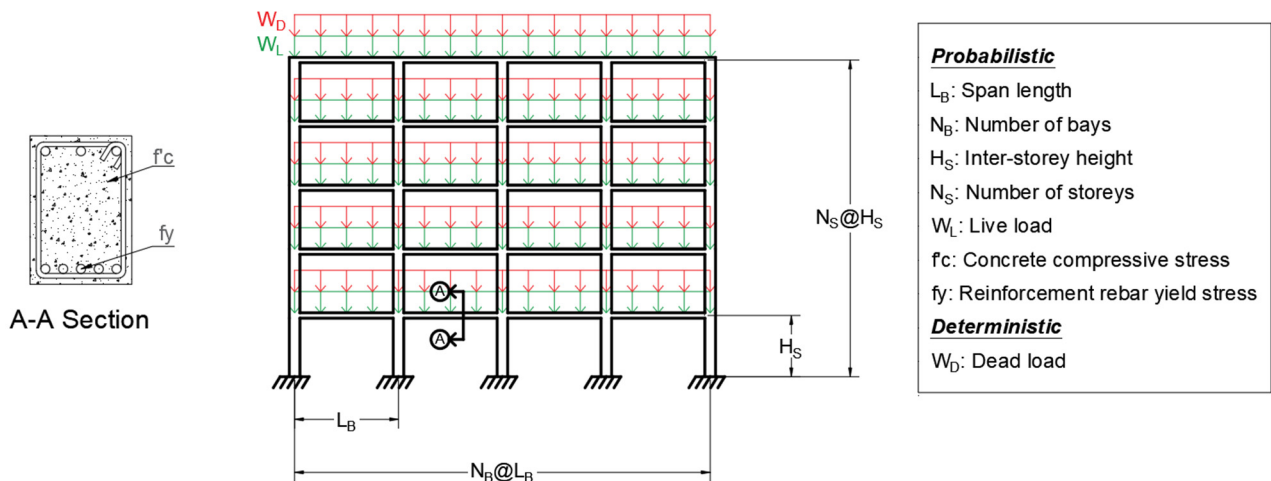


Fig. 2. Geometry and variables considered in the Monte Carlo simulation for the generation of the building portfolio used for the analyses (Perrone *et al.* (2019))

As mentioned in Section 2, the cascading approach was used in order to generate floor motions to be used as input for nonlinear time history analyses to be performed in Phase 5 of the methodology. Two sets of floor motions were constructed for two different return periods: 70 years and 475 years. The reasons for choosing these two values of return periods is discussed in Section 3.6. Only top floor motions were considered in this example for simplicity. A sets of 20 accelerograms were selected for a site close to the city of Cassino in Italy from the PEER NGA-West database [11] in order to perform the building nonlinear time history analyses. The records selection was performed based on spectral compatibility with a conditional mean spectrum following the methodology proposed by Jayaram *et al.* [12]. For more information regarding the records selection, the reader can refer to [10]. Once the records sets were defined, nonlinear time-history dynamic analyses were performed in order to obtain the floor acceleration time histories for all the 100 buildings of the portfolio. The median absolute acceleration floor response spectrum (FRS), along with the envelope of the minimum and maximum FRS, for the two return periods of the seismic intensity are presented in Figure 3.

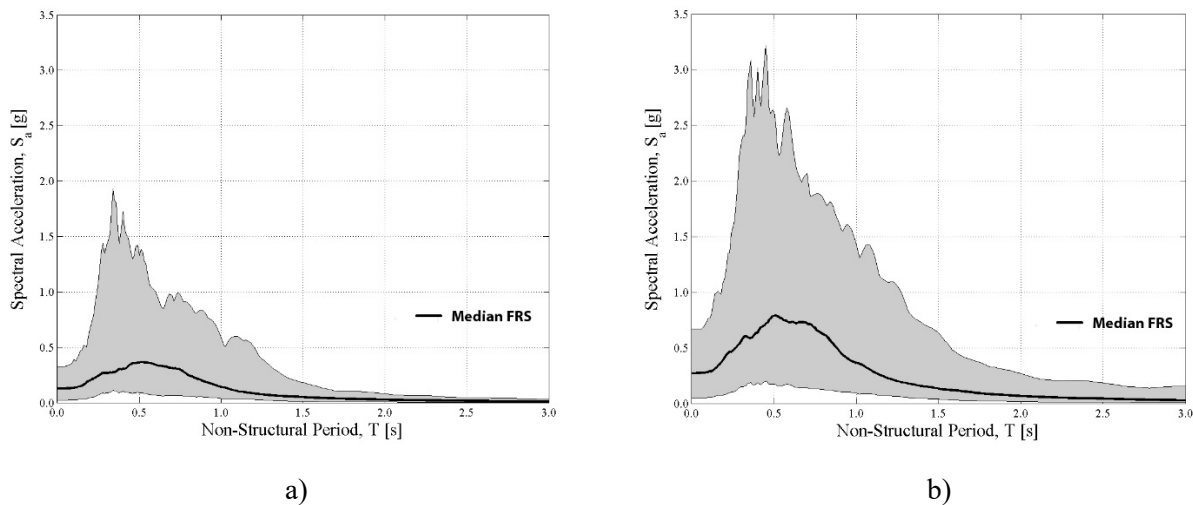


Fig. 3 – Median and envelope of the minimum and maximum absolute acceleration floor response spectra considered for a) 70 years return period and b) 475 years return period

3.3. Phase 3: Characterize behavior

The behavior characterization of a system requires the definition of the system archetypes. Ideally, the definition of the archetypes should reflect the range of design parameters and system features that can be obtained by the application of the design requirements and that affect the system response. This phase is of paramount importance in order to obtain a performance evaluation that is statistically representative of the intended design space. Several configuration and behavioral aspects must be considered. For the case of suspended piping restraint installations, among the most important issues that should be considered for the development of the system archetypes are:

- Configuration: The range of dimensions permitted within the design requirements, different typologies of piping restraint installations (for example, channel and rod trapezes) and amount of gravity load tributary to the system.
- Intended use: The type of suspended piping, the type of fluid to be carried by the piping and the possibility of considering live loads due to maintenance of the system.
- Component type: The typology of the connections to be used in the system. For example, hinged connections or fully restrained connections.
- Period: The range of periods allowed in the design requirements.
- Occupancy: The type of occupancy of the supporting structure.
- Design ground motion: The intensity of the ground motion used for the design.
- Gravity load: Low and high levels of gravity load.
- Strength: Components overstrength and capacity design requirements.
- Stiffness: Deformation limits within the design requirements.
- Inelastic deformation capacity: Redundancy, capacity design requirements and components detailing.



In order to account for the variability that all of these configurations and behavioral issues induce to the system, a large number of archetypes would be needed. Due to space limitations, and since the main objective of this illustrative example is to demonstrate the effectiveness of the proposed framework, only two archetypes were considered. One of the suspended piping restraint trapeze installations tested by Perrone *et al.* [8] was selected for this illustrative example. The selected configuration is a typical channel trapeze restraint installation used to provide lateral bracing in the transverse direction of suspended piping systems (Figure 4). As explained in the next section, two archetypes were designed and analyzed by varying the assumptions made to design the considered suspended piping restraint trapeze installation.

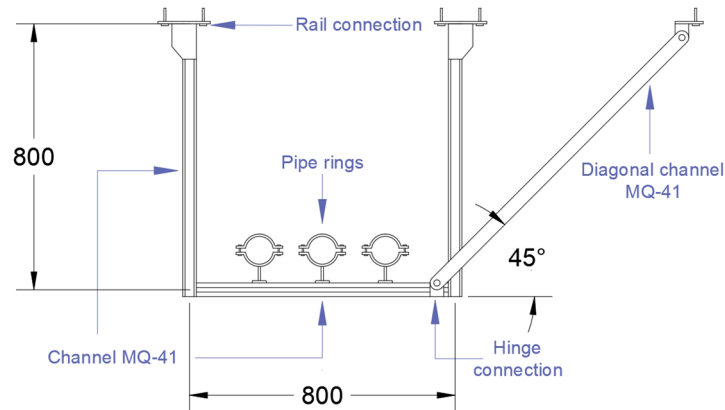


Fig. 4. Archetype selected for illustrative example of the framework. Units in millimeters

3.4. Phase 4: Nonlinear model development

The first part of the nonlinear model development phase consists in carrying out the design of the archetype models. As discussed in Section 3.1, the Eurocode 8 design requirements were selected for this illustrative example. In those provisions, the ratio of the period of the NSE to the fundamental period of the supporting structure (T_a/T_1) is needed in order to compute the seismic design force of the NSE per Equation (1). The design can be conducted for two limit conditions: 1) the ratio T_a/T_1 is assumed equal to 1, that is the case in which the NSE is in resonance with the fundamental vibration mode of the supporting structure; and 2) the ratio T_a/T_1 is equal to 0, that corresponds to the condition in which no amplification of the seismic demand is expected because the NSE is assumed to be rigid. With the purpose of evaluating the influence of this design assumption, these two extreme conditions were considered in the design of the archetype configuration previously selected in Section 3.3. Trial values of the behavior factor (q_a) equal to 1, 2, 3 and 4 were used to perform the design and run all the analyses. Currently, Eurocode 8 assigns a value of $q_a = 2$ for the seismic design of suspended non-structural systems in Equation (1). During the performance evaluation phase (Section 3.6), the most suitable q_a factor to be used in design can be obtained based on meeting or not the performance objectives. Since the maximum strength of the braced trapeze assembly is known from the experimental data, the design phase is carried out to evaluate the maximum length of the pipes, and the corresponding mass, for each of the case studies.

The second part of Phase 4 is the generation of the archetype nonlinear models. The accurate prediction of the mechanical behavior of a system can be done through several types of numerical models. In earthquake engineering, the two usual approaches are the full continuum finite element models and the phenomenological models. Full continuum finite element models use several discretization techniques that allows to represent more reliably the underlying mechanical response of a structural or non-structural system at the expense of higher computational burden. Phenomenological models, on the other hand, make use of spring elements to



describe previously calibrated nonlinear force-deformation responses. For the current example, the phenomenological modelling approach was selected, as it allows the implementation of simplified models that significantly reduces the computational overhead, making them more suitable to perform large number of analyses. The open source platform OpenSees [13] was used in order to construct the nonlinear models, which consisted in single-degree-of-freedom systems in which the mass was defined according to the archetype design and the hysteretic behavior was modelled with the “ZeroLength” element using the “Pinching4” material from the OpenSees element and material libraries. The inherent viscous damping was modelled by Rayleigh damping using a value of 2% of critical damping for the first and second mode, given that all the hysteric energy dissipation was directly simulated through the hysteretic behavior of the braced trapezes. The parameters required to reproduce the hysteretic response of the system archetype were obtained from [8]. The experimental and numerical hysteresis loops for the braced trapeze assembly considered in this illustrative example are presented in Figure 5.

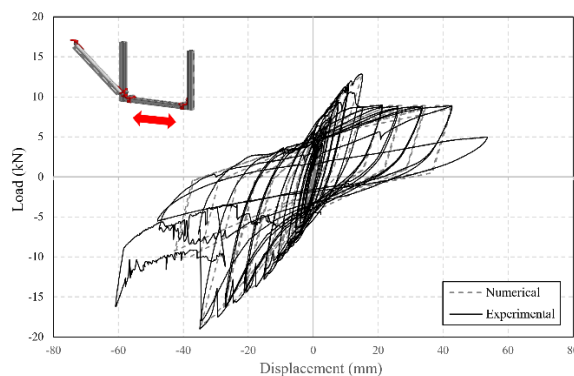


Fig. 5 Numerical and Experimental cyclic load-displacement response for the considered suspended piping restraint trapeze installation

3.5. Phase 5: Nonlinear analysis

Once the nonlinear models are developed, the next phase in the framework is to conduct the nonlinear time history dynamic analyses of the archetypes. As discussed in Section 3.2, two sets of 20 accelerograms each were applied to the 100 reinforced concrete frames developed by Perrone *et al.* [10] to generate top floor motions. These analyses resulted in a total of 4,000 floor motions, which were used to perform nonlinear time history dynamic analyses on the nonlinear braced trapeze models developed in Section 3.4. Given that two archetypes were designed, and a total of four different behavior factors (q_a) were evaluated, the total number of analyses rose to 32,000. Clearly, the large number of analyses required is one of the limitations of the approach chosen for the establishment of the seismic demands and will be further commented in Section 4. The summary of the obtain results for the whole set of analyses is presented in Table 1 in terms of median displacements of the braced trapeze at the level of the pipe rings (see Figure 4) and relative to the supporting top floors.



Table 1. Summary of results expressed in terms of median displacements relative to the supporting top floors

Median displacement assuming $T_a/T_1=0$ [mm]				
	$q_a=1$	$q_a=2$	$q_a=3$	$q_a=4$
$T_r=70$	3.0	8.1	13.7	19.1
$T_r=475$	7.3	31.2	58.8	59.5

Median displacement assuming $T_a/T_1=1$ [mm]				
	$q_a=1$	$q_a=2$	$q_a=3$	$q_a=4$
$T_r=70$	0.9	2.7	4.9	7.2
$T_r=475$	2.4	6.5	12.5	24.5

3.6. Phase 6: Performance evaluation

The final phase of the proposed framework is the performance evaluation of the non-structural system. At this stage, it becomes crucial to establish the performance objectives that are intended to be achieved. As shown in Table 2, two performance objectives corresponding to two different seismic hazard levels were investigated for illustrative purposes. The first performance objective is associated with damage prevention in the sway braced trapezes under frequent earthquakes so that their functionality is not compromised, while the second performance objective is related to life safety of the occupants of the building structure under earthquakes with intensities consistent with the design earthquake. These performance objectives are consistent with the recommendations provided in NIST GCR 18-917-43 [14] and were defined based on the available experimental data [8], in which the yield displacement and the ultimate displacement are provided for the tested braced trapeze assemblies. The yield displacement is defined as the ratio between the maximum load capacity (Q_m) and the initial stiffness, while the ultimate displacement as the deformation corresponding to a drop of 20% of Q_m in the post-peak range. As regards to the hazard levels, the frequent earthquake was defined according to the recommendations provided by the Italian Building Code [15]. According to this reference, a good characterization for a frequent earthquake corresponds to approximately a 60% probability of exceedance in 50 years, which is equivalent to a return period of 70 years. As for the design earthquake, a probability of exceedance of 10% in 50 years, which corresponds to a 475 years return period, was selected, as indicated in the ASCE 7-16 [16] and the Eurocode 8 provisions.

Table 2. Performance objectives for the braced trapeze assemblies

Performance Objective	Hazard Level	
	Frequent Earthquake (70 years return period)	Design Earthquake (475 years return period)
Not exceeding yield displacement	X	
Not exceeding ultimate displacement		X

Yield displacement (from cyclic test data) = 13.1 mm

Ultimate Displacement (from cyclic test data) = 24.9 mm



According to the established performance objectives, the results obtained from the nonlinear analyses were evaluated. As shown in Table 1, the cells highlighted in green represent the q_a values meeting the performance objectives. From this results, it can be observed that the values of the behavior factor (q_a) that fulfils the acceptance criteria are:

$$q_a = 1 \quad \text{for} \quad \frac{T_a}{T_1} = 0 \quad (3)$$

$$q_a = 4 \quad \text{for} \quad \frac{T_a}{T_1} = 1 \quad (4)$$

The two obtained values for the behavior factor (q_a) differ from the value of 2 specified by Eurocode 8. Furthermore, the results obtained illustrates the flexibility in the selection of adequate behavior factors based on different assumptions of the ratio T_a/T_1 . Because of this, one modification that could be considered in future design procedures of Eurocode 8, is to remove the ratio T_a/T_1 from the design equation and use different values of the behavior factor (q_a) depending on the typology of the NSE and how prone it is to be in resonance with the fundamental vibration mode of the supporting structure. Obviously, more suspended piping restraint trapeze installation archetypes would need to be evaluated before this recommendation can be implemented.

4. Conclusions

In this study, a framework for the quantification of seismic performance factors (SPFs) for non-structural elements (NSEs) was proposed based on the FEMA P695 methodology applicable to structural systems. The key features differentiating the performance evaluation process between building structures and NSEs were identified in order to develop a framework capable of addressing the main challenges of the performance assessment of NSEs. Based on the proposed framework, an illustrative example for the quantification of the behavior factor (q_a) for suspended piping restraint trapeze installations to be designed according to Eurocode 8 was carried out. From the proposed framework and the illustrative example, the following conclusions can be drawn:

- The proposed framework addresses all the main phases that have a direct impact on the global nonlinear response of a non-structural system, allowing for an accurate assessment of its performance.
- The results of the illustrative example indicate that a $q_a = 1$ would be adequate for the seismic design of suspended piping restraint trapeze installation when assuming no amplification of the NSE response with respect to the one of the supporting structure ($T_a/T_1 = 0$), while a $q_a = 4$ is more suitable when assuming resonance between the non-structural and structural responses ($T_a/T_1 = 1$). This demonstrates the flexibility of the proposed framework in which different design equations could be used to achieve similar performance objectives. It also demonstrates that some design variables difficult to evaluate accurately (in this case T_a/T_1) could be eliminated as variable of the design procedure.
- Even though only one typology of NSE was used to illustrate the procedure in this study, the proposed framework can be applied to any typologies of NSEs for which the proper information has been generated. Because of this, the proposed framework can be considered as a good basis for the future development of a comprehensive and standardized methodology for the quantification of SPFs of NSEs.
- As it was shown in the illustrative example, the approach proposed for the definition of the seismic demands requires a large number of analyses. In contrast with the assumption made for the building population in the illustrative example (which only considered reinforced concrete frames), comprehensive performance assessment of NSEs should consider the possibility of several types of seismic force-resisting systems containing the NSEs. For this reason, a simplified approach to evaluate



the seismic demand is required for future applications. The authors are currently investigating this issue.

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