Seismic demands on nonstructural components anchored to concrete accounting for structure-fastener-nonstructural interaction (SFNI)

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Summary
This paper investigates the influence of the hysteretic shear behavior of post-installed anchors in concrete on the seismic response of nonstructural components (NSCs) using numerical methods. The purpose of the investigation is to evaluate current design requirements for NSC and their anchorage. Current design guidelines and simplified methods, such as floor response spectra (FRS), typically approach the dynamics of the structure-fastener-NSC (SFN) system using simplified empirical formulae. These formulations decouple the structure from the NSC and neglect the behavior of the anchor connection, with the assumption of full rigidity. There is a lack of knowledge on the complex interaction between a host structure, the fastening system, and the NSC, herein referred to as structure-fastener-nonstructural interaction (SFNI). More specifically, it is important to investigate whether and how the actual hysteresis shear behavior that takes place in the anchorage could alter the seismic response of the SFN system and its components. Herein, the results of extensive nonlinear dynamic analyses (NLDA) with different models for the anchorage force-displacement relationship are presented and compared with those obtained with FRS procedures and current code provisions. The anchor models include (a) linear-elastic, (b) bilinear, and (c) a recently developed hysteresis rule. The results of the NLDA showed that the first two approaches are not able to reflect the behavior of an anchor loaded in dynamic shear. Moreover, when using the more refined hysteresis model, it appears that current code provisions might underestimate the component and anchor shear amplification factors for rigid NSC fixed to the host structure through anchors.

KEYWORDS
anchorage shear hysteresis, macro modeling, nonstructural components (NSCs), postinstalled fasteners in concrete, structure-fastener-nonstructural interaction (SFNI)
INTRODUCTION

Postinstalled anchors are commonly used in concrete construction for connecting structural components to each other and nonstructural components (NSCs) to their host structure. NSCs, often termed secondary systems, comprise elements attached to the floors, roof, and walls of a building and do not contribute to the load-bearing capacity of the primary structural system. According to Villaverde, NSCs can be broadly classified into (a) architectural components, (b) mechanical and electrical equipment, and (c) buildings contents. They constitute about 60% to 70% of the total construction costs in typical buildings and account for over 78% of the future estimated national annual earthquake loss in the United States.

The response of NSCs subjected to seismic actions strongly depends on the hysteresis behavior of the anchor connection. Compared with static loads, earthquake-induced actions result in more complex demands on the anchor including cyclic loading (tension or shear dominated, or a combination of both), cyclic cracking of concrete, high loading rates, and large crack widths (Eligehausen et al. 2006; Mahrenholtz 2013).

In the European Union, Qualification Guidelines issued by the European Organisation for Technical Approvals (EOTA) define the requirements for issuance of a European Technical Approval for a specific concrete anchor type (EOTA 2013). Only anchors with a “Seismic Approval” are allowed for seismic applications. The design guidelines presented in CEN/TS 1992-4-1, to be implemented in Eurocode 2 (EN 1992-4) in the future, define the design requirements and procedure.

Current code provisions for the seismic design of NSCs and their fastening system (e.g., EC8, ASCE/SEI 7-10, and CEN/TS 1992-4-1) rely upon either empirical formulae or simplified methods, such as the floor response spectra (FRS). In the second case, (elastic) floor spectra are built from the analysis of a structure (without NSCs) and are used to calculate the required design force for a given NSC. In the calculations, the anchor is typically assumed to be rigid, such that the NSC is decoupled from the host structure, ignoring the inelastic behavior of the anchor connection.

Over the past two decades, the primary focus of research into anchor behavior has been to improve the realism of testing albeit focused at the local, as opposed to system, level. In contrast, there is limited knowledge about how the actual hysteretic behavior that takes place in the anchor connection could affect the expected seismic demands on the NSC and on the anchors within a system level.

The research presented in this paper investigates the dynamics of the structure-fastener-nonstructural (SFN) system comprising (a) a NSC, (b) its host structure, and (c) the anchorage between them (see Figure 1), when subjected to seismic actions. The study is focused on, and limited to, single anchors resisting shear actions, whose hysteresis behavior is

FIGURE 1 Structure-fastener-nonstructural component (NSC) (SFN) system subjected to earthquake actions (figure adapted from)
characterized by pronounced pinching, stiffness degradation, and low energy dissipation capability (Eligehausen et al. 2006; Mahrenholtz 2013; Quintana Gallo 2018, 2019).6,12-14

Different approaches for modeling the anchor connection are used to include the interaction within the SFN system, referred herein as to SFN interaction (SFNI).4 The seismic demands experienced by the NSC and the anchor, within the particular case described before, are examined via nonlinear dynamic analyses (NLDA) of SFN systems with different degrees of freedom (DOF), including comprehensive sensitivity studies. Most of the numerical work includes the specific hysteresis rule for concrete anchors developed by Quintana Gallo et al.14 The demands on the NSC, as predicted with the NLDA, are compared with those obtained with simpler methods and code requirements, whose key features are reviewed in the following sections.

2 | RESEARCH SIGNIFICANCE

In the seismic design of anchors attaching NSC to structures, and the design of NSC themselves, it is assumed that the anchor connection is fully rigid, such that the NSC is decoupled from the host structure and the hysteresis neglected. Rather limited information is available on the effect of including a realistic model of the anchorage hysteretic behavior, to study an SFN system as a whole. To address this, a recently developed hysteresis model for single anchors subjected to shear actions14 was used to numerically investigate the response of NSC within an SFN system, using NLDA. This investigation demonstrates that the anticipated demands that NSC can face during earthquakes may be underestimated by the current code provisions considered.

3 | CODE PROVISIONS FOR THE SEISMIC DESIGN OF NSCS AND FASTENERS

In the following, the requirements of a group of selected international code provisions for the seismic design of NSCs and fasteners are summarized and compared.

3.1 | ASCE/SEI-711

ASCE/SEI-711 contains a relatively limited amount of requirements for the seismic design of NSCs, despite other codes, such as IBC15 adopt them. According to ASCE/SEI-7, the seismic demands on NSC, and hence on the anchorage, shall be determined with Equation (1):

$$0.3 S_{DS} I_p W_p \leq F_p = \frac{0.4 a_p S_{DS} W_p}{R_p} \left(1 + 2 \frac{z}{H}\right) \leq 1.6 S_{DS} I_p W_p,$$

where $S_{DS}$ is the short period spectral acceleration for a given soil type and seismic zone, $a_p$ is the component amplification factor, varying from 1 (rigid) to 2.5 (flexible), $(1 + 2z/H)$ is an amplification factor related to the height (z) where the NSC is located within the total height of the structure (H) (varies from 1 to 3 at the base and roof levels, respectively), $I_p$ is the component importance factor (1.0 and 1.5 for normal and critical components, respectively), $R_p$ is the component modification factor, which depends on the overstrength and inelastic deformation capability of the NSC, and its supports and attachments (1.5, 2.5, and 3.5 for low, limited and high deformability elements, respectively), $W_p$ is the NSC operating weight, and $0.4 S_{DS}$ represents the seismic coefficient at the base of the structure.

In this formulation, the seismic force ($F_p$) is independent of the dynamic characteristics of the host structure and of the NSC itself (except, obviously, for $W_p$). However, it does distinguish between flexible and rigid components, resulting in amplification factors 2.5 times larger for flexible than for rigid NSC (see Figure 2). The amplitude of the seismic action varies linearly with the height of the building. The maximum amplification factors are limited by an upper bound of four times the PGA, as shown in Figure 2 for flexible NSCs, referred to as ASCEflex.
3.2 Eurocode 8\textsuperscript{10}

Eurocode 8 (EC8)\textsuperscript{10} contains simplified expressions for the equivalent lateral force ($F_a$) used for designing NSCs. The expression generally used for NSC that do not contain hazardous materials is given in Equation (2).

$$F_a = \frac{S_a W_{NSC} \gamma_a}{q_a},$$  \hspace{1cm} (2)

where $F_a$ is the design lateral seismic force for the NSC, acting at its center of mass (CM) and in the most unfavorable direction; $S_a$ is the seismic coefficient for the NSC; $\gamma_a$ is an importance factor (1.0 and 1.5 for normal NSCs and NSCs which must remain operational after an earthquake or contain hazardous materials, respectively); and $q_a$ is the “behavior factor” of the NSC (see\textsuperscript{10}) (varying from 1.0 to 2.0).

The parameter $S_a$ in Equation (2) is given by Equation (3):

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

where $\alpha$ is the design peak ground acceleration for soil type A, $q_0$, divided by $g$; $S$ is a factor that depends on the soil type; $(1 + \frac{z}{H})$ is an amplification factor related to the height ($z$) where the NSC is located within the total height of the structure ($H$) (varies from 1 to 2 at the base and roof levels, respectively); $T_a$ is the fundamental period of vibration of the NSC; and $T_1$ is the fundamental period of vibration of the building in the direction of analysis.

*FIGURE 2* Normalized seismic amplification factors (PCAAF) code-requirements: (A) at ground level; (B) at roof level; (C) PCAA as a function of $z/H$. For EC8: $T_1 = \alpha = 1$; for ASCE 7: $W_{NSC} = R_p = I_p = 0.4S_a = 1.0$; for CEN/TS: $A_a = 3.0$ (flex.) and 1.5 (rigid) [Colour figure can be viewed at wileyonlinelibrary.com]
3.3 | **CEN/TS 1992-4⁸ and prEN 1992-4⁹**

The prescriptions of the standards CEN/TS 1992-4 and prEN 1992-4⁹ are identical to each other. They define the design forces for anchor connections with the simplified version of the EC8 formula, presented in Equation (4). This equation provides amplification factors \( A_a \) ranging from 1.5 to 3.0, if \( T_a \) and \( T_1 \) are unknown.

\[
S_a = \alpha S[(1 + z/H)A_a - 0.5] > \alpha S. \tag{4}
\]

Additionally, the forces associated to the vertical component of the ground motion are calculated with Equation (5).

\[
F_{va} = (S_{va}W_{NSC}/q_a), \tag{5}
\]

where \( S_{va} \) is the vertical seismic coefficient, \( S_{va} = \alpha v A_a \), with \( \alpha v \) the vertical design peak ground acceleration for soil type A divided by \( g \); and \( A_a \) is the “response modification factor,” defined in table E.1 of EC8.

3.4 | **Comparison**

Figure 2 illustrates the peak component acceleration amplification factors (PCAAF), obtained with the code provisions previously reviewed, for different design situations (ie, values of the parameters involved in the problem). Figure 2A,B presents the amplification factor of the NSC acceleration (PCAAF) for the ground and roof levels, respectively, as a function of \( T_a/T_1 \). Figure 2C, on the other hand, presents PCAAF for values of relative floor height \( z/H \).

As shown in Figure 2A,B, according to EC8, the seismic coefficients have a maximum when the two natural periods (structural and nonstructural) are equal, the case of resonance \( (T_a/T_1 = 1) \). All the other procedures define constant coefficients; ie, they are independent of \( T_a/T_1 \). Thus, EC8 provides a more refined formula than ASCE and CEN/TS for the case when \( T_a \) and \( T_1 \) are known.

The graphs presented in Figure 2 include PCAAF for flexible and rigid NSCs per ASCE (ASCEflex and ASCErig) and CEN/TS (CEN/TSflex and CEN/TSrig). As can be observed in Figure 2C, CEN/TS prescribes similar minimum (rigid) and maximum (flexible) values for PCAAF over the height of the structure, compared with EC8.

As depicted in Figure 2A, the resonance case of EC8 yields similar values of \( S_a \) compared with ASCEflex, while ASCE and CEN/TS define identical coefficients. The highest values of \( S_a \) at ground level \( (S_a = 5.0) \) are given by CEN/TS for flexible NSCs, when the factor \( \alpha_{gap} \) is included. The factor \( \alpha_{gap} \) accounts for the impact to the anchor resulting from the gap between the anchor and the anchor hole in the component, assuming that this gap has not been filled or bridged. It varies from 1.0 to 2.0. The factor \( \alpha_{gap} \) is typically used in the calculation of the resistance of the anchorage, but not of the NSC, despite both elements being affected by such a phenomenon.

According to ASCE, the value of the design seismic force for flexible NSCs is limited by the upper limit \( 1.6 S_{ds}I_p W_{NSC} \), which corresponds to four times the PGA (see Figure 2C). They are, therefore, smaller than the maximum corresponding values according to EC8 and CEN/TSflex, whose maximum \( F_p \) corresponds to an amplification of the PGA equal to 5.5. The largest values of \( S_a \) (11.0) are required by CEN/TS for flexible NSC, due to the inclusion of \( \alpha_{gap} \). It is worth noting that the impact (amplification) effects due to the anchor connection clearance are only considered in codes defining the anchor design forces (CEN/TS), but they are not included in the calculation of the seismic demand for the NSCs, which can thus be underprotected. This shows an apparent inconsistency between the design actions for NSCs and anchors according to current code provisions. Moreover, the reviewed procedures, in general, do not consider type and behavior of individual anchors, but assume that the connection is fully rigid, which is not representative of the anchorage behavior during dynamic shear¹²-¹⁴

In summary, current design codes apply empirical formulae that generate approximate FRS directly from a specified ground response spectrum and other factors. Code formulations suggest that design forces are larger for flexible components and flexible attachments, larger for items anchored higher in the building and smaller for items with high deformability or ductility capacities. Moreover, to enhance the survivability of these attachments, amplification of forces is suggested for items that contain hazardous materials, which is needed for life safety functions or for continued operations of an essential facility.
4 | NUMERICAL MODELING OF THE ANCHORAGE

This section reviews the experimental monotonic and cyclic-hysteretic behavior of single anchors subjected to shear loading and discusses alternative numerical models for its representation, including their required calibration process.

4.1 | Experimental behavior of single anchors under shear loading

Figure 3A presents the dimensions of the wedge-type expansion anchor of nominal diameter 12 mm (M12) under investigation, and Figure 3B presents the experimental force-displacement results when tested against monotonic and cyclic quasi-static shear loading.

As shown in Figure 3B, the monotonic behavior can be well approximated by a bilinear curve, defined by an initial linear segment with stiffness \( k_0 \), an equivalent yielding force \( F_y \), and a postyielding stiffness \( r k_0 \), where \( r \) is the postyielding stiffness factor.

The load-displacement behavior under reversed shear loading differs from monotonic loading response as shown in Figure 3B and as noted by others (Mahrenholtz, P. 2013). In the cyclic-loading case, the hysteretic behavior is characterized by pronounced pinching and stiffness degradation, due to the deterioration of the surrounding concrete when loaded by the anchor rod. In addition, such cycles present a slip region close to the origin, due to the existence of a clearance between the baseplate fixture and the anchor rod, as well as between the anchor and the surrounding concrete. This slip region also presents an enlargement with the increasing-amplitude of the loading cycles. A comprehensive description of this behavior can be found in Vintzéleou and Eligehausen,16 Mahrenholtz,12 and Quintana Gallo et al..14

In the following, the results of the three modeling approaches for implementing NLDA of structure-anchor-NSC interaction are presented. The three models vary in the degree of similitude as applied to the anchor load-displacement response.

4.2 | Modeling of the anchor connection

To model the reversed shear behavior of the anchor numerically, different approaches can be used. The first and most simplistic one is to model the anchor connection with a linear-elastic spring with stiffness \( k_{eff} \), which represents the secant or effective stiffness associated to the monotonic bilinear idealization, up to the mean failure resistance, corresponding in this case to 15 mm of lateral displacement (see Figure 3).

A second approach is to use a bilinear hysteresis model, whose backbone curve is defined by the monotonic curve presented in Figure 3. However, in this case, the unloading and reloading stiffness do not necessarily represent the experimental behavior of the anchorage accurately.

A third approach, hereafter referred to as FR or fastener rule, is to implement a mode defined load-displacement model that includes pinching and degradation; see, eg, Quintana Gallo et al..14 These three models, illustrated in Figure 4, are considered in this investigation.

FIGURE 3 Expansion anchor M12 (A) anchor dimensions, embedded length and fixture geometry (all dimensions in mm) (from14); (B) experimental quasi-static monotonic and cyclic force-displacement curves under shear loading
4.3 Calibration of the anchor models

The linear and bilinear models were calibrated with the results of the monotonic curves presented in Figure 3. In the latter case, only the envelope or backbone properties of the model are calibrated, whereas the hysteretic behavior cannot be calibrated. The FR model, on the other hand, was calibrated, in its symmetrical form, with the results of the cyclic loops shown in Figure 3. A description of such primary curve of the FR model is presented in Figure 5.

As depicted in Figure 5, the symmetric version of the model is determined by the following: (a) an initial elastic stiffness \( k_0 \); (b) an initial gap length \( d_{g0} \); (c) a loading stiffness \( k_l \); (d) a primary unloading stiffness \( k_{u1} \); (e) a secondary unloading stiffness \( k_{u2} \); (f) a force decrease, \( \Delta F \), which determines the end of the primary unloading path; and (g) the slip region stiffness \( k_s = \gamma k_0 \). The parameters \( k_0 \), \( d_{g0} \), and \( \Delta F \) are fixed numbers, whereas the stiffness are defined in Equations (6) to (8). In addition, the length of the gap or slip region is varied in time according to Equation (9).

\[
\begin{align*}
    k_l &= \beta_0 k_0 \left( \frac{d_m}{d_{g0}} \right)^{\alpha_1}, \\
    k_{u1} &= \beta_1 k_0 \left( \frac{d_m}{d_{g0}} \right)^{\alpha_1}, \\
    k_{u2} &= \beta_2 k_0 \left( \frac{d_m}{d_{g0}} \right)^{\alpha_2},
\end{align*}
\]

Figure 5 Fastener rule (FR) model, symmetrical form (adapted from\textsuperscript{14})
From these equations, it is noted that in addition to $k_0$, $d_{g0}$, $\Delta F$, and $\gamma$, the parameters $\beta_0$, $\beta_1$, $\beta_2$, $\beta_3$, and $\alpha_0$, $\alpha_1$, $\alpha_2$, $\alpha_3$ need to be determined for calibrating the model, summing up a total of 12 parameters. Figure 6 presents the numerical approximation of the experimental hysteresis loops of the anchor M12 (see Figure 3), using the FR model implemented with the parameters presented in Table 1.

The numerical force and displacement results are presented in the hysteresis form in Figure 6A, and in parametric in Figure 6B, and are overlapped with the experimental counterparts for comparison. In addition, Figure 6B shows the equivalent damping calculated for each cycle. For further details on the calibration process, see Pürgstaller.4

5 | SFN SYSTEMS DEFINITION AND INPUT MOTIONS FOR NLDA

5.1 | Systems definition

As mentioned before, code-formulae and simplified methods, such as FRS, decouple NSCs from the structure by neglecting the behavior of the anchorage and assuming a fully rigid connection between them. Consequently, the interaction between the structure, the fastener, and the NSC (SFNI) is not considered.

Figure 6 presents an illustration for the different systems analyzed in this contribution. Figure 7A shows the simplest case of an SFN system: a single-DOF (SDOF) structure supporting an NSC anchored to it, ie, effectively a two-DOF (2DOF) system. Figure 7B presents an idealization of such a system. In Figure 7, the springs represent the restoring capability provided by the structure ($k_{Str}$) and the fastener ($k_{Fast}$). The dashpot, on the other hand, represents the viscous damping associated with the system, calculated as that corresponding to the structure alone. Figure 7C shows a two degree of freedom structure with one NSC anchored to each floor. The resulting is the simplest case of a multi-degree of freedom SFN system. This model can be extended to multistory structures.

Figure 7D presents an SDOF system including the NSC only. The input, in this case, is not necessarily the ground motion, but can also be the motion of the floors of the structure subjected to a ground motion. The second case is the approach followed by code prescriptions for estimating the demands upon NSC, assuming a rigid connection between

| $k_0$, kN/mm | $d_{g0}$, mm | $\Delta F$ | $\gamma$ | $\beta_0$ | $\beta_1$ | $\beta_2$ | $\beta_3$ | $\alpha_0$ | $\alpha_1$ | $\alpha_2$ | $\alpha_3$ |
|--------------|--------------|------------|---------|--------|--------|--------|--------|--------|--------|--------|--------|
| 0.35         | 2.0          | 0.8        | 0.25    | 20     | 26.7   | 9.3    | 0.17   | 0.22   | 0.62   | 0.48   | 1.8    |

FIGURE 6  Calibration of FR model with quasi-static cyclic experiments of the anchor M12 [Colour figure can be viewed at wileyonlinelibrary.com]
the NSC and the host structure (i.e., $k_{\text{Fast}} \to \infty$). This research, on the other hand, includes the behavior of the anchor for calculating the response of the SDOF system.

Herein, a numerical study on SFN systems with (a) 2DOF systems with a SDOF structure and a NSC; (b) SDOF systems with the NSC subjected to ground and floor accelerations obtained with NLDA; and (c) a multiple-DOF (MDOF) system comprising a five-story frame structure with one NSC connected to each floor are presented.

### 5.2 Input ground motions

An ensemble of 15 ground motions representative of far-field earthquakes was considered. Table 2 presents the characteristics of these records. Following the Eurocode 8 recommendations, each of the records was scaled in amplitude such that (a) the square of the error between the single 5% damped response acceleration spectrum and the target design spectrum is minimized, (b) the average spectrum of the scaled records is always larger than 90% of the design spectrum within the period range of 0.15 to 2.25 seconds, and (c) the value of the average spectrum at period equal zero (PGA) is larger than the corresponding value.

For the construction of the design spectrum, besides a 5% critical damping ratio, a soil type C was assumed. Additionally, in Pürgstaller, the individual records were scaled as described before, to three different ground acceleration target spectra: 0.15g, 0.3g, and 0.5g. Figure 8 presents the acceleration response spectra of the selected records scaled to PGA = 0.15g, and the target design spectrum per EC8.

### 6 RESULTS OF NLDA AND COMPARISON WITH CODE REQUIREMENTS

#### 6.1 2DOF SFN systems

NLDA were conducted to comprehensively investigate the effects of the approaches used for modeling the anchor on the response of the NSCs, particularly on the maximum NSC acceleration. To quantify this, the component acceleration amplification factor ($a_p$), defined as the ratio between the peak acceleration of the NSC (PCA) and the peak floor acceleration (PFA), was considered.
## Table 2  Ground motions characteristics

| Year | Country | Earthquake                | $M_w$ | Station                        | $R_{jb}$, km | $R_{rup}$, km | PGA, g | $D_{lb}$, s | $D_{rb}$, s | $D_{lb}$, s | $A_I$, m/s | Record ID |
|------|---------|---------------------------|-------|--------------------------------|--------------|---------------|--------|-------------|-------------|-------------|-------------|-----------|
| 1992 | US      | Cape Mendocino            | 7.0   | Eureka – Myrte & West         | 40.2         | 42.0          | 0.15   | 0.18        | 20.8         | 19.8        | 7.6         | 8.4       | 44.0 44.0 | FF01 FF02 |
| 1992 | US      | Landers                   | 7.3   | Morongo Valley                | 17.3         | 17.3          | 0.19   | 0.14        | 31.0         | 32.1        | 37.5        | 35.6      | 70.0 70.0 | FF03 FF04 |
| 1992 | US      | Landers                   | 7.3   | North Palm Springs            | 26.8         | 26.8          | 0.14   | 0.13        | 36.3         | 37.0        | 38.9        | 34.7      | 70.0 70.0 | FF05 FF06 |
| 1992 | US      | Landers                   | 7.3   | Palm Springs Airport          | 36.1         | 36.1          | 0.08   | 0.09        | 39.4         | 35.8        | 28.1        | 32.8      | 60.0 60.0 | FF07 FF08 |
| 1999 | US      | Hector Mine               | 7.1   | Amboy                         | 41.8         | 43.0          | 0.18   | 0.15        | 27.5         | 25.2        | 25.8        | 19.9      | 60.0 60.0 | FF09 FF10 |
| 1976 | Italy   | Friuli (aftershock)       | 6.0   | ST33                          | 9           | -             | 0.11   | 0.10        | 8.1          | 11.2        | 5.0         | 6.5       | 26.4 26.4 | FF11 FF12 |
| 1997 | Italy   | Umbria-Marche             | 6.0   | ST223                         | 2.2          | -             | 0.17   | 0.11        | 25.1         | 28.7        | 10.4        | 7.2       | 55.3 55.3 | FF13 FF14 |
| 2010 | NZ      | Darfield                  | 7.1   | REHS                          | 37           | -             | 0.24   | 23.3        | 33.2         | 50.0        | 1.45        |           | FF15      |

*a*Joyner-Boore distance from surface projection of fault plane to site.

*b*Closest distance from fault plane to site.

*c*Significant duration of the motion between 5% and 95% of the maximum Arias Intensity.

*d*Bracketed duration for an acceleration threshold of 0.05g.

*e*Total duration of the motion.

*f*Arias Intensity.

*g*Distance to epicenter.
The model of the 2DOF SFN systems included the following fixed parameters: total mass of the structural system $M_{\text{Str}} = 20,000$ kN, representative for the RC building defined in Pürgstaller,\textsuperscript{4} and equivalent viscous damping $\xi_{\text{sys}} = 5\%$. The period of the structure, $T_{\text{Str}}$, was varied from 0.01 to 2.5 seconds, whereas the mass of the NSC $M_{\text{NSC}}$ was varied from 0.5 to 25.0 kN. All the ground motions, scaled to PGA = 0.15 g, were used as input for the analyses. The lower and upper values of the variables $M_{\text{Str}}$ and $M_{\text{NSC}}$ were selected such that they are representative of the typical design space for the expansion anchor M12 included in this research (see\textsuperscript{4} for details).

Figure 9 compares the mean values of the amplification factor $a_p$ as a function of $M_{\text{NSC}}$, for different values of $T_{\text{Str}}$, obtained for all the analyses. Figure 9A-C presents the results obtained with the 2DOF systems implemented with the linear-elastic, bilinear, and FR models, respectively. As depicted in Figure 9, the results of the analyses show that the system including FR predict significantly larger values of $a_p$, compared with the other two simpler models.

Figure 10A,B shows that the linear-elastic (LE) and bilinear (BL) models present a resonant behavior for $T_{\text{Str}} = 0.1$ and 0.2 seconds. These values are similar to $T_{\text{NSC}}$ calculated with the connection and NSC properties, such that it varies from 0.065 to 0.207 seconds and 0.05 to 0.15 seconds, for LE and BL, respectively. For the models with FR, on the other hand, $T_{\text{NSC}}$ ranges from 0.33 to 1.05 seconds (assuming $k_{\text{eff}} = 90$ kN/m for M12), greater than the maximum value obtained with LE and BL models. The results presented in Figure 10C show that the FR models predict significantly larger amplification factor $a_p$ compared with the LE and BL counterparts. The highest values of the NSC acceleration are obtained for $T_{\text{Str}}$ in the range of 0.3 to 0.6 seconds. In contrast to BL models, for FR, no clear resonance behavior was observed. This is believed to be a result of the slip behavior within the annular gap region and the constant and sharp stiffness changes of FR, such that the NSC cannot present a fully resonant behavior.

Figure 10 compares the component acceleration amplification factors $a_p$ predicted with the 2DOF systems (for $T_{\text{Str}}$ varying from 0.01 to 2.5 s) with the requirements of code provisions. This figure depicts how the mean amplification factors $a_p$ obtained with the analyses were in the range of 5 to 6 (with considerable scatter for different input motions), for rigid NSCs anchored with anchors subjected to shear loading. Current code prescriptions, on the other hand, require significantly smaller values than these. For rigid NSCs $a_p = 1$ in general, except for the case where the factor $a_{\text{gap}} = 2.0$ is included to account for impact effects as required by CEN/TS.\textsuperscript{8} For flexible NSCs, values in the order of $a_p = 2.5$ to 3 are prescribed, still smaller than $a_p$ predicted with SFNI models. Nevertheless, if the factor $a_{\text{gap}} = 2.0$ is used for flexible NSC to compute anchor forces directly from accelerations, CEN/TS provisions prescribe similar values to those obtained with the numerical analyses. This suggests that most of current code provisions might significantly underestimate the amplification of rigid NSC connected with anchors when loaded predominantly in shear.

### 6.2 SDOF SFN systems

SDOF SFN systems representing a rigid NSC anchored to a rigid structure (or, alternatively, a concrete foundation) with an anchor M12 (see Figure 3A) are considered herein. Figure 11 presents the results of the response amplification of...
FIGURE 9 2DOF SFN systems $a_p$ vs $M_{NSC}$; anchor models: (A) linear-elastic; (B) bilinear; (C) FR [Colour figure can be viewed at wileyonlinelibrary.com]

FIGURE 10 Factor $a_p$ predicted with a 2DOF SFN system (Figure 8C) and code requirements (PGA = 0.15g) [Colour figure can be viewed at wileyonlinelibrary.com]
NSC and anchors with respect to the ground motion, in terms of the amplification parameter $a_p = \frac{PCA}{PGA}$, as a function of $M_{NSC}$.

Figure 11 shows the maximum predicted values of $a_p$ when the system is subjected to the complete set of records, together with their mean averaged value. As observed in this figure, there is a significant scatter in the results of the individual analyses. The averaged values, however, are in the order of $a_p = 5$ to 6, similar to the most restrictive requirements for flexible NSCs with $\alpha_{gap} = 2.0$, required by CEN/TS and EC8 ($a_p = 6$ and 5, respectively).

In summary, as already discussed for 2DOF-SFN systems before, code provisions require significantly smaller amplification factors for rigid than flexible NSC connected with traditional anchors.

### 6.3 MDOF SFN systems

The results of NLDA of a model representing a multistory structure with NSCs anchored to each floor via a single anchor M12 are presented next. These results are compared with those obtained with the FRS method and the requirements of the selected code provisions. The considered structure comprises of a five-story RC frame building designed according to the current Italian seismic design provisions (Eurocode 2,\textsuperscript{20} NTC-2008\textsuperscript{21}), as illustrated in Figure 12.
The prototype building was assumed to be constructed in L’Aquila, Italy, and was designed under the assumption of high ductility class (CDA) for its structural members, and a behavior factor \( q = 5.85 \). Considering the symmetry of the structural plan layout, a planar (2-dimensional) model of the building responding in one of the principal directions was constructed in the computer program Ruaumoko2D, and subjected to the set of 15 earthquake records. The numerical modeling was based on lumped plasticity approach. For further details of the building geometry, mechanical properties, and numerical modeling the reader is referred to Pürgstaller.

Figure 13 presents PCAAF as a function of the story level, obtained with (a) the FRS method, using the floor accelerations of the structure obtained with the NLDA; and (b) the NLDA of the MDOF SFN system directly. With the first method, it is predicted that PCAAF < 3 for the NSCs of all floor levels. These values are similar to those calculated with the formulas for rigid NSC prescribed by CEN/TS, ASCE/SEI, and EC8. However, the results of the NLDA of the SFN system predict much larger values than these, with PCAAF in the order of 5.5 and 14 for the first and top stories, respectively.

As mentioned before, the provisions for flexible NSCs with \( \alpha_{gap} = 2 \) required by the codes above provide component amplification factors close to those predicted with the analysis of the SFN system. This is also indicated in Figure 14 for comparison.

7 | COMPARISON OF RESULTS AND DISCUSSION

Figure 15 presents the values of \( a_p \) as a function of \( T_{NSC}/T_{Str} \), obtained with (A) the FRS of the acceleration history of a SDOF structure (Figure 15A); and (B) a 2DOF SFN model implemented with the FR anchor model (Figure 15B). The graphs of Figure 15 show that larger \( a_p \) factors are predicted with the SFN model compared with the results obtained with the FRS of the SDOF structure. As presented in Figure 15, the maximum value obtained with FRS is \( a_p = 5.5 \), which is close to the mean value obtained with the SFN system.

As can be observed in Figure 15B, the results obtained with the SFN system and FR predict no clear resonance between the structure and the NSC, which, in turn, is predicted with the linear-elastic model (see Figure 15A). This is believed to be a result of the constantly changing stiffness of the FR anchor model, as mentioned before.

The maximum values predicted with the FRS method, at resonance, are similar to the mean average calculated with the results of 2DOF SFN for all \( T_{Str} \). All other results obtained with the spectral procedure are significantly smaller.

Figure 16 presents a comparison of the results for \( a_p \) obtained with the MDOF and 2DOF SFN systems. Figure 16 shows that the maximum results obtained with 2DOF models are significantly larger than those obtained with MDOF models. This can be attributed to the assumption of linearity in the restoring properties of the SDOF structure within the 2DOF SFN models included in this study, whereas the members of the MDOF structure were modelled with
Inelastic properties. It should be noted that the results presented here were gained using a high-ductile design for the building properties, whereas a lower ductility design could have given different results. This is in line with findings from other researchers, who showed, among other things, that the acceleration response of NSC is significantly influenced by the nonlinear behavior of the supporting structure.

Figure 17 compares the mean average of $a_p$ calculated with the SDOF, 2DOF, and MDOF SFN systems, implemented with the FR anchor model, as a function of $M_{NSC}$. The figure shows that for all the cases, $a_{p,avg}$ was in the range of 5 to 6. This suggests that independent of the building properties, large fastener amplification factors ($a_p \geq 5$) are predicted for rigid NSC anchored with an anchor to their host structure.

Figures 10, 11, and 16 suggest that current code provisions might significantly underestimate the amplifications effects of rigid NSC under the conditions of this investigation. It is noticed also that these differences are even greater for the NSC than for the anchorage itself, because the code prescriptions for these do not account for the $\alpha_{gap}$ amplification parameter included in the CEN/TS and EC8 provisions for the anchorage system, when an annular gap exists in the connection, as in the case of this investigation.
8 | FURTHER RESEARCH

To evaluate the presented numerical predictions, shake table tests at Structural Laboratory of La Sapienza University of Rome (Italy) have been prepared and were on-going at the time of the initial writing of this paper. Future numerical investigations must also aim to extend the applications to flexible NSC and possibly to structural connections.

9 | SUMMARY AND CONCLUSIONS

This investigation demonstrates that a more realistic nonlinear pinched hysteresis model for shear-loaded anchors (fastener rule [FR]) significantly affects the acceleration demands placed upon NSCs and fasteners. In contrast to the
more typically used linear (fastener) model, no clear resonance behavior was observed. Additionally, with a FR model, significantly larger maximum accelerations of the NSC, and anchorage forces and displacements, were predicted. The comparisons further suggest that a linear fastener model is not able to properly capture the behavior of an anchor loaded in dynamic cyclic shear and that it would further significantly and, unconservatively, underestimate the resulting demands. In the design of fasteners, the impact between the fixture and the rod are included by means of the $a_{gap} = 2$ factor, when the hole clearance between steel base plate and anchor shaft (for an M12 typically 2 mm) is not filled with mortar. This impact factor is not included in the design of NSC, despite both elements are affected by the same phenomenon (action-reaction). It is thus suggested that the $a_{gap}$-factor be included also in the design prescriptions for NSC. The effects of SFNI were studied on 2DOF and MDOF-SFNI models and compared with simpler FRS methods and code provisions. All the SFNI models analyzed predicted significantly greater fastener amplification factors, suggesting that current code provisions might significantly underestimate the component and fastener amplification factors for rigid NSCs fixed to their host structure by means of traditional anchors, such that they must resist shear loading. SFNI models predict amplification factors in the range of $a_p = 5$ to 6, while code provisions predict amplification factors of $a_p = 1$ and $a_p = 2$ (when considering the $a_{gap}$ factor) for rigid NSC attached to the structure with concrete anchors. Similar amplification factors to those predicted with SFNI models were only achieved when adding the impact factor ($a_{gap}$) and accounting for flexible NSC. It is thus suggested that NSC be considered as flexible (when designing acc. to current code prescriptions) if the anchorage behavior is taken into account. It is concluded that if a realistic (adhoc) hysteresis model is included to evaluate the response of NSC via NLDA, to account for the anchor connection behavior, the amplifications of accelerations increase in comparison with the reference values obtained with (a) current code provisions and FRS and (b) if linear elastic effective or bilinear models are used to model the anchor connection, and hence, current code provisions might underestimate the component amplifications for rigid NSC fixed to the host structure through anchors.

**NOTATION**

- $a_p$: component amplification factor $a_p = \frac{PCA}{PGA}$
- PCA: peak component acceleration (g)
- PCAAf: peak component acceleration amplification factor: PCAAf = PCA/PGA
- PFA: peak floor acceleration (g)
- PFAA: peak floor acceleration amplification factor: PFAA = PFA/PGA
- PGA: peak ground acceleration of ground motion (g)
- 2DOF: two degree of freedom system
- ASCEflex: ASCE provisions for flexible NSC
- ASCErig: ASCE provisions for rigid NSC
- CEN/TSflex: CEN/TS provisions for flexible NSC
- CEN/TSrig: CEN/TS provisions for rigid NSC
- EC8reson: EC8 provisions for NSC with $T_{Str} = T_{NSC}$ (resonance case)
- EC8rig: EC8 provisions for rigid NSC
- FRS: floor response spectra
- MDOF: multi-degree of freedom system
- MDOF-FRS: floor response spectra of multidegree of freedom system
- MDOF-SFNI: multiple DOF models for structure-fastener-nonstructural interaction
- SDOF: single degree of freedom system
- SDOF-FRS: floor response spectra of single degree of freedom system
- SDOF-SFNI: structure-fastener-nonstructural interaction of single degree of freedom system
- SFNI: structure-fastener-nonstructural interaction

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