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SAFETY FACTORS FOR THE ESTIMATION OF SEISMIC CAPACITIES OF ACCELERATION-SENSITIVE NONSTRUCTURAL ELEMENTS VIA SHAKE TABLE TESTING

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Abstract: *The assessment of seismic capacity of acceleration-sensitive nonstructural elements (NEs) is typically carried out through shake table testing, and this is mandatory for seismic qualification purposes. However, current codes and protocols do not provide any information regarding the expected reliability of the protocols, as well as no guidance is provided regarding the estimation of design measures of capacity from the empirical ones, directly associated with shake table testing. This is an extremely critical issue since seismic capacity assessment of NEs is mandatory in several countries, including Italy. In order to cover this gap, the paper provides a novel methodology and the preliminary findings associated with the assessment of reliable seismic capacities of acceleration-sensitive NEs by means of shake table testing. In particular, reliability-targeted safety factors are developed to estimate the seismic capacities by enforcing a desired level of reliability. The factors are estimated through incremental analysis of inelastic single-degree-of-freedom (SDOF) NEs, subjected to multiple shake table protocol and real floor motion input sets. Shake table protocols represent the key tool/procedure for the estimation of the seismic capacity, whereas real earthquake floor motions recorded in instrumented buildings are associated with a relatively representative and realistic capacities. The factors can be applied to reduce the shake table protocol capacities, supplying a desired level of reliability. A wide range of case study NEs are modelled, covering a representative frequency range of interest, and multiple incremental damage states (DSs) are considered. The safety factors are provided through closed-form equations, as a function of shake table protocol, DSs, model properties, and elastic frequencies.*

Introduction

Seismic capacity of nonstructural elements (NEs) can be assessed through different methods, i.e., experimental testing (Bianchi et al., 2021; Mosqueda et al., 2009), analytical/numerical methods (D'Angela et al., 2021a; Petrone et al., 2018), data observation/analysis (Del Gaudio et al., 2020; Perrone et al., 2019), and mixed methods (American Society of Civil Engineers, 2017; Zito et al., 2022b). Shake table testing represents the state of the art method for the assessment of seismic response and capacity of NEs, especially for elements that are sensitive to inertial effects (Zito et al., 2022b). Shake table tests are typically carried out by implementing artificial loading histories, defined according to protocols defined by regulations and codes. AC156 (International Code Council Evaluation Service (ICC-ES), 2020) and FEMA 461 (Federal Emergency Management Agency (FEMA), 2007) are the most used protocols for generic NEs, whereas several protocols are available for specific components and equipment, e.g., IEEE 693 (Institute of Electrical and Electronics Engineers, 2006) for substation equipment, GR-63-CORE (Network Infrastructure and Operations, 2006) for telecommunication equipment, and IEC-60068 (International Electrotechnical Commission et al., 2013) for mechanical and electrotechnical products.

Even though the abovementioned protocols are often used to assess, qualify, and certify NEs, especially in the US (American Society of Civil Engineers, 2017; Office of Statewide Health Planning and Development (OSHPD), 2007; Zito et al., 2022a), their reliability has never been

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assessed, unless very peculiar equipment is considered, i.e., rocking-dominated unanchored elements (D'Angela et al., 2021b). Some recent studies (Perrone et al., 2019; Petrone et al., 2016; Zito et al., 2022a) suggested that, in some cases, the seismic capacity assessment via the application of the abovementioned protocol might be unsafe, e.g., by comparing the spectral ordinates of the protocols to the floor response spectra associated with building structures. Furthermore, the available protocols do not recommend or suggest the use of safety factors in order to account for uncertainty measures associated with the capacity estimations, as it is typically done in the framework of the semi probabilistic assessment approaches.

The abovementioned code and literature gap is potentially associated with major consequences in the field of the seismic assessment and qualification/certification of NEs. As a matter of fact, (1) it is not known the level of reliability of the current protocols, and they might be unsafe, as it was found regarding peculiar conditions, (2) shake table testing is often performed to assess critical components and equipment and/or that are housed within critical facilities, and (3) the abovementioned protocols are increasingly being applying in the industry to certify the industrial product, and this will result soon in thousands of "certified" commercialized products, all over the world. The present paper addresses this issue and provides the preliminary results of an extended research study. Numerical analysis of NEs is carried out to estimate reliability-targeted safety factors to be applied to the abovementioned use of the current protocols. In particular, the case study NEs consist in single-degree-of-freedom (SDOF) systems, modelled as inelastic lumped plasticity cantilever elements.

Methodology

Aim and methodology outline

The aim of the study is to supply both a methodology and quantitative measures for a more reliable assessment of seismic capacities of NEs by means of shake table testing. In particular, the implemented methodology is based on numerical modelling and analysis of NEs and reliability-based data processing. Incremental analyses are carried out modelling NEs as SDOF systems, having a hysteretic response; the loading histories consisted in several sets of real floor motions recorded within instrumented US buildings and seismic inputs derived according to reference shake table protocols. The seismic response of the modelled NEs is characterized in terms of damage response and fragility, and the reliability of the investigated shake table protocols is estimated. Finally, reliability-targeted safety factors are estimated by setting optimum target reliability indexed; these factors are to be applied to reduce the seismic capacities of NEs assessed through shake table testing, in order to achieve a desired level of reliability, considering the real floor motions as a reference.

Case study models and numerical modelling

The case study models consisted in a set of cantilever elements having lumped mass at the free end. The inelastic response was implemented in OpenSees (McKenna et al., 2000) through the lumped plasticity approach, assigning the peak-oriented Ibarra-Medina-Krawinkler (IMK) model (Ibarra et al., 2005; Ibarra and Krawinkler, 2005). These elements are potentially representative of acceleration-sensitive NEs since NEs are often meant to be elastic (and inelastic) SDOF systems in the literature (Akkar and Bommer, 2007; Merino et al., 2020; Petrone et al., 2015), as well as this is implicit in considering an assessment approach based on (elastic) response spectra (British Standards Institution and European Committee for Standardization, 2005; Merino et al., 2020; Ministero delle Infrastrutture e dei Trasporti, 2019; Petrone et al., 2016).

The cantilevers had S275 steel square hollow sections, and the modelling parameters were derived from the study by Lignos and Krawinkler (Lignos and Krawinkler, 2010). The modelling and formulation details are omitted for the sake of brevity, and the reader is referred to the abovementioned studies. The moment-rotation (M-θ) backbone of the models included (a) a linear elastic branch (from 0-0 to My-θy), (b) an post-yielding hardening branch (from My-θy to Mc-θc), (c) a softening branch (from Mc-θc to Mr-θr), and (d) a perfectly plastic indefinite branch associated with a residual strength (from Mr-θr).

A set of 12 models was defined, varying cross section dimensions (*b and t are the section dimension and thickness*), cantilever's height (h), and applied mass (m). The models had elastic frequency (f_a) ranging within 1 – 9 Hz, which is a representative range for acceleration-sensitive NEs (American Society of Civil Engineers, 2017; Riley et al., 2006). The investigated models are reported in Table 1, where the models are grouped within frequency ranges.

Model ID	range [-]	f_a [Hz]	f_a [Hz]	b [mm]	t [mm]	h [m]	m [t]
M1a	I	~1.0	1.02	70	3.0	4.50	0.10
M1b			1.03	60	3.0	2.50	0.35
M1c			1.13	50	2.5	3.00	0.08
M2a	II	~1.5	1.48	70	3.0	3.50	0.10
M2b			1.52	60	3.0	2.50	0.16
M2c			1.52	60	2.5	3.00	0.08
M3a	III	~3.0	2.97	70	3.0	2.20	0.10
M3b			3.04	60	3.0	2.50	0.04
M3c			3.06	90	3.0	3.00	0.08
M4a	IV	> ~3.0	5.86	70	3.0	1.40	0.10
M4b			7.34	80	4.0	1.50	0.10
M4c			9.02	70	3.0	1.05	0.10

Table 1. NE models investigated in the study; f_a is the elastic frequency, b and t are the section dimension and thickness, h is the cantilever's height, and m is the applied mass.

Loading histories and incremental analysis

Two orders of seismic inputs were considered as loading histories: (1) shake table protocol inputs (STPIs) and (2) real seismic input recorded within instrumented buildings in the US, namely floor motions (FMs). The following protocols were considered to derive the analysis STPI sets: AC156 (International Code Council Evaluation Service (ICC-ES), 2020), FEMA 461 (Federal Emergency Management Agency (FEMA), 2007), IEEE 693 (Institute of Electrical and Electronics Engineers, 2006), and Zito et al. (Zito et al., 2022a). Furthermore, a modified version of AC156 was also implemented, i.e., AC156w/o, as well as an exception of Zito et al. was also considered. AC156w/o protocol is identical to AC156 except for the application of a limitation to the upper bound value of the required response spectrum (RRS) recommended by the code; further details can be found in (D'Angela et al., 2021b; Perrone et al., 2019; Zito et al., 2022a). Seven inputs were developed according to AC156, AC156w/o, and Zito et al., and Zito et al. exception protocols; three inputs were derived considering FEMA 461 as a reference; ten inputs were considered for IEEE 693 protocol. Further details regarding the generation or derivation of the protocol-compliant signals can be found in (D'Angela et al., 2021b, 2021a; Takhirov et al., 2017; Wilcoski et al., 1997; Zito et al., 2022a, 2022b).

FM sets were defined by considering records provided by Center for Engineering Strong Motion Data (CESMD) database (CESMD, 2017), which includes both real ground and floor motion records. In particular, the building scenario is associated with reinforced concrete (RC) buildings designed/built in the US from 1923 to 1975. The selected FMs have the following characteristics: (a) PGA larger than 0.05 g, (b) associated with near and far field ground motions, (c) maximum acceleration amplification (almost always at the roof floor), (d) associated with low-, medium-, and high-rise buildings. Four sets of FM were considered: (1) FM, including a number of 18 records, having an equal number of near and far field records and low-, medium-, and high-rise buildings; (2) SFM, including a number of seven records from FM set, having PGA larger than 0.2 g; (3) FFFM, including a number of nine records from FM set, related to far field ground motions; (4) NFFM, including a number of nine records from FM sets, related to near field ground motions. FM set was derived from (D'Angela et al., 2021a) but a number of records used in this latter study were not considered for their mildness with regard to the NE models (i.e., FMs #4, #8, #11, #16, #20, and #24). Figure 1 depicts both median and 84th percentile acceleration spectra (Sa) related to with FM and STPI sets, where peak floor acceleration (PFA) is set equal to 1.0 g.

Incremental dynamic analysis (IDA) (Vamvatsikos and Cornell, 2002) was carried out considered the abovementioned models, assigning Rayleigh damping to the elastic elements (5% damping ratio) and including P-Δ effects. The analyses were performed by scaling the abovementioned seismic inputs, considering PFA as an intensity measure (IM), from 0.05 g up to failure of all models.

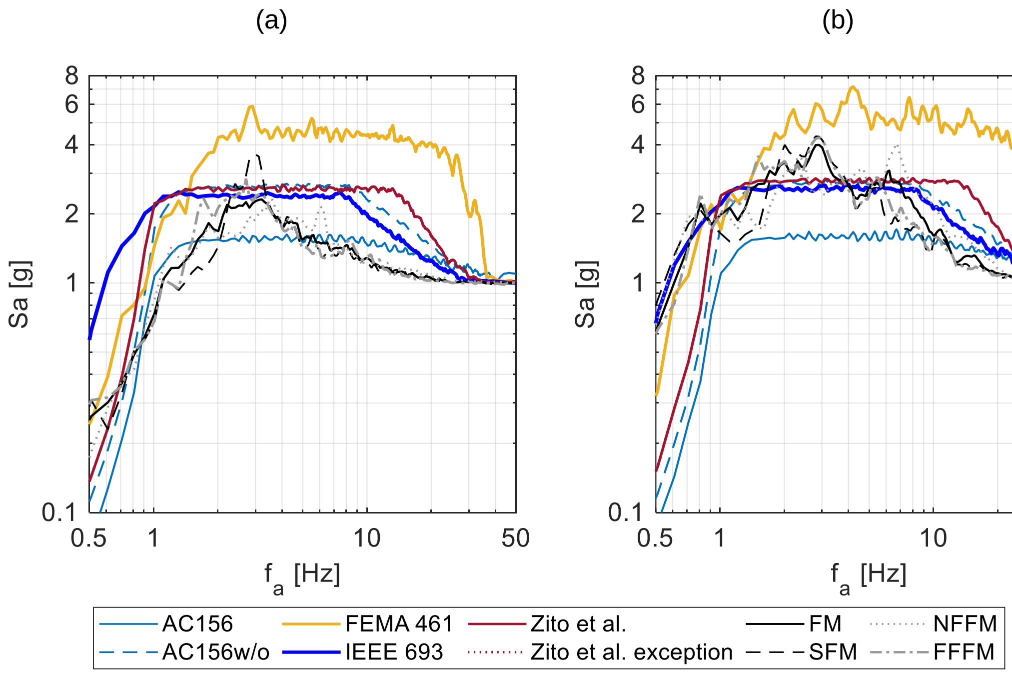

Figure 1. (a) Median and (b) 84th percentile acceleration spectra (Sa) related to with FM and STPI sets, setting PFA equal to 1.0 g.

Damage assessment

The damage assessment was performed by processing the results of the incremental analyses as described in the following. The SDOF mass displacement Δ was considered as an engineering demand parameter (EDP). Five damage states (DSs) were defined by thresholds of Δ: (DS1) halved yielding strength/displacement (Δ_{DS1}), (DS2) yielding (Δ_{DS2}), (DS3) capping strength (Δ_{DS3}), (DS4) post-capping 20% strength decrease (Δ_{DS4}), and (DS5) residual strength (onset of perfectly plastic response) (Δ_{DS5}); the Δ capacity thresholds associated with the investigated models were derived from the pushover curves including the P-Δ effects. Figure 2 shows the backbone of the modelled elements and the definition of the investigated DSs, where V is the shear force.

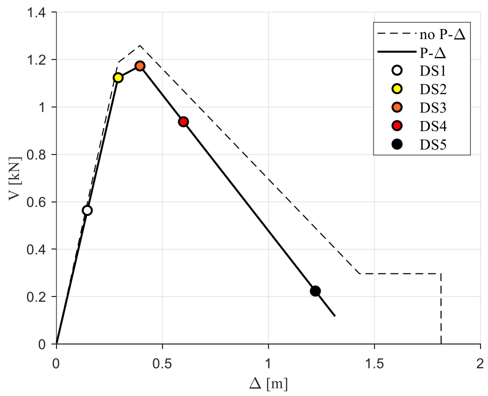

Figure 2. (a) Schematic backbone response and investigated DSs associated with the SDOF models.

The Δ capacities associated with the defined DSs (i.e., Δ_{DS}) are reported in Table 2. These DSs are representative of performance levels associated with NEs, i.e., DS1 can be associated with functioning/service conditions, DS2 with damage limitation conditions, DS3 with life safety conditions, and DS4/DS5 with collapse/failure conditions.

Model ID	Δ_{DS1} [m]	Δ_{DS2} [m]	Δ_{DS3} [m]	Δ_{DS4} [m]	Δ_{DS5} [m]
M1a	0.146	0.291	0.394	0.599	1.221
M1b	0.052	0.105	0.171	0.257	0.515
M1c	0.091	0.181	0.262	0.407	0.847
M2a	0.088	0.176	0.256	0.427	0.945
M2b	0.052	0.105	0.172	0.297	0.674
M2c	0.076	0.151	0.218	0.357	0.778
M3a	0.035	0.070	0.120	0.237	0.591
M3b	0.052	0.105	0.172	0.338	0.841
M3c	0.050	0.100	0.153	0.278	0.659
M4a	0.014	0.028	0.060	0.138	0.376
M4b	0.014	0.028	0.069	0.172	0.486
M4c	0.008	0.016	0.040	0.100	0.281

Table 2. Displacement capacities (Δ_{DS}) associated with the investigated models.

Reliability evaluation and reliability-targeted safety factor estimation

The reliability evaluation method was derived from a past study by the authors (D'Angela et al., 2021c). The concept beyond the methodology is that the reliability of a protocol, or more generally, of an engineering tool that estimates capacity measures, can be quantitatively expressed by the reliability index, estimated considering the statistical distribution of a margin (Z) identified between a nominal capacity measure, estimated considering the protocol, and a realistic capacity measure, associated with a representative scenario of capacities. In other words, traditional capacity (R) and demand (S) are referred to FM-based responses (i.e., realistic capacities) and protocol-based responses (i.e., nominal estimation capacities). The reliability index (β) was estimated according to a second-level first-order reliability method (FORM) (Schultz et al., 2010), and Equation (1) reports the formulation, where $x_{m,R}$, $x_{m,S}$, σ_R, and σ_S are median capacity measure, median demand measure, logarithmic standard deviation capacity measure, and logarithmic standard deviation demand measure. These measures were estimated according to the Porter method (Porter et al., 2007). Further details are omitted for the sake of brevity, and the reader is referred to the abovementioned literature studies.

$$\beta = \frac{\ln\left(\frac{x_{m,R}}{x_{m,S}}\right)}{(\sigma_S{}^2+\sigma_R{}^2)^{0.5}} \tag{1}$$

The (protocol) failure probability (P_f) can be defined as the probability that the protocol yields capacities that exceed the realistic capacities, or, technically, the probability that the margin (Z) is positive. P_f can be assessed as it is reported in Equation (2), where Φ is the cumulative standard normal distribution.

$$P_f = \Phi(-\beta) \tag{2}$$

If a target reliability index ($\bar{\beta}$) can be determined, Equation (1) can be used to derive the statistical distribution of the nominal capacity measures (i.e., protocol-based) that yield capacities associated with $\bar{\beta}$. Assuming that a safety coefficient k is applied to the nominal capacities measures, to estimate design capacities, and noting that the lognormal standard deviation of the resulting set of capacities (i.e., design capacities) is equal to the one associated with the set of nominal measures, k can be evaluated as reported in Equation (3).

$$k = \exp\left(\bar{\beta}\,(\sigma_S{}^2 + \sigma_R{}^2)^{0.5}\right)\left(\frac{x_{m,S}}{x_{m,R}}\right) \tag{3}$$

In this study, k was assessed for all protocols, assuming that $\bar{\beta}$ is equal to one, associated with target P_f equal to 16%. This condition represents an reasonable target reliability (D'Angela et al., 2021c), given that the assessment is aimed at NEs, that that, for structural elements and collapse, target failure probabilities are often associated with 10% (American Society of Civil Engineers, 2017).

Results, discussion, and concluding remarks

The reliability-targeted safety factors (k) are depicted in Figure 3, as a function of the elastic frequency (f_a) of the models, for all investigated DSs; in particular, the k values are associated with the largest envelop over the four investigated FM sets, i.e., FM, SFM, FFFM, and NFFM, in order to identify conservative estimations. In addition to the estimated k values, depicted by markers, fitting lines were identified. These lines are III-degree polynomial equations associated with coefficients of determinations R^2 larger than 0.6, with a median R^2 equal to 0.87. These criteria are meant to be first tentative estimations, buy they can be referred to for expeditious but relatively reliable capacity estimations of design capacities, from nominal capacities related to the application of the relevant shake table protocols. In case k are extremely large, the use of the related protocols is discouraged by the authors, who refer to the protocols associated with relatively reduced k values, e.g., lower than 2.5 – 3.

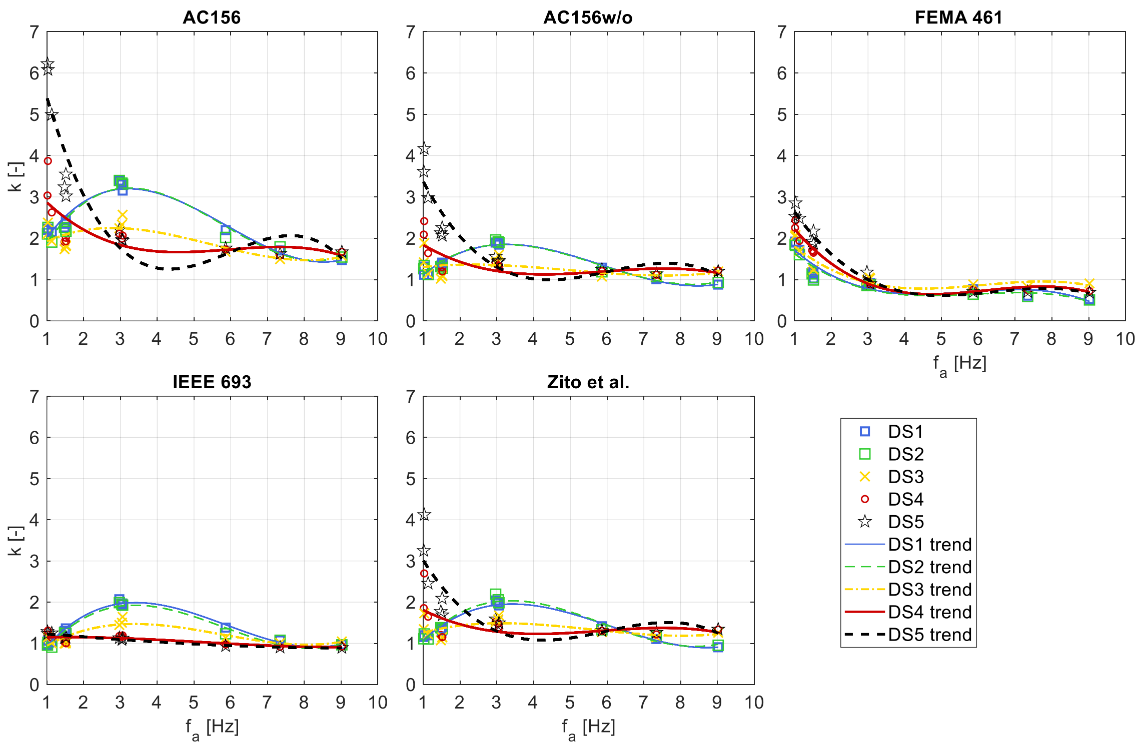

Figure 3. (a) Reliability-target safety factor (k) as a function of the NE elastic frequency (f_a) and fitting curves, evaluated for investigated DSs.

The key results are summarized in the following.

- IEEE 693 protocol is overall associated with the lowest k values, for all DSs, being lower than 2, whereas AC156 produces k values that can be extremely large, e.g., larger than 3. The other protocols provide k values that are within a reasonably range of values, e.g., within 1 – 2.5, except for very few cases of lager k value, which are associated with the lowest frequencies.<

- The influence of DS on k values strongly depends on the protocol, but DS1/DS2 and DS3 to DS5 are associated with similar k values. For all protocols but IEEE 693, DS5 is associated with the largest k values, which corresponds to the lowest frequencies.

- Despite AC156 protocol was found to be significantly more unreliable than other protocols for all DSs and models, the AC156 k values are not significantly larger than the other protocols, especially for frequencies larger than 1 Hz and for FEMA 461 results. This can be explained by recalling that k increases as the dispersion associated with the protocol grows, as it can be seen by analysing Equations (1) and (3); as a matter of fact, AC156 protocol was found to be associated with a relatively low dispersion, if compared with other protocols, with particular regard to FEMA 461.

- The provided fitting curves could be considered as a reference for determining, in an expeditious but relatively reliable manner, the safety factors k to be applied to estimate reliable capacities of acceleration-sensitive nonstructural elements that can be modelled as inelastic SDOF systems.
- The fitting curves are based on the assumption of a unitary target reliability index, but the methodology can be easily applied considering lower or higher levels of target reliability.

The study is limited to the case studies, i.e., inelastic SDOF systems analysed considering IMK model. Further studies should be carried out considering different NE scenarios and modelling approaches. An experimental validation should strengthen and confirm the findings, in order to define ready-to-use safety factors, possibly generalizable to a wider range of acceleration-sensitive NEs.

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