DOI: 10.1111/mice.12951

INDUSTRIAL APPLICATION

COMPUTER-AIDED CIVIL AND INFRASTRUCTURE ENGINEERING WILEY

A shake table protocol for seismic assessment and qualification of acceleration-sensitive nonstructural elements

Martino Zito[1] | **Danilo D'Angela**[1] | **Giuseppe Maddaloni**[2] | **Gennaro Magliulo**[1,3]

[1]Department of Structures for Engineering and Architecture, University of Naples Federico II, Naples, Italy

[2]Department of Engineering, University of Benevento Sannio, Benevento, Italy

[3]Construction Technologies Institute, National Research Council, Naples, Italy

Correspondence
Gennaro Magliulo, Department of Structures for Engineering and Architecture, University of Naples Federico II, Via Claudio 21, 80125 Naples, Italy.
Email: gmagliul@unina.it

Funding information
Italian Department of Civil Protection (DPC), Grant/Award Number: DPC - ReLUIS 2019—2021 WP17; Italian Ministry of University and Research (MUR), Grant/Award Number: PRIN 2020

Abstract

A shake table protocol for seismic assessment and qualification of acceleration-sensitive nonstructural elements (NEs) is developed. The paper critically reviews existing protocols and highlights their criticalities, pointing out the need for the development of novel assessment and qualification approaches and protocols. The protocol is developed in light of these criticalities, considering the most recent advances in the field and the specific expertise of the research team. The most significant and contributing parts of the developed protocol consist of the definition of novel required response spectra and the generation of signals for seismic performance evaluation tests. The reliability and robustness of the protocol are evidenced in the paper considering real-floor motions as a reference, also proving the superiority of the developed protocol with respect to the reference alternatives. The defined approach and procedures are generally applicable and easily extendable to different case studies, as the process is highly versatile and modifiable. The implementation of the developed approach and protocol in the literature and in practice will significantly enhance seismic assessment and qualification of acceleration-sensitive NEs. This will possibly have a strong impact on public safety and economy.

1 | INTRODUCTION

1.1 | Technical background

The estimation and mitigation of the effects of catastrophic events such as earthquakes on infrastructural and structural systems are among the most challenging aims of the modern era and are increasingly pushing researchers and engineers to develop innovative solutions (Javadinasab Hormozabad et al., 2021). Design and assessment of nonstructural elements (NEs) according to current regulations, codes, and guidelines are typically based on performance-based earthquake engineering (PBEE; ASCE, 2016; CEN, 2005; INN, 2015; MIT, 2018; NRCC, 2015; NZS, 2009; UTCB, 2013). These documents often provide methods and criteria for both behavior assessment and performance evaluation of NEs; seismic qualification and certification procedures are also defined in ASCE 7–16 (ASCE, 2016). Seismic performance evaluation of NEs typically consists in (a) assessing the seismic capacity of NEs associated with relevant damage states (DSs) and (b) correlating this capacity to consistent measures of seismic demand or targets, often defined by regulations/codes. The definition of both seismic capacity and demand is based on the performance levels of interest. The capacity to demand evaluation is typically performed via statistical-based approaches, possibly accounting for relevant uncertainty sources, considering effective engineering demand parameters for the quantitative assessment. Seismic qualification of NEs is an assessment and evaluation process aimed at satisfying specific NE performance levels, according to strict requirements and rules typically defined by regulations and codes. The seismic qualification process includes the assessment of the seismic behavior of the NEs, involving the dynamic identification of NEs, which is an essential task for robust seismic assessment of engineering systems (Amezquita-Sanchez et al., 2017; Perez-Ramirez et al., 2016). The seismic certification procedure typically includes seismic qualification processes carried out according to specific certification requirements and criteria, aimed at achieving the highest possible level of credibility, and operated through the intervention of subjects accredited by the institutions/authorities that regulate and issue the certifications. Seismic certification, meant as the outcome of a certification procedure, is a standardized certificate recognized by reference institutions. ASCE 7–16 establishes requirements and criteria for performing (special) seismic certification of NEs. In particular, mechanical and electrical equipment that has to be functioning under the design earthquake ground motion, NEs with hazardous substances, and NEs with an importance factor equal to 1.5 (ASCE 7–16, Section 13.1.3) shall be certified by the manufacturer considering operativity/functioning as a performance level. Other national regulations and industrial codes define requirements and criteria for seismic performance evaluation and certification of NEs (BSI, 2013; IEEE PES, 2013). Overall, seismic qualification, or seismic certification, when required, shall be carried out via one of the following methods: (a) analysis; (b) (experimental) testing; (c) experience data; and (d) a combination of methods (a), (b), and (c).

Experimental testing (i.e., method (b)) is the most common method to qualify/certify NEs, as this is typically considered to be the most reliable and robust. Experimental qualification is performed according to strict protocols and requirements. Quasi-static and dynamic testing procedures are typically associated with the qualification of displacement- and acceleration-sensitive NEs, respectively. However, dynamic testing is generally preferred (a) for NEs that show marked sensitivity to multiple demand parameters (e.g., both acceleration- and displacement-sensitive) and (b) in the case of NEs expected to exhibit a complex and irregular dynamic response that can (only) be identified and characterized through dynamic loading procedures, which are more representative of actual seismic demand. Dynamic testing also allows the assessment of the dynamic properties of the elements (e.g., dynamic identification). Shake table testing represents state-of-the-art dynamic testing (Bianchi et al., 2021; Filiatrault et al., 2004; Ghith et al., 2019; Jun et al., 2020; Kim & Shin, 2021; Lee & Jung, 2020; Luo et al., 2021; Magliulo et al., 2014; Qi et al., 2021; Tian et al., 2015; T. Zhou et al., 2021).

Numerical/analytical assessment of NEs is typically complex but represents a powerful method, often economic and fast, and easily implementable if the relevant models are effective and relatively reliable. However, these conditions are not often verified due to limited use and inadequate knowledge. According to IEEE 344 (IEEE PES, 2013), analysis is not recommended for equipment and systems that cannot be modeled in an adequate manner. Similar provisions regarding active mechanical and electrical equipment are supplied in ASCE 7–16 (ASCE, 2016). Qualification through experience data is even less typical than using analysis methods. There are extremely limited supporting data and information, especially considering the wide variability of the characteristics of NEs, buildings, and ground motions. Therefore, experimental testing is generally preferential and considered to be more reliable and applicable among the qualification methods.

1.2 | Inadequacy of existing shake table protocols

The technical definition of both loading and testing protocols is of paramount importance for the seismic qualification of NEs through shake table testing. However, both regulation provisions and literature criteria defining loading/testing protocols for NEs are inadequate, as it is motivated by the following.

As a first comment, it should be noted that seismic demands on NEs became a topical issue for research only in the very last years. Before, the existence of seismic demand formulations, despite these were reliable or not, was already an achievement since NEs were meant to be objects of "second rank" of interest. In particular, the existing shake table protocols used for seismic assessment and qualification of NEs were often developed considering insolated and self-referential approaches, often associated with peculiar applications (e.g., a specific type of equipment).

Both response and performance of NEs subjected to seismic events are significantly conditioned by loading history, which cannot generally be univocally correlated with given values of intensity measures (IMs) associated with the seismic events (e.g., peak accelerations; Cao & Ronagh, 2014). The seismic input properties significantly affect the response of NEs. For example, the time-varying frequency content may significantly affect the system response, and a nonstationary earthquake ground motion model could guarantee a relatively reliable seismic assessment, capturing the temporal nonstationarity of realistic earthquake scenarios (Li et al., 2016; Zhou & Adeli, 2003a).

The definition of the protocols, with particular regard to required response spectra (RRS), was often not based on the evaluation of the pre-existing methods/formulations, and in many cases, it did not even consider reference responses to calibrate these formulations. Conversely, safe, consistent, and updated formulations of the seismic demand need to be considered to develop reliable qualification protocols, with particular regard to RRS. The existing protocols and reference RRS except for AC156 (ICC-ES, 2012) are not clearly associated with real or analytical demand formulations, also taking into account potential building scenarios (e.g., FEMA, 2007; Telcordia, 2006). Perrone et al. (2019) modified ISO 13033 (BSI, 2013) formulation to be compliant with the Eurocode 8 (CEN, 2005) in terms of seismic demand on NEs, even though they considered AC156 provisions for the corner frequencies. Their study stresses the lack of adherence between the protocol and the compliant seismic demand formulations and highlights the need for further studies. AC156 RRS, or equivalently ASCE 7–16 seismic demand formulation, was found to potentially underestimate the seismic demands on building acceleration-sensitive NEs (Anajafi & Medina, 2018; D'Angela et al., 2021c; Perrone et al., 2019). Furthermore, standards and specific rules should be defined by the protocols to guarantee analysis, finalization, and extension of qualification outcomes in a robust and reliable manner. The existing protocols do not often provide univocal and consistent rules and criteria (e.g., BSI, 2013) or provide difficultly implementable and extremely complex procedures (e.g., FEMA, 2007), especially regarding the generation and processing of the protocol-compliant testing protocol and program. However, baseline generation and signal processing procedures define the features of the seismic inputs and appropriate generation/processing methods/techniques allow to characterize and enhance the key characteristics of the seismic inputs (e.g., Zhou & Adeli, 2003b). For example, the analytical procedures for enforcing the spectrum compatibility are rarely defined within the protocols, especially regarding the signal processing/adjustment methods and techniques (Amiri et al., 2012; Hancock et al., 2006).

Finally, the seismic qualification is typically referred to specific critical elements (e.g., telecommunication equipment; Telcordia, 2006), and, more importantly, the relevant rules are not general and strongly depend on the developer's discretion. In the opinion of the authors, this should not be acceptable for seismic qualification procedures. These should be as general and universal as possible. Qualification should be possibly applicable to any NE and the related protocols should be provided by minimizing any conflict of interest. Both loading inputs and testing protocols should be validated considering their representativeness and reliability concerning severe real ground and floor motions, as these later represent the most essential reference for comparison purposes. Accordingly, current approaches for performing seismic qualification need major revision and testing/loading protocols urge technical updating and significant enhancement.

1.3 | Aim, objectives, and organization of the paper

This study is motivated by both the inadequacy of the existing protocols and the critical need for reliable and consistent protocols. After a technical review of existing protocols, a novel assessment and qualification protocol are developed in this study. The loading input and testing procedure are defined to solve the criticalities associated with the current codes, considering (a) a novel assessment and qualification approach recently implemented in the literature, (b) a consistent code-compliant formulation of the seismic demand on acceleration-sensitive NEs, (c) a series of technical consolidated rules and criteria for developing and filtering the loading input, and (d) an evaluation and validation process.

Section 2 reports a solid and robust technical background/methodology description, from the elementary ingredients to the newly defined methodology framework. The literature contribution associated with this section is expressed in terms of original synthesis, assessment, evaluation, discussion, and validation of the state-of-the-art shake table qualification of NEs, with particular regard to the methodological and procedural levels. Section 3 defines a novel code-compliant shake table protocol by particularizing the methodology framework representing an outcome in terms of valuable product/outcome/item. Signal-based evaluation and validation of the developed protocol are reported in Section 4, where alternative protocols and real-floor motions are considered as a reference.

2 | METHODOLOGY

2.1 | Case-study nonstructural elements (NEs) and shake table protocols

The case-study NEs consist in a linear elastic single degree of freedom (SDOF) systems, which are assumed to not interact with the structure (and with the shake table).

The protocol is suitable for real-acceleration-sensitive elements, which might be associated with complex and nonlinear responses. The interaction between the shake table and the tested NEs is typically not accounted for or neglected since the basic hypothesis of no expected interaction is often reasonably assumed to be valid. However, depending on the mass and stiffness—and, more generally, dynamic properties of NEs—and on the element to shake table connections, a non-negligible interaction might be exhibited in real cases. With regard to experimental testing, the assessment of the interaction is investigated in terms of actual experimental response, whereas the shake table protocols do not account for the interaction. Therefore, the application can be considered to be reliable in this context.

Floor response spectra and shake table protocols generally assume NEs as linear elastic SDOF systems. Indeed, acceleration-sensitive NEs are generally meant to be SDOF systems in the literature (Akkar & Bommer, 2007; Merino et al., 2020; Petrone et al., 2015), and the assessment methodology, also including the seismic demand estimation, is based on SDOF hypotheses and spectral responses according to most authoritative national and international regulations and codes (ASCE 7–16; BSI, 2013; MIT, 2018). Representative examples of NEs compatible with SDOF systems include, but are not limited to, operating lights, projectors, antennas, base-anchored cabinets, and museum artifacts.

AC156 (ICC-ES, 2012) is intended to support data for seismic certification of systems that are sensitive to the accelerations, that is, architectural, mechanical, electrical, and other nonstructural systems attached to structures. FEMA 461 (FEMA, 2007) establishes a protocol for shake table testing of structural members and NEs that are sensitive to the dynamic effects of motion transferred to the component through a single point of attachment (acceleration-sensitive); the protocol also includes the methodology for the PBEE assessment via fragility estimation. ISO 13033 (BSI, 2013) defines the procedure to derive seismic actions and seismic performances of NEs. This code is not intended for mechanical and electrical equipment of industrial facilities, including nuclear power plants. However, the standard might be applied to these facilities.

AC156, FEMA 461, and ISO 13033 are intended for generic (acceleration-sensitive) NEs, whereas other protocols describe methods and criteria for seismic qualification of specific or peculiar NEs, such as mechanical and electrical equipment. GR-63-CORE (Telcordia, 2006) provides a protocol for shake table testing of telecommunications equipment, systems, or service facilities. IEEE 344 (IEEE PES, 2013) describes methods for seismic qualification of nuclear power plant equipment. The protocol can be used to perform the seismic/dynamic evaluation of NEs: from the tests to the analysis and up to the experienced-based evaluations. The code encloses common methods currently used in seismic qualification. IEEE 693 (IEEE PES, 2018) provides the minimum requirements for seismic qualification of electrical substation equipment. Regulatory Guide 1.60 (USNRC, 2014) (referred to below as RG-1.60) of the US Nuclear Regulatory Commission establishes design response spectra for the seismic design of nuclear power plants. The international standard IEC 60068-2-57 (IEC, 2013) outlines methods and standards for testing components, equipment, and electrotechnical products including the testing procedure for seismic applications. It extends the general requirements for seismic testing described in a separate standard, IEC 60068-3-3 (IEC, 2019).

2.2 | Loading input

2.2.1 | Outline

The definition of input signals according to assessment and qualification/certification protocols and compliant codes/guidelines is a complex process that often involves several phases and multiple key parameters and features. Typically, shake table signals are artificially obtained (FEMA, 2007; ICC-ES, 2012), but they can also be defined following empirical approaches (IEEE PES, 2018; IEC, 2013). For example, two groups of strong ground motion records and one group of artificial waves were used for the input excitations by Lu et al. (2018) and Luo et al. (2021). In Takhirov et al. (2017), a set of an earthquake and synthetic strong-motion records were generated according to IEEE 693 for the seismic qualification of NEs.

The definition process of artificially obtained shake table inputs is generally based on the definition of the following features: (a) IM, (b) baseline signal, (c) RRS, (d) compliance/compatibility criteria and rules, and (e) instrument characteristics and capacities. It should be mentioned that no studies or literature documents define or describe these features, which were systematically defined and discussed in this study, according to personal experience regarding shake table testing and literature/code references.

2.2.2 | Intensity measure (IM)

The most common IM typically consists of peak table acceleration, and this is representative of peak ground acceleration (PGA) or peak floor acceleration (PFA). The reference IM of the input test signal defined according to AC156 is the design earthquake spectral response acceleration parameter at short periods, that is, S_{DS} (ASCE, 2016). FEMA 461 recommends the use of spectral acceleration at the appropriate natural frequency of the NE as an IM, that is, $(S_a(T_a))$. IEEE 693 employs the site-specific hazard method, considering PGA. GR-63-CORE supplies four earthquake risk zones as a parameter to be used for defining input test signal intensity level.

2.2.3 | Baseline signal

The key features of the baseline signal are discussed in the following reference protocols and literature. (1) The baseline time interval envelope typically includes (a) acceleration ramp-up or *rise part* (RP), (b) hold time or *strong motion part* (SMP), and (c) de-acceleration ring downtime or *decay part* (DP). AC156 and IEEE 693 require that the total duration of the input motion shall have at least 20 s of SMP, whereas signal duration should be equal to 60 and 32 s according to FEMA 461 and GR-63-CORE, respectively. IEC 60068 requires the duration of SMP to be a given percentage of the total duration, whereas the typical total duration is 30 s with a minimum SMP duration equal to 20 s. (2) Fixed sampling rate/frequency is typically assumed for the baseline; AC156 and IEEE 693 do not define that, whereas FEMA 461 and GR-63-CORE recommend a sampling rate equal to 100 Hz and equal to or larger than 200 Hz, respectively. (3) The energy content (Luise & Vitetta, 2009) of the theoretical input signal is typically associated with given frequency ranges and resolutions. Energy content should be ranging from 1.3 to 33.3 Hz with one-third-octave and one-sixth-octave bandwidth resolution corresponding to analog and digital synthesis equipment, respectively, for AC156, whereas FEMA 461 provides signals with frequency contents ranging from 0.5 to 32 Hz and one-third-octave bandwidth resolution. IEEE 693 requires that the input motion includes the lower corner point frequency of the RRS equal to 1.1 Hz. GR-63-CORE inputs should have frequency contents ranging from 0.5 to 50 Hz. The time history obtained according to IEC 60068 shall be generated by the composition of frequencies included within a specified range (typically frequency range from 1 to 35 Hz; Takhirov et al., 2004) and through an appropriate resolution as a function of the specimen damping: the larger the specimen damping (ν), the lower the frequency resolution.

2.2.4 | Required response spectra (RRS)

RRS or target spectra can be obtained through empirical, analytical, and standard code approaches. Empirical methods consist of assessing reference RRS according to (ground or floor) time histories recorded during the seismic events (Takhirov et al., 2004, 2017). The analytical approach typically involves numerical analysis of structures and/or NEs for the estimations of RRS (Blasi et al., 2021; Singh & Gupta, 2021). Finally, the approach based on standard code provisions uses closed-form RRS formulations, depending on relevant parameters of buildings and/or NEs and often also referred to for NE design purposes; this latter approach is the one typically recommended by shake table protocols (BSI, 2013; ICC-ES, 2012). The procedure used to define standard code RRS is not typically described/provided by the relevant codes and guidelines. It should be noted that code-based floor demand estimations might not be reliable according to recent studies, where numerical estimations (Chichino et al., 2021; D'Angela et al., 2021a; Di Domenico et al., 2021; Salari et al., 2022) or real records (Anajafi & Medina, 2018; D'Angela et al., 2021b; Pürgstaller et al., 2020) were considered as a reference, stressing the need for technical revision of code prescriptions.

AC156 defines 5%-damped RRS according to the formulation of the horizontal seismic design force and the definition of flexible and rigid NEs provided by ASCE 7–16. RRS were defined considering two regions: amplified region and zero period acceleration (ZPA) region, separated by 8.3 Hz; for the computation of the spectral demand acceleration, 16.7 Hz is considered as a threshold for defining flexible and rigid NEs; however, this discrepancy only seems to be formal and does not affect the RRS. FEMA 461 does not define RRS but provides some rules regarding the response spectra of the input signals. The response spectra should be defined considering 5% damping. The signal shall be scaled to have (a) acceleration response spectra amplitude equal to 1 g within 2–32 Hz and (b) the displacement response spectra would be approximately uniform below 2 Hz. IEEE 693 supplies two RRS, associated with high- and moderate-performance levels; these spectra are provided as a function of the NE damping, considering a range of values ranging from 2% to 20%. GR-63-CORE provides RRS associated with four earthquake zones (Zones 1 to 4 in the United States), considering a damping value equal to 2%. Unlike other cases, GR-63-CORE RRS spectra have shapes varying for different areas. ISO 13033 follows an approach similar to the one recommended in AC156 for the definition of RRS; however, the former can also be extended to other regulations and building codes. For example, in Perrone et al. (2019), the approach proposed by ISO 13033 was modified to provide RRS compatible with the design horizontal equivalent static force evaluated according to Eurocode 8 (CEN, 2005). RG-1.60 provides RRS associated with five damping ratios, that is, 0.5%, 2.0%, 5.0%, 7.0%, and 10% (for different damping ratios, a linear interpolation should be used). These RRS correspond to a

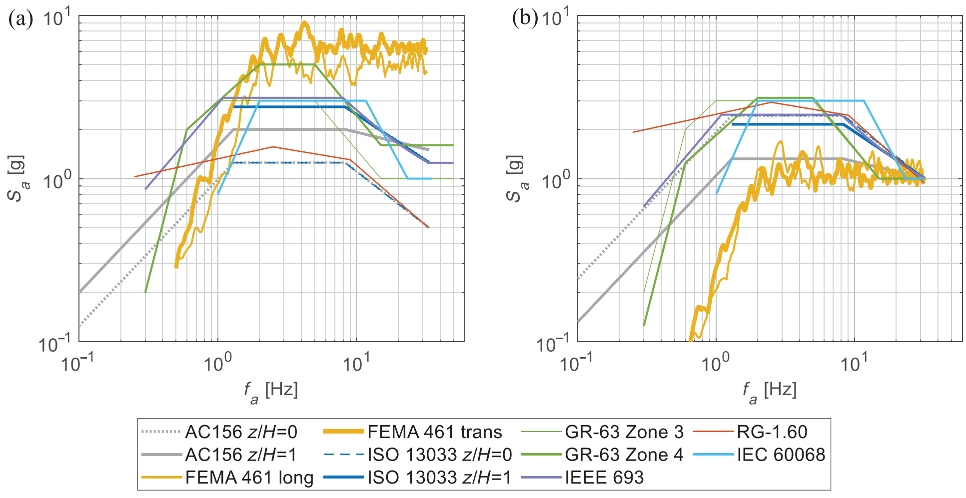

FIGURE 1 Comparison among required response spectra (RRS) and input response spectra related to reference protocols considering (a) peak ground acceleration (PGA) equal to 0.50 g and (b) spectral ordinate S_a corresponding to 32 Hz equal to 1.0 g

PGA equal to 1.0 g and peak ground displacement equal to 0.91 m. For different design earthquakes, the RRS can be linearly scaled in proportion to the specified PGA (e.g., PGA = 0.5 g). IEC 60068 defines three RRS, associated with 2%, 5%, and 10% damping ratios. These RRS exhibit a generalized form that is based on simple correlations among the corner frequencies, and the specific corner frequencies depend on assumptions regarding the frequency range of sensitivity of the NE.

Figure 1a shows a comparison among the response spectra related to the protocols of interest, where FEMA 461 long and trans stand for FEMA 461 signals along horizontal longitudinal and transversal directions. All spectra but GR-63-CORE RRS (a) are related to 5% damping and (b) were computed considering PGA equal to 0.50 g and assuming a building acceleration amplification factor according to the specific protocol (a value equal to 2.5 was assumed for FEMA 461, which does not provide this ratio). Both *z/H* equal to zero and one conditions are depicted for AC156 and ISO 13033 RRS. IEEE 693 RRS are associated with a moderate-performance level, and FEMA 461 spectra were obtained by scaling the protocol input signals. In particular, these seismic inputs were provided by the protocol considering specific levels of acceleration (e.g., PFA equal to about 0.2–0.25), and in this study, these signals were scaled to obtain PGA equal to 0.50 g (Figure 1a) and spectral ordinate S_a corresponding to 32 Hz equal to 1.0 g (Figure 1b). GR-63-CORE RRS are associated with 2% damping, as the protocol only considers this damping condition. For this latter protocol, RRS spectra associated with earthquake risk Zones 3 and 4 are depicted, which correspond to expected PGA in the range of 0.2 – 0.4 g and 0.4 – 0.8 g, respectively.

FEMA 461 spectra exhibit higher spectral ordinates than all other RRS for frequencies higher than about 2 Hz, whereas for lower frequencies, they cross all other RRS and become the lowest spectra over the lowest frequencies (e.g., lower than 0.7–1.0 Hz). AC156 and ISO 13033 are identical for *z/H* equal to zero condition, whereas ISO 13033 presents significantly higher ordinates for *z/H* equal to one condition. In particular, *z/H* equal to zero spectra present the lowest ordinates for frequencies larger than about 1 Hz. GR-63-CORE and IEEE 693 RRS are the most amplified spectra over most frequencies, but it is recalled that the former is associated with 2% damping, whereas the latter spectra are associated with 5%. The spectral shape is quite similar among the different RRS, whereas FEMA 461 inputs present a different trend of the ordinates over the frequencies. Sections of the response spectra can be identified for FEMA 461 signals: (a) increasing branch up to about 2 Hz and (b) plateau for larger frequencies. The increasing branch crosses all other spectra, whereas the plateau has ordinates significantly larger than the other RRS ones.

Figure 1b depicts a comparison among the protocol spectra considering a different comparison criterion. In particular, the spectra are calculated under the assumptions described in Figure 1a, but they are scaled to have spectral ordinate equal to 1.0 g corresponding to 32 Hz, which represents the highest frequency (lowest period) common to all protocols. The spectral response associated with this frequency is related to the response of an approximately rigid NE. Accordingly, Figure 1b highlights the spectral amplification associated with the flexible to the rigid response of NEs. All spectra but FEMA 461 and AC156 *z/H* = 1 ones are quite similar among them, whereas these latter provide significantly lower ordinates. GR-63-CORE spectra (2% damping) and IEC 60068 present the highest plateau ordinate, while IEEE 693, AC156 $z/H = 0$, and ISO 13033 spectra (5% damping) present a similar, lower, ordinate, still significantly larger than AC156 $z/H = 1$ and FEMA 461 ones. Further comments are omitted for the sake of brevity.

2.2.5 | Spectrum compatibility

Test response spectra (TRS) should be compatible with RRS to satisfy specific target levels, considering both theoretical signals and recorded signals. IEEE 693 supplies different compatibility rules for theoretical and recorded inputs. For all other protocols, no distinction is made between theoretical and recorded input compatibility rules. In general, the spectrum-compatibility rules include: (a) the spectral resolution definition of TRS, (b) the frequency range against which to perform the spectrum-compatibility check, (c) spectrum ordinate amplitude tolerance range, expressed in terms of RRS, that quantifies the compatibility spectrum ordinate check for TRS (e.g., inferior or superior ordinate tolerance). The spectral resolution represents the interval between two frequency data points of spectral analysis. AC156 protocol states that TRS must envelop the RRS at 5% damping based on a maximum-one-sixth octave bandwidth resolution over the frequency range from 1.3 to 33.3 Hz. TRS should not exceed the RRS by more than 30% over the amplified region of the RRS (i.e., frequencies lower than or equal to 8.3 Hz). The protocol provides exemption rules applicable to both the amplified region and ZPA region (i.e., frequencies larger than or equal to 8.3 Hz).

Theoretical TRS to RRS compatibility should be checked at 24 divisions per octave resolution or higher, and TRS ordinate should be within ±10% of the RRS at 5% damping; TRS shall include the lower corner point frequency of the RRS (1.1 Hz) for comparison with the RRS. IEEE 693 defines compatibility criteria as less restrictive for recorded inputs. In particular, the shake table output TRS shall envelop the RRS within a –10%/+50% tolerance band at 12 divisions per octave resolution or higher. Exemptions are provided regarding both upper and lower limitations. According to GR-63-CORE, TRS must meet or exceed RRS for the applicable earthquake risk zone in the range from 1.0 to 50 Hz. In particular, TRS evaluated considering 2% damping should not exceed RRS by more than 30% in the frequency range of 1 to 7 Hz. A test may be invalid if an equipment failure occurs when the TRS exceeds the RRS by more than 30% in this frequency range. ISO 13033 and RG-1.60 do not provide criteria regarding the spectrum compatibility. IEC 60068 establishes that the TRS shall be checked in the specified range at least in one-sixth octave bandwidth resolution in the general case, that is, specimen damping lying between 2% and 10%. The tolerance to be applied to the RRS shall be in a range between 0% and 50%. Moreover, after the plateau zone of the RRS, a tolerance of more than 50% is permitted.

While protocols often provide spectrum-compatibility verification rules, the procedure to achieve or enforce this condition is not typically addressed. In order to achieve the best possible spectrum compatibility, Crewe (2012) recommends that (a) the iterative matching process for each time history should be continued beyond initial convergence to capture later iterations that may be a much closer match to the RRS, (b) the spectra-matching procedure should be always conducted at a minimum of one-24th-octave points, (c) high-pass filtering of input motions should not be used to limit the demand placed on the shake table by TRS, and (d) matching over a reduced frequency range is more effective and results in a TRS that matches the RRS more closely.

The response spectrum compatibility is often performed in the literature using software products and tools based on analytical methods and formulations. Zaghi et al. (2012) and Tran et al. (2021) generated an artificial earthquake using the SIMQKE software (Gasparini & Vanmarcke, 1976). In Magliulo et al. (2012) and Di Sarno et al. (2019), the signal was enhanced using the spectrum-matching procedure of the RSPMatch software (Hancock et al., 2006). Yazdani and Takada (2009) developed a method for modifying many realistic earthquake ground motions through linear/nonlinear response spectra and energy matching. Amiri et al. (2014) introduced a method to generate a suite of artificial near-fault ground motion time histories for specified earthquakes based on the superposition of a coherent extracted velocity pulse with a random acceleration record corresponding to a wavelet-based nonstationary model and multiplied by a time-modulating envelope function. Several other authors conducted shake table tests and implemented artificial acceleration time histories using the STEX program of MTS (Jeon et al., 2021; Takhirov et al., 2004).

2.2.6 | Signal processing and instrumentation compatibility

Capacities and limitations of shake table and testing instrumentation must be met by the spectrum-compatible signals, and this should be checked prior to performing the tests. Among the possible parameters to be checked, maximum accelerations, velocities, and displacements expected to be achieved by the table should be assessed and compared to the shake table and instrumentation capacities.

For example, low-frequency content in the input signal typically imposes large displacement demands on the table, which can often exceed the shake table displacement capacity; this limitation is typically critical to verify (Takhirov et al., 2017). If the capacity compatibility is not achieved, the input signal might be adjusted. In particular, the acceleration time history of the theoretical input motion could be filtered to meet the capacities of the shake table.

AC156 recommends that the general requirement for enveloping RRS by the TRS can be modified under certain conditions. When no resonance response phenomena exist below 5 Hz, TRS is required to envelop the RRS down to 3.5 Hz (instead of 1.3 Hz), whereas TRS is required to envelop the RRS only down to 75% of the lowest frequency of resonance (instead of 1.3 Hz) if resonance below 5 Hz exists. According to IEEE 693, the theoretical input may be high-pass-filtered at frequencies lower than or equal to 70% of the lowest fundamental frequency of the specimen but not higher than 2 Hz. The lowest fundamental frequency of the specimen should be assessed through experimental tests (i.e., dynamic identification tests as described in the following sections). GR-63-CORE requires that the cutoff of the high-pass filter does not exceed 0.20 Hz, while the cutoff of the low-pass filter should not be below 50 Hz. FEMA 461 and ISO 13033 do not establish a procedure to process the signals to obtain compatibility with the shake table limitations. However, FEMA 461 provides a procedure for filtering input motions to remove energy contents close to the excitation frequency that has already caused a DS to occur or, more generally, that is not of interest (i.e., *notch filtering*).

The filtering procedure used to reduce shake table displacement demands often consists in applying a high-pass filter in the frequency domain. However, it might be necessary to also reduce the high-frequency contents of the input signals according to other capacities and limitations of the shake table (e.g., 50 Hz). Therefore, a band-pass filter is often applied to solve both problems of maximum displacements and high-frequency contents. In several studies (Jun et al., 2020; Magliulo et al., 2012; Petrone et al., 2014, 2017), the acceleration time histories were filtered through low-pass and band-pass filters to meet the instrument and facility capacities, for example, to reduce the maximum shake table displacements. Takhirov et al. (2017) proposed several filtered options suitable for most of the shake tables worldwide for the seismic qualification of NEs according to IEEE 693.

2.2.7 | Analytical and experimental validation

The decisive step of the loading input definition should be the signal verification and the validation of the experimental procedure, especially if further filtering procedures were implemented. Generally, adherence to both theoretical and recorded signals is considered to be sufficient for verifying and validating the experimental qualification procedure, and this is based on spectrum-compatibility criteria. However, the protocol compliance of (recorded) signals might be critical since shake table and testing instrumentation might not generally reproduce a compliant signal given to instrumental and diverse reasons, and this stresses the need for strict verification and validation rules and criteria (e.g., Maddaloni et al., 2011).

2.3 | Testing procedure

2.3.1 | Outline

The testing program typically involves pretest and testing phases. For the pretest phase, FEMA 461, AC156, IEEE 693, and GR-63-CORE require pretest inspection and functional verification to be documented. The testing phase generally consists of dynamic identification tests and seismic performance evaluation tests. FEMA 461 includes an additional testing type, named failure tests. Failure tests are carried out to induce DSs that could pose life safety risks and DSs corresponding to the incipient failure of the test specimen. Failure tests are typically performed as part of the performance evaluation tests.

The current testing protocols implicitly recommend that the dynamic identification tests should be performed prior to seismic performance evaluation tests. However, FEMA 461, which recommends an incremental performance evaluation test procedure, establishes that dynamic identification tests should be conducted prior to and after each performance evaluation test. In literature, several studies followed this approach. For example, in Cosenza et al. (2015) and Petrone et al. (2017), the dynamic identification was carried out over the incremental tests, and both dynamic properties and exhibited damage were correlated to the testing intensities.

2.3.2 | Dynamic identification tests

To perform an exhaustive assessment, vibration modes, fundamental periods/frequencies, and damping ratios evolution should be associated with the damage process evolution, considering undamaged, partially, and fully damaged conditions (or at least all the DSs reached during the tests). Another parameter that also accounts for the dynamic properties of NEs is the dynamic component amplification factor, often defined a_p by regulations and codes (ASCE, 2016; CEN, 2005; INN, 2015; MIT, 2018;

NRCC, 2015; NZS, 2009; UTCB, 2013). In most regulations and codes, a_p is a key parameter for computing seismic demand forces on NEs (ASCE, 2016; CEN, 2005; INN, 2015; MIT, 2018; NRCC, 2015; NZS, 2009; UTCB, 2013). This parameter is typically expressed by conservative formulations or values and is rarely estimated with regard to specific NEs through experimental or numerical procedures. Reference shake table protocols do not require estimations of this parameter even though it is essential for reliable estimations of RRS or seismic demands.

Disregarding a_p, the input of the dynamic identification tests is generally expressed by a low-intensity acceleration time history signal, which is defined by the reference protocol in some cases. FEMA 461 establishes that single-axis identification tests should be carried out along each principal direction of the test specimen, considering four alternative types of tests: white noise tests, single-axis acceleration-controlled sinusoidal sweep tests, resonance tests, and static pull-back tests. AC156 and GR-63-CORE recommend single-axis acceleration-controlled sinusoidal sweep tests, IEEE 693 indicates sine sweep or random noise excitation test, and ISO 13033 includes the dynamic tests but does not describe the testing procedure and input.

Several testing methods were used in the literature to identify the dynamic characteristics of NEs. Random noise excitation tests (Cosenza et al., 2015; Di Sarno et al., 2019; Fiorino et al., 2019; Jun et al., 2020; Lu et al., 2018; Luo et al., 2021; Petrone et al., 2017; Qi et al., 2021) and sine sweep tests (Kim & Shin, 2021; Lee & Jung, 2020; Lin et al., 2014; Son et al., 2020) are among the most used ones, and no studies, to the knowledge of the authors, identified the differences among the different methods in terms of dynamic properties assessment results or supplied motivations for preferring the use of one specific method. Even though the methods defined within the relevant protocols or in the literature can be considered to be relatively reliable, the absence of a preferred method and the lack of standardized definitions might condition the robustness of the estimations, especially considering comparison purposes. This evidence stresses the need for defining a unique reliable and robust dynamic identification test method that (a) complies with specific technical procedures for defining the input signal, (b) minimizes the analyst bias, (c) is widely applicable, (d) strengthens the accuracy of the estimations, and (e) fosters consistent comparisons and result in extrapolations.

2.3.3 | Seismic performance evaluation tests

According to most protocols, seismic performance evaluation tests are defined by the tests performed considering the seismic intensity associated with the target performance level(s) that the specimen should meet. ISO 13033, AC156, IEEE 693, and GR-63-CORE do not specify a minimum number of performance evaluation tests to perform or do not provide recommendations for defining a testing program. Conversely, FEMA 461 supplies criteria to define the testing program according to an incremental approach. In particular, this protocol requires at least three different shaking intensities and indicates that the intensities of the performance evaluation tests should be defined to induce relevant DS occurrence, that is, functioning interruption and repair/replacement intervention, for seismic performance evaluation tests, and severe damage, incipient failure, and life-threatening risk, for failure tests. FEMA 461 also defines the minimum intensity step increment between consecutive intensity-level tests, which is equal to 25%.

Seismic performance evaluation tests are generally carried out by applying the input motions simultaneously along the principal axes of the specimens. In particular, AC156, IEEE 693, GR-63-CORE, and FEMA 461 establish that the performance evaluation tests (and failure tests for FEMA 461) should be performed through triaxial tests with input motions applied simultaneously along all principal axes of the test specimen; alternatively, multiple biaxial tests can be used (along horizontal and vertical directions) according to an exhaustive approach. FEMA 461 establishes that horizontal tests (biaxial or uniaxial) could be performed if the effect of vertical motion on the seismic response of the test specimen is negligible; the other protocols do not address this condition, which can be quite common for typical NEs (e.g., partitions or infill panels). In particular, FEMA 461 recommends that this condition may be acceptable if the vertical fundamental frequency of vibration of the test specimen is at least 10 times larger than horizontal fundamental frequencies or if the vertical natural frequency of the test specimen falls outside the frequency range of the input motions. AC156 allows uniaxial tests, which should be performed along each of the three principal directions of the specimens.

In several literature studies, shake table tests were performed through incremental procedures despite only FEMA 461, among several protocols, recommending performing multiple (incremental) tests for assessing the seismic performance and qualifying NEs. It is worth recalling that this latter protocol is not intended for seismic certification, and therefore this approach (i.e., incremental tests) is not required to be applied for seismic certification purposes. In some literature studies, shake table inputs compliant with protocols other than FEMA 461 (e.g., AC156) were scaled according to relatively dense incremental procedures (Di Sarno et al., 2019; Fiorino et al., 2019; Magliulo et al., 2014; Petrone et al., 2017), that is, FEMA 461 approach was applied considering seismic inputs other than this latter approach. This stresses the lack and inconsistency of the current seismic qualification approaches and protocols. In particular, they do not seem to give significance to incremental procedures, which are certainly associated with more reliable and robust assessment and evaluation, also compliant with the PBEE approach (Porter, 2021).

2.3.4 | Representativeness of qualification and certification

The test specimens should effectively represent the class or type of components intended to be qualified/certified by the manufacturer to achieve the target representativeness of the qualification/certification; this representativeness strongly depends on the objective of the assessment or qualification. A relatively adequate number of shake table tests should be performed considering a minimum number of test specimens. AC156 provides the criteria for defining test specimen configuration requirements for an element product line. The selection can be achieved based on the least seismic capacity offered by the structural configurations, mounting configurations, mass distribution, and specimen components and subassemblies. Other current protocols do not provide requirements or information on the representativeness of the qualification/certification procedure. GR-63-CORE only provides recommendations regarding installation conditions for equipment and systems. Only FEMA 461 recommends a minimum number of specimens to test, which is equal to three.

2.4 | Critical evaluation of existing protocols

Table 1 summarizes the key parameters and features that are essential for the definition and implementation of qualification procedures defined by the reference protocols. Most of the parameters reported in Table 1 were described and discussed in previous sections; therefore, redundant comments are omitted for the sake of brevity, and the focus of this section is on the most significant comparisons and critical evaluation. AC156, FEMA 461, and ISO 13033 are intended for any type of element (i.e., generic NEs), whereas IEEE 693, GR-63-CORE, and RG-1.60 are defined for specific equipment or systems. However, it is not clear how the type of target specimens conditioned the definition of the protocol characteristics, especially for the specific equipment protocols. Even regarding the sensitivity of the target specimens, there are no clear requirements for several protocols (e.g., specific equipment protocols). However, the authors believe that there is a common skepticism about using shake table testing (protocols) to assess and qualify NEs that are (also) sensitive to displacements/drifts, as it can also be identified in AC156 criteria. Indeed, the fact that shake table testing is the best option to assess dynamic effects/response should not limit the use of the reference protocols to assess NEs that are also sensitive to drifts/deformations or that, more generally, have a complex response that can only be reasonably assessed through dynamic tests. This critical issue will be addressed further in the following section, where the proposed approach and the novel protocol are described. Regarding the boundary conditions of the target specimens, most protocols are intended to provide criteria for anchored or attached elements. However, due to the absence of reliable protocols intended for unanchored or freestanding equipment, the reference protocols were often used in the literature also to assess these peculiar NEs (e.g., Di Sarno et al., 2019; Ghith et al., 2019; Huang et al., 2021; Prota et al., 2022; Wittich & Hutchinson, 2014)).

Sinusoidal sweep and white noise tests are the most referenced by the protocols, and these methods are the most used in the literature as previously discussed. However, the protocols do not provide clear information and technical guidance on how to develop the dynamic identification test inputs, or rather they do not follow general and widely applicable approaches.

Regarding RRS, the reference protocols provide quite dispersed and varied requirements and information, which are associated with significant differences among the protocols. These differences might be significant in terms of RRS details/specification/information, site dependency, scalability of RRS, frequency range and corner frequencies, and component and floor amplification factors.

This parameter variability might significantly condition the reliability of the protocols as tools to qualify the NEs. For example, the plateau to ZPA amplification provided by AC156 RRS (related to z/H) is significantly lower than the one related to other protocols. This was already proven to underestimate seismic demands associated with strong floor motions (D'Angela et al., 2021b). This is likely due to an upper limitation criterion defined by the protocol, which was already criticized by literature studies (D'Angela et al., 2021a; Perrone et al., 2019; Petrone, et al., 2016). Another critical definition of RRS is related to IEEE 693 criteria, which seem to not intrinsically account for the building acceleration amplification, which should be included by RRS amplifications by the signal analysts.

RRS definition should be based on consolidated and consistent formulations of seismic ground and building demands on NEs, which should be reported by the protocols and proven to be reliable and robust and also relatively simple to implement.

TABLE 1 Comparison among reference protocols considering most significant features and parameters

		AC156	FEMA 461	ISO 13033	IEEE 693	GR-63-CORE	RG-1.60	IEC-60068
Target specimens	Type	Any	Any	Any	Electrical substation equipment	Telecommunications network equipment	Nuclear power plants equipment	Components, equipment, and electrotechnical products
	Behavior/EDP sensitivity	Not specified[a]	Dynamic effects, velocity, strain-rate effects	Acceleration/ displacement	Not specified	Not specified	Not specified	Not specified
	Boundary conditions	Anchored[b]	Anchored	Anchored	Anchored	Anchored	Not specified	Not specified
Dynamic identification test	Sinusoidal sweep	X	X	–	X	X	–	
	White noise		X		X			
	Resonance		X					X
	Static pull-back		X					
Required response spectra (RRS)	Damping ratio ν (%)	5	Not applicable	5	$2 \div 20$	2	0.5,2,5,7,10	2,5,10
	Site-dependency	X		X				
	Maximum building acceleration amplification factor	1.6	Not applicable	Not applicable	2.5	Included	Not applicable	2.0
	Frequency range (Hz)	0.1–33.3	Not applicable	1.3–33.3	0.3–50	0.3–50	0.25–33.0	1–35
	Plateau range (Hz)	1.3–8.3	Not applicable	$1.3 \div 2.5 - 7.5 \div 8.3$	1.1–8.0	2.0–5.0 (Zone 4)	2.5–9.0	2–11.7
	Spectral acceleration at plateau, in fractions of zero period acceleration (ZPA)	$f(z/H)$: $2.5(z/H = 0)$	Not applicable	2.5	$f(\nu)$: $3.24(\nu = 2\%)$	f(Zone): 3.13 (Zone 4)	$f(\nu)$: 2.9[c] (at 5%)	$f(\nu)$: 3.0 (at 5%)
Seismic performance evaluation test input	Total (strong motion) duration (s)	30 ± 6 (20 +6/−0)	60	Not applicable	(≥20)	32	Not applicable	30
	Strong part to duration ratio (%)	Not applicable	Not applicable	Not applicable	≥30	Not specified	Not applicable	25,50,75
	Sampling rate (Hz)	Not applicable	100	Not applicable	Not specified	>200	Not applicable	Not specified

(Continues)

TABLE 1 (Continued)

		AC156	FEMA 461	ISO 13033	IEEE 693	GR-63-CORE	RG-1.60	IEC-60068
	Energy content (Hz)	1.3–33.3	0.5–32	Not applicable	1.1–33	Not specified	Not applicable	Not specified
	Resolution bandwidth (octave)	1/3 or 1/6	1/3	Not applicable	Not specified	1/6	Not applicable	Not specified
Spectrum compatibility	Tolerance above RRS (%)	30	Not applicable	Not applicable	50[d]	30	Not applicable	50
	Tolerance below RRS (%)	10[e]	Not applicable	Not applicable	10[f]	0	Not applicable	0
	Tolerance range applicability (Hz)	1.3–8.3	Not applicable	Not applicable	< 15	1.0–7.0	Not applicable	Not specified
	Tolerance resolution (octave)	1/6	Not applicable	Not applicable	1/12[g]	1/6	Not applicable	$f(\nu)$: 1/6(ν = 2%–10%)
	Compatibility range applicability (Hz)	75% f_a[h] (or 3.5) – 33	Not applicable	Not applicable	70% f_a[h] (or 2) – 33	1.0–50.0	Not specified	Not specified

[a]The protocol is not intended to evaluate effects of relative displacements on NEs.
[b]The protocol is intended for anchored elements but this is not clearly and univocally stated within the document.
[c]The plateau spectral ordinate varies from 3.13–2.61 (average).
[d]The tolerance above RRS for the theoretical response spectrum is equal to 10%.
[e]In both the ZPA region and the amplified region of RRS, a maximum of two individual points up to 10% below the RRS can be acceptable provided the adjacent one-sixth octave points are at least equal to the RRS.
[f]A –10% deviation is allowed, provided that the width of the deviation on the frequency scale, measured at the RRS, is not more than 12% of the center frequency of the deviation and not more than five deviations occur at the stated resolution.
[g]The spectrum matching procedure should be conducted at one-24th octave resolution or higher for the theoretical response spectrum.
[h]f_a is the natural frequency of NE.

Significant variability is also associated with the protocol definition of the test input to perform seismic performance evaluation tests, and similarly to the case of RRS, this can significantly affect the reliability of the assessment, qualification, or certification procedure. The present paper has already stressed the significance of the seismic loading history characteristics on the reliability of the seismic evaluation. The spectrum-compatibility criteria are more comparable among the different protocols and follow more common approaches. However, some non-negligible differences can also be identified as can be seen in Table 1.

As a conclusive comment, the authors believe that the seismic performance of NEs should be assessed through incremental procedures of excitation by scaling the input signals to be representative of the seismic scenario actions that would potentially excite the specimen at the structure-to-element interfaces. This is compliant with the PBEE and is recommended in FEMA 461. In particular, it is desirable that the results of the qualification tests are documented for each significant intensity level and regarding relevant DSs and used for fragility or vulnerability assessment. These aspects are not addressed by reference protocols except FEMA 461. Therefore, novel approaches and protocols should be defined to favor reliable and robust assessment, qualification, and certification procedures according to incremental procedures and evaluations based on the PBEE.

3 | DEFINITION OF A NOVEL CODE-COMPLIANT TESTING PROTOCOL

3.1 | Outline

A novel code-compliant shake table testing procedure, namely, *testing protocol*, is defined in this section. The protocol defines the procedure and requirements for seismic assessment of acceleration-sensitive NEs by shake table testing, with particular reference to seismic qualification and certification processes. The protocol is applicable for NEs having fundamental frequencies greater than or equal to 1.0 Hz. In the following, the seismic input used for the seismic performance evaluation test is also referred to as the *loading protocol*, as this represents the most significant feature of the testing protocol.

3.2 | Seismic qualification approaches

Two different approaches can be considered for qualifying NEs: (a) *specific performance-level qualification* and (b) *extensive qualification*. The former approach is inspired by the procedures typically defined by existing shake table protocols (BSI, 2013; ICC-ES, 2012; IEEE PES, 2018; Telcordia, 2006). Accordingly, the qualification is aimed at checking whether the component fulfills (or not) a specific performance level, associated with a target level of a relevant (seismic) intensity parameter (i.e., pass or fail test). The seismic intensity parameter and the relevant target levels are typically defined by regulations or codes. Acceleration spectral response S_a is typically considered to be a seismic intensity parameter for the assessment of acceleration-sensitive elements and is referred to within several national and international codes (ASCE, 2016; CEN, 2005; INN, 2015; MIT, 2018; NRCC, 2015; NZS, 2009; UTCB, 2013). However, PGA or PFA might also be considered, if appropriate, as they might be equally reliable as seismic intensity demands. Target seismic intensity parameters should not be confused with testing IMs, even though testing IMs might also be considered to be seismic intensity parameters, when possible and appropriate. It is fundamental that the target intensity parameter measures are established from the basic parameters (e.g., hazard or soil conditions) following consistent and robust approaches and formulations/specifications, which should also be compatible with the qualification protocol. In particular, the target levels can be defined according to specific site-building-component scenarios or can be more general and referred to entire regions and wide representative scenarios. For example, if the building site and NE installation height (over building height) are known, NE could be qualified considering the specific scenario associated with this location, according to reference seismic demand formulations. Regional or national maximum demand scenarios are typically considered for the identification of the qualification intensity level(s), especially if the qualification is carried out by manufacturers. Further comments on target seismic intensity parameters are omitted as their definition should be addressed, through conventional decisions, by regulations, codes, and technical guidelines.

The extensive qualification reflects the technical–scientific requirement for a more exhaustive assessment and evaluation of the seismic performance of NEs, not only meeting a conventional requirement target but developing novel technical and applicative knowledge. This approach was developed in light of the recent literature in the field, where incremental complex shake table procedures were implemented (Cosenza et al., 2015; Di Sarno et al., 2019; Fiorino et al., 2019; Ghith et al., 2019; Kim & Shin, 2021; Tian et al., 2015; Zaghi et al., 2012). In particular, the extensive qualification is aimed at characterizing the behavior and the damage response of NEs considering multiple incremental intensity levels, identifying the NE capacity thresholds corresponding to the significant performance levels. Target levels of seismic intensity parameters should be defined for the relevant performance levels as it is described for specific performance-level qualification.

The extensive qualification is a complete and exhaustive identification and characterization of the NE in terms of seismic behavior, capacity, and performance, providing robust and reliable capacity thresholds to evaluate the fulfillment of the target performance levels, whereas the specific performance-level qualification is only associated with the fulfillment of a conventional performance-level requirement. The use of one approach over another should be regulated by national and international regulations and qualification or certification rules/standards. In particular, the importance and representativeness of the NE should be among the most significant key parameters for providing these criteria.

The manufacturer could prefer to perform an extensive qualification even if a specific performance level is required by the relevant regulatory requirements, as this would shed light on the complete performance of the NE, providing significant additional technical information to the specific performance-level check. For example, the specific performance-level qualification does not allow determining the safety conditions regarding the performance-level demand, as the result of this qualification process is checking that the performance level is satisfied, without quantifying the capacity and safety margins. This specific margin quantification could be considered to be of primal importance; for example, in the case of nuclear power facilities or hospitals, where safety is expected to matter more than the economic aspects. Moreover, identifying the capacity margins associated with the relevant performance levels would be essential for an efficient design of the NE. Finally, an incremental qualification procedure would also allow the assessment of the seismic fragility and vulnerability associated with NE, essential features for the PBEE. For these and other reasons, particular focus is given to the extensive qualification approach in this study.

3.3 | Damage states (DSs) and limit states

Four representative DSs associated with the NE can be defined to perform seismic qualification through both possible approaches: absent damage DS0, minor damage DS1, moderate damage DS2, and major damage DS3. DS1 achievement implies the need for minor repair interventions and/or rearrangement of specimens to restore the original conditions; in general, DS1 does not affect the functioning of the element. DS2 implies that the test specimen is damaged so it should be partially replaced, and this results in loss of functioning. DS1 and DS2 are typically associated with serviceability limit states (CEN, 2005). DS3 implies that the damage level is such that the test specimen needs to be totally replaced/repaired and life safety is not ensured. DS3 is typically associated with ultimate limit states (CEN, 2005).

The technical definition of the damage level associated with DS, which is defined by damage-to-DS criteria or correlations (viz., *damage scheme*), strongly depends on the type, features, and arrangement of NEs. An example of the DS definition for seismic qualification of a temporary partition wall was proposed by Petrone et al. (2017). In this context, the DS definition and related consequences are based on the definition given by Taghavi and Miranda (2003). In particular, the correlation between each DS and the loss can be expressed in terms of the three "D" (FEMA, 2012): (a) human casualties (*Deaths*), (b) direct economic loss due to the repair or replacement of NCs (*Dollars*), and (c) occupancy or service loss (*Downtime*). Damage schemes should be defined for each type of damage and for each significant component of the test specimen (e.g., panels, studs, horizontal elements, rails, and screws in Petrone et al., 2017). The more the damage scheme is defined by quantitative and univocal engineering parameters and measures, the more this is efficient and robust.

Regarding shake table tests, damage to NEs should be observed after each seismic performance evaluation test by inspecting the physical conditions of the test specimen, and an appropriate damage survey form should be compiled. The achievement of DSs should be identified by analysis of the damage survey forms according to the criteria defined within the damage scheme, and the DSs should be correlated to efficient intensity parameter measures that are representative of the seismic demands. The limit state verifications should be performed according to the qualification approach and the compliant regulations.

3.4 | Test specimens and loading program

The selection of the test specimen should follow the aim of the manufacturer and the requirements of the relevant regulations. In particular, the test specimen should be more or less representative of a production line according to the expected wideness and representativeness of the qualification results. Generally, the test specimen should represent a conservative condition to obtain the least seismic capacity associated with the system of interest. The number of specimens to be tested should also be compliant with the relevant requirements and should reasonably depend on (a) desired qualification robustness and (b) potential uncertainty associated with the production and response of the specimen.

The loading program consists of a series of dynamic tests, including both dynamic identification tests and seismic performance evaluation tests. According to the extensive qualification approach, the seismic performance evaluation tests should be performed through an incremental procedure. The initial, incremental, and final testing IM level should be chosen according to the expected behavior and damage exhibited by the specimen with regard to the performance target (e.g., operativity conditions) and should be compatible with the capacities of the shake table and instrumentation. Before and after each significant performance evaluation test, a dynamic identification test should be conducted along each principal direction of the test specimen. Generally, the performance evaluation tests should be performed via triaxial tests, with motions applied in the principal directions of the NEs. However, biaxial (horizontal) tests may be carried out if the element can be reasonably assumed to be not sensitive to the accelerations along the vertical direction; this condition could be considered to be applicable if the vertical fundamental period is at least an order of magnitude lower than a maximum horizontal fundamental period or if the vertical fundamental period is outside the significant frequency range of the signal.

3.5 | Dynamic identification tests

White noise tests are recommended in this study to identify natural frequencies and damping of the test specimen (Cosenza et al., 2015; Di Sarno et al., 2015; Fiorino et al., 2019; Jun et al., 2020; Lu et al., 2018; Luo et al., 2021; Magliulo et al., 2014; Qi et al., 2021). In practice, white noise is a theoretical idealization since no system can generate a uniform spectrum for all frequencies extended from zero to infinity. In real applications, white noise signals present spectral ordinates having values oscillating around the reference spectral value over a range of frequencies. Typically, the white noise signals present greater amplitude at low frequencies and a smaller amplitude tending to zero at higher frequencies (Luise & Vitetta, 2009).

The random noise excitation should be obtained by a uniform random stationary process (Clough & Penzien, 2003). The acceleration peaks of the signal shall be at most of 0.10 ± 0.05 g; this intensity threshold should prevent causing damage to the specimen; however, in some cases, a lower (or higher) intensity might be considered. The signal should have a significant energy content ranging from 1 to 32 Hz, a minimum duration of 60 s, and a sampling frequency of 200 Hz. The baseline can be filtered to provide the abovementioned frequency contents to the random noise excitation or to eliminate frequencies not compatible with the instrumental facility capacity; in the latter case, it should be verified that the cut of the critical frequencies does not affect the reliability of the signal and the robustness of the dynamic identification (e.g., by proving that the specimen does not exhibit significant sensitivity to those frequencies). In particular, a fourth-order low-pass Butterworth filter with a cutoff frequency of 40 Hz was used in the context of the proposed protocol, according to the literature review and expertise of the authors. The signal was then modified with a window function to have a rise time and decay time equal to 5% of the signal duration.

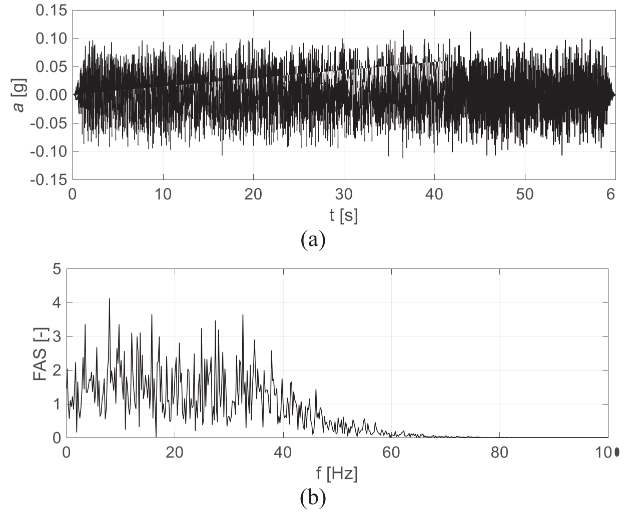

FIGURE 2 Example of random noise excitation history: (a) acceleration time history; (b) Fourier amplitude spectrum (FAS)

Figure 2a shows a representative random noise excitation developed according to this procedure. Figure 2b shows the corresponding Fourier amplitude spectrum (Zhou & Adeli, 2003a) that extends from 1.0 to 40 Hz and envelops the frequency range of interest.

3.6 | Seismic performance evaluation tests

3.6.1 | Required response spectra (RRS)

The proposed RRS were developed according to the NTC 2018 (MIT, 2018) formulation and literature studies (Petrone et al., 2015, 2016). NTC 2018 defines the total design horizontal force on NE, F_a, defined as

$$F_a = \frac{S_a W_a}{q_a} \tag{1}$$

In particular, F_a is the horizontal seismic design force applied at the component's center of gravity and distributed relative to the component's mass distribution, S_a is the horizontal spectral design acceleration of the NE attached at level i of the building structure for the limit state in question, W_a is the weight of the NE, and q_a is the NE response modification factor or behavior factor, that is, a factor aimed at reducing the elastic design forces accounting for the expected inelastic response; q_a can be specified according to the ductility and overstrength of the NE, referring to regulations/codes (ASCE 7–16, 2016; CEN, 2005; MIT, 2018) and/or literature studies (Johnson & Dowell, 2017; NIST, 2018; Kazantzi et al., 2020).

The floor response spectrum (S_a) used to calculate the horizontal equivalent static force is given by:

$$S_a = \begin{cases} \max\left\{ \alpha S\left(1+\frac{z}{H}\right)\left[\frac{a_p}{1+(a_p-1)\left(1-\frac{T_a}{aT_1}\right)^2}\right], \alpha S \right\} \text{for} T_a < aT_1 \\ \alpha S\left(1+\frac{z}{H}\right) a_p \text{for} aT_1 \leq T_a < bT_1 \\ \max\left\{ \alpha S\left(1+\frac{z}{H}\right)\left[\frac{a_p}{1+(a_p-1)\left(1-\frac{T_a}{bT_1}\right)^2}\right], \alpha S \right\} \text{for} T_a \geq bT_1 \end{cases} \quad (2)$$

according to NTC commentary (MIT, 2019); this formulation was derived from Petrone et al. (2015, 2016) and was already proven to be reliable by recent literature studies, considering bare and infilled RC buildings as a reference (Chichino et al., 2021; Di Domenico et al., 2021). In particular, α is the ratio between the design PGA on stiff soil for the relevant limit state and acceleration of gravity; S is the soil amplification factor; z is the height of the building point of attachment of the component, measured from the foundations; H is the average roof height of the building measured from the foundations; T_a is the fundamental period of the component-attachment system, T_1 is the fundamental period of the building; and a, b, and a_p are parameters defined according to the fundamental period of the building (tab. C7.2.II, Section C7.2.3, NTC commentary).

If the dynamic proprieties of the building are not defined, it is not possible to evaluate the RRS through Equation (2). In particular, regarding (generic) seismic qualification, NEs should be assumed to be installed in different types of buildings, and the RRS should not depend on specific dynamic characteristics of the building. In order to supply a valid and applicable qualification, a novel RRS formulation was developed considering a wide and representative range of building fundamental periods, that is, from 0.1 to 2.0 s; this range was defined according to representative European building scenarios (Housner et al., 1953; Rodriguez et al., 2021). Figure 3 shows the dimensionless floor response spectra for $z/H = 1$, obtained considering the range of periods of interest (5% damping). The proposed RRS envelop 0.1 to 2.0 s building period floor response spectra evaluated according to:

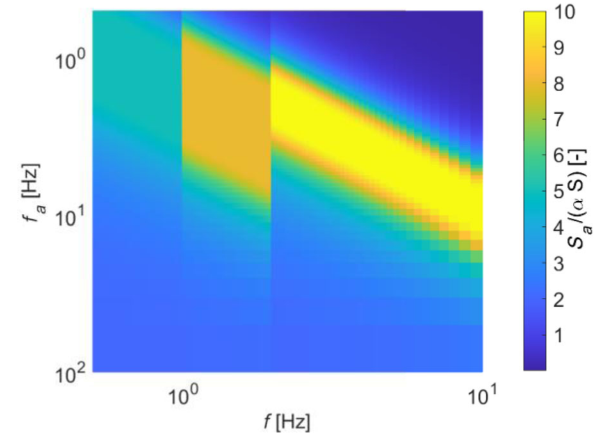

FIGURE 3 Dimensionless response spectra for the range building fundamental periods from 0.1 to 2.0 s, expressed as a function of fundamental nonstructural element (NE) frequency (f_a) and fundamental frequency of primary structure (f)

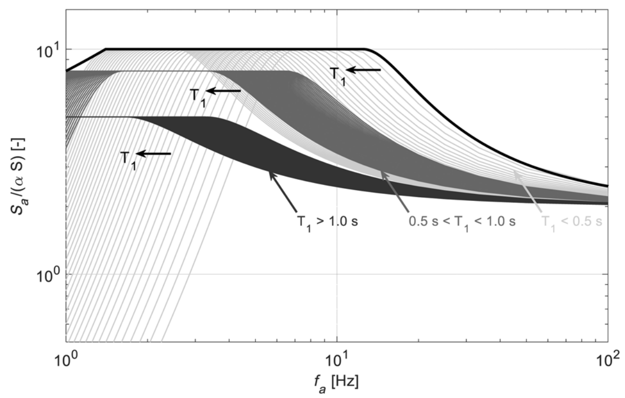

FIGURE 4 RRS (5% damping) derivation according to the proposed protocol

$$\frac{S_a}{(\alpha S)} = \begin{cases} 4\left(1+\frac{z}{H}\right) + \frac{\left(1+\frac{z}{H}\right)}{(f_1-f_0)}(f_a - f_0) & \text{for } f_a < f_1 \\ 5\left(1+\frac{z}{H}\right) & \text{for } f_1 \leq f_a < f_2 \\ \left[\frac{5\left(1+\frac{z}{H}\right)}{1+4\left(1-\frac{f_2}{f_a}\right)^2}\right] & \text{for } f_a \geq f_2 \end{cases} \quad (3)$$

as it is depicted in Figure 4. The formulation is reported in Equation (3), where f_0, f_1, and f_2 are set equal to 1.00, 1.40, and 12.5 Hz, respectively.

Considering the most relevant z/H condition (i.e., equal to unity), the proposed RRS are compared with reference protocol RRS and input spectral responses (for FEMA 461) in Figure 5, following the same comparison approach used in Figure 1. Considering PGA equal to 0.50 g, the proposed RRS is the most conservative RRS, whereas it is among

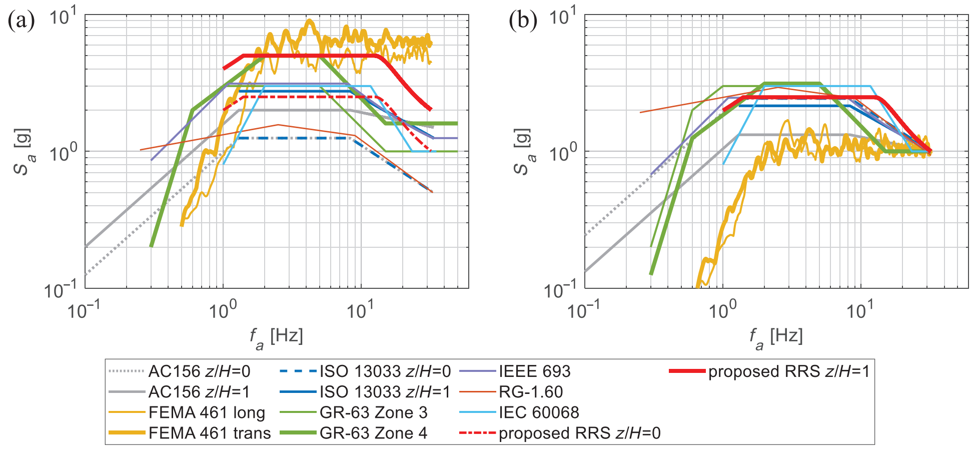

FIGURE 5 Comparison between proposed RRS and RRS and input response spectra related to reference protocols considering (a) PGA equal to 0.50 g and (b) spectral ordinate corresponding to 32 Hz (S_a(32 Hz)) equal to 1.0 g

the most conservative RRS if PFA equal to 1.0 g is considered. The plateau frequency range of the proposed RRS is larger than other protocols. Even if few reference RRS provide slightly higher ordinates corresponding to narrow frequency ranges, the proposed RRS is overall the most conservative one, especially considering the reference RRS envelope and both PGA equal to 0.50 g (Figure 5a) and S_a(32 Hz) equal to 1.0 g (Figure 5b) conditions. The reference spectra that exceed the proposed RRS ordinates in some regions are associated with significantly lower responses in other regions. It should be noted that the proposed RRS matches significantly well the plateau of FEMA 461 input spectra. The proposed RRS associated with z/H equal to zero is also more severe than other reference protocol RRS, especially considering PGA equal to 0.50 g condition (Figure 5a).

3.6.2 | Seismic performance evaluation test input

The generation and processing of the seismic performance evaluation test input in terms of acceleration time history were implemented considering the RRS described in Section 3.6.1. In this specific case, the RRS provided by Formula (3) was detailed assuming 5% damping, $z/H = 1$, and $\alpha^{\bullet}S = 0.4$ g, which is representative of high seismicity in Italy. However, the procedure is general and easily applicable considering different seismic demand formulations or RRS.

An artificial procedure was carried out through three phases: (1) *baseline generation*, (2) *RRS spectrum-compatibility enforcement*, and (3) *further signal processing*, including (4) *exceptions*, which are described in the following.

The procedure is described with regard to horizontal components, but it is generalizable for the vertical direction. According to the literature, the authors recommend that if triaxial tests are to be performed, the response spectra of the vertical input should be compatible with 80% of the horizontal RRS.

1. Baseline generation: The baseline signal was generated by enforcing the following features: nonstationary random signal with an energy content ranging from 1.0 to 32.0 Hz; one-sixth octave bandwidth resolution, that is, for each octave, two consecutive frequencies have a ratio equal to $2^{1/6}$; sampling rate of 400 Hz; total duration equal to 30 s; at least 20 s of strong motion; nonstationary time history with rise (RP), strong motion (SMP) and decay (DP) parts of 5, 20, and 5 s, respectively. For each frequency (f_i) a sinusoidal wave with a duration of 30 s was defined as follows:

$$x_i\,(t) = A \sin\left(2\pi f_i t + \varphi_i\right) \tag{4}$$

In particular, A is the amplitude of the sinusoidal wave; the time step (t) is the reciprocal of the sampling rate, that is, equal to 0.0025 s; φ_i is the phase angle of the sinusoidal wave, defined according to:

$$\varphi_{i+1} = \frac{a\pi}{n_f} + \varphi_i \tag{5}$$

The phase angle of the first sinusoidal wave was set equal to $\varphi_1 = a\,\pi/n_f$, where n_f is the number of frequencies in the range from 1.0 to 32.0 Hz, and a is a harmonizing factor that modifies the phase of the strong motion of baseline. In particular, a defines the quantitative manner of combination of the elementary frequency contents in terms of harmonic functions. This parameter is

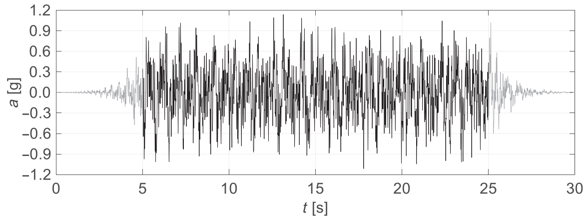

FIGURE 6 Example of baseline for developing a seismic performance evaluation test input. The gray part represents rise part and decay part of the signal, whereas the black one represents the strong motion part

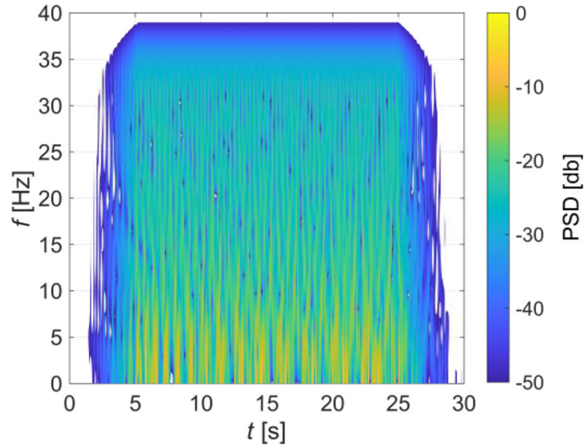

FIGURE 7 Power spectral density (PSD) of the baseline

responsible for the unicity of the baseline and accounts for the "random" character. The value of a for each baseline was assumed by implementing a random function in Matlab, which selects, randomly, real numbers (for further details regarding this issue, see Bendat & Piersol, 2010; Clough & Penzien, 2003). This factor allows to obtain a smooth signal and to avoid abrupt discontinuities of the baseline. The baseline was obtained by adding the three parts, that is, RP, SMP, and DP: SMP ($y_{SMP}(t)$) was determined as the mean of all sinusoidal waves for each time step in the range from 5 to 25 s; RP was defined with a growth exponential signal in the range from 0 to 5 seconds, according to:

$$y_{RP}(t) = \frac{y_{SMP}(t)\, e^{t}}{b} \tag{6}$$

where b is the harmonizing factor of RP of the baseline; finally, DP was defined with a negative exponential signal in the range from 25 to 30 s using:

$$y_{DP}(t) = \frac{y_{SMP}(t)\, e^{-(t-25)}}{c} \tag{7}$$

where c is the harmonizing factor of the DP of the baseline.

Figure 6 shows an example of a baseline signal, where RP and DP are depicted in gray and SMP in black. Figure 7 shows the spectrogram (power spectral density) of the baseline highlighting how the frequency content varies with time. The maximum spectral power is concentrated in the frequency range of interest and in the SMP.

2. RRS spectrum-compatibility enforcement: The spectrum-compatibility enforcement was carried out using RSPMatch (Hancock et al., 2006). In particular, the procedure was applied through a time-domain modification of the baseline signal to enforce the RRS spectrum compatibility. The signal matching procedure was implemented according to the recommendations provided by Hancock et al. (2006). In particular, various wavelets were added to the signal acceleration time history in the time domain, according to Suárez and Montejo (2005), that is, sinusoidal corrected displacement compatible wavelet using explicit integration (model 14, according to RSPMatch manual).

The spectrum compatibility was enforced by considering the frequency's sixths octaves used to generate the baseline to optimize the procedure. In particular, the seismic performance evaluation test input should be associated with response spectra that envelop RRS considering a maximum one-sixth-octave bandwidth resolution over the frequency range from 1 to 32 Hz. The amplitude of each matched spectrum ordinate should be independently adjusted until the response spectrum envelops the RRS. The response spectrum ordinates should not be lower than RRS and larger than 1.3 times RRS. For these reasons, the RSPMatch reference RRS was obtained by considering a 10% increase of the RRS described in Section 3.6.1.

The spectrum compatibility should be checked considering both signals to assign to the table (*theoretical signals*) and signals recorded by the table during the seismic performance evaluation tests (*actual signals*). Theoretical signals associated with spectra that fall below the RRS ordinates are generally not acceptable, whereas the spectrum-compatibility criteria to check the actual (i.e., reproduced/recorded) signals are less stringent. In the latter case, a maximum of two of the one-sixth octave analysis points may be below RRS, in terms of spectral ordinate, by 10% or less, provided that, for each point, the adjacent one-sixth-octave points are at least equal to RRS. This condition can occur in both the amplified region of the RRS (frequencies less than or equal to 12.5 Hz) and ZPA region (frequencies greater than 12.5 Hz).

3. Further signal processing: The maximum accelerations, velocities, and displacements associated with the

theoretical inputs should be estimated considering the maximum expected levels of shaking intensity defined in the loading program. These estimations should be compared with the capacities of the instrumentation and shake table to guarantee consistent tests and to guarantee the reliability of the results. In the case of exceedance of the capacity thresholds, the input might be subjected to further filtering processing. The filtering procedure is described in the following considering a representative case-study application.

4. Exceptions: There might be cases in which signal processing procedures are not able to guarantee full compatibility of the signal with the limitations/capacities/features, and the signal cannot be adequately reproduced by the shake table (i.e., the theoretical signal is spectrum-compatible and the reproduced signal is not). These difficulties might be associated with two crucial issues: significantly larger peak displacements of the tables and/or major resonance of the shake table with testing facilities or infrastructures. When the filtering procedures do not solve the abovementioned problems, a novel approach could be used to operatively solve the problem. The first step is the detection of the unique frequency range that is associated with the abovementioned problems, if it exists, for example, typically lower frequencies for higher displacements or facility/infrastructure fundamental frequencies for resonance issues. Once the frequency range is identified, if this is sufficiently reduced, that is, it does not exceed a one-sixth octave interval, the baseline can be generated by assuming a parameter A corresponding to these two one-sixth octave elementary harmonics that is lower than the value assumed for all other harmonics; this value might even be set equal to zero. The spectrum-matching procedure is then carried out considering the modified baseline as an input, and if the spectrum compatibility is fully achieved, the output signal could be fully considered to be compliant with the protocol. As a matter of fact, pilot studies carried out by the authors found that this procedure lowers the Fourier transform amplitude corresponding to the critical frequencies (for a maximum of two one-sixths octave) still enforcing the full spectrum compatibility. In particular, the matching procedure adds wavelets that also correspond to the two critical one-sixths octave to achieve the compatibility. Lowering the transform amplitudes eases the reproducibility of the signal since the energy content associated with the critical frequencies is lower, even though the signal is fully compliant with the RRS. In particular, the lowering of A should be balanced by an enhancement of the signal reproducibility by the shake table. The value of parameter A to assume depends on the criticality of the reproducibility issues and should be calibrated by iterative signal generation processes and experimental calibrations/tests at the discretion of the analysts. This exception does not affect the signal severity since the presence of the energy contents related to the critical frequencies is guaranteed (wavelets added by matching procedure) and the spectrum compatibility is achieved.

Considering the present application and concerning the facilities of the Laboratory of the University of Naples Federico II (Italy), the maximum displacement limits were assumed to be equal to ±25.0 cm. These limits are likely to be compatible with most shake tables and earthquake simulators (Takhirov et al., 2017). However, the procedure described in the following is generalizable and applicable to different capacity limits. Since displacement time histories typically present peaks due to long-period components of the accelerograms, a low-cut filter can be applied to the signals to reduce maximum displacements (Trifunac, 1971). The obtained matched record was, then, filtered with a band-pass Butterworth filter, order equal to four, over the range of frequency 0.4 ÷ 40 Hz. This filter is among the most used in literature, as well as it can be considered among the most effective and robust ones for reducing the long-period noise in accelerograms (Boore & Bommer, 2005). The need to keep the signal energy content ranging from 1.0 to 32.0 Hz justified the use of a band-pass filter. In particular, the lower cutoff frequency was determined by the need to correct the signal according to the procedure by Boore and Bommer (2005) and to keep the energy content from 1 Hz, whereas the higher cutoff frequency was determined considering the frequency limit of the shake table (in this specific case equal to 50 Hz) and the upper limit of the energy content of the signal. The acausal filter was used to not produce any phase distortion in the signal (Boore & Bommer, 2005). Moreover, the low-frequency content was eliminated from the test signals records for not exceeding the displacement and velocity capacities of the shake table.

The definite signals should be verified to be spectrum compatible, and preliminary (empty table) tests should be performed to fully check the experimental reproducibility. In case the conditions associated with Exceptions apply, the previously proposed procedure can be used to foster the generation of fully reproducible signals.

The acceleration, velocity, and displacement time histories of the test signal before and after the filtering procedure are shown in Figure 8. The difference between filtered and unfiltered velocities and accelerations is negligible, while the maximum displacement was reduced to about 200 mm, lower than the table limit, and the mean deviation was zeroed. Figure 9 depicts the spectrum-compatibility check performed with respect to the protocol

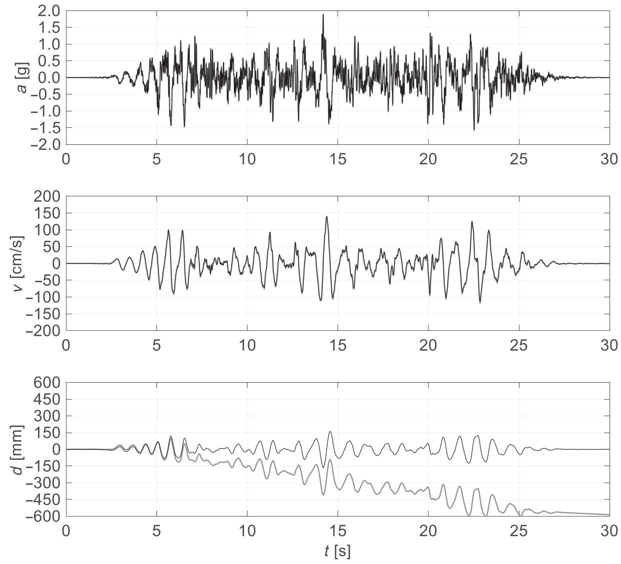

FIGURE 8 Acceleration, velocity, and displacement time histories of the test signal: output by RSPMatch (gray), and output after the filtering procedure (black)

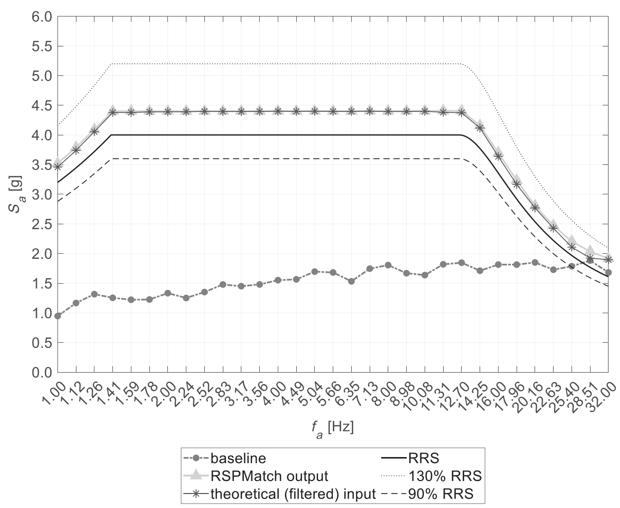

FIGURE 9 Spectrum-compatibility check of the test response spectrum (TRS) with RRS and RRS limits: TRS of the baseline signal, TRS of the RSPMatch output signal, and TRS of the (further) filtered signal

requirements, considering baseline, signal after spectrum compatibility, and signal after further filtering.

4 | EVALUATION AND VALIDATION

4.1 | Methodology

In this section, the proposed protocol is evaluated and validated considering a set of seven representative performance evaluation test acceleration signals. These

TABLE 2 Definition of strong floor motion duration (SFMD), specific energy density (SED), and predominant period (T_m)

$\boldsymbol{SFMD} = \boldsymbol{t}_{95} - \boldsymbol{t}_5$	$\boldsymbol{t_X} = \bar{\boldsymbol{t}} \mid \boldsymbol{I_a}(\bar{\boldsymbol{t}}) = \frac{X}{100}\boldsymbol{I_a}(\boldsymbol{T_D})$ $I_a(\bar{t}) = \frac{\pi}{2g}\int_0^{\bar{t}}[a(t)]^2 dt$
$\boldsymbol{SED} = \int_0^{T_D}[\boldsymbol{v}(\boldsymbol{t})]^2 \boldsymbol{dt}$	
$\boldsymbol{T_m} = \frac{\sum_i C_i^2(1/f_i)}{\sum_i C_i^2}$	

Note: T_D is the total duration of the signal, I_a is the Arias intensity (Arias, 1970), C_i are the Fourier amplitude coefficients, and f_i are the discrete fast Fourier transform (FFT) frequencies between 0.25 and 20 Hz.

signals are referred to as novel protocol (acceleration) signals (NPSs). NPSs are tested through a multi-level criteria approach, which is associated with signal-based assessment.

Time history assessment: Acceleration, velocity, and displacement time histories of NPSs are analyzed, also referring to their spectral response.

Seismic parameter assessment: Representative seismic parameters typically correlated with seismic damage of dynamic systems are computed for NPSs, and they are assessed considering representative real-floor motions as a reference, referred to as FMs. In particular, Table 2 reports the seismic parameters considered for the analysis, that is, strong floor motion duration (SFMD) (Rodriguez et al., 2021; Trifunac & Brady, 1975), peak floor velocity (PFV) to PFA ratio (D'Angela et al., 2021b; Kramer, 1996), specific energy density (Cao & Ronagh, 2014; Nguyen et al., 2020), and predominant period (T_m; Nguyen et al., 2020; Rathje et al., 2004). These parameters were generally found to be well-correlated with both (seismic) damage potential and exhibited damage of structures and NEs, even though they cannot be considered to be exhaustive.

Spectral assessment: Elastic acceleration response spectra of NPSs are assessed considering FMs and alternative protocols as a reference, considering 5% damping.

Time history and seismic parameter assessments do not imply the assumption of specific models for the case-study acceleration-sensitive elements, whereas the spectral assessment procedure implicitly assumes a linear elastic SDOF response, which is consistent with the case-study elements (see Section 2.1). It should be mentioned that a damage-based evaluation should be carried out and an experimental validation should be performed to fully validate the protocol for regulation/code implementation purposes.

FMs are signals recorded in instrumented US buildings and derived from Center for engineering strong motion data (CESMD) database (Partner Data Centers & Networks, 2017); for each seismic event and building, the most amplified (acceleration) response was selected over the building floors. In particular, reinforced concrete

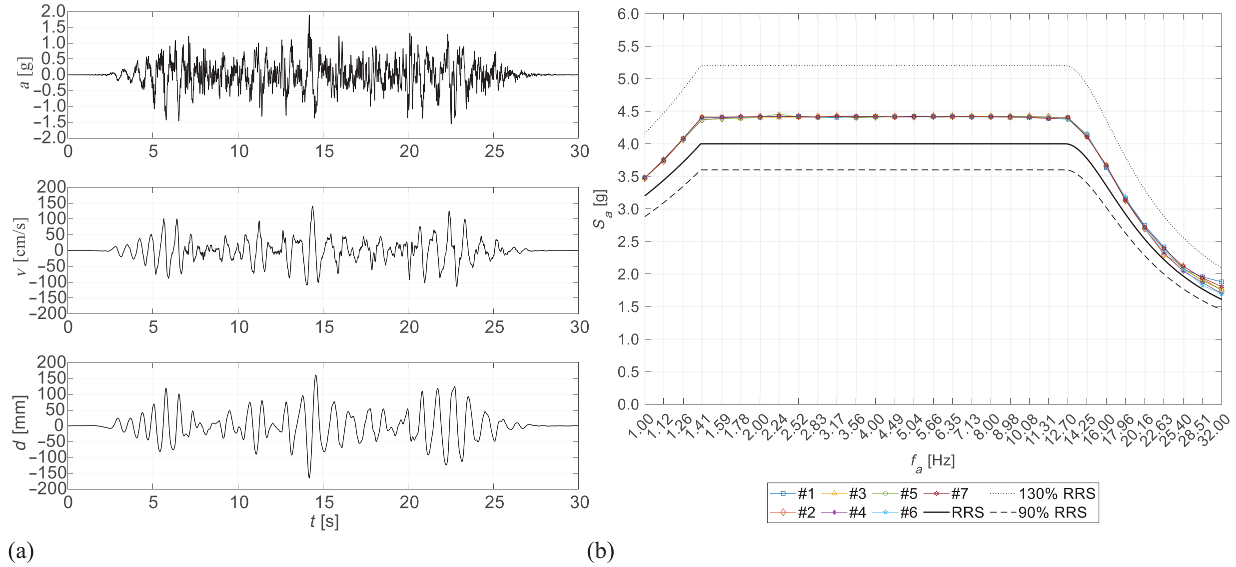

FIGURE 10 (a) Acceleration, velocity, and displacement time histories related to novel protocol signal (NPS) #1 and (b) response spectra related to reference protocol signals (time histories NPS #2 to #7 are reported in the Appendix). NPSs are related to RRS having PGA equal to 0.40 g and assuming *z/H* equal to one

buildings designed/built from 1923 to 1975 are considered as a reference. Two sets of FMs are considered for both seismic parameter and spectral assessment: (Set 1 FMs) 24 records related to an equal number of low-, medium-, and high-rise buildings, equally including near- and far-field ground motions, with PGA ranging in 0.05 to 0.45 g; (Set 2 FMs) seven records related to low-, medium-, and high-rise buildings, including both near- and far-field ground motions, with PGA larger than 0.20 g. Set 2 is included within Set 1. Further details on the selected floor motions are omitted as the same FM sets were used in D'Angela et al. (2021b).

4.2 | Results

Figure 10a depicts a representative NPS (#1) expressed in terms of acceleration, velocity, and displacement time histories; the other reference NPSs are reported in the Appendix. The response spectra associated with the developed time histories are shown in Figure 10b. NPSs are developed considering PGA equal to 0.40 g and assuming *z/H* equal to one. Qualitatively, the time histories are not dissimilar to real ground and floor records (D'Angela et al., 2021b), as well as they are quite similar to the ones developed according to other protocols, such as AC156 ones (Kim & Shin, 2021; Luo et al., 2021; Petrone et al., 2017). RP, SMP, and DP are quite regular, and several significant peaks are observed in SMP, especially in the first and last part. The time histories have (multiple) significantly high peaks; PFA, PFV, and peak floor displacement range in 1.45–1.88 g, 1.15–1.40 m/s, and 0.146–0.184 m, with median values equal to 1.76 g, 1.21 m/s, and 0.156 m, and coefficient of variation equal to 0.098, 0.076, and 0.081, respectively. The response spectra are overall relatively smooth, even though a minor (genuine) dispersion can be observed among the different spectra, especially in the amplified frequencies region. The spectrum-compatibility criteria determine spectral ordinates overall slightly larger than RRS ones.

Figure 11 depicts the comparison between NPSs and (Set 1 and Set 2) FMs in terms of (a) SFMD, (b) PFV/PFA, (c) SED, and (d) T_m. The results are reported considering each signal and percentile/median threshold for NPSs and FMs, respectively.

Considering all parameters, NPSs provide values larger (smaller) than the median (86th percentile) related to Set 1 FM ones, whereas NPS values match very well (are larger than) median values of set 2 FM considering PFV/PFA and T_m (SFMD and SED). A higher parameter value is typically associated with higher damage potential for the investigated parameters. NPSs provide a reduced dispersion, associated with limited uncertainty and variability due to the signal generation/development process. These findings confirm the reliability of the protocol procedure and prove that the protocol loading histories are potentially associated with relatively high and representative damage severity, according to efficient seismic parameters.

Figure 12 shows the acceleration response spectra of NPSs normalized considering (a) PFA and (b) PGA, compared with (1) Set 1 and (2) Set 2 FMs, respectively, whereas Figure 13 depicts the spectral comparisons among

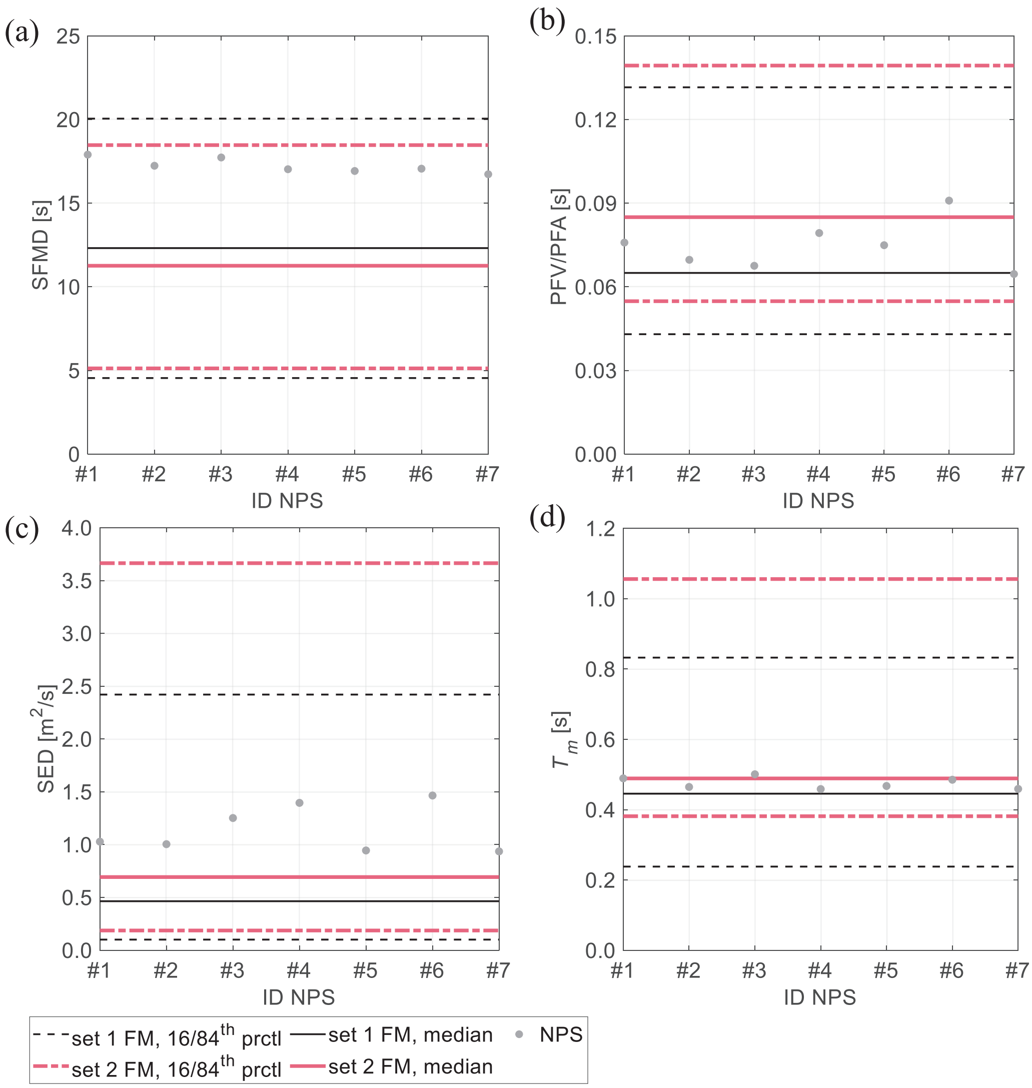

FIGURE 11 Comparison between NPSs and (Set 1 and Set 2) floor motions (FMs) considering (a) strong floor motion duration (SFMD), (b) peak floor velocity (PFV)/peak floor acceleration (PFA), (c) specific energy density (SED), and (d) predominant period (T_m)

reference protocol inputs, NPSs, and FMs (PGA equal to 0.50 g).

Considering the component amplification, that is, looking at spectral response normalized using PFA, the response spectra related to NPSs envelop very well to the median spectrum of Set 1 FMs (Figure 12a1); moreover, they also envelop the 84th percentile spectrum except for few peak responses, associated with 1.59 to 4 Hz. However, enveloping the 84th percentile would certainly be too conservative, as the median spectral response (over seven spectra) is typically considered to be reliable for (structural assessment) spectrum compatibility (e.g., CEN, 2005). Therefore, NPSs are conservative but not in an excessive manner, accounting for a wide and representative range of low-to-high seismicity hazard, building, site, and soil type scenarios. Regarding Set 2 FMs, median FM spectra exceed NPS spectra only in the narrow vicinity of 3 Hz, even though with a magnitude not larger than 20% (Figure 12a2); in other frequency ranges, NPS spectra are significantly higher than median FM ones. However, it is to be noted that unscaled (natural) Set 2 FMs are associated with an average PGA equal to 0.32 g, which is consistent with high-to-very high seismicity in Europe. Set 2 FM PGA is 88% higher than the value associated with Set 1 FMs. Furthermore, unscaled (natural) Set 2 FM also present PGV, PFA, and PFV (120%, 76%, and 100%, respectively) larger than related values of Set 1 FMs. For further details regarding Set 1 and Set 2 FMs, refer to D'Angela et al. (2021b).

Considering both building and component amplification, that is, looking at spectral response normalized considering PGA, NPS spectra present ordinates significantly higher than both Set 1 (Figure 12b1) and Set 2 (Figure 12b2) FMs, whereas FM 84th percentiles slightly

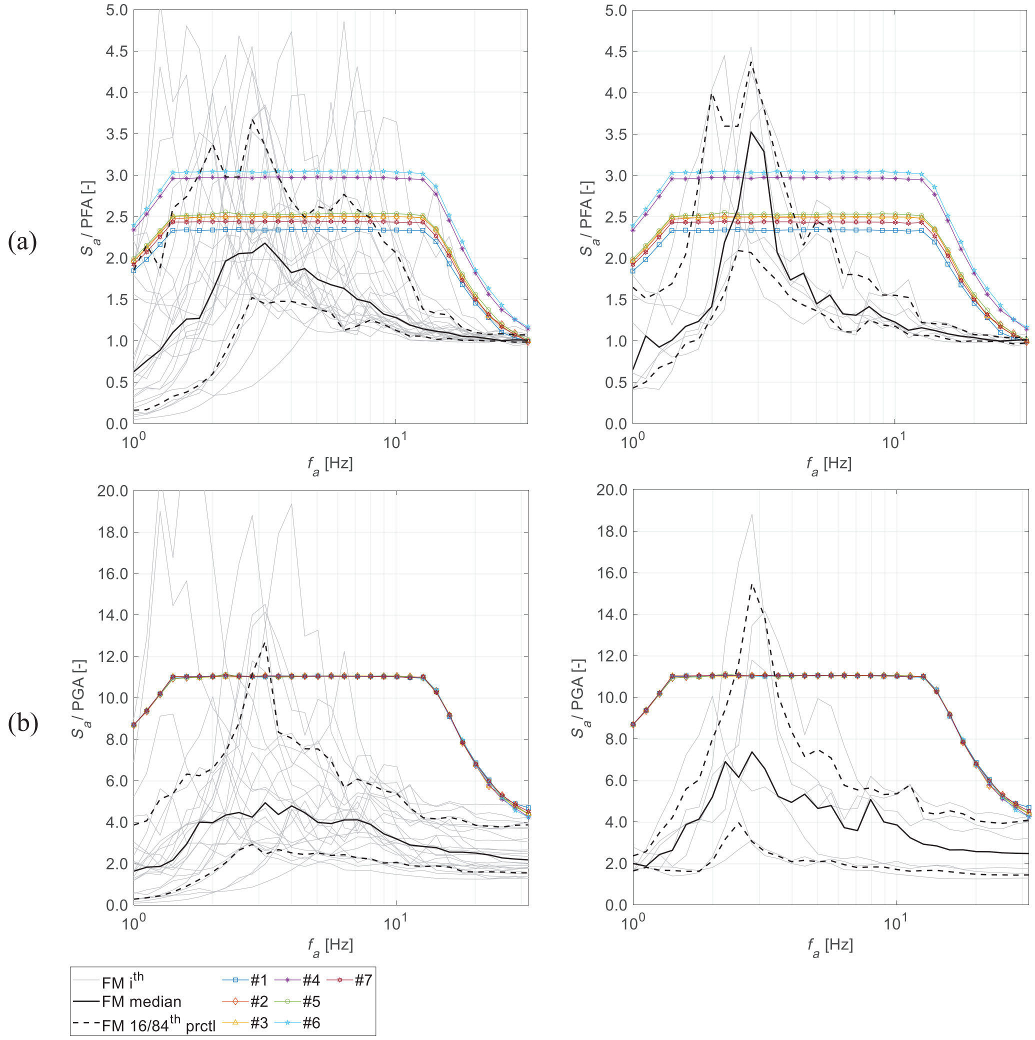

FIGURE 12 Acceleration response spectra of NPSs normalized considering (a) PFA and (b) PGA, compared with (1) Set 1 and (2) Set 2 FMs. All spectra are related to 5% damping

exceed and exceed NPS spectra in the narrow vicinity of 3 Hz considering Set 1 and Set 2 FM, respectively. Further comments on the conservativity associated with 84th percentiles of FMs are omitted as this was previously discussed.

The safe compatibility between NPSs and FM stresses the reliability of the developed protocol with regard to seismic and building scenarios considered to develop the seismic demand associated with RRS. Moreover, this evidence proves the generality and wide applicability of the developed protocol. Extensive comparisons between RRS of NPSs and alternative reference protocols were reported in the previous sections. However, as an additional evaluation and validation means, reference protocol spectra (RRS and input spectra) are compared with NPS spectra and FMs in Figure 13a,b, respectively.

GR-63-CORE RRS is not reported as this is defined for 2% damping (and all other spectra, including NPS ones refer to 5% damping). The spectra are reported considering PGA equal to 0.50 g since this allows assessing both building and component amplification response. Reference protocols provide spectral ordinates significantly lower than the NPS ones (Figure 13a) and, in some cases, lower than Set 1 and Set 2 FM median responses. This points out the superiority of the developed RRS and NPSs and, overall, of the developed protocol.

5 | CONCLUSION

The study addresses seismic assessment and qualification of NEs by means of shake table testing; in particular, a novel testing protocol is developed and analytically validated. The protocol development is based on the synthesis among (a) technical critical evaluation of reference existing protocols, (b) recent advances in the field, also

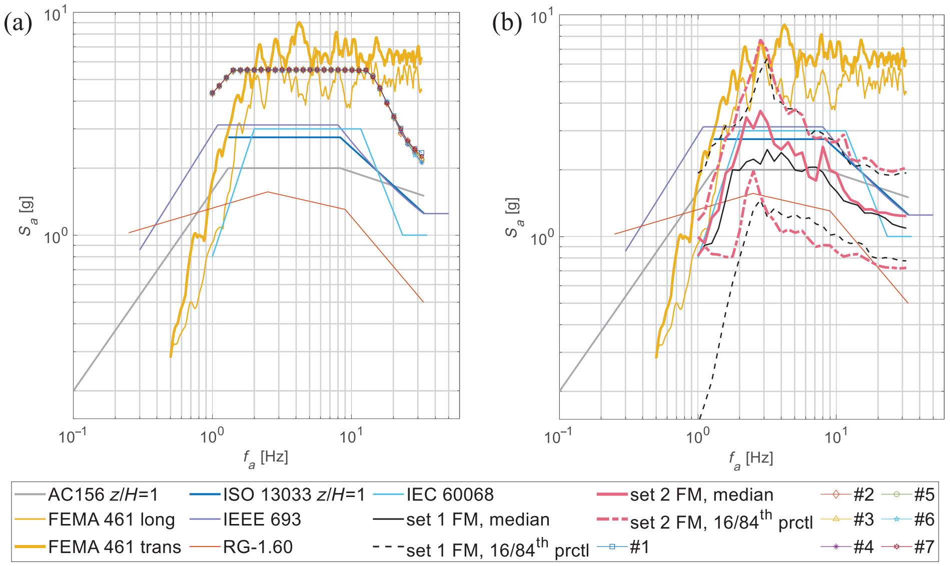

FIGURE 13 Comparison between RRS/input response spectra related to reference protocols and (a) NPS spectra and (b) FM spectra, considering PGA equal to 0.50 g. All spectra are related to 5% damping

accounting for latest literature studies and testing applications, and (c) expertise and experience gained in the field by the research team.

Novel testing approaches are developed toward seismic assessment and qualification performed following the PBEE. Technical criteria are developed for defining a robust qualification protocol. This protocol is demonstrated to provide more reliable testing procedures, promisingly associated with more robust assessment and qualification outcomes.

The most significant and substantial feature of the protocol is associated with a consistent code-compliant definition of the loading histories to perform seismic evaluation tests. This definition follows the extension of a recently developed seismic demand formulation, which is compliant with reliable estimations and proven reliability. This formulation is implemented considering an innovative approach, which accounts for a wide variability of building periods. The most critical part of the seismic signal development is associated with the analysis and processing procedures that are carried out through consolidated methodologies, which are clearly described and discussed in the paper, also providing technical and detailed guidance for implementation.

The reliability and the general applicability of the developed protocol are assessed and confirmed by an extensive evaluation and validation process based on analysis of time history signals, seismic parameters, and spectral response. Representative real-floor motions are considered as a reference for the validation, considering both seismic parameter and response spectra assessment. The superiority and generality of the developed signals and protocol are also confirmed by considering the existing protocols as a reference.

The developed protocol, with particular regard to developed RRS and signal generation/processing, overcomes the technical inadequacies highlighted in the paper and within the literature studies. This is due to (1) a more reliable formulation of RRS, (2) more consistent signal development (e.g., clear baseline definition, robust matching criteria, and consistent signal processing techniques), (3) spectral and signal-based superiority considering existing protocols, and (4) quantitative validation considering real-floor motions as a reference.

The use and development of the proposed protocol promisingly result in more reliable seismic assessment and qualification processes, completely renewing the existing assessment and qualification methodologies and procedures. The developed approach and protocol are extendible to different case studies, suiting specific needs or requirements (e.g., different seismic demand formulations or specific testing facility capacities). Further studies should be carried out to validate the protocol by means of a damage-based validation procedure.

ACKNOWLEDGMENTS

This study was funded by (a) the Italian Department of Civil Protection (DPC) in the framework of the national project DPC–ReLUIS 2019–2021 WP17: “Code contributions for nonstructural elements” and (b) the Italian Ministry of University and Research (MUR) in the framework of national project PRIN 2020: “ENRICH project: ENhancing the Resilience of Italian healthCare and Hospital facilities.”

ORCID

Martino Zito https://orcid.org/0000-0002-5632-5651
Danilo D'Angela https://orcid.org/0000-0002-8096-5202
Giuseppe Maddaloni https://orcid.org/0000-0002-3173-0669
Gennaro Magliulo https://orcid.org/0000-0002-1812-4357

REFERENCES

Akkar, S., & Bommer, J. J. (2007). Prediction of elastic displacement response spectra in Europe and the Middle East. *Earthquake Engineering & Structural Dynamics*, *36*(10), 1275–1301.

Amezquita-Sanchez, J. P., Park, H. S., & Adeli, H. (2017). A novel methodology for modal parameters identification of large smart structures using MUSIC, empirical wavelet transform, and Hilbert transform. *Engineering Structures*, *147*, 148–159.

Amiri, G. G., Abdolahi Rad, A., Aghajari, S., & Khanmohamadi Hazaveh, N. (2012). Generation of near-field artificial ground motions compatible with median-predicted spectra using PSO-based neural network and wavelet analysis. *Computer-Aided Civil & Infrastructure Engineering*, *27*(9), 711–730.

Amiri, G. G., Rad, A. A., & Hazaveh, N. K. (2014). Wavelet-based method for generating nonstationary artificial pulse-like near-fault ground motions. *Computer-Aided Civil & Infrastructure Engineering*, *29*(10), 758–770.

Anajafi, H., & Medina, R. A. (2018). Evaluation of ASCE 7 equations for designing acceleration-sensitive nonstructural components using data from instrumented buildings. *Earthquake Engineering & Structural Dynamics*, *47*(4), 1075–1094.

Arias, A. (1970). Measure of earthquake intensity. In R. J. Hansen (Ed.), *Seismic design for nuclear power plants* (pp. 438–483). Massachusetts Institute of Technology Press.

ASCE 7–16. (2016). *Minimum design loads and associated criteria for buildings and other structures*. American Society of Civil Engineers (ASCE).

Bendat, J. S., & Piersol, A. G. (2010). *Random data: Analysis and measurement procedures* (4th ed.). Wiley.

Bianchi, S., Ciurlanti, J., Perrone, D., Filiatrault, A., Costa, A. C., Candeias, P. X., Correia, A. A., & Pampanin, S. (2021). Shake-table tests of innovative drift sensitive nonstructural elements in a low-damage structural system. *Earthquake Engineering & Structural Dynamics*, *50*(9), 2398–2420.

Blasi, G., Perrone, D., Aiello, M. A., & Pecce, M. R. (2021). Seismic performance assessment of piping systems in bare and infilled RC buildings. *Soil Dynamics & Earthquake Engineering*, *149*, 106897.

Boore, D. M., & Bommer, J. J. (2005). Processing of strong-motion accelerograms: Needs, options and consequences. *Soil Dynamics & Earthquake Engineering*, *25*(2), 93–115.

British Standards Institution (BSI). (2013). *Bases for design of structures-loads, forces and other actions-seismic actions on non-structural components for building applications*. British Standard Institution.

Cao, V. V., & Ronagh, H. R. (2014). Correlation between seismic parameters of far-fault motions and damage indices of low-rise reinforced concrete frames. *Soil Dynamics & Earthquake Engineering*, *66*, 102–112.

Chichino, B., Peloso, S., Bolognini, D., Moroni, C., Perrone, D., & Brunesi, E. (2021). Towards seismic design of nonstructural elements: Italian code-compliant acceleration floor response spectra. *Advances in Civil Engineering*, *2021*, 1–18.

Clough, R. W., & Penzien, J. (2003). *Dynamics of structures*. Computers & Structures, Inc.

Comité Européen de Normalisation (CEN). (2005). *Eurocode 8: Design of structures for earthquake resistance*. British Standards Institution.

Cosenza, E., di Sarno, L., Maddaloni, G., Magliulo, G., Petrone, C., & Prota, A. (2015). Shake table tests for the seismic fragility evaluation of hospital rooms. *Earthquake Engineering & Structural Dynamics*, *44*(1), 23–40.

Crewe, A. J. (2012). Generation of improved artificial earthquakes for seismic qualification testing. In *15WCEE* (pp. 1.10). Lisbon, Portugal.

D'Angela, D., Magliulo, G., & Cosenza, E. (2021a). Towards a reliable seismic assessment of rocking components. *Engineering Structures*, *230*, 111673.

D'Angela, D., Magliulo, G., & Cosenza, E. (2021b). Seismic damage assessment of unanchored nonstructural components taking into account the building response. *Structural Safety*, *93*, 102126.

D'Angela, D., Magliulo, G., & Cosenza, E. (2021c). Incremental dynamic analysis of rigid blocks subjected to ground and floor motions and shake table protocol inputs. *Bulletin of the New Zealand Society for Earthquake Engineering*, *55*(2), 64–79.

Di Domenico, M., Ricci, P., & Verderame, G. M. (2021). Floor spectra for bare and infilled reinforced concrete frames designed according to Eurocodes. *Earthquake Engineering & Structural Dynamics*, *50*(13), 3577–3601.

Di Sarno, L., Magliulo, G., D'Angela, D., & Cosenza, E. (2019). Experimental assessment of the seismic performance of hospital cabinets using shake table testing. *Earthquake Engineering & Structural Dynamics*, *48*(1), 103–123.

Di Sarno, L., Petrone, C., Magliulo, G., & Manfredi, G. (2015). Dynamic properties of typical consultation room medical components. *Engineering Structures*, *100*, 442–454.

FEMA 461. (2007). *Interim testing protocols for determining the seismic performance characteristics of structural and nonstructural components*. FEMA.

FEMA E-74. (2012). *Reducing the risks of nonstructural earthquake damage-a practical guide*. FEMA.

Filiatrault, A., Kuan, S., & Tremblay, R. (2004). Shake table testing of bookcase–partition wall systems. *Canadian Journal of Civil Engineering*, *31*(4), 664–676. https://doi.org/10.1139/l04-031

Fiorino, L., Bucciero, B., & Landolfo, R. (2019). Evaluation of seismic dynamic behaviour of drywall partitions, façades and ceilings through shake table testing. *Engineering Structures*, *180*, 103–123.

Gasparini, D. A., & Vanmarcke, E. H. (1976). *SIMQKE: A program for artificial motion generation. User's manual and documentation*. MIT Department of Civil Engineering.

Ghith, A., Ezzeldin, M., Tait, M., & El-Dakhakhni, W. (2019). Shake table seismic performance assessment of auxiliary battery power systems using the FEMA 461 protocol. *Journal of Structural Engineering*, *145*(8), 04019080.

Hancock, J., Watson-Lamprey, J., Abrahamson, N. A., Bommer, J. J., Markatis, A., McCoy, E. M. M. A., & Mendis, R. (2006). An improved method of matching response spectra of recorded earthquake ground motion using wavelets. *Journal of Earthquake Engineering*, *10*, 67–89.

Housner, G. W., Martel, R. R., & Alford, J. L. (1953). Spectrum analysis of strong-motion earthquakes. *Bulletin of the Seismological Society of America*, *43*(2), 97–119.

Huang, B., Günay, S., & Lu, W. (2021). Seismic assessment of freestanding ceramic vase with shaking table testing and performance-based earthquake engineering. *Journal of Earthquake Engineering*, *26*(15), 7956–7978.

International Code Council-Evaluation Service (ICC-ES). (2012). *AC156 Acceptance criteria for seismic certification by shake-table testing of nonstructural components*. Brea, California, USA. https://icc-es.org/acceptance-criteria/ac156/

IEEE Power & Energy Society (IEEE PES). (2013). *Standard for seismic qualification of equipment for nuclear power generating stations—IEEE 344*. IEEE Std344–2013 (Revision of IEEE Std 344–2004), 10016–5997.

IEEE Power & Energy Society (IEEE PES). (2018). *IEEE recommended practice for seismic design of substations*. IEEE 693–2018, 10016–5997.

Instituto Nacional de Normalización (INN). (2015). *NCh3357: Seismic design of non-structural components and systems* (in Spanish). https://ecommerce.inn.cl/nch3357201545948

International Electrotechnical Commission (IEC). (2013). *Environmental testing–Part 2–57: Tests–Test Ff: Vibration—Time-history and sine-beat method*. IEC/TC 104 - IEC_TC_104, Geneva, Switzerland, 1–43.

International Electrotechnical Commission (IEC). (2019). *Environmental testing–Part 3-3: Supporting documentation and guidance—Seismic test methods for equipment*. IEC 60068-3-3:2019, Geneva, Switzerland.

Javadinasab Hormozabad, S., Gutierrez Soto, M., & Adeli, H. (2021). Integrating structural control, health monitoring, and energy harvesting for smart cities. *Expert Systems*, *38*(8), e12845.

Jeon, B. G., Son, H. Y., Eem, S. H., Choi, I. K., & Ju, B. S. (2021). Dynamic characteristics of single door electrical cabinet under rocking: Source reconciliation of experimental and numerical findings. *Nuclear Engineering & Technology*, *53*(7), 2387–2395.

Johnson, T. P., & Dowell, R. K. (2017). Evaluation of the overstrength factor for nonstructural component anchorage into concrete via dynamic shaking table tests. *Journal of Building Engineering*, *11*, 205–215.

Jun, S. C., Lee, C. H., Bae, C. J., & Lee, K. J. (2020). Shake-table seismic performance evaluation of direct- and indirect-hung suspended ceiling systems. *Journal of Earthquake Engineering*, *26*(9), 4833–4851.

Kazantzi, A. K., Miranda, E., & Vamvatsikos, D. (2020). Strength-reduction factors for the design of light nonstructural elements in buildings. *Earthquake Engineering & Structural Dynamics*, *49*(13), 1329–1343.

Kim, H. J., & Shin, D. H. (2021). Shake table test program of cold-formed steel in-plane partition walls. *Structures*, *30*, 503–517.

Kramer, S. L. (1996). *Geotechnical earthquake engineering*. Prentice Hall.

Lee, S. M., & Jung, W. Y. (2020). Evaluation of anchorage performance of the switchboard cabinet under seismic loading condition. *Advances in Mechanical Engineering*, *12*(5), xx–xx.

Li, Y., Conte, J. P., & Barbato, M. (2016). Influence of time-varying frequency content in earthquake ground motions on seismic response of linear elastic systems. *Earthquake Engineering & Structural Dynamics*, *45*(8), 1271–1291.

Lin, F. R., Chai, J. F., Lai, Z. Y., Chou, P. F., & Wu, Y. C. (2014). Experimental study for seismic demands on in-cabinet equipment in NPPs. *Journal of the Chinese Institute of Engineers, Transactions of the Chinese Institute of Engineers, Series A*, *37*(7), 833–844.

Lu, Y., Mosqueda, G., Han, Q., & Zhao, Y. (2018). Shaking table tests examining seismic response of suspended ceilings attached to large-span spatial structures. *Journal of Structural Engineering*, *144*(9), 04018152.

Luise, M., & Vitetta, G. M. (2009). *Teoria dei segnali*. (McGraw-Hill, Ed.) (in Italian). Paolo Roncoroni.

Luo, Z., Xue, J., Zhou, T., Qi, L., & Zhao, X. (2021). Shaking table tests and seismic design suggestions for innovative suspended ceiling systems with detachable metal panels. *Engineering Structures*, *232*, 111830.

Maddaloni, G., Ryu, K. P., & Reinhorn, A. M. (2011). Simulation of floor response spectra in shake table experiments. *Earthquake Engineering & Structural Dynamics*, *40*(6), 591–604.

Magliulo, G., Pentangelo, V., Maddaloni, G., Capozzi, V., Petrone, C., Lopez, P., Talamonti, R., & Manfredi, G. (2012). Shake table tests for seismic assessment of suspended continuous ceilings. *Bulletin of Earthquake Engineering*, *10*(6), 1819–1832.

Magliulo, G., Petrone, C., Capozzi, V., Maddaloni, G., Lopez, P., & Manfredi, G. (2014). Seismic performance evaluation of plasterboard partitions via shake table tests. *Bulletin of Earthquake Engineering*, *12*(4), 1657–1677.

Merino, R. J., Perrone, D., & Filiatrault, A. (2020). Consistent floor response spectra for performance-based seismic design of nonstructural elements. *Earthquake Engineering & Structural Dynamics*, *49*(3), 261–284.

Ministero delle Infrastrutture e dei Trasporti (MIT). (2018). *Aggiornamento delle Norme tecniche per le costruzioni Decreto 17 gennaio 2018, 23*–35 (in Italian). https://www.gazzettaufficiale.it/

Ministero delle Infrastrutture e dei Trasporti (MIT). (2019). *Istruzioni per l'applicazione dell'«Aggiornamento delle Norme tecniche per le costruzioni» di cui al decreto ministeriale 17 gennaio 2018* (in Italian). https://www.gazzettaufficiale.it/

National Research Council of Canada (NRCC). (2015). *National Building Code of Canada* (Vol. 1). National Research Council of Canada (NRCC).

New Zealand Standard (NZS). (2009). *Seismic performance of engineering systems in buildings*. NZS, 4219:2009, 1–114.

Nguyen, D. -D., Thusa, B., Han, T. -S., & Lee, T. -H. (2020). Identifying significant earthquake intensity measures for evaluating seismic damage and fragility of nuclear power plant structures. *Nuclear Engineering & Technology*, *52*(1), 192–205.

NIST GCR 18-917-43. (2018). *Recommendations for improved seismic design and installation of nonstructural components*. Applied Technology Council.

Partner Data Centers & Networks. (2017). *Center for engineering strong motion data (CESMD)*. https://www.strongmotioncenter.org/

Perez-Ramirez, C. A., Amezquita-Sanchez, J. P., Adeli, H., Valtierra-Rodriguez, M., Camarena-Martinez, D., & Romero-Troncoso, R. J. (2016). New methodology for modal parameters identification of smart civil structures using ambient vibrations and synchrosqueezed wavelet transform. *Engineering Applications of Artificial Intelligence*, *48*, 1–12.

Perrone, D., Brunesi, E., Dacarro, F., Peloso, S., & Filiatrault, A. (2019). Seismic Performance of non-structural elements seismic assessment and qualification of non-structural elements in europe: A critical review. *SPONSE 2019 Conference*, Pavia, Italy.

Petrone, C., Magliulo, G., & Manfredi, G. (2014). Shake table tests for the seismic assessment of hollow brick internal partitions. *Engineering Structures*, *72*, 203–214.

Petrone, C., Magliulo, G., & Manfredi, G. (2015). Seismic demand on light acceleration-sensitive nonstructural components in European reinforced concrete buildings. *Earthquake Engineering & Structural Dynamics*, *44*(8), 1203–1217.

Petrone, C., Magliulo, G., & Manfredi, G. (2016). Floor response spectra in RC frame structures designed according to Eurocode 8. *Bulletin of Earthquake Engineering*, *14*(3), 747–767.

Petrone, C., Magliulo, G., & Manfredi, G. (2017). Shake table tests on standard and innovative temporary partition walls. *Earthquake Engineering & Structural Dynamics*, *46*(10), 1599–1624.

Porter, K. (2021). A beginner's guide to fragility, vulnerability, and risk. University of Colorado Boulder, 1–139. https://www.sparisk.com/pubs/Porter-beginnersguide.pdf

Prota, A., Zito, M., D'Angela, D., Toscano, G., Ceraldi, C., Fiorillo, A., & Magliulo, G. (2022). Preliminary results of shake table tests of a typical museum display case containing an art object. *Advances in Civil Engineering*, *2022*, 1–18.

Pürgstaller, A., Gallo, P. Q., Pampanin, S., & Bergmeister, K. (2020). Seismic demands on nonstructural components anchored to concrete accounting for structure-fastener-nonstructural interaction (SFNI). *Earthquake Engineering & Structural Dynamics*, *49*(6), 589–606.

Qi, L., Kurata, M., Ikeda, Y., Kunitomo, K., & Takaoka, M. (2021). Seismic evaluation of two-elevation ceiling system by shake table tests. *Earthquake Engineering & Structural Dynamics*, *50*(4), 1147–1166.

Rathje, E. M., Faraj, F., Russell, S., & Bray, J. D. (2004). Empirical relationships for frequency content parameters of earthquake ground motions. *Earthquake Spectra*, *20*(1), 119–144.

Rodriguez, D., Perrone, D., Filiatrault, A., & Brunesi, E. (2021). A probabilistic strong floor motion duration model for seismic performance assessment of non-structural building elements. *Earthquake Engineering & Structural Dynamics*, *50*(15), 4161–4179.

Salari, N., Konstantinidis, D., Mohsenzadeh, V., & Wiebe, L. (2022). Demands on acceleration-sensitive nonstructural components in special concentrically braced frame and special moment frame buildings. *Engineering Structures*, *260*, 114031.

Singh, S., & Gupta, A. (2021). Seismic response of electrical equipment subjected to high–frequency ground motions. *Nuclear Engineering & Design*, *374*, 111046.

Son, H., Park, S., Jeon, B. G., Jung, W. Y., Choi, J., & Ju, B. S. (2020). Seismic qualification of electrical cabinet using high-fidelity simulation under high frequency earthquakes. *Sustainability (Switzerland)*, *12*(19), 1–14.

Suárez, L. E., & Montejo, L. A. (2005). Generation of artificial earthquakes via the wavelet transform. *International Journal of Solids & Structures*, *42*(21-22), 5905–5919.

Taghavi, S., & Miranda, E. (2003). *Response assessment of nonstructural building elements*. PEER 2003/05.

Takhirov, S. M., Fenves, G. L., Fujisaki, E., & Clyde, D. (2004). *Ground motions for earthquake simulator qualification of electrical substation equipment*. PEER Report No. 2004/07, University of California, Berkeley.

Takhirov, S. M., Fujisaki, E., Power, B., Vancouver, A., Riley, W. M., & Low, B. (2017). *Development of time histories for IEEE693 Testing and Analysis*. PEER Report No. 2017/10 (including seismically isolated equipment), University of California, Berkeley.

Telcordia Technologies. (2006). *NEBS requirements: Physical protection*. Telcordia Technologies Generic Requirements, GR-63-CORE. 08854–41821.866.672.6997 (USA).

Tian, Y., Filiatrault, A., & Mosqueda, G. (2015). Seismic response of pressurized fire sprinkler piping systems I: Experimental study. *Journal of Earthquake Engineering*, *19*(4), 649–673.

Tran, T. T., Salman, K., & Kim, D. (2021). Distributed plasticity approach for nonlinear analysis of nuclear power plant equipment: Experimental and numerical studies. *Nuclear Engineering & Technology*, *33*(9), 3100–3111.

Trifunac, M. D. (1971). Zero baseline correction of strong-motion accelerograms. *Bulletin of the Seismological Society of America*, *61*(5), 1201–1211.

Trifunac, M. D., & Brady, A. G. (1975). A study on the duration of strong earthquake ground motion. *Bulletin of the Seismological Society of America*, *65*(3), 581–626.

Universitatea Tehnica de Constructii Bucuresti (UTCB). (2013). *Cod de proiectare seismică. partea i–prevederi de proiectare pentru clădiri*. P100-1. Bucharest, Romania (in Romanian).

US Nuclear Regulatory Commission (NRC). (2014). *Regulatory guide 1.60: Design response spectra for seismic design of nuclear power plants*. US Nuclear Regulatory Commission (NRC).

Wittich, C., & Hutchinson, T. (2014). Development of a rocking-period centered protocol for shake table testing of unattached stiff components. *Tenth U.S. National Conference on Earthquake Engineering*, Anchorage, AK.

Yazdani, A., & Takada, T. (2009). Wavelet-based generation of energy- and spectrum-compatible earthquake time histories. *Computer-Aided Civil & Infrastructure Engineering*, *24*(8), 623–630.

Zaghi, A. E., Maragakis, E. M., Itani, A., & Goodwin, E. (2012). Experimental and analytical studies of hospital piping assemblies subjected to seismic loading. *Earthquake Spectra*, *28*(1), 367–384.

Zhou, T., Wang, X., Liu, W., Zhang, Z., Ma, B., & Tan, W. (2021). Shaking table tests on seismic response of discontinuous suspended ceilings. *Journal of Building Engineering*, *43*, 102916.

Zhou, Z., & Adeli, H. (2003a). Wavelet energy spectrum for timefrequency localization of earthquake energy. *International Journal of Imaging Systems & Technology*, *13*(2), 133–140.

Zhou, Z., & Adeli, H. (2003b). Time-frequency signal analysis of earthquake records using Mexican hat wavelets. *Computer-Aided Civil & Infrastructure Engineering*, *18*(5), 379–389.

How to cite this article: Zito, M., D'Angela, D., Maddaloni, G., & Magliulo, G. (2022). A shake table protocol for seismic assessment and qualification of acceleration-sensitive nonstructural elements. *Computer-Aided Civil and Infrastructure Engineering*, 1–28. https://doi.org/10.1111/mice.12951

APPENDIX

The time history signals associated with NPSs #2 to #7 are depicted in Figure A1.

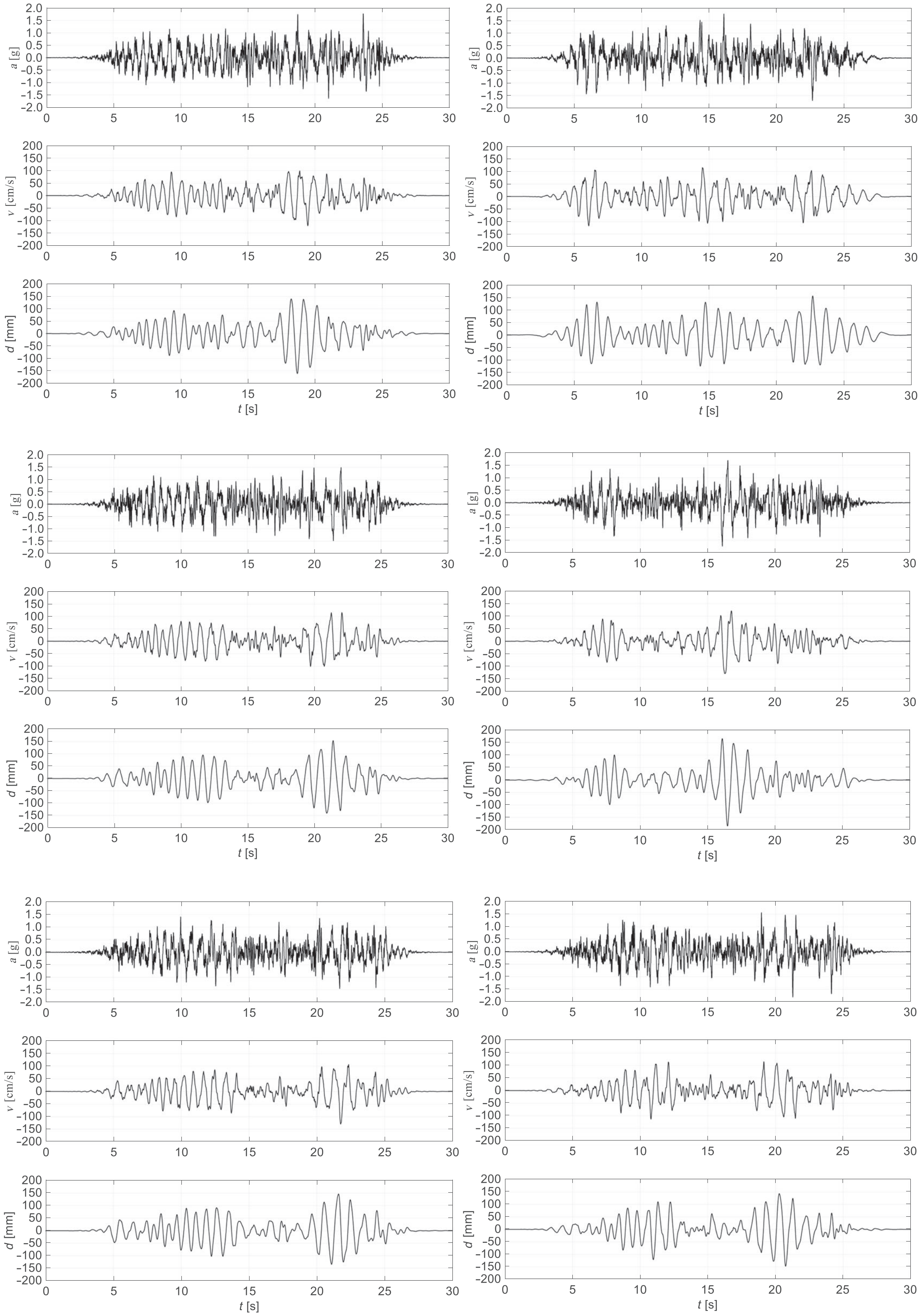

FIGURE A1 Acceleration, velocity, and displacement time histories related to NPS #2 to #7 (sorted from the left to the right and from the top to the bottom). NPSs are related to RRS having PGA equal to 0.40 g and assuming *z/H* equal to one