

## Seismic monitoring of structures: characterization of building response by analyzing nonlinear elasticity and slow dynamics

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## THÈSE

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préparée au sein de l'Institut des Sciences de la Terre dans l'École Doctorale Terre, Univers, Environnement

Surveillance sismique des structures: caractérisation de la réponse des bâtiments en analysant l'élasticité nonlinéaire et la dynamique lente

Seismic monitoring of structures: characterization of building response by analyzing nonlinear elasticity and slow dynamics

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## Résumé

La surveillance de la réponse structurale est fondamentale pour estimer la performance des bâtiments et réduire les pertes lors de futurs séismes. Un moyen pratique de détecter les changements de comportement structural consiste à analyser les variations des propriétés élastiques lors d'excitations dynamiques. Dans ce travail, on montre que les variations de la fréquence fondamentale des bâtiments lors de tremblements de terre (faibles à forts) pourraient être expliquées par des processus élastiques non linéaires qui se produisent à l'intérieur du matériau, et qui finalement affectent le comportement macroscopique global des bâtiments. Ces processus élastiques non linéaires sont responsables de la diminution temporaire ou permanente de la rigidité structurale, pouvant expliquer les processus de récupération des propriétés élastiques observés à la suite d'événements sismiques. Cette étude comble le fossé entre des expériences de laboratoire à l'échelle microscopique et des observations sismologiques à l'échelle macroscopique, où l'élasticité non linéaire est également observée. Dans un premier temps, une base de données sismiques établie dans le cadre de cette thèse est présentée, incluant des réponses de bâtiments instrumentés de façon permanente dans le monde: des milliers d'enregistrements de mouvements sismiques et plusieurs bâtiments du Japon et des États-Unis ont été traités, apportant des connaissances utiles pour le domaine du génie parasismique, notamment pour la prédiction empirique de la réponse structurale en fonction de mesures d'intensité du mouvement au sol. Les incertitudes associées à la prédiction d'endommagement sont présentées, ainsi que l'évaluation de la vulnérabilité d'un bâtiment sous forme de courbes de fragilité. Ensuite, la base de données est utilisée pour analyser les signatures élastiques non linéaires dans les bâtiments, en particulier les effets de la dynamique lente (ou relaxation). Les variations des fréquences de résonance sont étudiées à court et à long terme, en estimant la contribution du sol à la réponse du système sol-structure. Différents états structuraux sont déduits en fonction des amplitudes de chargement et propriétés observées via les enregistrements. Des modèles de relaxation développés en laboratoire sont ensuite adaptés aux données des bâtiments afin de caractériser la densité de fissuration et les hétérogénéités, en effectuant des comparaisons entre les états structuraux avant et après de fortes excitations telles que le séisme de 2011 (Mw=9) de Tohoku (Japon). Les effets des chargements sont observés lors de la récupération des séquences de répliques. Les résultats sont étendus à différentes typologies de bâtiments, en analysant l'influence du matériau et des caractéristiques de chargement, notamment les taux de déformation. Enfin, quelques conclusions générales sont présentées, ainsi qu'une perspective de travail utilisant des outils de machine learning pour prédire la réponse de bâtiments en fonction de signatures élastiques non linéaires observées.

**Mots clés:** surveillance sismique des structures, elasticité nonlinéaire, dynamique lente, base de données sismiques, réponse dynamique des bâtiments.

## Seismic monitoring of structures: characterization of building response by analyzing nonlinear elasticity and slow dynamics.

Monitoring structural response is fundamental for evaluating the performance of buildings and reducing losses during future earthquakes. One practical way to detect changes in structural behavior is analyzing variations of elastic properties during dynamic excitations. Here we show that variations in the fundamental frequency of buildings during (weak -to- strong) earthquakes might be explained by nonlinear elastic processes carried out within the structural material, which affect the global macroscopic structural behavior. These nonlinear elastic processes are responsible for both transitory and permanent structural softening, and might explain the intriguing recovery effects observed in the fundamental frequency of buildings following seismic events. This study bridges the gap between microscale laboratory experiments and macroscale seismological observations, where nonlinear elasticity is also observed. In the first part of this study, a new seismic database of building responses is presented: thousands strong motion recordings and several buildings from Japan and US were processed, providing useful tools for the earthquake engineering community, notably for the empirical prediction of structural response as a function of several ground motion intensity measures. Examples of uncertainties associated to damage prediction are presented, as well as the vulnerability assessment of a building throughout fragility curves. Next, the seismic database is used to analyze nonlinear elastic signatures in buildings, particularly the slow dynamics or relaxation effects. Variations of resonant frequencies are monitored at both short and long-term, estimating the contribution of soil in the response of the system soil-structure. Different levels of damage are inferred according to loading amplitudes and structural states. Some laboratorybased models of relaxation are adapted to the building data in order to infer crack-density and heterogeneities over time, making comparisons between structural states before and after large excitations such as the Mw 9 Tohoku earthquake. Conditioning effects are observed during the backbone recovery of aftershocks sequences. The results are extended to different building typologies, analyzing the influence of structural material and loading features, notably strainrates. Finally, some general conclusions are presented, together with a perspective work using machine learning to predict building response based on nonlinear elastic signatures.

**Keywords:** seismic structural health monitoring, nonlinear elasticity, slow dynamics, earthquake database, dynamic building response.

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## Glossary

Bond system: set of cracks, voids, defects and other heterogeneities between the particles that constitute a material.

Drift ratio, or structural drift: maximum relative displacement between the top and the bottom of a building normalized by its height. The drift ratio is considered a proxy of structural deformation, and therefore, of structural damage.

Nonlinear Elasticity: atypical nonlinear behavior characterized by nonlinear effects manifested at very small deformations, within the elastic range.

Relaxation, or recovery, or Slow dynamics: process in which the elastic properties of a material recover over time after being altered due to an external loading. Some relaxation models intend to describe this process (i.e. Snieder et al., 2017; Shokouhi et al., 2017a; Ostrovsky et al., 2019).

Relaxation parameters: parameters computed from the relaxation models applied to the earthquake data recorded in buildings. Relaxation parameters are:

- $\tau_{min}$  characteristic initial time of the relaxation process
- $\tau_{max}$  characteristic final time of the relaxation process
- *p* slope of the relaxation process
- $\tau_c$  characteristic time of the relaxation spectrum
- A<sub>max</sub> maximum amplitude of the relaxation spectrum
- *bw* bandwidth of the relaxation spectrum
- *a* and *G* proxies for the elasticity before and during the relaxation, respectively

## Introduction

Earthquakes are the natural events most likely to have devastating consequences. However, the occurrence of earthquakes alone does not explain the number of victims caused during these events. Approximately 75% of the fatalities attributed to earthquakes are caused by the collapse of buildings (Coburn and Spence, 2002). Economic losses also depend more on the construction's quality than on the severity of the seismic shaking. To reduce both casualties and economic losses due to earthquakes it is therefore essential to guarantee earthquake-resistant structures. We must therefore develop our knowledge of seismic hazard to improve building design and to update seismic building codes. All of these actions require detailed understanding of structural behavior under seismic excitation, which is a complex and multi-disciplinary problem, involving a high degree of uncertainty.

To assess the expected performance of structures subjected to earthquakes, the Pacific Earthquake Engineering Research Center (PEER) proposes Performance-Based Earthquake Engineering (*PBEE*), a robust methodology aimed at improving seismic risk decision-making (i.e. Moehle and Deierlein, 2004). The methodology divides the assessment process into four main elements (Fig. 1): 1) Definition of a ground motion Intensity Measure, IM, which characterizes in probabilistic terms the input *ground motion* features that affect structural response (for example, peak ground or spectral values); 2) Calculation of Engineering Demand Parameters, EDP, which describe the *structural response* to the input ground motion (e.g. structural deformations or peak accelerations); 3) Description of Damage Measures, DM, to characterize the condition of the structure in relation to the EDP (for example cracking, spalling, collapse); and 4) Calculation of Decision Variables, DV, which is the translation of damage into quantities that can be used in risk management decisions (i.e. repair costs, casualty rates, downtime). Underlying the PBEE methodology, there is a probabilistic framework representing the uncertainties inherent to each step of the earthquake performance assessment process (Figure 1).

In the context of seismic risk assessment, the estimation of the fundamental -or natural or resonant- frequency is a relevant issue. The fundamental frequency is the primary dynamic property of a building system. Dependent on mass and stiffness, this parameter is used in both the design of new buildings and the assessment of existing ones. The fundamental frequency describes the behavior of buildings under different loadings, including earthquakes (i.e.

Trifunac et al., 2001; Kohler et al., 2005; Clinton et al., 2006; Guéguen et al., 2016; Chiauzzi et al., 2012; Astorga et al., 2018 and 2019). In most building codes, the fundamental period appears in the equation to calculate the design base shear and lateral forces. Given that the structural period cannot be measured for a structure that has not yet been built, most codes provide empirical formulae to estimate the natural period based on the building's typology (i.e. material, structural system, height), based on past experience and recorded responses of existing buildings (i.e. Goel and Chopra, 1996; Todorovska et al., 2006).



Figure 1. Underlying probabilistic framework of PBEE (according to Moehle and Deierlein, 2004).

A linear-elastic response is generally assumed for structures below a certain level of deformation; this is usually around 10<sup>-3</sup> for common buildings. Below this threshold, the stress vs. strain relationship is assumed to be linear, with no alteration of fundamental frequency. Above this threshold, the structural response is expected to enter the nonlinear range, usually accompanied by non-reversible, plastic deformations (i.e. damage). However, several authors have proved that the behavior of fundamental frequency in buildings is more complex, manifesting both nonlinear and elastic properties under different loading conditions (i.e. Kohler et al., 2005; Johnson, 2006; Clinton et al., 2006; Guéguen et al., 2016; Michel and Guéguen, 2010; Chiauzzi et al., 2012; Astorga et al., 2018 and 2019).

For example, Clinton et al., (2006) observed that the natural frequencies of the Millikan Library in California changed significantly during strong shaking, but also showed measurable changes during minor earthquakes and weather conditions at strain levels below 10<sup>-6</sup> (well below the threshold generally defined for nonlinear behavior). Similarly, Kohler et al., (2005) detected frequency variations during earthquakes and ambient vibrations in the UCLA Factor Building; Todorovska et al., (2006) observed analogous behavior in the responses of 21 buildings during weak and strong earthquakes. The above authors observed that the building frequencies recovered somewhat after moderately severe shaking events, which is a sign of elasticity.

Transient frequency variations during seismic excitation have been interpreted as a transitory nonlinear softening in system stiffness, attributed to the nonlinear response of the soil-structure interaction (i.e. Todorovska and Trifunac, 2008, Todorovska, 2009) or to nonlinearities in the system caused by the opening/closing of preexisting cracks in the superstructure (i.e. Luco et al., 1987; Clinton et al., 2006; Michel and Guéguen, 2010; Mikael et al., 2013). These variations might occur in the short or long term, and may or may not be reversible (i.e. Clinton et al., 2006; Todorovska and Al Rjoub, 2006). Permanent frequency variations might appear if structural damage occurs (i.e. Celebi et al., 1993; Dunand et al., 2006; Clinton et al., 2006). In this case, the frequency recovery is only partial, and does not reach the pre-excitation value. For example, Figure 2 shows the fundamental frequency variations of a building in Ecuador during the Mw 7.8 earthquake in April, 2016 (after Guéguen, 2016, personal communication). Higher acceleration causes frequency drops, followed by total recovery (i.e. during the foreshock and the aftershock) or partial recovery (i.e. during the mainshock). The nonlinear structural softening and recovery observed in buildings after earthquakes are not clearly understood and have not been properly characterized. However, analogous behavior has been observed in laboratory tests on granular materials. Despite significant progress, the underlying physical processes are still unknown.



**Figure 2.** Fundamental frequency variations of a building in Ecuador during the Mw 7.8 earthquake in April, 2016. Frequency drops corresponding to loading acceleration are observed followed by partial or total recovery, according to permanent or transient structural softening, respectively. (After Guéguen, 2016, personal communication)

Several studies (i.e. McCall and R.A. Guyer, 1994; Guyer et al., 1995; Guyer and Johnson, 1999; Johnson and Sutin, 2005) in heterogeneous media such as rocks, soils, concrete, and most of the materials on Earth, have reported anomalous nonlinearity that cannot be explained by

conventional theory (Landau and Lifshitz, 1986). The authors observed unexpected nonlinearity and recovery effects with hysteresis and discrete memory under quasi-static and dynamic loadings. In such media subjected to wave amplitudes in the order of  $10^{-6}$  in deformation, a rapid drop (~  $\mu$ s) of the elastic properties is observed, temporarily shifting the material from its equilibrium state. When the stress disappears, the material slowly recovers its elastic properties (~ minutes, hours), returning to a state of equilibrium. This recovery is called *slow dynamics*. A phenomenological behavior has been introduced to describe these *nonlinear*-*elastic* materials (i.e. Guyer and Johnson, 1999 and 2009; Johnson and Sutin, 2005), considering the origin of the nonlinear response to be at the scale of the microstructure. The physical phenomena linked to this behavior are not yet fully understood, but they are believed to be related to the presence of microcracks and to processes occurring in the contacts between grains within the material (i.e. the *bond system*). The authors observed that the nonlinear-elastic response governs the behavior of highly heterogeneous materials, such as concrete. A simplified scheme of this nonlinear elasticity is shown in Figure 3.



**Figure 3.** Schematic representation of nonlinear elasticity, originated at the microstructure scale, where the bond system (i.e. contacts between grains, cracks, etc.) controls the behavior. The system, initially in a state of equilibrium, is disturbed by an excitation (i.e. wave), causing frequency to fall sharply. The system is in temporary disequilibrium. When the excitation ceases, frequency slowly shifts back to its previous value as the particles within the microstructure are rearranged. The system returns to its original state (if no damage occurred) or to a new state of equilibrium (in the case of residual damage). (According to McCall and R.A. Guyer, 1994; Guyer et al., 1995; Guyer and Johnson, 1999; Ostrovsky and Johnson, 2001; Johnson and Sutin, 2005).

Sens-Schönfelder et al. (2018) suggest that the observation of nonlinear elasticity is related to the physics of friction. The authors associate softening with small-scale damage due to shear motion of internal contacts; they relate healing (recovery) to a thermally-activated process of connections creation (i.e. capillary bridges, chemical bonds) across the internal contacts. Likewise, several other studies (Johnson and Jia, 2005; Snieder et al., 2017; Delrue et al., 2018;

Ostrovsky et al., 2019) also relate non-classical nonlinear elasticity to friction conditions and thermally-activated processes between particles within the material. In the Earth's crust, nonlinear drops in elastic properties are also observed during earthquakes (i.e. Rubinstein and Beroza, 2004; Brenguier et al., 2008; Brenguier et al., 2014; Bindi et al., 2018), together with long-term recovery of the shallow layers and fault zone: a healing of cracks that is attributed to the same slow dynamic process as that observed in laboratory studies. These conclusions from laboratory tests and seismological observations imply that it might be possible to characterize variations of internal properties -which might be related to damage- by analyzing nonlinear elastic processes.

Currently, most methods to identify and evaluate damage in materials are based on nondestructive testing (NDT), which is particularly effective to detect discontinuities and differences in the material's characteristics. Different NDT techniques can be used to generate, propagate and receive ultrasound waves that travel throughout the material specimen, revealing wave distortions to detect, localize and characterize damage features. Some authors have employed NDT to identify damage in concrete (Goueygou et al., 2008; Garnier et al., 2013; Payan et al., 2014; Shokouhi et al., 2017b), and several other materials, including sandstone, Plexiglas, and aluminum (i.e. Van Den Abeele et al., 2000b and 2001; Jin et al., 2018; TenCate and Johnson, 2019). Evidence of nonlinear signatures has been found, including hysteresis, transient elastic softening and slow relaxation, indicating micro-scale damage. Legland et al., (2017) used NDT to detect and characterize defects using a controlled-damage protocol performed in a meter-scale concrete structure. The authors detected an increase in nonlinearity levels due to crack generation. After post-tensioning, the crack closes, which can be confirmed by the residual nonlinearities.

In this study, we analyze real earthquake data recorded in real buildings. Specifically, variations of fundamental frequencies in buildings of different typologies are monitored to detect nonlinear elastic processes that might be related to damage. We found surprising similarities with previous observations. Despite the different scales and techniques used, different loading and boundary conditions, different natures and levels of complexity, clear signatures of nonlinear elastic behavior and slow dynamics are apparent. As with NDT studies, we also detect elastic parameter frequency drops followed by recovery, and evidence of the opening and closing of cracks after seismic -damaging- excitation.

Given the resemblances with previous observations, it is likely that structural response under different earthquake loadings is controlled by similar micro-scale processes to those seen in various materials at laboratory and seismological scales. Furthermore, the use of real earthquake data allows actual structural behavior to be characterized, enabling the development and calibration of models based on real responses, thus reducing the uncertainties linked to assumptions made during modeling. Data-driven measurements are also useful to understand the interaction between building response and ground motion, and thus to define the parameter that best characterizes the damaging nature of earthquakes. Another advantage of in situ measurements is that actual building vulnerability can be analyzed at different damage levels, provided a sufficiently large range of earthquake loadings is available.

The main objective of this study is therefore to provide data-driven elements about the response of buildings to real earthquakes, while explaining the internal nonlinear elastic processes occurring within the structures, and how they are linked to damage.

In the **first chapter** of this study, we introduce the NDE1.0 database, used throughout the study. This new database compiles information on IM and EDP parameters computed from thousands of earthquakes recorded in several buildings, providing fundamental information for earthquake engineering applications (i.e. PBEE). For example, what is the link between structural response and ground motion? Which IM parameter provides the least variability in the prediction of EDP?

In the **second chapter**, we identify the nonlinear elastic signatures detected in a building monitored over a 20-year period. Fundamental frequency drops and slow dynamics, correlated with loading parameters and structural state, are observed between and during seismic events. What causes this behavior? How does the soil contribute to the soil-structure response?

The **third chapter** presents a detailed analysis of slow dynamics in buildings, studied through the recovery of fundamental frequencies after a single earthquake or a sequence of seismic events. Different relaxation models are adapted and applied to the building data. Hysteresis and discrete memory are observed during aftershock cycles. The behavior of parameters linked to relaxation, or recovery, helps to explain how slow dynamics can be a proxy of structural health.

In the **fourth chapter**, we analyze nonlinear elastic response in buildings of different typologies, monitored over long periods of time, showing frequency variations and relaxation

parameters over time and different damage levels. Is it possible to extrapolate the results observed in one building to other buildings with different characteristics? What variability is associated with the structural material? What is the influence of loading amplitude and loading rate on the response observed?

Finally, our conclusions are presented, along with perspectives for future works.

# 1

## NDE1.0 – A NEW DATABASE OF EARTHQUAKE DATA RECORDINGS FROM BUILDINGS FOR ENGINEERING APPLICATIONS

In this chapter we introduce a new database of earthquakes and building responses that is used throughout the thesis. After the description of the data, basic statistics and relationships between intensity measures and engineering demand parameters are processed for earthquake engineering purposes. As example, experimental fragility curves are presented, including analysis of uncertainties. Performance Based Earthquake Engineering solution is tested using experimental data for building specific performance assessment. This chapter is published in the *Bulletin of Earthquake Engineering*.

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#### Abstract

Over the last two decades, seismic ground motion prediction has been significantly improved thanks to the development of shared, open, worldwide databases (waveform and parametric values). Unlike seismic ground motion, earthquake data recorded in buildings are rarely shared. However, their contribution could be essential for evaluating the performance of structures. Increasing interest in deploying instrumentation in buildings gives hope for new observations, leading to better understanding of behavior. This manuscript presents a flat-file containing information on earthquake responses of buildings. Herein, we present the structure of the NDE1.0 flat-file containing structural response parameters (i.e. drift ratio, peak top values of acceleration, velocity and displacement, pre- and co-seismic fundamental frequencies) computed for several intensity measures characterizing ground motion (i.e. peak ground values of acceleration, velocity and displacement, spectral values, Arias intensity, strong motion duration, cumulative absolute velocity, destructive potential). The data are from real earthquake recordings collected in buildings over the years. Some building, site and earthquake

characteristics are also included (i.e. structural design, shear wave velocity, magnitude, epicentral distance, etc.). This 1.0 version contains 8,520 strong motion recordings that correspond to 118 buildings and 2,737 events, providing useful information for analyses related to seismic hazard, variability of building responses, structural health monitoring, nonlinear studies, damage prediction, etc.

Keywords: structural response, seismic database, earthquakes, buildings.

#### 1.1. Introduction

The probabilistic estimation of losses due to earthquake damage of buildings requires the analysis of different and independent components. The underlying probabilistic framework proposed by the Pacific Earthquake Engineering Research (PEER) for performance-based earthquake engineering (PBEE) divides the analysis into four main components related to seismic hazard, earthquake response of structures, probability of damage, and losses/repair costs. Each component includes uncertainties that affect the final loss estimates. Baker and Cornell (2008) propose a procedure for propagating uncertainties between the components of the approach, combining both random and epistemic uncertainties to quantify total uncertainty in seismic loss estimation.

The uncertainty resulting from the characterization of seismic ground motion has been shown to make a significant contribution to the results of probabilistic seismic hazard analysis (PSHA) (Porter, 2003; Douglas, 2003), affecting the outcomes of structural response analysis and the final result of the PBEE process. Al Atik et al., (2010) provide a comprehensive description of the variability in ground motion prediction models and their standard deviations, considering components of inter- (between) and intra- (within) event residuals. In turn, performance assessment related to building/earthquake behavior also adds uncertainties to the analysis, because of assumptions related to material properties, boundary conditions, and structural state for existing buildings. For structures in design, most recent analyses are carried out by incremental dynamic analysis (IDA) using a set of synthetic or real recordings to obtain numerical models of structural responses and to determine uncertainty in performance assessment (Iervolino, 2017). Numerical methods are efficient but they are often not representative of actual structural behavior, and might not be cost-effective for large-scale analyses. The data recorded in civil engineering structures, having been established many years ago, notably by a number of seminal papers (e.g., Housner, 1959; Jennings and Kuroiwa, 1968), have made a significant contribution to our understanding of the physical processes involved.

These data enable the calibration of numerical models, as well as the development of new empirical models, just as equations have been used for predicting ground motion.

The recent worldwide growth of seismological networks has certainly boosted the development of ground motion prediction equations (i.e. Douglas, 2011). They are based on flat-files of parameters created by national or international research programs (i.e. the NGA-West2 database, the IRIS earthquake browser, the Engineering Strong Motion ESM database, etc.). However, earthquake data in buildings have not benefitted from this progress. The California Strong Motion Instrumentation Program (CSMIP, www.conservation.ca.gov/cgs/smip), and to a lesser extent the French National Building Array Program (NBAP, Péquegnat et al., 2008; Limongelli et al., 2019) are the only open-access data programs known. Other networks exist, including Japanese, Italian and Turkish programs, but their non-open data policy in a uniform and comprehensive format does not allow the same development of extensive research as ground motion prediction data has enjoyed. However, Perrault and Guéguen (2015) have shown the relevance of such data for improving engineering demand parameter (EDP) prediction models for a given intensity measure (IM), by testing the sufficiency and effectiveness of the IM and considering the associated uncertainties.

The aim of this manuscript is to present the New Earthquake Data recorded in buildings (NDE1.0), i.e. a flat-file providing post-processed data for IM values (e.g. peak ground values, spectral values, duration), EDP values (e.g. structural drift, acceleration, displacement or velocity at the top floor), as well as building details (e.g. design, resonance frequency, co-seismic frequency, number of floors) and earthquake characteristics (e.g. magnitude, distance). The data from the Japanese and Californian networks used in this study are presented first. The methodology for processing IM and EDP parameters is then described, and some cross-correlated analysis and uncertainty values are discussed. Finally, the usefulness of the NDE1.0 flat-file for engineering seismology and earthquake engineering is illustrated using examples of possible applications and analyses.

#### 1.2. Data description

The information in the flat-file of this study was obtained by processing accelerometric timehistory files downloaded from the US Centre for Engineering Strong Motion Data (CESMD) website and the Japanese Building Research Institute (BRI) within the framework of a twoparty agreement with the authors. BRI has operated the building instrumentation program since 1957. BRI currently operates more than eighty strong motion instrumented buildings. The BRI website (https://smo.kenken.go.jp) provides information on the instrumented buildings, a technical description of the building array, acceleration waveforms and response spectra of earthquake data. Accelerometric time histories at different floors of the building can be provided by the BRI upon request. In the US, CESMD (<u>https://strongmotioncenter.org</u>) provides raw and processed worldwide strong-motion data for earthquake engineering applications. CESMD is a cooperative center integrating data from the CGS California Strong Motion Instrumentation Program, the USGS National Strong Motion Project, and the Advanced National Seismic System, which includes building array data.

Figure 1.1 shows the locations of the earthquakes and buildings (1.1a) included in NDE1.0, the distribution of data by magnitude (1.1b) and the amount of data for each building characteristic (1.1c). In total, 118 buildings and 8,520 recordings from 2,737 events were processed. The buildings are mainly steel frame (S) (56) or reinforced concrete (RC) structures (37). There are also 12 steel-reinforced concrete (SRC) buildings, 9 masonry (M) and 4 wooden (W) structures. The buildings are also ranked according to the Hazus' building classification (FEMA, 1999) based on building height: low-rise (1 to 3 floors) corresponding to 26% of the data, mid-rise (4 to 7 floors) corresponding to 31% of the data, and high-rise buildings ( $\geq$  8 floors) corresponding to 43% of the data. This study focuses only on accelerometric data on the ground and top floors, recorded in the two horizontal components, corresponding to the longitudinal and transversal directions of the structures.



**Figure 1.1** a) Geographical distribution of the events (circles) and buildings (squares) in the database. b) Normalized histograms showing event distribution according to magnitude M. c) Number of buildings in the database according to construction material.

NDE1.0 contains earthquakes of magnitudes ranging from 2.4 to 9.1 (JMA magnitude) and 3.5 to 7.3 (moment magnitude Mw) for the Japanese and US data, respectively. The normalized distribution of events according to magnitude is shown in Figure 1.1b. The mean value corresponds to magnitude 5.1 and 68% of the data are between magnitude 4.0 and 6.2. The strongest magnitude corresponds to the Mw. 9.1 Tohoku earthquake (March 11, 2011) recorded by almost all Japanese building arrays. The US data include seminal earthquakes, such as the Mw 7.2 Landers event in 1992 and the Mw 7.3 Baja California earthquake in 2010. According to Ambrasey et al. (2005), 10% of the dataset fall within the criteria of interest for earthquake engineering, i.e. magnitude M $\geq$ 5 and epicentral distance R  $\leq$ 100 km (Fig. 1.2a). Moreover, 50% of the data have epicentral distances of less than 142.2km and 50% have magnitudes above 5.1. The remaining data also provide interesting information to help understand the IM/EDP relationships involved in weak-to-moderate shaking. Figure 1.2b shows the Magnitude/Distance distribution for the Japanese ANX building which contributed the most data to the NDE1.0 flat-file. With 1,630 earthquakes recorded in both directions over a monitoring period of 20 years (June 1998-July 2018), this building has been the subject of specific analyses in this study.



**Figure 1.2** Magnitude-distance relationship corresponding to a) the entire database and b) the specific ANX (Japan) building. Magnitude and distance distributions are shown at the right and top of each plot, respectively. a) The red crosses correspond to M $\geq$ 5 and R  $\leq$ 100km. The black dashed curve corresponds to all data and the red solid curve corresponds to earthquakes with M $\geq$ 5 and R  $\leq$ 100. b) The red dot represents the Tohoku earthquake for the ANX building.

Accelerometric time histories were processed according to Boore's recommendations (2005). Firstly, trends and mean values were removed, and a Butterworth filter of order 2 was applied between 0.1 Hz and the Nyquist frequency. Velocities and displacements were then computed by numerical integration of the acceleration records. Figure 1.3 shows the log-normal distribution of the peak ground acceleration (PGA), velocity (PGV) and displacement (PGD) values from our database, ranging from 0.07cm/s<sup>2</sup> to 333cm/s<sup>2</sup>, 2x10<sup>-3</sup>cm/s to 42.4cm/s and

 $3x10^{-4}$ cm to 12.7cm, respectively. At the top floor, peak top acceleration (PTA), velocity (PTV) and displacement (PTD) vary from  $9x10^{-2}$  to 908cm/s<sup>2</sup>, from  $3x10^{-3}$  to 159.6cm/s, and from  $3x10^{-4}$  to 45.8cm, respectively. Logarithmic means and standard deviations are equal to  $\mu = 0.773$  and  $\sigma = 0.500$  for PTA,  $\mu = -0.314$  and  $\sigma = 0.587$  for PTV and  $\mu = -1.170$  and  $\sigma = 0.675$  for PTD. The relationship between peak ground and peak top values is given in Figure 4. Unlike the ASCE Standard (2006) and FEMA 450 (2004), which define the value of the PTA/PGA ratio of 3.0 regardless of building typology, the PTA/PGA value observed is equal to 2.0, and velocity and displacement ratio values are 2.4 and 1.7, respectively, considering both Japanese and US buildings.

Figure 1.3d shows also the log-normal distribution of the drift values ( $\Delta$ ) computed for the buildings in the database, ranging from  $3.47 \times 10^{-7}$  cm/cm to  $1.16 \times 10^{-2}$  cm/cm. Structural drift is computed as the relative displacement between the top and bottom signals divided by the intersensor distance. This parameter is generally linked, in the first order, to a damage index, considered to indicate structural deformation. Drift ratio thresholds related to damage states are defined according to building typology (i.e. material, height, or structural system), performance level and seismic design. For example, the Hazus technical handbook (FEMA, 1999) defines four  $\Delta$ -based damage states for RC-framed, high-rise buildings: 0.0025, 0.0043, 0.0117, and 0.03 for slight, moderate, extensive and complete damage levels, respectively. The percentages of data in the NDE1.0 flat-file exceeding these Hazus thresholds are 0.25% and 0.09% for slight and moderate damage, respectively (Figure 1.3d). Total drift ratios, however, represent the response assuming a single degree of freedom (SDOF) system, and inter-story drifts must be computed using intermediary height sensors.



**Figure 1.3** Overview of several intensity measures and engineering demand parameters from the NDE1.0 flat-file. a) peak ground acceleration values (PGA), b) velocity (PGV), c) displacement (PGD), and d) drift ratios, including all the flat-file data. Mean and standard deviation values are indicated (in log). The vertical red dashed lines show the mean+/- one standard deviation, and the dotted black lines (d) indicate the slight and moderate damage thresholds for RC high-rise buildings.



**Figure 1.4** Relationship between peak ground and peak top values for US (gray crosses) and Japanese (black dots) buildings. a) acceleration, b) velocity, and c) displacement. The linear regression model +/- $\sigma$  (red lines) corresponds to log(Y) = a + b log(X). The PTX/PGX ratio is given by slope 'b' and indicated in each plot.

#### 1.3. Data processing: the Wigner-Ville distribution

Values of fundamental frequency of the buildings were computed at the beginning and during each seismic event. Using time-frequency distributions (Neild et al., 2003), we monitor instantaneous small frequency variations that occur over time in the presence of external perturbations. We used a Cohen's class distribution (Cohen, 1989), also called energy distribution. The energy of a signal x(t) can be deduced from the squared modulus of either the signal or its Fourier transform, i.e.

$$E_{x} = \int_{-\infty}^{+\infty} |x(t)|^{2} dt = \int_{-\infty}^{+\infty} |X(\omega)|^{2} d\omega$$
 (1.1)

We can interpret  $|x(t)|^2$  and  $|X(\omega)|^2$  as energy densities, respectively in time *t* and in frequency  $\omega$ . The time-frequency energy distribution is covariant by translation in time and in frequency (Cohen, 1989). This means that if we integrate the time-frequency energy density along one variable, we obtain the energy density corresponding to the other variable. Applied to buildings, where energy is carried by resonance modes, Cohen's class tracks the energy variation at the considered modal frequency in time. In this study, we considered the Wigner-Ville (WV) distribution designed for the analysis of non-stationary signals, and defined thus:

$$WV_{x}(t,\omega) = \int_{-\infty}^{+\infty} x\left(t + \frac{\tau}{2}\right) x^{*}\left(t - \frac{\tau}{2}\right) e^{-j2\pi\omega\tau} d\tau$$
(1.2)

This is one of the simplest methods to obtain the time-frequency distribution of the energy of a signal. The definition of WVx requires knowledge of the quantity  $x(t + \tau/2) x^2(t - \tau/2)$  over  $[-\infty, +\infty]$ , which can be problematic in practice with discrete signals producing interferences.

We therefore considered a windowed version of the Wigner-Ville distribution, called the pseudo Wigner-Ville distribution, noted  $P_{WV}$ , given by the following equation:

$$P_{WV_{x}}(t,\omega) = \int_{-\infty}^{+\infty} h(\tau)x\left(t+\frac{\tau}{2}\right)x^{*}\left(t-\frac{\tau}{2}\right)e^{-j2\pi\omega\tau}d\tau$$
(1.3)

where  $h(\tau)$  is a classical windowing function. This windowing is equivalent to a frequency smoothing of the WV function that attenuates oscillation of the interferences compared with WV. However, this formulation shows the classical time-frequency distribution trade-off between time and frequency resolutions: by selecting a short time window h, the smoothing function will be narrow in time and wide in frequency, and vice versa. P<sub>WV</sub> is then only controlled by the short-time window h(t). We can therefore add a degree of freedom to the distribution, by considering a separable smoothing function g(t), to allow progressive and independent control in both time and frequency of the smoothing applied to the P<sub>WV</sub>. This new distribution is called the smoothed-pseudo Wigner-Ville distribution (sP<sub>WV</sub>), as follows:

$$sP_{WV_x}(t,\omega) = \int_{-\infty}^{+\infty} h(\tau) \int_{-\infty}^{+\infty} g(s-t)x\left(s+\frac{\tau}{2}\right) x^*\left(s-\frac{\tau}{2}\right) ds \ e^{-j2\pi\omega\tau} d\tau.$$
(1.4)

The previous time-frequency trade-off is now replaced by a compromise between the joint timefrequency resolution and the amplitude of the interferences. However, Eq. 1.4 shows that a time-frequency domain in the vicinity of each  $(t,\omega)$  point is delimited, within which the weighted average of the time-frequency values is computed. The energy band of the distribution remains broad, making it difficult to distinguish slight variations of the frequency carrying the maximum energy. A reassignment method has been proposed (Kodera et al., 1976; Auger and Flandrin, 1995) to solve the trade-off between the reduction of misleading interferences and a sharp concentration of the signal component. This method states that there are no physical reasons why the WV values should be symmetrically distributed around each point  $(t,\omega)$ , which is considered as the geometrical center of the domain. Consequently, their average must be assigned at the center of gravity, which is much more representative of the local energy distribution of the signal than the geometric center of this domain. In practice, the reassignment method ( $\delta$  operator) shifts each value of the time-frequency distribution from any point (t, $\omega$ ) to the center of gravity  $(\hat{t}, \hat{\omega})$  of the signal energy distribution around  $(t, \omega)$ . The mathematical implementation of the reassignment proposed by Kodera et al. (1976) is developed in Auger and Flandrin (1995) for several classical time-frequency methods. Equation 1.4 is then modified to the reassigned rsP<sub>WV</sub> distribution, whose value at any point (t',  $\omega$ ') is the sum of all the distribution values reassigned to this point, i.e.:

$$rsP_{WV}(t',\omega';h) = \int_{-\infty}^{+\infty} \int_{-\infty}^{+\infty} sP_{WV}(t,\omega;h)\delta(t'-\hat{t}(x;t,\omega))\delta(\omega'-\hat{\omega}(x;t,\omega))dt\,d\omega$$
(1.5)

Several other time-frequency distribution methods are available in the literature but Michel and Guéguen (2010) compared several time-frequency distributions of accelerograms recorded in buildings, with and without reassignment. They concluded on the efficiency of the rsP<sub>WV</sub> distribution for our application, i.e., for tracking the fast dynamic variation of the energy position in resonance frequency during earthquakes, reducing both interferences and the aforementioned trade-off. In this study, rsP<sub>WV</sub> was applied automatically to the whole dataset, using a 4<sup>th</sup> order of decimation in frequency and 2,048 frequency points (N). Time *h* and frequency *g* smoothing windows were taken to be default Hamming windows, with N/10 and N/4 points, respectively. Furthermore, instantaneous maximum energy value variations are tracked using a 3<sup>rd</sup> order Savitzky-Golay (polynomial smoothing window) filter with a size equal to 15% of the triggered window (i.e. time window of the duration of the record). Figure 1.5 shows an example of the rsP<sub>WV</sub> distribution and the curve corresponding to the Savitzky-Golay filter. The characteristic values of the resonance frequency observed during each earthquake are automatically picked (and manually checked) from this curve for efficient application to the whole dataset.



**Figure 1.5** a) Time-frequency distribution during an earthquake, computed using the Wigner-Ville distribution. b) Reassigned smoothed-pseudo Wigner-Ville distribution (i.e.  $rsP_{WVx}$ ) applied to the plot in a). The solid black line corresponds to the Savitzky-Golay function. Pre-seismic  $f_i$  and co-seismic  $f_{min}$  fundamental frequencies are indicated. The color scale represents the energy intensity. c) Acceleration time history at the top of the building, with *idx\_fi* as the first arrival time.

- The pre-seismic value *f<sub>i</sub>* associated with the elastic fundamental frequency is computed by the Fourier transform of the zero-padded (16,384 samples) pre-event noise window. The first arrival time (i.e. *idx\_fi*) of the earthquake is obtained by applying the STA/LTA algorithm (Short Term Average to Long Term Average), with STA/LTA = 2.
- $f_{min}$  corresponds to the co-seismic minimum value of the fundamental frequency computed from the time-frequency distribution of each record. Indeed, co-seismic variations of frequency are reported as related to structural health and nonlinear processes, generally associated with the opening/closing of preexisting cracks (i.e. Clinton et al., 2006; Todorovska and Trifunac, 2007; Astorga et al, 2018, 2019). In the flat-file, f<sub>min</sub> corresponds to the average value of ± 10 samples around the minimum value observed in the smoothing function.

#### 1.4. Structure of the NDE1.0 flat-file

The flat-file is divided into 4 information levels corresponding to (1) building and earthquake characteristics, (2) building response and engineering demand parameters, (3) ordinate and spectral values of intensity measures, and (4) duration of ground motion.

#### 1.4.1. Building and earthquake characteristics

Building information (i.e. geographical coordinates, number of floors, structural material) is summarized in Table 1.1, as given by the BRI and CESMD websites (*network\_ID*).

*Building\_ID* is the unique code of each building based on the nomenclature of data providers. *B\_lat* and *B\_long* correspond to the latitude and longitude of the building location, given in decimal degrees.

*Vs30* is the equivalent shear wave velocity for the first 30 meters of the uppermost soil layer. *Vs30* was obtained from the Japan Seismic Hazard Information Station (<u>j-shis.bosai.go.jp</u>) and the USGS Vs30 map viewer application (<u>http://usgs.maps.arcgis.com</u>).

*Height* is the distance between the top and bottom sensors, used to compute structural drift. *Floors* is the total number of floors in the building, excluding basements and penthouses. *Material* describes the material of the structural elements of the building: Reinforced Concrete (RC), Steel (S), Steel-Reinforced Concrete (SRC), Masonry (M) and Wood (W), completed by the structural system when available.

#### Table 1.1 General building information in the NDE1.0 flat-file

Building information		
ID	Description	Units
Network_ID	Strong motion network operator	
Building_ID	Building identification code. Subscript 'x' or 'y' corresponds to the two horizontal, orthogonal directions of the building	
B_lat	Geographic latitude of the building	degree
B_long	Geographic longitude of the building	degree
Vs30	Average shear wave velocity of the site at a depth of between 0 and 30 meters	m/s
Height	Distance between the top and bottom sensors	cm
Floors	Total number of floors in the building	
Material	Structural material of the building	

Table 1.2 contains a summary of earthquake characteristics, with *E\_Lat*, *E\_Long*, *Magnitude* and *Epicentral distance* related to epicentral information about the earthquake, i.e., latitude, longitude, magnitude and distance to the building; all information was provided by the network operator.

*Record\_ID* identifies the accelerometric datafile according to earthquake occurrence and sensor position. For example, in *Record\_ID 201103111446\_ANX\_8FE\_180\_1*:

- Field 1 (12 digits) corresponds to the date and time of the earthquake in the format YYYYMMDDhhmm (Y: Year, M: Month, D: Day, h: hour, m: minute). The example refers to the earthquake occurring on March 11, 2011, at 14:46 hours, local time.
- Field 2 (3 or 4 digits) corresponds to the *Building\_ID*.
- Field 3 specifies the floor and location of the sensor. For example, '8FE' corresponds to a sensor situated on the 8<sup>th</sup> floor, east-oriented. 'BF' indicates the basement.
- Field 4 (3 digits) corresponds to the azimuth of the sensor, measured in degrees from the north.
- Field 5 indicates whether the orientation of the sensor coincides (1), or not (0), with one of the main directions of the building.

Earthquake information		
ID	Description	Units
Record_ID	Name of the record containing the building and earthquake information	
E_lat	Latitude of the epicenter	degree
E_long	Longitude of the epicenter	degree
Magnitude	Magnitude of earthquake	$M_w \ or \ M_{JMA}$
Epicentral distance	Distance of the epicenter from the building	km

 Table 1.2 Earthquake information in the NDE1.0 flat-file

#### 1.4.2. Building response and engineering demand parameters

The parameters related to building response (Table 1.3) are PTA, PTV, PTD and  $\Delta$ .

In the NDE1.0 flat-file, the total drift ratio  $\Delta$  reported in Table 1.3 corresponds to the maximum value computed in four ways:

- Maximum relative displacement between the top and bottom
- Relative displacement computed between the maximum values at the top (PTD) and bottom (PGD)
- Relative displacement at the time of the PTD.
- Relative displacement between top and bottom at the time corresponding to the coseismic frequency (i.e. f<sub>min</sub>).

Table 1.3 Parameters related to building response in the NDE1.0 flat-file

Building responses		
ID	Description	Units
$\mathbf{f}_{i}$	Elastic frequency, i.e. pre-seismic fundamental frequency	Hz
$\mathbf{f}_{\min}$	Minimum value of fundamental frequency during the earthquake, i.e. co-seismic fundamental frequency	Hz
PTA	Peak acceleration recorded by the top sensor	cm/s <sup>2</sup>
PTV	Peak velocity computed at the top of the building	cm/s
PTD	Peak displacement computed at the top of the building	cm
drift_ratio	Maximum relative displacement between the top and bottom sensors normalized by building height	cm/cm

The NDE1.0 flat-file provides the pre-seismic  $f_i$  and co-seismic  $f_{min}$  fundamental frequency values required to compute the spectral IM and to analyze nonlinear elastic processes in the following chapters. Values of  $f_i$  and  $f_{min}$  were computed as explained in the section 1.3.

#### 1.4.3. Intensity of ground motion

The NDE1.0 flat-file includes several ground motion intensity parameters, classified as ordinary and spectral intensity measures (Tables 1.4 and 1.5, respectively). In addition to PGA, PGV and PGD, Arias Intensity (i.e. *Ia*, Arias, 1970), Destructive Potential (DP, Araya and Saragoni, 1984), and Cumulative Absolute Velocity (i.e. CAV, EPRI, 1988) are computed from the acceleration time histories a(t). *Ia* is an energy-based parameter that includes both amplitude and duration of the seismic shaking. It is often linked to the cumulative damage experienced by a structure, where damage is considered to be proportional to the energy dissipated per unit weight during the overall duration of the motion. *Ia* is defined as:

$$Ia = \frac{\pi}{2g} \int_0^{tf} a^2(t) \, dt \tag{1.6}$$

where g is the acceleration due to gravity and tf is the total duration of the recording. DP is a modification of Ia, where the frequency content of the earthquake is considered, as follows:

$$DP = \frac{Ia}{v_0^2} \tag{1.7}$$

where  $v_0^2$  is the number of zero crossings per unit of time. *CAV* is assumed to reflect the damaging potential of seismic loading. *CAV* is given by:

$$CAV = \int_0^{tf} |a(t)| dt \tag{1.8}$$

where |a(t)| is the absolute value of the acceleration time history.

Ordinate intensity measures		
ID	Description	Units
PGA	Peak acceleration recorded by the bottom sensor	cm/s <sup>2</sup>
PGV	Peak velocity computed at the base of the building	cm/s
PGD	Peak displacement computed at the base of the building	cm
Ia	Arias intensity	cm/s
DP	Destructive potential	cm*s
CAV	Cumulative absolute velocity	cm/s

Table 1.4 Ordinary IMs in the NDE1.0 flat-file

In Table 1.5, spectral value-based IMs are provided for a 5% damping ratio. The frequency information suggests that spectral values should be more closely related to damage potential than peak values. We use the algorithms given by Papazafeiropoulos (2015), based on Newmark and Hall (1982) to generate response or pseudo-response spectra values, considering both frequencies: pre-seismic (Sa1, Sv1 and Sd1) and co-seismic (Sa2, Sv2 and Sd2) frequencies for acceleration, velocity and displacement, respectively. In order to take into consideration the frequency shift during the seismic loading, the NDE1.0 flat-file also includes the mean spectral values computed between  $f_{min}$  and  $f_i$  (Avg\_Sa, Avg\_Sv and Avg\_Sd). This approach has been used by Bommer et al. (2004) and Perrault and Guéguen (2015) to take into account the co-seismic nonlinear response of buildings, also reducing the uncertainties in the prediction of EDP (Perrault and Guéguen, 2015).

<b>Table 1.5</b> Spectral values of IMs in the NDE1.0 flat-file	
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Spectral intensity measures		
ID	Description	Units
Sa1	Pseudo-spectral acceleration for the spectral frequency of $f_i$ ( $\xi$ =5%)	cm/s <sup>2</sup>
Sv1	Pseudo-spectral velocity for the spectral frequency of $f_i$ ( $\xi$ =5%)	cm/s
Sd1	Spectral displacement for the spectral frequency of $f_i$ ( $\xi$ =5%)	cm
Sa2	Pseudo-spectral acceleration for the spectral frequency of $f_{min}$ ( $\xi$ =5%)	cm/s <sup>2</sup>
Sv2	Pseudo-spectral velocity for the spectral frequency of $f_{min}$ ( $\xi$ =5%)	cm/s
Sd2	Spectral displacement for the spectral frequency of $f_{min}$ ( $\xi$ =5%)	cm
Avg_Sa	Mean pseudo-spectral acceleration between Sa1 and Sa2	cm/s <sup>2</sup>
Avg_Sv	Mean pseudo-spectral velocity between Sv1 and Sv2	cm/s
Avg_Sd	Mean spectral displacement between Sd1 and Sd2	cm

#### 1.4.4. Strong motion duration

Ground motion duration is often considered as a key parameter determining structural damage (i.e. Bommer and Martínez-Pereira, 1999; Araya and Saragoni, 1980). However, most recent analyses related to duration as a damage predictor are based on numerical simulations rather than experimental data (i.e. Chandramohan et al. 2016; Barbosa et al. 2017). The seminal definition of strong motion duration was proposed by Trifunac and Brady (1975), who consider that the significant duration is achieved at 95% of *Ia*. In this study, we compute different strong motion durations, summarized in Table 1.6, and defined as follows:

- Bracketed duration, D<sub>B</sub> (Fig. 6a): total time between the first and the last exceedance of a specific acceleration threshold (i.e. a<sub>0</sub>). Four acceleration thresholds are defined: 0.05g (i.e. D<sub>B1</sub>), 0.1g (i.e. D<sub>B2</sub>), 0.15g (D<sub>B3</sub>), and 0.20g (D<sub>B4</sub>).
- Effective duration,  $D_E$  (Fig. 6b): defined by  $D_E = t_f t_0$ , where  $t_0$  corresponds to the time at which 0.01m/s of cumulative energy is reached in the Husid diagram (i.e. energy build-up plot, Ia) and  $t_f$  corresponds to the time at which Ia=0.125 m/s.
- Uniform duration, D<sub>U</sub>: sum of the time intervals during which acceleration exceeds a specific acceleration threshold. Four acceleration thresholds are considered in the flat-file: 0.05g (i.e. D<sub>U1</sub>), 0.1g (i.e. D<sub>U2</sub>), 0.15g (i.e. D<sub>U3</sub>), and 0.20g (i.e. D<sub>U4</sub>).
- Significant duration,  $D_S$  (Fig. 6b): defined as the time interval over which a specific percentage of total energy is accumulated on the Husid diagram. Intervals corresponding to (5-75)% and (5-95)% of total energy are considered, indicated as  $D_{Sa1}$  and  $D_{Sa2}$ , respectively. Durations based on cumulative energy computed from velocity (i.e.  $D_{Sv1}$  and  $D_{Sv2}$ ) and displacement signals (i.e.  $D_{Sd1}$  and  $D_{Sd2}$ ) are also determined, as suggested by Trifunac and Brady (1975).

- Zhou and Xie (1984) duration, D<sub>ZX</sub>, defined as:

$$D_{ZX} = 2 \sqrt{\frac{\int_{0}^{t_{r}} (t-t_{c})^{2} \cdot a^{2}(t)dt}{\int_{0}^{t_{r}} a^{2}(t)dt}} \quad \text{where} \quad t_{c} = \frac{\int_{0}^{t_{r}} t \cdot a^{2}(t)dt}{\int_{0}^{t_{r}} a^{2}(t)dt}$$
(1.9)

 $t_c$  is the center of gravity of  $a^2(t)$  along the time axis, a(t) is the acceleration at a given time t, and  $t_r$  is the total duration of ground motion.



**Figure 1.6** Schematic view of several duration definitions given for two different acceleration time histories. a) Bracketed duration. The horizontal dashed line corresponds to the threshold level of acceleration. b) Significant strong motion duration computed for (5-75) % and (5-95) % of total energy based on the Husid diagram.

Strong motion duration		
Parameter	Description	Units
D <sub>E</sub>	Time interval between 0.01m/s and Ia=0.125 m/s using the Husid diagram	S
DB (1,2,3 or 4)	Total time between the first and the last exceedance of the acceleration threshold, i.e. $0.05g (D_{B1}), 0.10g (D_{B2}), 0.15g (D_{B3})$ and $0.20g (D_{B4})$	S
DU (1,2,3 or 4)	Sum of the time intervals during which acceleration exceeds the acceleration threshold, i.e. 0.05g (D <sub>U1</sub> ), 0.10g (D <sub>U2</sub> ), 0.15g (D <sub>U3</sub> ) and 0.20g (D <sub>U4</sub> )	S
DSa (1 or 2)	Duration corresponding to $(5-75)$ % (i.e. $D_{Sa1}$ ) and $(5-95)$ % (i.e. $D_{Sa2}$ ) of total energy associated with ground motion acceleration.	S
DSv (1or 2)	Duration corresponding to $(5-75)$ % (i.e. $D_{Sv1}$ ) and $(5-95)$ % (i.e. $D_{Sv2}$ ) of total energy associated with ground motion velocity.	S
D <sub>Sd (1or 2)</sub>	Duration corresponding to $(5-75)$ % (i.e. $D_{Sd1}$ ) and $(5-95)$ % (i.e. $D_{Sd2}$ ) of total energy associated with ground motion displacement.	S
Dzx	Zhou and Xie (1984) duration	S

 Table 1.6 Definitions of strong motion duration in the NDE1.0 flat-file

## **1.5.** Empirical prediction of building response P(EDP|IM) and associated uncertainties

One key step in the PBEE framework is the prediction of EDP for a given IM (i.e. P(EDP|IM)). These relationships are based on statistical regressions between IM and EDP. EDP follows a lognormal distribution for a uniform distribution of ground motion parameters (Perrault and

Guéguen, 2015) and therefore a log-linear functional form is considered to estimate the value of EDP | IM. The functional form proposed corresponds to a first-degree polynomial written as:

$$\log(\Delta) = a + b \log(IM) + \varepsilon$$
(1.10)

where a and b are coefficients obtained by linear regression, and  $\varepsilon$  is the standard error used to determine the efficiency of the IM in predicting  $\Delta$  (Shome and Cornell, 1999; Luco, 2002).

In Figure 1.7, the functional form is applied to estimate  $\Delta$  variability of the buildings as a function of PGA and PGV for the Japanese (1.7a) and US (1.7b) data. We observe that it is PGV and not PGA that gives the smallest EDP variability ( $\sigma = 0.346$  and  $\sigma = 0.437$  for the Japanese and US data, respectively; compared with PGA values of  $\sigma = 0.518$  and  $\sigma = 0.518$  for Japanese and US data, respectively). This has also been reported by several previous studies in different contexts (i.e. Wald et al. 1999, Bommer and Alarcon, 2008; Akkar and Bommer, 2007; Lesueur et al., 2013; Perrault and Guéguen, 2015). This observation does not depend on building design.



**Figure 1.7** Correlation between drift ratio ( $\Delta$ ) and ordinary intensity measures (left) PGA and (right) PGV for a) all Japanese data and b) all US data. The red solid and dashed lines correspond to the mean  $\pm \sigma$  of the functional form log( $\Delta$ ) = a + b log(IM) +  $\epsilon$  with  $\sigma$ =mean( $\epsilon$ )

Figure 1.8 shows the plot of the same results using spectral averaged values Avg\_Sa and Avg\_Sv. For the Japanese data (Fig. 1.8a), average spectral values reduce the uncertainties of predicted EDP compared with peak values (Fig. 1.7) ( $\sigma$ =0.352 for Avg\_Sa and  $\sigma$ =0.293 for

Avg\_Sv). However, for the US buildings (Figure 8b), the standard deviations are only slightly reduced, suggesting additional parameters controlling P(EDP|IM), such as building typology or earthquake parameters (Perrault and Guéguen, 2015). Further analyses using the NDE1.0 flat-file could be carried out on the efficiency and sufficiency of each IM (see Table A1 in the appendix for the results for all IMs in the NDE1.0 flat-file). Table A1 shows reduced variability for US and Japanese buildings after sorting by material (particularly wooden US structures and steel Japanese structures).



**Figure 1.8** Correlation between drift ratio ( $\Delta$ ) and spectral intensity measures Avg\_Sa and Avg\_Sv for a) all Japanese data and b) all US data. The solid and dashed lines correspond to the mean  $\pm \sigma$  of the functional form  $\log(\Delta) = a + b \log(IM) + \varepsilon$  with  $\sigma$ =mean( $\varepsilon$ )

Figure 1.9 shows the prediction of co-seismic frequency  $f_{min}$  as a function of PGA, according to functional form (Eq. 1.10), assuming  $f_{min}$  as a proxy of the co-seismic capacity of the structure related to non-ductile/ductile transition (Table A2 in the appendix summarizes all the  $\sigma$  values). The variability in the prediction of  $f_{min}$  for all Japanese data (Fig. 1.9a) is 0.212, which is reduced to 0.086 and 0.048 when the data are sorted by material type, such as SRC (Fig. 1.9c) or S (Fig. 1.9d). For RC buildings (Fig. 1.9b), the data are more scattered, due to different characteristics (i.e. number of floors, different structural systems, such as shear-walls, frames, etc.) but also because heterogeneous and fractured RC materials are more sensitive to loading. Indeed, several authors have demonstrated the influence of the microstructure on nonlinear elastic behavior, proving that granular/heterogeneous materials (e.g., rocks, concrete, damaged
materials) show a wide range of nonlinear elastic responses (Guyer and Johnson, 1999; TenCate et al., 2000; Rivière et al., 2015 and 2016). This was also confirmed in by Astorga et al. (2019) for civil engineering structures.

In table A2, the results of the variability of functional form suggest that considering fundamental frequency as EDP yields a lower variability than  $\Delta$ , suggesting that fundamental frequency could be used for predictive models of structural response. Finally, by testing and combining the most effective IMs, further studies could be conducted on P(EDP | IM) in an attempt to reduce EDP prediction uncertainties, such as structural drift or co-seismic resonance frequency.



**Figure 1.9**  $P(f_{min}|PGA)$  models. a) all Japanese data, b) Japanese RC buildings, c) Japanese SRC buildings, and d) Japanese steel buildings. The solid and dashed lines correspond to the mean  $\pm \sigma$  of the functional form indicated in the title of the figures.  $\sigma$  values are also indicated. The Y-axis corresponds to values of  $f_{min}$  normalized by the minimum value of each data group.

#### 1.6. Uncertainties related to building-specific damage prediction

For structural performance analysis, Luco and Cornell (2007) defined a sufficient *IM* as the ground motion parameter that renders building response, then associated seismic damage, conditionally independent of earthquake magnitude and source-to-building distance. With respect to seismic ground motion variability, most GMPEs activities have focused on understanding its origin, making the most of the large amount of data published over the past decade. Al Atik et al. (2010) described the uncertainties of seismic ground motion by decomposing total variability into between-event ( $\tau$ ) and within-event ( $\phi$ ) variability. Along the same lines as Al Atik et al., (2010), a preliminary analysis is presented herein to examine the

origin of the variability in the structural response, using the so-called Building Damage Prediction Equation (BDPE), first introduced by Perrault and Guéguen (2015) and corresponding to Eq. 1.10. The total standard deviation of the BDPE is thus given by:

$$\sigma = \sqrt{\tau^2 + \phi^2} \tag{1.11}$$

 $\sigma$  represents the total variability of the BDPE, in this case associated with  $\Delta$  regardless of the magnitude and distance parameters.  $\tau$  represents the median deviation of the BDPE for a given magnitude considering all observations, and  $\phi$  corresponds to the deviation of individual observations from the earthquake-specific median prediction (given a magnitude).

Figure 1.10 shows the ( $\Delta$ = f(R) | M [5.0, 5.5, 6.0, and 6.5]) relationship, illustrating the equivalent between-event and within-event variability of  $\Delta$  as a function of R. For Japanese buildings, the three components of the variability ( $\phi$ ,  $\tau$  and  $\sigma$ ) increase with magnitude, i.e. structural response (for EDP= $\Delta$ ) does not appear to be conditionally independent of magnitude and distance. The  $\phi$  component is of the same order of magnitude as the  $\sigma$  associated with total variability, i.e. the within-event component contributes most to total variability of the building damage prediction equation.  $\phi$  is also one order of magnitude higher than  $\tau$ , indicating that, for EDP= $\Delta$ , between-event variability contributes less to the prediction of  $\Delta$  regardless of ground motion and building typology.



**Figure 1.10** Between-event and within-event components of drift variability given epicentral distance for Japanese data with magnitudes M = [5.0, 5.5, 6.0 and 6.5]. Black dashed lines are the medians of the data ranked by magnitude. The red line is the total median model considering all the data. The between-event variability is quantified by  $\tau$ , which represents the distance between the mean model for a given magnitude and the total median model. The within-event variability is quantified by  $\phi$ , corresponding to the distance between the markers and the corresponding model in magnitude.

In the same way, the variability of EDP between and within different building materials (e.g., S or RC), for a given IM, is shown in Figure 1.11. For S buildings (m=S) and given PGV values (Fig. 11a), the within-material variability (i.e.  $\phi_m$ ), between-material variability (i.e.  $\tau_m$ ) and total material variability (i.e.  $\sigma_m$ ) are one order of amplitude smaller than for reinforced concrete (m=RC) in the drift ratio prediction. The effect of the structural material on the expected response can be quantified, in addition to the results in Figure 1.9 and Tables A1 and A2, as well as the larger dispersion observed for RC structures, as mentioned above. The BDPE must be also cross-analyzed with IM sufficiency regardless of typology (Luco and Cornell, 2007). The sufficiency of IM is evaluated by analyzing the residuals for a given M and R (i.e. Buratti, 2012), as shown in figures 1.11b and 1.11c. In these figures, the residuals increase slightly with distance and magnitude when considering PGV as IM, i.e. the building response ( $\Delta$ ) is not totally conditionally independent of magnitude and distance.



**Figure 1.11** a) Between-material and within-material variability in the prediction of drift ratio as a function of PGV. Linear regressions of  $\Delta = f$  (PGV) are presented for Japanese S (dashed line) and RC (dashed-dotted line) buildings. The solid line corresponds to the median prediction model for all Japanese data.  $\sigma$ ,  $\tau$  and  $\phi$  values are indicated for each building-class model. b) Analysis of PGV residuals with respect to M. c) Same with respect to R.

#### 1.7. Seismic vulnerability assessment and performance-based analysis

The NDE1.0 flat-file provides useful information for the development of experimental vulnerability functions and performance-based approaches for existing buildings. These functions include lognormal fragility curves that give the probability of a structure exceeding different states of damage. Damage states can be based on drift thresholds, for example (FEMA, 1999). The general form of a fragility curve is a cumulative lognormal distribution defined by:

$$P[ds|IM] = \varphi \left[ \frac{1}{\beta_{ds}} \ln \left( \frac{IM}{\overline{IM}_{,ds}} \right) \right]$$
(1.12)

where  $\overline{IM}_{,ds}$  is the median value of the IM at which the building reaches the threshold of the damage state,  $ds_{.}\beta_{ds}$  is the standard deviation of the natural logarithm of the IM for the given damage state, and  $\varphi$  is the standard normal cumulative distribution function.

Figure 1.12 shows the fragility curves for the ANX building, i.e. the building that experienced the most earthquakes in NDE1.0. Several IMs are tested to estimate the probability of exceeding the slight damage state (i.e.  $\Delta$ =0.0025). Fragility curves are computed using the procedure given by Porter et al. (2007), where the mean and standard deviation values (i.e.  $\overline{IM}_{,ds}$  and  $\beta_{ds}$ , respectively) are computed as follows:

$$\overline{IM}_{,ds} = exp\left(\frac{1}{N}\sum_{i=1}^{N}\log(r_i)\right) \qquad \beta_{ds} = \sqrt{\frac{1}{N-1}\sum_{i=1}^{N}(\log(r_i/\overline{IM}_{,ds}))^2}$$
(1.13)

with *N* the number of samples exceeding  $\Delta = 0.0025$ , and  $r_i$  the EDP value at which damage occurs in sample *i*. In the case of the ANX building (Fig. 1.12), spectral velocity values (i.e. *Sv1*) and the average spectral displacement value (i.e. Avg\_Sd) provide smaller standard deviations in the prediction of slight damage (i.e.  $\sigma = 0.386$  and  $\sigma = 0.363$ , respectively). Among the peak ground motion values, *PGV* is the best predictor (i.e.  $\sigma = 0.439$ ). These data enable building-specific vulnerability assessment, with the advantage of considering the actual responses of the structures and their evolution over time. Indeed, Astorga et al. (2018) reported the time variation of the ANX building fundamental frequency, due to the cumulative effects of earthquakes. Consequently, the efficiency of IMs in the prediction of the building response may also be time-dependent.

Iervolino (2017) assesses uncertainties for PBEE considering both the IM approach and the EDP approach. The author used 30-recorded accelerograms and utilized the IDA method to compute building model failure. For each record, the minimum IM-value causing the unwanted structural response was used, and the estimators of equation (1.13) (i.e. mean and standard deviation) were applied to obtain the fragility curves. Figure 1.13 shows an example of the experimental IM-approach applied to the Japanese SRC buildings, considering slight damage and Avg Sa as IM. In NDE1.0, five buildings exceed the slight damage threshold ( $\Delta$ =0.0025).



Figure 1.12 Fragility curves for the ANX building in Japan considering slight damage ( $\Delta = 0.0025$ ). Mean and standard deviation values associated with the lognormal distribution are shown in each plot.

In Figure 1.13, the crosses indicate the spectral values above which the damage threshold is exceeded, and the (log-normal) probability distribution of these values is displayed. The corresponding cumulative distribution function (i.e. fragility curve) is shown in the right plot, with a lognormal mean of  $\mu = 5.25$  and standard deviation (Eq. 1.13) equal to  $\beta = 0.24$ . The

corresponding probabilities of exceeding the slight damage state for this typology-specific analysis (SRC Japanese buildings) are:

- P [ds=slight | Sa = 149.90 cm/s<sup>2</sup> ( $\mu$   $\beta$ )] = 0.28
- P [ds=slight | Sa = 190.57 cm/s<sup>2</sup> ( $\mu$ )] = 0.50
- P [ds=slight | Sa = 242.26 cm/s<sup>2</sup> ( $\mu$  +  $\beta$ )] = 0.92

Furthermore, for Sa = 210 cm/s<sup>2</sup>, Japanese SRC buildings have a probability of 0.66 of exceeding the slight damage threshold (shaded area in Fig. 1.13). Structural fragility can actually be determined given different IMs and structural performances, resulting in the ( $\mu$ ,  $\beta$ ) pair of fragility parameters for given failures, and quantifying the between-record variability of response for a same-class building. Record-to-record variability means that the parameters ( $\mu$ ,  $\beta$ ) are expected to change when a different sample is used. Finally, structural fragility could also be integrated into the underlying PBEE framework along seismic hazard, yielding an annual failure rate for the structure/building-class in a specific seismic prone region.



**Figure 1.13** (Left) Example of the IM-approach for fragility assessment, using the correlation between spectral acceleration (i.e.  $Avg_Sa$ ) and building response (i.e. drift ratio) for 5 SRC-buildings from the database. The vertical dashed line indicates the threshold of the slight damage state (i.e. drift = 0.0025). The horizontal dashed line indicates a hypothetical value of the expected ground motion IM (i.e. 210 cm/s<sup>2</sup>). The log-normal distribution of the IM/EDP relationship is displayed, with the shaded area indicating the probability of exceeding the damage threshold given the Sa value. (Right) Fragility curve corresponding to the left plot. The probability of exceeding the damage threshold is given as a function of mean and standard deviation values.

### **1.8.** Conclusions

Buildings with permanent instrumentation can make a significant contribution to improving our understanding of the physical response of structures to earthquakes. Data collected in civil

engineering structures are fundamental to the validation and calibration of existing models and the physical concepts used for structures in design, and to the development of empirical relationships for the prediction of real structural behavior. In particular, a key feature of performance-based earthquake engineering is the estimation of the uncertainties involved in the (ED|IM) relationship. The inclusion of data from real earthquakes in building-specific studies makes it possible to imagine data-driven damage prediction relationships, associated with the reduction of epistemic uncertainty. The epistemic uncertainties in (EDP|IM) are not yet processed properly, and such uncertainties contribute to the overall prediction of damage and loss parameters. Even at low levels of deformation, real earthquake data might enable better understanding of the physical processes occurring in structures.

This study presents the first version of a new flat-file of earthquake recordings for building damage prediction (NDE1.0). Some applications are given as examples of experimental evaluations of structural response and correlated performance. We tested the variability between some IMs and EDPs, and developed empirical fragility curves using NDE1.0 data. Using a quantitative assessment, we confirmed the reduction in variability associated with the (EDP|IM) relationship when PGV or spectral values are considered. However, parameters characterizing building response, such as co-seismic frequency, could also be considered to estimate structural degradation and earthquake-related damage rather than structural drift, with lower variability.

With NDE1.0, we initiated an analysis of within-event and between-event variability as well as within-typology and between-typology variability. Identification of the various components of building-response variability is a promising step towards improving PBEE. Indeed, some apparent variability can be changed to epistemic uncertainty. Considering that full-scale observations in structures are much more representative of physical and natural activated processes than even the most sophisticated laboratory or numerical experiments, the only way to improve earthquake building response prediction is to create extensive and comprehensive flat-files or databases with earthquake recordings from permanently instrumented structures. With more data and more information on building response, such as co-seismic frequency, building features and IM efficiency, epistemic uncertainties could be reduced or even eliminated. If epistemic uncertainty is not reduced, the mean building response considered for vulnerability functions or equivalent IDA approaches will be unchanged, broadening the building response fractile. This would mean that the fractiles of the loss function would be increased for PBEE and would have to be included in a logic-tree based approach.

Further studies are required in the fields of efficiency and sufficiency of IMs, the relationship between drift and frequency reduction, and the prediction of structural drift, with special emphasis on the estimation of uncertainties. This approach is still useful for studying the different components of the uncertainty related to building response and identifying strategies for further studies to reduce this uncertainty. Additional data will be integrated in an extended version of the flat-file, starting with French National Building Array program. The objective is to draw the attention and interest of a broader research community, and to involve data contributions from additional national networks, as well as to increase the number of instrumented structures worldwide in order to build a sound and comprehensive program of data-driven structural performance-based analysis.

# 2

# NONLINEAR ELASTICITY OBSERVED IN BUILDINGS: THE CASE OF THE ANX BUILDING (JAPAN)

In this chapter we provide evidence of nonlinear elastic behavior in buildings, analyzing the particular case of the ANX building in Japan, where nonlinear elastic signatures are observed at short and long-term monitoring of earthquake data. We focus on the variations of resonance frequencies to study transitory and permanent modifications of structural properties. The possible origins of this behavior are discussed, including the effects of soil, to characterize structural damage. The content of this chapter is published in the *Bulletin of the Seismological Society of America*.

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## Abstract

Nonlinear elasticity is evidenced by the variation of the elastic properties of a system under slight strain and their recovery after loading. These characteristics have been previously observed at the scale of the laboratory sample and at the Earth's crust immediately after strong earthquakes. In this chapter, we observe that the nonlinear elastic response can be observed in a similar manner in civil engineering structures. Herein, we study a Japanese building under permanent monitoring since 1998 and during the Tohoku (Mw 9.0) 2011 seismic sequence. First, we observe that under low strain, the resonance frequency of the building decreases continuously until a plateau value is reached, characterizing the continuous damage process with repeated dynamic loading forces conditioning the structure. After each earthquake, intraevent recovery of the resonance frequency is observed. A strong variation of the resonance frequency is seen after the Tohoku earthquake, characterizing the damage state of the building, but this is followed by long-term recovery (over about six months) of the elastic properties. By analyzing the site response, we conclude on the moderate contribution of the nonlinear soil-

structure interaction to the slow dynamics observed after the Tohoku earthquake, and we assume that the degree of heterogeneities (cracks) present in the structural elements of the building governs the nature of the nonlinear elastic response. Analysis of these variations might represent an easy and suitable way of monitoring structural health.

**Keywords:** nonlinear elasticity, slow dynamics, resonance frequency, ANX building, Japan, sequence of earthquakes, cracks, monitoring structural health.

### 2.1. Introduction

In earthquake engineering and structural design, the idealized response of a civil engineering structure to a pattern of loading (i.e., earthquake) may be simplified by an equivalent bi-linear elastoplastic system. Within the linearly elastic range, the system has a natural vibration frequency. Deformation is proportional to loading according to a linear relationship, which is in turn proportional to the stiffness of the structure. Elasticity implies that the unloading-reloading path is reversible. After the yield point, a nonlinear response (plastic) is observed and the loading-deformation relationship for the structural elements undergoing cyclic deformations becomes nonlinear and hysteretic. This is typically accompanied by a variation of the natural vibration frequency, associated with a certain level of degradation or damage. However, another type of behavior, known as nonlinear elasticity, has been observed in different materials and at different scales, ranging from laboratory tests (i.e. Johnson and Rasolofosaon, 1996; Guyer and Johnson, 1999; Ostrovsky and Johnson 2001; Guyer and Johnson, 2009; TenCate, 2011; Rivière et al., 2015) to the crust of the Earth (i.e. Brenguier et al., 2008; Brenguier et al., 2014).

Laboratory tests on materials such as rocks, soils and concrete have shown a nonlinear response in the stress-strain domain, corresponding to resonance frequency shifts at low strain values, and a slow dynamic pattern (Guyer and Johnson 1999; Johnson and Sutin, 2005; TenCate, 2011; Renaud et al., 2014). Recently, nonlinear elasticity was also observed on the scale of buildings Guéguen et al. (2016) assuming that the variation of the natural vibration frequency at lowstrain values (<10<sup>-5</sup>) was associated with the elastic properties. Moreover, a recovery process, called slow dynamics, that returns the material properties to the original or a new equilibrium state, usually accompanies nonlinear elasticity after a damaging event (TenCate and Shankland, 1996; Guyer and Johnson, 2009). The recovery of the observables (i.e., frequency, velocity, etc.) in many experiments varies logarithmically with time (Johnson and Sutin, 2005a; Guyer and Johnson 2009; TenCate, 2011). A similar log-time recovery of observables has also been reported after strong earthquakes in the Earth's crust (Sawazaki et al., 2006; Brenguier et al. 2008, Brenguier et al. 2014) or in the shallow upper layers of sediments, corresponding mostly to a change in shear velocity (Wu et al., 2009a; Sawazaki et al., 2009; Bonilla et al., 2011; Takagi and Okada, 2012).

Snieder et al. (2017) assumed the recovery of observables is a consequence of different combined relaxation mechanisms operating on different spatial and temporal scales, including healing rates, breaking of capillarity bonds of different sizes within fractures. Irrespective of the mechanisms involved, the nature of the relaxation is governed by the extent of the heterogeneities in the material (Guyer and Johnson, 1999), which can also represent the level of damage. In buildings, Kohler et al. (2007), Bodin et al. (2012) and Guéguen et al. (2016) have shown a recovery process of the elastic properties of the structures under strong and weak motions. In these cases, relaxation was monitored by the time-variation of their resonance frequency during earthquakes or weak loading. Assuming an invariant scale mechanism (i.e., from the lab to the Earth's crust) associated with the relaxation, this behavior can be analyzed as a proxy of the health of structures. Considering that data from real-scale buildings are much more relevant compared to even the most sophisticated laboratory experiments or numerical simulations, the use of such data makes it possible to improve our knowledge of the mechanisms involved in civil engineering structures, with a view to predicting their dynamic response or monitoring structural health.

In this chapter, we use approximately 20 years of seismic recordings from the ANX building (Japan) to analyze the fluctuations of the fundamental frequency under strong motions that induce different levels of deformation in the structure. The main objective is to confirm the presence of nonlinear elasticity in buildings and to observe the short- and long-term recovery of the structure's elastic properties during an extensive earthquake sequence. The first section presents a brief description of the ANX building and the data analyzed. Then, the tracking of the fundamental frequency of the building is discussed and interpreted as a characteristic of nonlinear elasticity. Finally, some conclusions and discussions are provided on the observation of structural dynamics and health monitoring during earthquake sequences.

### 2.2. Description of the ANX building and data

Located in Tsukuba, approximately 60km northwest of Tokyo (Japan), the Annex (ANX) building hosts the Japanese Urban Disaster Prevention Research Center. ANX is an 8-storey, steel-framed reinforced concrete (SRC) structure, with one basement floor (Kashima, 2004;

Kashima, 2014). Figure 2.1 shows the plan and section views of the ANX building, as well as the location of the sensors whose data were used in this study. The building rests on soft soil through a spread foundation (8.2m deep). The soil column is layered with clay and sandy-clay materials to a depth of at least 40m. Its construction was completed in March 1998 and the building has been monitored since its completion by a dense network of instruments operated by the Building Research Institute (BRI). This monitoring system is composed of eleven 3C accelerometric sensors oriented along the main horizontal and vertical directions, spread over the height of the building. Another seven 3C accelerometers complete the ANX network, located in the nearby ground on the surface and in boreholes at three different depths.



**Figure 2.1** Picture, plan and section views of the ANX building. The triangles represent the 2 horizontal components of the accelerometric sensors used in this study, (BFE = Basement East corner; 8FE = 8th Floor East corner; A01 = Ground Level sensor; A14 = borehole sensor 14m deep). Figure simplified from Kashima (2014).

Since 1998, the ANX building array has recorded a large number of earthquakes. In this study, a total of 1630 earthquakes occurring between June 1998 and May 2018 were considered for each direction of the building, including the main shock of the great Tohoku earthquake (Mw 9.0) in March of 2011 and its aftershocks sequence, which caused slight structural damage that included cracking in several walls and broken expansion joints, leading to retrofitting works some months after the event (Kashima, 2014). The data consist in triggered time histories of accelerometric sensors, sampled at 100Hz. Figure 2.2 shows the geographic location of the building and the epicenter positions of the earthquakes (2.2a), the magnitude (JMA) – epicentral distance distribution (2.2b), and the acceleration at the bottom of the building versus the structural deformation (2.2c). Structural deformation or structural drift ( $\Delta$ , chapter 1) was calculated as the relative displacement between top and basement, divided by the building height, 34 m. The data correspond to earthquakes of magnitudes ranging from 2.6 to 9.0 (Mw), and epicentral distances from 1 to 1726km. Peak Ground Acceleration (PGA) recorded by the nearby free-field station (sensor A01) ranges from 0.4 to 279.3cm/s<sup>2</sup>, corresponding to Peak Top Accelerations (PTAs) from 0.39 to 596.80cm/s<sup>2</sup>. Obviously, the highest values of PTA and

PGA correspond to the largest Mw 9.0 Tohoku earthquake in March 2011, located 330km northeast of the ANX building (Fig. 2.2a). All these data have produced total structural drifts ranging from  $10^{-7}$  to  $10^{-3}$  (cm/cm). We analyzed all pairs of sensors between the top and the bottom in both horizontal directions. Herein, because of results similarity, we only consider one pair of sensors, that is, eighth floor east corner (8FE) at the top, and basement east corner (BFE) in the basement (Fig. 2.1). The elastic frequency of the building is 1.85Hz (+/- 0.04Hz) in the X-270 direction and 1.63Hz (+/- 0.08Hz) in Y-180. These values were estimated from the first ten events recorded at the beginning of the measurements in 1998.



**Figure 2.2** a) Epicenter location of the earthquakes used in this chapter. The square indicates the position of the ANX building. The star indicates the epicenter of the Tohoku Mw 9.0 earthquake. b) Magnitude (Japan Meteorological Agency [JMA]) versus the epicentral distance of the earthquakes with respect to the ANX building. The red dot corresponds to the 2011 Tohoku event c) Peak value of acceleration at the bottom of the ANX building, PBA, versus maximum structural deformation (or maximum structural drift)  $\Delta_{max}$ , computed for the ANX building during each earthquake (using the BFE and the 8FE sensors).

#### 2.3. Data processing

As explained in the chapter 1, data were processed according to the Boore (2005) recommendations for processing strong-motion accelerograms. Velocities and displacements were obtained by integrating the acceleration data, after removing the mean and trend from the time-histories. The signals were filtered using a Butterworth filter of order 2 between 0.1Hz and the Nyquist frequency (50Hz). The signals were then tapered with a Tukey cosine window (10%). Figure 2.3 shows the time history of accelerations, velocities and displacements, as well as the accelerometric Fourier spectrum for the Tohoku earthquake recorded by the sensor 8FE (X direction). In this study, the Engineering Demand Parameters (EDPs) considered are PTA,  $\Delta$  or both. The maximum interstory drift ratio is usually considered to represent the structural response and damage level. In structural design, the damage-control limit state can be defined by material strain limits and by design drift limits intended to restrict damage. It is also

comparatively straightforward to compute drift limits from strain limits (Presley, 2000). By analogy with the laboratory experiments,  $\Delta$  will be considered in this study as a proxy of the strain value of the system.



**Figure 2.3** a) Acceleration, velocity, and displacement time histories of the  $M_w 9.0$  Tohoku earthquake recorded by sensor 8FE in the X direction. b) Fourier transform in acceleration.

The nonlinear response of the ANX building is studied by monitoring the co-seismic variation of its resonance frequency (Fig. 2.4). The coseismic frequency (i.e. f<sub>min</sub>) is the minimum value observed when computing the reassigned version of the Wigner-Ville time-frequency distribution (See chapter 1 for details of the procedure to obtain  $f_{min}$ ). Figure 2.4a shows an example of the transitory fluctuations of resonant frequency during short-term monitoring (i.e. within the duration of one earthquake). The behavior is defined by an initial drop of frequency corresponding with the arrival and strongest portion of the excitation (i.e. from the pre-seismic fundamental frequency  $f_{iapp}$ , to the co-seismic value  $f_{min}$ ), followed by the recovery of frequency from f<sub>min</sub>. This is a similar behavior to that observed in laboratory experiments (i.e. Rivière et al., 2015 and 2016). Moreover, the dynamic nonlinear elastic behavior observed at the laboratory scale is related to the pattern of loading (i.e. Guyer and Johnson, 1999, 2009). Therefore, we use the duration of the strongest motion (i.e. DSM, chapter 1) as a parameter linking the nonlinear elastic response to the pattern of loading. In engineering seismology, the potential for damage of seismic ground motion is usually associated with the DSM, containing the maximal loading energy and much shorter in time than the total duration. The most classical definition is given by Trifunac and Brady (1975), that is, DSM corresponds to the time between 5% and 95% of the total cumulative energy. Two additional frequencies are therefore also considered (Fig. 2.4a), equivalent to f<sub>95</sub> and f<sub>99</sub> and corresponding to the time at 95% and 99.9% of the total signal energy, respectively. The total energy of the signal is computed by the Arias intensity (Ia) function (Arias, 1970), explained in the chapter 1. An example of the cumulative energy is given in Fig. 2.4c. Note that because of the triggered nature of the data we call 'apparent' values the initial and final frequencies ( $f_{iapp}$  and  $f_{fapp}$ , respectively) to indicate initial and final values observed in the triggered earthquake window. It is noticeable that the first part of the recovery process is faster, and falls within the DSM region (i.e. 'Segment 1' in figure 2.4a). Therefore, it is likely affected by conditioning/loading effects. On the other hand, we make the hypothesis that the recovery after  $f_{95}$  is not affected by on-going loading, but rather is related to the state of the structure. This second part of the recovery is slower than the previous one. The slow dynamics is studied in detailed in the chapter 3, by analyzing the 'Segment 2' (i.e. from  $f_{95}$  to  $f_{fapp}$ ).



**Figure 2.4** Example of the analysis applied to the recordings, corresponding to a moderate earthquake (15 March 2011) recorded by the sensor 8FE (X direction of the ANX building). a) Time-frequency representation obtained by the reassigned smoothed-pseudo Wigner-Ville distribution (rsPwv, chapter 1). Two segments are observed in the recovery: segment 1, between  $f_{min}$  (minimum value of fundamental frequency) and  $f_{95}$  (fundamental frequency estimated at 95% of the earthquake energy) and segment 2, from  $f_{95}$  to  $f_{99app}$  (i.e. apparent final fundamental frequency, at 99.9% of the earthquake energy). The first vertical line (from left to right) corresponds to the time of the first arrival wave (i.e.  $f_{iapp}$ ). The other vertical dashed lines correspond to the limits of segments 1 and 2, between  $f_{min}$  and  $f_{95}$  and  $f_{99app}$ . The solid red line is the Savitsky–Golay smoothing function applied to the maximum energy values (scale min–max at the right). b) Time history of acceleration corresponding to the rsPWV distribution shown in (a). c) Cumulative energy given by the Arias intensity (Ia) distribution, with reported characteristic values of energy and the position of the duration of strongest motion (DSM). The first vertical dashed line (from left to right) corresponds to the time of the 5% of the Ia.

#### 2.4. Variation of the resonance frequency

By computing the time-frequency distribution for each strong motion, we are able to monitor the variations of frequency with time (i.e. over years), and to correlate them with EDPs such as PTA or  $\Delta$ . Figure 2.5 shows the evolution of the minimum value of fundamental frequency

(fmin) in both directions of the ANX building, recorded over 20 years of measurements, assuming that f<sub>min</sub> is related to the instantaneous variation of the stiffness for equivalent input loading. Between 1998 and ~2005 the frequency decreases progressively in both directions of the building. The fundamental frequency values shifted from 1.85Hz (+/- 0.04Hz) to 1.39Hz (+/- 0.06Hz) in the direction X-270, representing a mean drop of approximately 25%. In Y-180 the drop is about 16.5%, i.e. from 1.63Hz (+/- 0.08Hz) to 1.36Hz (+/- 0.06Hz). Note that the drop of frequency was more prominent in the X-270 direction. During this long-term seismic sequence, the frequency decrease for equivalent loading reflects the immediate post-built coseismic and slow softening of the structure resulting from the increasing number of cracks opening and closing during each earthquake: the building was built in 1998, therefore the appearance of new cracks was very susceptible to loading during this period. In the same manner as for the following years, the dispersion of the frequency value for a given date is governed by the loading value, the smallest frequency values corresponding to the strongest PTA (i.e. colorscale). In October 2004, a sharp decrease of frequency is observed, coincident with the Mw. 6.9 Niigata-Chūetsu earthquake, followed by a rapid recovery up to a somewhat flat span between 2005 and 2011, with frequency equal to 1.33Hz in both directions, +/- 0.06Hz in Y-180 and +/- 0.14Hz in X-270: the system reached a state of equilibrium (~8 years after its construction), softer than in the previous years, reflecting a period during which the number of cracks remained stable. Frequency thus remains stable for a given level of shaking. Next, a sharp decrease in frequency is observed in 2011 (up to ~0.78Hz), during the Mw 9.0 Tohoku earthquake. This earthquake caused further softening because of the creation of new and wider permanent cracks, as reported by occupants (T.K. Kashima, personal comm., 2015) and postearthquake observations (Kashima, 2014), and leading to a new state of the structural elements. Thus, a slight tendency to recover is observed after this earthquake, which may be linked to slow dynamic effects. Despite a partial recovery immediately after the 2011 earthquake, seven years after the event the fundamental frequency did not reach the value measured prior to the earthquake. In 2018, the fundamental frequency of ANX building was stable around 1.06Hz (+/- 0.08Hz) and 1.04Hz (+/- 0.07Hz) in the Y-180 and X-270 directions, respectively, which represents a 35% and 43% drop relative to the initial elastic frequencies.



**Figure 2.5** Variation of the minimum value of the fundamental frequency  $f_{min}$ , in the ANX building during the period 1998-2018, obtained for each earthquake in the a) Y-180 direction b) X-270 direction. The color bar represents the peak acceleration at the top of the building, PTA (note the log scale). The vertical dashed lines represent the limits of the periods discussed in the text: 1998~2005, 2005-March2011, March-September 2011, After October 2011. The white symbols correspond to the average computed by bins of 20 consecutive events. Error bars represent one standard deviation of frequency values.

The long-term recovery of the ANX building after the Tohoku earthquake is revealed when the main event and some of its aftershocks caused significant transient changes in the elastic properties of the building. This long-term recovery occurs over approximately six months (i.e. shaded area in Fig. 2.5). Previous studies have concluded that elastic properties are determined by the bond system of the material (i.e. Guyer and Johnson, 1999, 2009; Van Den Abeele et al., 2000b; Ostrovsky and Johnson, 2001). Such a system is composed of asperities and cracks in the interfaces between rigid particles, given that they are heterogeneous in shape and size. As a result, the material shows nonlinear elastic behavior. This response manifests itself in a variety of ways, including resonant frequency shifts at low levels of strain and slow dynamics, as well as other effects (Guyer and Johnson, 1999; Ostrovsky and Johnson, 2001). Slow dynamic effects appear in nonlinear elastic materials after the excitation (Guyer and Johnson, 1999), as observed at short- and long-term earthquake monitoring (i.e. Fig. 2.4 and Fig. 2.5, respectively). Although this behavior has not been studied in structures in any detail, it may be related to the presence of weaknesses and/or heterogeneities (e.g. cracks, dislocations, etc.) and to the structural health of the buildings, in the same manner as in rocks and soils. The following chapter (Chapter 3) is dedicated to the analysis of slow dynamics in buildings.

#### 2.5. Discussion of the origin of the nonlinearities

The long-term variation of the response of the ANX building, including the long-term recovery process after the Tohoku earthquake, is shown in Fig. 2.5. In this figure, we considered the fundamental frequency computed at the top of the building, that is, representing the soilstructure system. In this case, the origin of the variation is governed either by the response of the structure or the response of the soil, or both the building and soil system together, that is, influencing the soil-structure interaction. Several authors have shown the nonlinear response of the soil under strong shaking by directly measuring the change of the seismic ground motion (i.e. Field et al., 1997; Johnson et al., 2009) or the reduction of the site's resonance frequency (Wu et al., 2009a; Bonilla et al., 2011) or shear wave velocity (Chandra et al., 2015, 2016), characterized by a medium-to-long term recovery process of the elastic properties of the soil (Wu et al., 2009a). In Japan, after the Tohoku earthquake, whereas Bonilla et al. (2011) observed very long recovery of the central frequency of the site corresponding to the response of the uppermost soil layer at the accelerometric station, Brenguier et al. (2014) reported long recovery of the pre-Tohoku wave velocity of the crust at low frequency. In both cases, the authors associated the recovery rate with the presence of heterogeneities in the soil layer and the fractured state of the crust, respectively.

In the case of the ANX building, we compared the resonance frequency of the soil-structure system (Fig. 2.5) with the response of the equivalent fixed-base structure and the site response (Fig. 2.6). First, the value of the fixed-base frequency ( $f_1$ ) can be obtained by deconvolution between the recordings from the top and bottom, thus removing the soil-structure interaction (Snieder and Safak 2006, Todorovska 2009; Michel et al., 2010). In our case, deconvolution was obtained using the water-level regularization technique (Clayton and Wiggins, 1976). Knowing the input and output signals recorded at the bottom and top of the structure, respectively, the transfer function H( $\omega$ ) corresponding to the fixed-base structure is computed as follows:

$$H(\omega) = \frac{O(\omega)}{\max\left\{I(\omega), k\left(|I(\omega)|, \frac{I(\omega)}{|I(\omega)|}\right)_{max}\right\}}$$
(2.1)

where *O* and *I* are the output and input signals, respectively, and *k* is the water level coefficient (k=0.10 in our case). We choose k=0.1 because we found in Guéguen et al. (2016) and Michel and Guéguen (2018) this is the smallest value of the k parameter that gives stable deconvolved

wave in buildings. The resonance frequency of the structure is then obtained by picking the value of the first peak of the transfer function.

In the same manner, the shear wave velocity Vs of the uppermost layer characterizes the site response. As applied by several authors (Mehta et al., 2007; Nakata and Snieder, 2012; Chandra et al., 2015, 2016), Vs is obtained by taking the time delay of the impulse wave obtained between two sensors throughout the depth of the borehole using seismic interferometry by deconvolution. This consists in computing the inverse Fourier transform of  $H(\omega)$  (Eq. 2.1). This solution was successfully applied to vertical arrays in order to analyze the nonlinear response of the soil under strong shaking (Chandra et al., 2015, 2016). Herein, only the first 14m deep layer, between sensors A01 and A14 (Fig. 2.1), was analyzed, considering only the superficial nonlinear effect and topmost soil-structure interaction.

Figure 2.6 shows the response of the structure system considering the pair of stations 8FE and BFE in the X and Y directions, and the site response since 1998 until 2014, given that the borehole data were available until that year. First, we observe a clear effect of the soil-structure interaction that softens the soil-structure system response (Fig. 2.5). This interaction is the more marked because the ground is flexible. In our case, shear-wave velocity (Vs) is 188m/s (+/-3m/s). The frequency variation of the structure (Fig. 2.6a and 2.6b) has the same pattern as the variation of the soil-structure system, confirming that the degradation observed between 1998 and ~2005 and between 2005 and 2011 is indeed of structural origin, reflecting the degradation of the structure health during this long seismic sequence. Over the same periods, the velocities Vs are almost stable, with slight fluctuation around the mean value (Fig. 2.6c). After Tohoku, there is a comparable variation in the response of the structure and the soil-structure system, characterized by slow recovery after Tohoku until an asymptotic value lower than that of the pre-Tohoku period. This residual frequency variation is a proxy of the degree of damage to the structure. In the soil, variation of Vs is also observed, representing the degradation of the shear modulus during the strongest soil strain, characteristic of the nonlinear behavior of the soil. However, this variation is around 5%, well below that the one observed in the structure. In the same way, the characteristic time of recovery for Vs is visibly shorter than that of the resonance frequencies f<sub>1</sub> and f<sub>min</sub>. Again, this observation confirms the structural origin of the slow dynamics observed after Tohoku, in relation to the level of damage (or the degree of fracturing) of the structural elements.



**Figure 2.6** Variation of the structural frequency  $f_1$  (fixed-base) of the ANX building obtained after deconvolution considering the sensor 8FE a) in the direction Y-180 and b) in the direction X-270. c) Variation of the shear-wave velocity (Vs) obtained by seismic interferometry by deconvolution for bins of 20 frequency values using the borehole sensors A01 and A14 (see Fig. 2.1). The colorbar represents the strain proxy computed as the relative displacement between two sensors (see text). Error bars represent one standard deviation of frequency values. The right column shows a zoom-in of each plot at the left, centered over the Tohoku earthquake's sequence (indicated in shaded areas).

One common way to represent the damage level in buildings is to consider the structural drift (or structural strain) as an EDP. Several seismic codes and technical manuals (i.e. Federal Emergency Management Agency [FEMA], 1999; Eurocode 8) provide threshold values for interstory drift, corresponding to the yield and ultimate deformation, equivalent to certain levels of damage. For the ANX building typology, the average value of drift defining slight damage is about  $3x10^{-3}$  (FEMA, 1999). However, Figure 2.7 shows that nonlinear elastic behavior starts at very low levels of deformation, around  $10^{-5}$ , already reported by Guéguen et al. (2016) for buildings of different design. Table 2.1 provides the mean, standard deviation, and coefficient of variation (COV) of the frequencies over the four periods and corresponding to strain values ranging between  $10^{-6}$  and  $5x10^{-6}$ . In figure 2.7, several observations can be made:

1. Over the period 1998~2005, the distribution of frequency  $f_1$  (i.e. the fixed-base structure) for a given deformation is very dispersed. This reflects the period of severe degradation already mentioned, during which the structure is damaged in the same way as a conditioning process. For example, for a deformation in the range [10<sup>-6</sup>, 5x10<sup>-6</sup>], the mean ( $\mu$ ) frequency is around 2.21Hz with a standard deviation ( $\sigma$ ) of 0.20Hz (COV =  $\sigma/\mu$ =8.9%), with a relatively uniform distribution of values. After 2005, the data are

less scattered. For the same range of deformation, the mean frequency is around 1.99Hz with a sigma of 0.11Hz (COV = 5.6%), but the distribution is normal. After Tohoku, this dispersion is reduced even more, with  $\mu$ =1.39Hz;  $\sigma$ =0.09Hz with a COV of 6.6%, following a normal-like distribution. Finally, it is observed that the dispersion of the data increases again for the period following the post-Tohoku reparations ( $\mu$ =1.48Hz;  $\sigma$ =0.12Hz; COV=7.9%). These periods show that the more the structure is damaged, the more stable its response to strain over time. This can be explained by a gradual damaging mechanism, in which the number of cracks increases, and the energy required to reduce the stiffness (i.e. to open the cracks) is less important: after the structural elements have been cracked, the same number of cracks acts for a given deformation. The effects of the level of damage on the variations of nonlinear behavior have been previously reported by Van Den Abeele et al. (2000b) confirming our observations. Obviously, after Tohoku, this degradation is more important, but the dispersion of the data is reduced.

- 2. The soil-structure system exhibits more or less similar behavior to the fixed-base structure, confirming the predominance of the response of the structure in the response of the observed system, and minimizing in this case the effect of the soil for levels of deformation below 10<sup>-5</sup>. Moreover, Fig. 2.7c shows that the soil behaves in a linear manner up to this deformation value. Beyond about 10<sup>-5</sup>, the soil degrades, giving a nonlinear curvature to the strain-Vs data. It is also found that the soil recovers its elastic property (Vs) after the Tohoku earthquake sequence.
- 3. Finally, it can be seen that, beyond  $10^{-5}$ , there is a difference between figures 2.7a and 2.7b, that is, between the response of the structure and the soil-structure system. An inclination of the curve is visible for  $f_{min}$ , indicating that around the deformation value  $10^{-5}$ , non-linearity in the soil contributes to the frequency reduction of the soil-structure system. Although this contribution is less than that of the structure, it is not negligible and reflects the non-linearity of the soil-structure interaction under strong solicitation.



**Figure 2.7** Strain proxy versus the variation of the resonance frequency of a) the fixed-base structure,  $f_1$ , in the Y-direction, b) the soil-structure system,  $f_{min}$ . The distribution of the frequencies for the four periods are given (before 2005, 2005-Mar.2011, Mar.2011-Sept.2011, and Oct.2011-2014) corresponding to strain values ranging between  $10^{-6}$  and  $5 \times 10^{-6}$  (corresponding to the vertical dashed lines). c) Strain proxy versus the variation of the shear-wave velocity (Vs) in the GL0m-GL14m uppermost soil layer (GL: ground level). The colorscale represents the date (years).

**Table 2.1.** Mean values ( $\mu$ ), Standard Deviation ( $\sigma$ ) and Coefficient of Variation (COV=  $\sigma/\mu$ ) of f<sub>min</sub> and f<sub>1</sub> in the Y direction corresponding to strain values ranging between 10<sup>-6</sup> and 5x10<sup>-6</sup>.

		$f_1$			$\mathbf{f}_{min}$	
	μ	σ	COV	μ	σ	COV
	(Hz)	(Hz)	(%)	(Hz)	(Hz)	(%)
1998-2005	2.21	0.20	8.9	1.50	0.11	7.3
2005-2011	1.99	0.11	5.6	1.34	0.04	2.8
Mar-Oct 2011	1.39	0.09	6.6	1.13	0.08	6.8
Oct 2011-2014	1.48	0.12	7.9	1.14	0.08	7.4

#### 2.6. Observations in another building: the case of the THU building

The THU building is located in Tohoku, Japan. It is a 9-story, steel-reinforced concrete structure, irregular in the vertical direction (i.e. setback at the third floor, Fig. 2.8). It serves as a faculty department of the University of Tohoku. The structure was built in 1969 and significantly repaired and retrofitted in 2000-2001 (Celebi et al., 2012). Since the retrofit, the building faced several significant earthquakes (i.e. the Mw 8.0 Tokachi Oki in 2003, the Mw

7.2 Miyagi in 2005, the Mw 6.9 Iwate event in 2008, and the Mw.9.0 Tohoku event in 2011) causing considerably high accelerations and drift values (Fig. 2.9). The great earthquake in 2011 occurred 177 km away, triggering a PTA equal to 908.24cm/s<sup>2</sup> and a maximum drift of  $1.06 \times 10^{-2}$ , which is close the threshold defined for severe damage in this building typology (i.e. 1.17 x10<sup>-2</sup>, according to FEMA, 1999; EC8). Significant variations of the fundamental period of the structure were observed after the 2011 event, together with numerous damages reported by Kashima et al., 2012 and Celebi et al., 2012 during post-seismic observations. This included four columns heavily crushed and severe cracking in shear walls. The loading history of the THU building and its dynamic behavior has been analyzed by Motosaka et al., 2004; Motosaka and Mitsuji, 2012 and Kashima et al., 2012. The high excitations faced by this structure are evident in Fig. 2.9c and Fig. 2.10. The THU shows double PTAs, as well as drift values that are one order of magnitude higher than those in the ANX building. The plan and section views of the THU building are shown in figure 2.8, indicating the pair of sensors used in this study, located at the first (i.e. 01F) and the top floor (i.e. 09F). The geographical location of the building is shown in Fig. 2.9a, the magnitude-distance distribution of the events analyzed is shown in Fig. 2.9b, and the PBA-drift relationship is displayed in Fig. 2.9c.



**Figure 2.8** Picture, plan and section views of the THU building. The triangles represent the 2 horizontal components of the accelerometric sensors used in this study,  $(01F = \text{first floor}; 09F = 9^{\text{th}} \text{Floor})$ . Figure simplified from Kashima (2014).

We used 317 earthquakes recorded from 2003 to 2012 to monitor the frequency variations as a function of time and EDPs (Fig. 2.10). Sharp frequency drops and slow dynamics are pretty well observed during the aftershock's sequences following strong earthquakes, notably in 2005, 2008 and 2011 (i.e. dashed lines in Fig. 2.10). As well as the THU, all the buildings included in the database introduced in the chapter 1, show evidence of nonlinear elasticity, proving that this response is manifested independently on building typology, loading amplitude, soil and environmental conditions. The recovery of slow dynamics is observed from small to very strong excitations, from seconds to years, confirming the universality and multiscale feature of this

behavior, already observed by several previous studies (i.e. Johnson and Sutin, 2005; Brengier et al., 2008; Guyer and Johnson, 2009; Averbakh et al., 2009; Guéguen et al., 2016).



**Figure 2.9** a) Epicenter location of the earthquakes used in this chapter and position of the THU building. The star indicates the epicenter of the Tohoku Mw 9.0 earthquake. b) Magnitude (JMA) versus the epicentral distance of the earthquakes with respect to the THU building. c) Peak value of acceleration at the bottom of the THU building, PBA, versus maximum structural deformation  $\Delta_{max}$ , computed for the building during each earthquake (using the 01F and the 09F sensors).



**Figure 2.10** Variation of the minimum value of the fundamental frequency,  $f_{min}$ , of the THU building during the period 2003-2012, obtained for each earthquake in one of the horizontal directions of the structure. The color bar represents the peak acceleration at the top of the building, PTA (note the log scale). The vertical dashed lines represent the time of occurrence of strong earthquakes. Notably dynamics are observe following these significant events.

#### 2.7. Conclusions

This chapter presents an analysis of low-to-high amplitude earthquake records of 1630 events recorded in the ANX building in Japan over a period of about 20 years (1998–2018). The earthquake recordings were analyzed by applying a time-frequency distribution based on Wigner-Ville function to estimate the variation with time of the building's resonance frequency. The system frequency (soil-building) was estimated as the minimum frequency of the distribution, and the fixed-base building response was estimated by deconvolution between the top and bottom, and the nearby site response by seismic interferometry between depths of 0m and 14m. The main goals of this chapter were (1) to confirm the presence of nonlinear elasticity

in real buildings (2) to improve our current knowledge of the variability of system and fixedbase frequency during earthquakes associated with damage, and (3) to contribute to the usefulness of the resonance frequency-based methods for structural health monitoring after strong earthquakes.

The ANX building shows the clear signature of a nonlinear elastic response, in the manner of what is observed in laboratory experiments and in the Earth's crust. The long- and short-term variations of the fundamental frequency of the soil-structure system are observed even for very low levels of strain. Moreover, a slow dynamic phenomenon is also present in the ANX building, characterized by the recovery of the fundamental frequency to previous values at the end of loading. Slow dynamics are observed during the Tohoku aftershock sequence, during which the building is conditioned by successive loadings. In previous works, authors proposed that changes in the resonance frequency measured using ambient vibration immediately after earthquakes could be used as an effective and easy way to indicate damage in buildings. In the case of the ANX building after the Tohoku earthquake, the soil-structure system frequency recovered from ~0.8Hz to ~1.05Hz, ultimately reducing the level of damage characterized by the frequency drop alone. In a real-time health monitoring system, recovery must be integrated in the decision-making phase. It is also important to note that the nonlinear response observed in the structure system is mainly controlled by the structural elements, that is, with a slight but nonnegligible contribution of the nonlinear soil response. In the total system response, the soil's contribution to the strain-frequency relationships is visible for the largest strain value.

Based on previous studies, we also note that such a frequency-based decision applied to buildings would have resulted in the wrong decisions for the ANX building. For example, Dunand et al. (2004) and Vidal et al. (2014) have shown that slightly damaged buildings are characterized by a frequency drop of less than 30%, corresponding to buildings classified green by post-emergency visual screening. Trifunac et al. (2010) also concluded that a resonance frequency drop of more than 30% may not be systematically associated with structural damage. In the ANX building, during the first two periods, system frequencies show aprox. 20% of reduction, without damage being reported. However, the equivalent bond system is cracked during this period, corresponding to a constant damage process, which is more evident in the X-270 direction of the building. Moreover, seven years after the Mw 9.0 Tohoku earthquake, the fundamental frequency seems to be stable around 1.05Hz in both horizontal directions, that is, a permanent frequency drop of about 65% and 57%, respectively, since the construction of the structure. Despite this high value, the building is operative and still occupied.

We conclude that analyzing weak-to-strong earthquake recordings in real scale buildings can provide key information about the seismic response of structures. Nonlinear elastic response is a process that must be better understood to improve our knowledge of the dynamics of civil engineering structures. The only way the earthquake engineering and engineering seismology community can improve seismic response predictions for structures is using real data from actual structures experiencing earthquakes. Such information is also essential for the development and calibration of realistic and effective models or empirical models, useful for predicting the response of real-scale buildings as well as for calibrating the decision-tree tools associated with structural health monitoring. Most structural health monitoring algorithms are based on detecting changes relative to initial conditions, and their efficiency is highly dependent on the knowledge we have of the variation of observable values with respect to the health variation. These algorithms can be improved by including nonlinear elasticity or by integrating a full nonlinear modal identification, integrating higher modes in the process. Additional works should also include the use of vertical arrays in buildings (i.e., considering sensors at different levels) to localize the maximal deformation and to isolate different sections in the building by deconvolution between sensors, as well as to determine the maximal strain-stress relationships, a key information for structural health monitoring application.

# 3

# SLOW DYNAMICS (RECOVERY) USED AS A PROXY FOR SEISMIC Structural Health Monitoring

In this chapter we adapt relaxation models developed in laboratory to the recovery of fundamental frequency in buildings after earthquakes. Slow dynamics recovery is studied during short-term and long-term frequency variations, as well as during conditioning cycles caused by aftershocks sequences. Relaxation parameters computed from the models are monitored over time, and their behavior before and after damaging earthquakes is analyzed to make the link with structural damage. The content of sections 3.1 to 3.5 is published in *Structural Health Monitoring*. The section 3.6 is under review in *Engineering Research Express*.

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Astorga A. and Guéguen, P. (2019). Seismic structural health from drop and recovery of resonance frequencies in buildings. *Engineering Research Express*. Under review.

## Abstract

Elastic properties of civil engineering structures change when subjected to a dynamic excitation like earthquakes. The modal frequencies show a rapid decrease followed by a relaxation, or slow recovery, that is dependent on the level of damage. In this chapter, we analyze the slow recovery process applying relaxation models to fit real earthquake data recorded in a Japanese building (ANX) that shows variant structural state over 20 years. Despite the differences in conditions, the different scales and the complexity of a real-scale problem, the models originally developed for laboratory experiments are well adapted to real building data. The relaxation parameters (i.e. frequency variation, recovery slope, characteristic times and their amplitudes, and range of relaxation times, proxies of elasticity) are able to characterize the structural state, given their clear connection to the degree of fracturing and mechanical damage to the building.

The recovery process following strong seismic deformation, could, therefore, be a suitable proxy to monitor structural health.

**Keywords**: Slow dynamics, earthquakes, nonlinear elasticity, recovery, resonance frequency, cracks, structural health monitoring.

#### **3.1. Introduction**

Granular consolidated materials such as rock and concrete display unusual nonlinear behavior when subjected to dynamic loadings. For instance, a transient drop in elastic modulus - assessed through changes in resonance frequency or wavespeed - can be observed even at very low (dynamic) strain, order 10<sup>-6</sup>. After the loading, the modulus recovers to its original or new equilibrium state at rates that can last from a few seconds to several days, many months, and years (i.e. Guyer and Johnson, 2009; Wu et al., 2009a, 2009b). This recovery effect, or slow dynamics (i.e. TenCate and Shankland, 1996) is a phenomenon associated with slow relaxation of the elastic properties once the dynamic disturbance terminates. This nonlinear elastic response is considered a universal behavior, given that slow dynamics effects have been observed in a large variety of rocks and geomaterials, and in scales ranging from laboratory tests (i.e. Johnson and Rasalofosaon, 1996; TenCate et al., 2000; Ostrovsky and Johnson, 2001; Johnson and Sutin, 2005; Guyer and Johnson, 2009; Shokouhi et al., 2017a) to seismological observations at the surface and crust of the Earth (Rubinstein and Beroza, 2004; Peng and Ben-Zion, 2006; Brenguier et al., 2008; Wu et al., 2009a, 2009b; Johnson et al., 2009; Renaud et al., 2014) where the recovery process can be observed for years. It is also independent of the material or method used (i.e. Shokouhi et al., 2017a).

Analogously, atypical nonlinear response was also observed in civil engineering structures. Guéguen et al., (2016) observed a rapid decrease in resonance frequency of the UCLA Factor building (California) under seismic and environmental loadings, followed by a slow recovery to the initial elastic properties. Astorga et al., (2018) analyzed nonlinear elasticity in the Annex (ANX) building (Japan) throughout variations of its fundamental frequency during a long sequence of earthquakes. They reported recovery effects at short and long-term monitoring and found that the recovery rate is linked to the structural health. Although the underlying physical mechanisms of the slow dynamics are not fully understood, previous results suggest a single origin based on internal strains related to the material damage (i.e. TenCate et al., 2000). What Guyer and Johnson (1999) first called the bond system (i.e. structure of defects, inter-grain contacts, dislocations, cracks at different scales, etc.) is believed to play a fundamental role in

the recovery effect (i.e. Ostrovsky and Johnson, 2001; Baisch and Bokelmann, 2001; Brenguier et al., 2008; Tremblay et al., 2010; Korobov et al., 2013; Guéguen et al., 2016; Astorga et al., 2018).

Logarithmic-time dependence of the recovery was observed in several laboratory studies (i.e. TenCate et al., 2000; Vakhnenko et al., 2004; Tremblay et al., 2010). This behavior was also observed in the relaxation of fractured fault zone materials after earthquakes (Wu et al., 2009a, 2009b; Rubinstein and Beroza, 2004; Peng and Ben-Zion, 2006). Shokouhi et al., 2017a describe the recovery of slow dynamics with a relaxation spectrum that quantifies different relaxation dependencies occurring during the recovery process, meaning that at early time, the recovery is not logarithmic. Snieder et al., (2017) proposed a relaxation model that predicts non-logarithmic behavior at early and late times, which allows us to estimate the initial and the end times of the process, with a log-linear tendency. Ostrovsky et al., (2019) suggested another exponential model to represent the relaxation process, capturing relaxation mechanisms from the very beginning of the recovery.

In this chapter, we apply the relaxation models to real earthquake data collected in a Japanese building over a 20-year period. Apart from testing the models with in situ data, we compare the relaxation parameters and the level of loading and structural state, intending to find the evidence for a nexus between the extension of the bond system and the relaxation process after earthquakes that could be used to infer the structural health of buildings. In the following, we first describe the applied models. The recovery of the slow dynamics is analyzed throughout the evolution of the fundamental frequency with time, that is, a proxy of the elastic properties. Relaxation parameters are then studied as a function of engineering demand parameters (i.e. peak top acceleration -PTA- and structural drift,  $\Delta$ ), and results are discussed in connection to structural damage. In the section 3.6 we study slow dynamics effects during sequences of earthquakes, as well as during conditioning cycles, in the long-term monitoring of two buildings: the ANX and the THU, comparing different levels of damage. Finally, we present some conclusions.

### 3.2. Methodology

The methodology followed to analyze slow dynamics is described in the chapter 2 (section 2.3 'Data processing'). In short, the time-frequency distribution was applied to each earthquake recording, and the recovery of fundamental frequency  $f_{min}$ , was analyzed -as a function of time-from the end of the strongest motion (after the 95% of the earthquake energy, i.e. 'Segment 2'

in Fig. 2.4). Relaxation models were adapted to this segment, under the hypothesis that this portion is not governed by on-going loading effects but rather is related to the state of the structure.

### 3.3. Relaxation models

There is not currently a model able to fully describe slow dynamic effects. This phenomenon deserves special attention because the physical origin remains unknown, and it generally comes together with hysteresis and discrete memory (Guyer et al., 1999; Guyer and Johnson, 2009; Lebedev and Ostrovsky, 2014), which add more complexity to the modeling and interpretation. Slow dynamics effects can be present for several orders of magnitude in time, and therefore, several authors have described the recovery of elastic properties with a log-linear function of the form  $y=b \log(x)+c$  (TenCate, 2000). This function describes most part of the recovery, including the main range of mechanisms acting in the process. However, the log-linear model is based purely on observations, with no physical explanation. Besides, this model does not converge at the earliest and latest relaxation times, preventing us from obtaining important information regarding the beginning and the final portion of the recovery.

#### 3.3.1. Relaxation function

(Snieder et al., 2017) consider that the macroscopic relaxation of materials is a combination of different relaxation mechanisms that take place on different temporal and spatial scales. They proposed a *relaxation function* to describe this multi-scale phenomenon, assuming that the total relaxation is a superposition of decaying exponentials. The function depends on two parameters, the minimum and the maximum relaxation times (i.e.  $\tau_{min}$  and  $\tau_{max}$ , respectively) between which the relaxation mechanisms are distributed and display log-time behavior (i.e. linear slope in log-time). The authors define the relaxation function, R(t), as a perturbation to a physical observable O(t) of a system:

$$O(t) = O_0 (1 + SR(t))$$
(3.1)

with  $O_0$  the equilibrium value of O, and S a scale factor. The observable might be, for example, the seismic velocity, the material density, the elastic modulus (such as the fundamental frequency, in this study), etc. The total relaxation is given by the following superposition of relaxation processes:

$$R(t) = \int_{\tau_{min}}^{\tau_{max}} \frac{1}{\tau} e^{-t/\tau} d\tau$$
(3.2)

The expression (3.2) must be scaled with (3.1) to describe time-dependent material properties. The weight factor  $1/\tau$  is explained by the Arrhenius' law: assuming a relaxation process with activation energy *E* (i.e. the minimum energy required to start a chemical reaction), the corresponding relaxation time is given by:

$$\tau = A \exp\left(\frac{E}{k_B T}\right) \tag{3.3}$$

where A is a constant, *T* is the absolute temperature and  $k_B$  is the Boltzmann constant (Snieder et al., 2017). Suppose the activation energy has a density of states N(E) meaning that the number of activation mechanisms between *E* and *E*+*dE* is equal to N(E)dE. The density of states  $P(\tau)$  for the relaxation times satisfies  $P(\tau) = N(E)dE/d\tau$ . Using the expression (3.3)  $d\tau/dE = (A/k_BT)\exp(E/k_BT) = \tau/k_BT$ , hence:

$$P(\tau) = \frac{k_B T}{\tau} N(E)$$
(3.4)

N(E) is constant when the distribution of the activation energy is uniform between a minimum  $E_{min}$  and a maximum  $E_{max}$ , and according to expression (3.4) the density of states for a relaxation process with relaxation time,  $\tau$ , is for a fixed temperature, T, proportional to  $1/\tau$  (Snieder et al., 2017). Following the expression (3.3), the minimum and maximum relaxation times are related to the minimum and maximum activation energies, by:

$$\tau_{min} = A \exp^{\left(\frac{E_{min}}{k_B T}\right)} \quad , \quad \tau_{max} = A \exp^{\left(\frac{E_{max}}{k_B T}\right)} \tag{3.5}$$

Thus,  $\tau_{min}$  indicates the time at which the relaxation process begins, controlled by the relaxation mechanisms on the smallest spatial scale that require less energy. On the other hand,  $\tau_{max}$  is associated to the time at which the relaxation stops (i.e. when an equilibrium between external and internal stresses is reached; beyond this time no relaxation mechanism contributes).  $\tau_{max}$  depends on the perturbation that triggers the relaxation and ambient conditions (Snieder et al., 2017). Consequently, analyzing the dependence of  $\tau_{max}$  as a function of these factors might be potentially useful to diagnose healing mechanisms.

The relaxation function can be obtained by numerically evaluating the integral of expression (3.2). The minimum relaxation time  $\tau_{min}$  can be estimated from a relaxation curve, given that the transition to the logarithmic dependence (i.e. beginning of relaxation) occurs at  $t \approx \tau_{min}$ . However, previous laboratory experiments (Lobkis and Weaver, 2009) have noted that the log-time recovery can start at very early times, to the point that in some cases it is not possible to obtain reliable values of  $\tau_{min}$  (i.e., a much greater time resolution would be needed). On the

other hand, the maximum relaxation time  $\tau_{max}$  can be inferred by estimating the point where the curve flattens off. But again, the final transition to a non log-time behavior often occurs after several minutes or hours (TenCate, 2011), and might not be captured by non-continuous monitored systems.

To test the relaxation function (Eq. 3.2) Snieder et al. (2017) presented a model of pillars describing the closing of a fracture. Although this model does not satisfy the Arrhenius' law, it follows the function pretty well, so that the relaxation function might be applicable to different models. Similarly, no thermally activated processes are included in the present study. However, Guéguen et al., (2019) showed that the relaxation function (Eq. 3.2) describes building data reasonably well too.

Figure 3.1 shows an example of the relaxation function applied to the data (i.e. time-frequency variation in the segment 2 of Fig. 2.4, chapter 2) of one earthquake recorded at the top of the ANX building. Values of  $\tau_{min}$  and  $\tau_{max}$  are 1.70s and 144.6s respectively, representing the beginning and end of the relaxation process for this example. At intermediate times (i.e. between  $\tau_{min}$  and  $\tau_{max}$ ) the relaxation function follows a log-time tendency, where the estimated slope of recovery p = 0.03, as shown in Fig. 3.1b. This slope was computed by fitting a first-degree polynomial in the data between  $\tau_{min}$  and  $\tau_{max}$ . Y-axis in Fig. 3.1 represents the frequency recovery normalized with respect to the final apparent value,  $f_{fapp}$ . X-axis corresponds to time, with a logarithmic scale.



**Figure 3.1** Relaxation function following one earthquake registered in the ANX building and corresponding to the segment 2 in Fig. 3.4. The solid line is the fit of relaxation. The estimated minimum and maximum relaxation times are indicated with vertical dashed lines ( $\tau_{min} = 1.70s$  and  $\tau_{max} = 144.6s$ ). b) Log-linear fitting of the data between  $\tau_{min}$  and  $\tau_{max}$ , with slope p=0.03 and Df=0.07. Y-axis is the fundamental frequency normalized to the final value ( $f_{fapp}$ ). Note the logarithmic scale on the X-axis.

Note that it is challenging in Fig. 3.1 to precisely determine when the curve flattens off at late times and therefore estimate  $\tau_{max}$ . This is again due to the use of triggered windows that do not allow us to monitor the behavior of the relaxation continuously in time and prevent us from determining  $\tau_{max}$  unequivocally in some cases.

Previous results suggest that the slope of the log-time segment actually changes under different circumstances that can be related to the type of material, environmental conditions, level of loading, or damage (TenCate et al., 2000; Ostrovsky and Johnson, 2001; Johnson and Sutin, 2005; Shokouhi et al., 2017a; Astorga et al., 2018; Guéguen et al., 2019). In addition to p,  $\tau_{min}$  and  $\tau_{max}$ , the maximum transitory variation of fundamental frequency, Df, was computed with respect to the final apparent frequency,  $f_{fapp}$ . It has been observed that the variation of elastic parameters is strongly linked to the loading amplitude (Shokouhi et al., 2017a). Likewise, we notice a solid correlation in our data between Df and PTA and  $\Delta Max$  values (Fig. 3.2). To remove the effects of loading on the variation of frequency, we normalized Df to the maximum drift,  $\Delta Max$ . In this way, the computation of p is not conditioned to loading amplitudes.



**Figure 3.2** Correlation between the fundamental frequency variation *Df*, and maximum values drift  $\Delta Max$ , and acceleration, *PTA*.

#### 3.3.2. Relaxation time spectrum

Shokouhi, et al., (2017a) introduced the concept of *relaxation time spectrum* to quantify the recovery process in consolidated granular systems, measuring speed. The authors assumed the recovery to be represented as a sum of discrete exponential decays each having an amplitude  $A_n$  and time constant  $\tau_n$ . In this way, complementary to the model of Snieder et al., (2017), the relaxation time spectrum characterizes the recovery process over several orders of magnitude in time. It is given by:

$$\zeta(t) = \sum_{n=0}^{m \times N} A_n e^{\frac{-t}{\tau_n}}, \quad A_n \ge 0$$
(3.6)

where *t* is measured from  $t_0$  (t<sub>0</sub> is the time corresponding to the end of the external strain, i.e. the time corresponding to f<sub>95</sub>). *N* is the number of exponentials used to fit the function (i.e. N=10 in this study). The time constants  $\tau_n$  are assigned a priori and the amplitudes  $A_n$  are determined by minimizing the least squares objective function:

$$\epsilon = \int_{t_0}^{t_{max}} \left[ \frac{\Delta c(t)}{c_0} - \zeta(t) \right]^2 dt$$
(3.7)

where  $\frac{\Delta c(t)}{c_0} = (c(t) - c_0)/c_0$  is the relative shift of wavespeed (equivalent to frequency changes in our study), with  $c_0$  the wavespeed before the beginning of loading, and c(t) is the wavespeed at time t.  $\tau_n$  is chosen such that there are m logarithmically spaced time constants in each decade<sup>10</sup>, as follows:

$$\tau_n = \Delta T b^n, \ b = 10^{\frac{1}{m}}, \ n = 0, ..., m \times N$$
 (3.8)

The result given by Eq. 3.6 is a spectrum of values A (i.e. set of values  $A_0$ ,  $A_1$ ,  $A_2$ , ...,  $A_n$ ). Shokouhi, et al., (2017a) found the relaxation time spectrum to be independent of the amplitude of loading and of the environmental conditions. Hence, the relaxation spectrum can be considered as the signature of the slow dynamics recovery process. Even when the understanding of the connection between the shape of the relaxation spectra and the internal characteristics of the material is missing, this approach has the potential to unravel the underlying physical mechanisms of the slow dynamics (Shokouhi, et al., 2017a).

The main scheme to obtain the relaxation time spectrum is shown in Fig. 3.3 for one earthquake recorded in 2015 at the top of the ANX building. The behavior of the normalized fundamental frequency ( $Df/f_{fapp}$ ) as a function of time is shown in Fig. 3.3a. The set of exponential decays that compose the total relaxation spectrum is shown in Fig. 3.3b and their corresponding amplitudes A<sub>i</sub> are displayed in Fig. 3.3c, composing the relaxation time spectrum. The fitting of all the exponential decays is shown Fig. 3.3a. It is noticeable that mechanisms having relaxation times between  $\tau$ ~2s and  $\tau$ ~100s are present in this example, where the maximum amplitude,  $A_{max}$ , corresponds to relaxation times around 13s (i.e. green dataset in Fig. 3.3c).



**Figure 3.3** General scheme used to obtain the relaxation time spectrum. Example for an earthquake occurring in 2015 and recorded at the top of the ANX building. a) Normalized fundamental frequency  $(Df/f_{fapp})$  as a function of time during the recovery segment, after the strongest loading is finished.  $Df=f_{fapp}$ ; where  $f_i$  is the value of fundamental frequency at time  $t_i$  during the recovery. The circles represent the earthquake data evenly spaced in log-time, and the solid thick line is the fit corresponding to the sum of exponential decays (equation 3.6). b) Exponential decay terms that compose the fit shown in (a). c) Amplitudes  $A_i$  of the different exponential decays observed in (b). These amplitudes compose the relaxation spectrum. Maximum amplitude  $A_{max}$ , characteristic time  $\tau_{c}$ , and bandwidth *bw* are also represented in this example.

In this study we extract the following three parameters from the relaxation time spectrum:

- $A_{max}$  corresponds to the maximum amplitude observed in the relaxation spectrum.
- The characteristic time,  $\tau_c$  corresponds to the centroid, in the time axis, of the spectrum. It was determined using  $\tau_c = \frac{\sum A_i \times \tau_i}{\sum A_i}$ , where  $\tau_i$  and  $A_i$  are the relaxation times and their corresponding amplitudes, respectively, for the different exponentials used in the fitting (Fig. 3.3c).
- *The bandwidth, bw*, assesses the range of dominant relaxation times, as commonly used in signal processing (in the frequency domain typically), to estimate the damping ratio and quality factor of structures for instance (Clough and Penzien, 1993). Here the bandwidth is defined using  $1/\sqrt{2}$  of the maximum amplitude (Fig. 3.3c).

Once applied the relaxation models to fit our data, we are able to obtain the following relaxation parameters, listed herein for summary:

•  $\tau_{min}$  and  $\tau_{max}$ : beginning and end of the relaxation, respectively, they represent energyrelated parameters linked to smallest and largest scale mechanisms (i.e. smallest and largest cracks, respectively). They are obtained by iterative process using the equation 3.2.

- *p*: recovery slope. It is related to the rapidity with which the particles in the material rearrange until reaching an equilibrium state. It is estimated using a first-degree polynomial between τ<sub>min</sub> and τ<sub>max</sub>.
- τ<sub>c</sub>: characteristic time. Typical relaxation time related to the typical size of crack manifested for an earthquake.
- $A_{max}$ : maximum amplitude. It corresponds to the extent of cracks with a certain size.
- *bw*: bandwidth. It corresponds to the extension of the bond system / cracks-size variety.
- Ratio τ<sub>max</sub> / τ<sub>min</sub> can be interpreted as the development of new type/size of cracks, i.e. representing the cracks-size variety.

#### 3.4. Evolution of relaxation parameters over time

Long-term monitoring of the fundamental frequency in both directions of the ANX building is shown in chapter 2, Fig. 2.5. For that building, Astorga et al., (2018) have defined four time periods according to the behavior of the fundamental frequency in time: 1998–2005, 2005 to March 2011, March 2011-September 2011, and after October 2011. From now on, referred to as P1, P2, P3, and P4, respectively. We reproduce the figure in Fig. 3.4, showing the average  $\pm$  standard deviation values per period. Additionally, table 3.1 lists several great earthquakes recorded in the building, which have contributed to the observed response.



**Figure 3.4** Long-term variation of the minimum value of the fundamental frequency in the ANX building a) Y-180 direction b) X-270 direction. The color bar represents the peak acceleration at the top of the building (note the log scale). Error bars indicate the average (white markers) and standard deviation values (vertical black lines) computed by bins of 20 consecutive events. Shaded areas indicated as P1, P2, P3 and P4 refer to the structural periods observed in Astorga et al., (2018) according to the behavior of the frequency variation. Mean and standard deviation values per period are indicated in the text boxes at the right of each plot.
Earthquake	Date	Magnitude	$PTA (cm/s^2)$		$\Delta Max$ (cm/cm)		
		(Mw)	X-270	Y-180	X-270	Y-180	
Niigata-Chūetsu	23-Oct-2004	6.9	79.30	67.75	3.01x10 <sup>-4</sup>	2.11x10 <sup>-4</sup>	
Miyagi	16-Aug-2005	7.2	112.55	74.59	4.48x10 <sup>-4</sup>	2.78x10 <sup>-4</sup>	
Igate-Miyagi	14-Jun-2008	6.9	94.73	90.20	4.03x10 <sup>-4</sup>	3.64x10 <sup>-4</sup>	
Izu Islands	09-Aug-2009	6.6	64.09	32.73	2.57x10 <sup>-4</sup>	1.10x10 <sup>-4</sup>	
Tohoku	11-Mar-2011	9.1	505.18	596.84	0.0027	0.0028	

**Table 3.1** Strong earthquakes recorded in the ANX building.  $\Delta Max$ : maximal total structural drift; *PTA*: Peak Top Acceleration. Both horizontal directions X-270 and Y-180 are reported.

Astorga et al., (2018) explained the response of the structure along these periods and concluded that the origin of such a behavior is mostly related to the evolution of the bond system, that is, extension of cracks causing structural softening, as evidenced by permanent changes in the elastic properties. The response of the ANX building during the periods P1, P2, P3 and P4, can be summarized as follows:

During P1 there was slow structural softening due to the progressive increase of number of cracks during each seismic event. The drop of frequency was more prominent in the X-270 direction, suggesting a preferential distribution of cracks along elements in this direction. At the end of P1 a sharp decrease of frequency was observed, followed by a rapid recovery up to the stable period P2 (i.e. mean frequency is constant and similar in both directions). This sharp decrease is coincident with the Niigata-Chūetsu earthquake that occurred in October 2004 (Table 3.1), which confirms the correlation between the co-seismic variation of resonance frequency and the amount of cracks in structural elements. Similar behavior was observed after the great Tohoku earthquake in March 2011, that is, period P3, where the recovery took longer to reach the stable period P4 because of the long conditioning sequence produced by aftershocks. Seven years after the Tohoku event, the fundamental frequencies did not reach the pre-seismic value. The residual frequency variation indicates a permanent change in the elastic properties and suggests permanent structural damage has occurred.

Data dispersion represented by standard deviation in periods P2 and P4 are smaller than for P1 and P3, respectively. Astorga et al., (2018) found that the more the structure is damaged, the more stable its response to loading over time, characterized by a smaller dispersion (Fig. 3.4). This corresponds to a gradual damaging mechanism in which the number of cracks increases (which reduces the stiffness) and the energy required to open new cracks lessens.

A correlation between the relaxation parameters and the structural state is observed: the mean tendency of each parameter changes from one period to the other. Results for  $\tau_{\min}$ ,  $\tau_{\max}$ , p,  $\tau_{c}$ , *bw* and *A<sub>max</sub>* indicate a sudden increase from P1 to P2 in both directions (Fig. 3.5 and table 3.2). Significant variations are also observed from P2 to P3 and from P3 to P4 (except for  $\tau_{min}$ ). The mean minimum relaxation time  $\tau_{min}$  is 5.6s during the first period in the Y-180 direction. At the beginning of 2005 (i.e. the transition from P1 to P2), the value shifts to 9.9s and does not show significant variation until the end of P3. During P4 the mean value of  $\tau_{min}$  increases to 11.9s. This increase in  $\tau_{min}$  represents a delay in the beginning of the relaxation process. Results indicate that the more cracked the structure, the longer it takes for the relaxation to start. The variations of  $\tau_{min}$  from P1 to P2 and from P3 to P4 (Fig. 3.5a) are undoubtedly related to the transition from unstable periods of constant softening (i.e. P1 and P3) to stable periods (i.e. P2 and P4) where the system is cracked due to the expansion of the bond system during the period immediately before. Same tendency is observed in the X-270 direction, but with larger mean values of  $\tau_{min}$  for all the periods. This is also consistent with the stronger degradation in the X-270 direction with respect to Y-180 observed since the beginning of the measurements in 1998 (Fig. 3.4).



**Figure 3.5** Evolution of the different relaxation parameters as a function of time for the Y-180 (circles) and X-270 (squares) directions of the ANX building. a) Minimum relaxation time  $\tau_{min}$  b) Maximum relaxation time  $\tau_{max}$  c) Ratio  $\tau_{max} / \tau_{min}$  d) Log-time slope *p* of the interval  $\tau_{min} - \tau_{max}$  e) Characteristic relaxation time  $\tau_c$  f) bandwidth *bw* and g) Maximum spectral amplitude *A<sub>max</sub>*. Each dot represents the averaged value of 20 consecutive events. Vertical dashed lines correspond to the occurrence of great earthquakes listed in Tab. 3.1. Shaded areas divide the structural periods observed in Fig. 3.4 (i.e. P1, P2, P3 and P4).

Values of  $\tau_{max}$  are likely linked to the maximum size of cracks open during an earthquake (Guéguen et al., 2019), inasmuch as it represents the relaxation mechanisms operating at the largest scales. The constant increase of the mean value of  $\tau_{max}$  from P1 to P2 and from P2 to P3 (Fig. 3.5b) indicates the emergence of increasingly larger cracks and/or increasing crack density. The decrease of the mean  $\tau_{max}$  from P3 to P4 might be related to some retrofitting carried out in the building after the 2011 Tohoku earthquake (Kashima, 2014). The mean behavior of the maximum relaxation time  $\tau_{max}$  is analogous for both directions of the building, and once again the direction X-270 shows greater values, signifying larger cracks in this direction.

**Table 3.2** Mean values of the relaxation parameters for each structural period and for both directions of the building. Graphic representation of these values is shown in Fig. 3.5.

	$ au_{min}$	(S)	$ au_{max}$	(s)	$ au_{\text{max}}$	/ τ <sub>min</sub>	p	(%)	$ au_{c}$	(S)	bw	(s)	A <sub>max</sub>	· (%)
	Y-180	X-270	Y-180	X-270	Y-180	X-270	Y-180	X-270	Y-180	X-270	Y-180	X-270	Y-180	X-270
P1	5.6	8.2	13.1	17.3	12.8	9.2	1.1	1.2	9.9	12.6	12.5	15.0	3.3	4.6
P2	9.9	17.0	29.2	38.4	12.9	5.4	1.8	1.8	17.2	24.7	23.5	28.4	4.2	6.6
Р3	9.9	17.6	56.7	69.7	15.1	16.1	1.9	2.5	16.5	31.1	21.9	37.0	5.8	6.2
P4	11.9	19.7	34.8	62.8	9.8	14.0	3.1	2.3	19.4	31.6	20.5	34.1	5.9	5.6

Since  $\tau_{min}$  and  $\tau_{max}$  represent the initial and final relaxation times, respectively, the ratio between these values might be directly linked to the emergence of new relaxation mechanisms operating at different relaxation times. This ratio could then be interpreted as the development of new type/size of cracks. Values of  $\tau_{max}/\tau_{min}$  are very dispersed (Fig. 3.5 and 3.6): on one hand, there are earthquakes for which  $\tau_{min}$  and  $\tau_{max}$  are very different, that is, related to the creation of new size of cracks. On the other hand, there are earthquakes that did not create new types of cracks, and therefore  $\tau_{max}/\tau_{min} = 1$ . Guéguen et al., (2019) evaluated the recovery of the fundamental frequency of a building in Ecuador during the Mw 7.8 earthquake that occurred in April 2016. Results showed that, during the recovery after the mainshock, the value of  $\tau_{max}/\tau_{min} > 1$ , suggesting the expansion of the bond system with new size of cracks. However,  $\tau_{max}/\tau_{min}$  was equal to 1 during the relaxation process (when considering a foreshock and an aftershock with moderate shaking), implying that new sizes of cracks were created only during the mainshock.

We observe ratios  $\tau_{max}/\tau_{min} = 1$  or  $\tau_{max}/\tau_{min} > 1$  over the periods P1, P2, P3 and P4, with no apparent relationship with loading parameters (Fig. 3.6) or structural states (Fig. 3.5).

Nevertheless, in Fig. 3.6 we show the sequence of events between 2005 and 2010 and results indicate that a series of earthquakes with  $\tau_{max}/\tau_{min} = 1$  occurs after one or a few consecutive events with  $\tau_{max}/\tau_{min} > 1$ . This may be equivalent to that observed by Guéguen et al., (2019). In Fig. 3.6 the color scale indicates the maximum drift,  $\Delta Max$ . For  $\tau_{max}/\tau_{min} > 1$ , the size of the markers is related to the number of consecutive earthquakes, where  $\tau_{max}/\tau_{min} > 1$  (i.e. bigger markers correspond to a greater amount of successive earthquakes generating new types of cracks). For example, in August-September 2005, we observe three consecutive earthquakes with  $\tau_{\text{max}}/\tau_{\text{min}} > 1$  and mean  $\Delta Max = 10^{-4}$  (yellow dot in the dashed rectangle, Fig. 3.6). Thus, new sizes of cracks were created during these events. These are followed by several earthquakes, where no new sizes of cracks emerged,  $\tau_{max}/\tau_{min} = 1$ . However, we observe that earthquakes with big  $\Delta Max$  might not produce new types of cracks (i.e. yellow dots with  $\tau_{\text{max}}/\tau_{\text{min}} = 1$ , Fig. 3.6), and events with low  $\Delta Max$  could open new types of cracks (i.e. blue dots with  $\tau_{max}/\tau_{min} > 1$ , Fig. 3.6). Hence, there is no obvious relationship between the creation of new sizes of cracks and the maximum drift,  $\Delta Max$ . Much more effort is consequently needed to explain the fact that some earthquakes create new type of cracks and some others do not. However, we observe a decrease of the ratio  $\tau_{max}/\tau_{min}$  with time (Fig. 3.6), which characterizes the tendency toward uniformity of the crack features, generating fewer new cracks when successive earthquakes occur.

We should note that the value of  $\tau_{max}/\tau_{min}$  is sensitive to the segment selected for analyzing slow dynamics. We choose the segment after 95% of the earthquake energy (chapter 2), assuming that we can capture residual cracks that might represent the structural state. If the recovery is analyzed from a different initial point, that is, before the end of the strongest motion, we might observe different values of  $\tau_{max}/\tau_{min}$ , but they would not be representative of the structural state because the structure is still under strong excitation.



**Figure 3.6** Ratio  $\tau_{max}/\tau_{min}$  for the events recorded between 2005 and 2010 in the Y-180 direction of the ANX building. For  $\tau_{max}/\tau_{min} = 1$  (note the log-scale on the Y-axis) each dot represents one event. For  $\tau_{max}/\tau_{min} > 1$  the size of the markers is related to the number of consecutive events where  $\tau_{max}/\tau_{min} > 1$  (shown in the inset legend). The color scale represents the maximum drift. The dashed rectangle encloses three consecutive earthquakes creating new types of cracks, followed by some events where no new types of cracks appeared in the last portion of the recovery.

TenCate et al., (2000) reported different recovery slopes for different materials, including intact and damaged concrete. The authors observed an increase of the recovery slope for damaged concrete with respect to intact concrete, and they interpreted this increase by the fact that the rate of recovery in time is related to the rapidity with which the particles in the material rearrange until reaching an equilibrium state, that corresponds to the end of the recovery. Thus, materials whose bond system is more 'damaged' might be characterized by higher slopes because of the open spaces between the solid particles to be filled in the rearrangement carried out during the relaxation. We observe a slight but constant increase in the recovery slope with time (Fig. 3.5d), which is consistent with the building becoming progressively damaged over the years. The evolution of  $\tau_c$ , *bw* and *A*<sub>max</sub> is more or less analogous inasmuch as they show clear variations in each period (Fig. 3.5e-g).

A clearer picture of the evolution of these parameters with respect to the structural state is shown in Fig. 3.7 and 3.8, for the Y-180 and the X-270 directions, respectively. In these figures the mean relaxation spectrum is only computed for the stable periods P2 and P4, that is, before and after the 2011 Tohoku earthquake. In these figures, the data were grouped in five increasing ranges of drift amplitudes (i.e.  $\Delta_1 = 1 \times 10^{-6} - 5 \times 10^{-6}$ ,  $\Delta_2 = 5 \times 10^{-6} - 1 \times 10^{-5}$ ,  $\Delta_3 = 1 \times 10^{-5} - 5 \times 10^{-5}$ ,  $\Delta_4 = 1 \times 10^{-5} - 5 \times 10^{-5}$ ,  $\Delta_4 = 1 \times 10^{-5} - 5 \times 10^{-5}$ ,  $\Delta_5 = 1 \times 10^{-5} - 5 \times 10^{-5}$ ,  $\Delta_6 = 1 \times 10^{-5} - 5 \times 10^{-5}$ ,  $\Delta_7 = 1 \times 10^{-5} - 5 \times 10^{-5}$ ,  $\Delta_8 = 1 \times 10^{-5} - 5 \times 10^{-5} - 5 \times 10^{-5} - 5 \times 10^{-5}$ ,  $\Delta_8 = 1 \times 10^{-5} - 5 \times$  $5x10^{-5}-1x10^{-4}$ ,  $\Delta_5 = 1x10^{-4}-5x10^{-4}$ ), in order to separate loading amplitude effects from structural state. Before Tohoku (period P2) the variation of frequency *Df* and log-time recovery slopes increase with strain amplitude (Fig. 3.7a top). Therefore, the corresponding amplitudes  $A_{max}$  of the relaxation spectra also increase with the strain level (Fig. 3.7a bottom). Analogous results are observed in laboratory experiments when analyzing the recovery of sandstone at different strain amplitudes (Shokouhi et al., 2017a). Besides, Shokouhi et al., (2017a) obtained more or less equivalent spectrum shapes at different strain amplitudes. In our case, a smooth transition of the characteristic relaxation time  $\tau_c$  to higher values is observed as the drift increases (Fig. 3.7 bottom). However, maximum strain amplitudes in laboratory are in the order of  $10^{-5}$ , which would be comparable to our first two ranges of drift (i.e. yellow and green curves), where the shapes of spectra are rather similar.

For the same ranges of drift, there is an evident increase of the frequency drop after the Tohoku earthquake, i.e. in P4, with respect to P2 (Fig. 3.7b top). The slope values also increase after the earthquake, which is coherent with the aforementioned results on the relationship between the recovery slope and the state of the bond system. Characteristic relaxation times  $\tau_c$  also shift to higher values after Tohoku for all drift categories. This indicates that the typical size of cracks

activated during P4 is larger than the characteristic size of cracks opened during P2 for equivalent loading strains (Fig. 3.7b bottom).



**Figure 3.7** (Top) Recovery of the fundamental frequency with time for the Y-180 direction of the ANX building and different strain amplitudes, during a) period P2, before the Tohoku earthquake and b) period P4, after the Tohoku earthquake. Curves represent the mean frequency recovery *Df*, normalized to the final frequency  $f_{fapp}$ . Slopes corresponding to the time-logarithmic segment are represented in the insets. (Bottom) Mean relaxation spectra for different strain amplitudes during (left) P2 and (right) P4. Relaxation spectra were obtained from the mean exponential fit shown in the top plots. Different colors correspond to five drift ranges ( $\Delta_1=1x10^{-6}-5x10^{-6}, \Delta_2=5x10^{-6}-1x10^{-5}, \Delta_3=1x10^{-5}-5x10^{-5}, \Delta_4=5x10^{-5}-1x10^{-4}, \Delta_5=1x10^{-4}-5x10^{-4}$ ). Mean values of  $\tau_c$  and *bw* per period and per strain level are indicated in the legend. Note that time (X-axis) is displayed in log-scale.

Not only the size of the typical crack enlarged in P4, but also the number of cracks generated with respect to P2, especially for lower strain earthquakes ( $\Delta Max < 5x10^{-5}$ ). This is revealed by the higher values of maximum amplitude  $A_{max}$  observed in Fig. 3.7b (bottom) for the green, yellow and black spectra. In addition, the bandwidth may be a proxy of the different types/sizes of cracks created under certain level of loading. After the Tohoku earthquake an increase of the bandwidth values occurred, being more evident at higher strains (Fig. 3.7b bottom). This is related to the susceptibility of the structure to create a wider range of crack sizes at a given loading value, in comparison to those opened during P2. This bandwidth value finally characterizes the changes in the bond system, or in other words, the structural health.

In X-270 direction (Fig. 3.8), no significant variation between P2 and P4 is observed: frequency drops and slope values are approximately equivalent (Fig. 3.8 top). The spectra shapes at different strain amplitudes are similar and range of relaxation times *bw*, maximum amplitudes  $A_{max}$ , and characteristic times  $\tau_c$  are also rather equivalent (Fig. 3.8 bottom), specially for P4. The data dispersion is also very similar between P2 and P4 but also for the different range of strain values. This behavior might be linked to the fact that the structure was already

significantly cracked along the X-270 direction prior to the 2011 event, as a result of the largest variation of frequency observed during P1 in Fig. 3.4.



Figure 3.8 Similar to figure 3.7 but for the X-270 direction of the building.

Several post-earthquake observations confirm the relation between the evolution of the relaxation parameters and the pattern of cracks in the ANX building. For example, Kashima, (2014) reported damage in the building after the Tohoku earthquake in 2011. Visual post-earthquake structural surveys detected damage around the expansion joints, splits in the plasterboard of partitioning walls and several cracks in the concrete walls.

## 3.5. Relaxation parameters and structural damage

The long-term monitoring of the ANX building (Figure 3.4) shows clear variations of the fundamental frequency in both horizontal directions, with different trends in four periods, as already defined in the chapter 2. We also observe an increase of softening (i.e. a decrease of the frequency due to a decrease in the modulus) for increasing values of PTA. The origin of the observed variations in the elastic properties is mainly due to transient and permanent changes of the structural stiffness, which is controlled by the bond system (i.e. Guyer and Johnson, 1999), which in this case is the system of cracks and other heterogeneities creating weakness in the medium. The structural state evolution reported for the ANX building manifests itself in the slow dynamic recovery process following earthquakes. Different parameters describing these relaxation effects show distinctive behaviors that are concomitant with the structural state:

 $-\tau_{min}$  and  $\tau_{max}$  (energy-related parameters) values increase with structural softening (i.e. Fig. 3.5a and 3.5b). A stiff medium (i.e. ANX at the beginning of the measurements) is very sensitive to energy: opening and closing of cracks is a fast process manifesting at any loading amplitude. As the system becomes softer due to increased mechanical damage, the energy

required to activate cracks is less important. Hence, a cracked medium takes longer to react and to activate the relaxation process; once this process begins it also takes longer for the energy to be totally released. This explains the delay in the beginning of the relaxation (i.e. the increase of  $\tau_{min}$ ) and the elongation of the maximum relaxation time (i.e. the increase of  $\tau_{max}$ ) with the increase of cracking (i.e. from P1 to P2 in figures 3.5a and 3.5b). Mean values of  $\tau_{min}$  and  $\tau_{max}$  also show a punctual rise coincident with the occurrence of large earthquakes within the stable structural period P2 (i.e. Fig. 3.5a and 3.5b), which is related to a significant transitory decrease of stiffness during these events. Snieder et al., (2017) determined that loading is one parameter controlling  $\tau_{max}$ .

-The bandwidth *bw* and the maximum amplitude *Amax* of the relaxation spectrum model are directly connected to the extension of the bond system. Whereas the former reflects the variety in crack sizes, the latter is an indicator of the number of cracks of the same type/size. The increase of cracking in the ANX building due to the Tohoku earthquake and subsequent aftershocks is evident in Fig. 3.7 (bottom), where a wider range of crack types is expected even at low-strain amplitudes, and the number of smaller cracks considerably increased in comparison to the period preceding the 2011 event. The strong shaking of this great earthquake opened new structural cracks and enlarged pre-existing ones, which in turn reduced fundamental frequencies (and therefore stiffness) permanently (Fig. 3.4). Analogous conclusions were drawn by Guéguen et al., (2019) using lab experiments and real data in buildings; as well as laboratory experiments showing the effects of progressively increasing damage by Van Den Abeele et al., (2000a); or by Rubinstein and Beroza, (2004) when analyzing seismic velocity reductions in rock after strong motions.

- Initial softening observed in the building (Fig. 3.4) seems to have affected mostly the X-270 direction. Although the reasons for this have not been analyzed, it might be related to a preferential distribution of heterogeneities along this direction due to differences in the structural design, connections, cracking, and so on. The X-270 direction is softer than the Y-180 and this is also manifested in the recovery process by the behavior of the relaxation parameters, that is, a similar response for *Df*, bandwidth, maximum amplitude, before and after Tohoku was observed in Fig. 3.8.

-Just as bandwidth *bw*, the ratio  $\tau_{max}/\tau_{min}$  is related to the extension of the bond system with respect to the crack sizes. Stable periods (i.e. P2 and P4) show lower mean values of  $\tau_{max}/\tau_{min}$  with respect to the previous period P1 and P3, respectively, suggesting that new types of cracks were more prone to emerge during constant-softening periods (Tab. 3.2). This result is coherent with the observation made in by Astorga et al., (2018), who concluded that new

cracks were created during P1 and P3. Moreover, and similarly to the results reported by Guéguen et al., (2019) it is observed that some earthquakes are able to create new sizes/types of cracks, while some others are not (i.e. Fig. 3.6). This might be due to differences in loading conditions. However, no clear correlation was found between the maximum strain ( $\Delta Max$ ) and the events originating new types of cracks. We observed that the occurrence of one or few events creating new types of cracks is followed by several consecutive events where no new types of cracks emerge. Note that the latter case does not imply that the bond system is not expanded. For example, the creation of new cracks with the same size of the existent ones is manifested by the increase of  $A_{max}$ , which is clearly observed in Fig. 3.7 (bottom) after the Tohoku earthquake. More analyses are needed to determine the possible link between intensity measurements and/or engineering demand parameters and the creation of new sizes/types of cracks.

# 3.6. The multi-scale feature of frequency recovery in buildings

Recent studies have analyzed non-linear elastic responses of buildings during earthquakes (i.e. Astorga et al., 2018, 2019), with short-term (seconds) to long-term (months) transitory variations of their resonance frequencies seen. This behavior has been related to the elastic features of the buildings and the co-seismic opening of pre-existing cracks, which can cause transient material softening at different time scales (Fig. 3.9). This is shown by the rapid co-seismic decrease in their frequency that is followed by their immediate slow recovery (Fig. 3.9a). Without earthquake damage, the initial properties will be fully recovered. This reflects the coalescence of the particles within the damaged material over time, into an equilibrated arrangement (Ostrovsky and Johnson, 2001; Guyer and Johnson, 2009), which results in the closing of cracks. During this process, many thermodynamic and mechanical factors control the number of contacts within the cracks over time, and consequently, the duration of the recovery (Guyer and Johnson, 2009, Snieder et al., 2017a).

Over months after a large earthquake (Fig. 3.9c), we can observe slow recovery over a long time scale (i.e., of the order of several months, to a few years), in the manner of long-term relaxation of the crustal properties of the Earth after large earthquakes (Brenguier et al., 2008 and 2014). Strong shakings can open cracks, which might gradually close due to frictional contact between the particles in the damaged zones. Equivalent shaking caused by later smaller earthquakes might contribute to the growth of these contacts, to increase the pressure and friction between the grains, and consequently to favour the recovery process. The recovery of the elastic properties, however, can also be affected by conditioning effects (Johnson and Sutin,

2005; Johnson and Jia, 2005). This is observed in Figure 3.9b, where the slow dynamics were accompanied by hysteresis and discrete memory during the aftershock sequence of the 2011 Mw 9.1 Tohoku earthquake. The origin of these effects is in the bond system (Ostrovsky and Johnson, 2001; Johnson and Sutin, 2005; Guyer and Johnson, 2009), and particularly in the spatial arrangements of stress chains (Gist, 1994; Peters et al., 2005; Daniels et al., 2008; Gao et al., 2019), which represent groups of multi-size contacts that relay the strongest stresses. Structural cracking generates stress-chain rearrangements that represent the mechanism for energy dissipation during each event. The energy dissipation depends on the excitation amplitude: small events generally correspond to variations of local stress chains, whereas larger events can cause changes at a global scale, which results in a new complex anisotropic network of cracks that dominates the backbone recovery (i.e. the outer loop) shown in Figure 3.9b. Internal recovery cycles (i.e. hysteresis) are due to local stress changes that are generated by the strongest aftershocks, without any changes to the general response of the system, and thus with maintenance of the backbone (i.e. the discrete memory). In Figure 3.9b, the backbone, therefore, describes the recovery of the structural state, which is controlled by the maximum co-seismic strain state of the main shock.



**Figure 3.9** Different time scales of the slow dynamics observed for buildings. a) The drop and recovery of the resonance frequency (bottom panel) during a single earthquake (top panel). Red line, the co-seismic value of the resonance frequency extracted from the time–frequency distribution diagram. b) Hysteretic recovery during a sequence of aftershocks of the Tohoku earthquake (bottom panel). Each symbol indicates the co-seismic frequency computed during an earthquake, where the stars correspond to significant aftershocks. Solid blue line, the backbone recovery. The maximum acceleration at the top of the building (PTA) for the aftershock sequence is also shown (top panel). c) Long-term frequency recovery (bottom panel) during randomly spaced earthquakes (top panel). The event of Aug/2005 was a large-amplitude earthquake (~330 cm/s<sup>2</sup>), whereas the events shown from Oct/2005 to July/2007 were of the same order of lower amplitude (i.e. the PTAs did not exceed 10% of the large-event PTA).

#### 3.6.1. Backbone recovery curve and hysteresis during aftershocks

Figure 3.10 shows the co-seismic fundamental frequencies of two Japanese buildings of similar typology (i.e. ANX and THU, described in chapter 2) between August 2005 and September 2012 (Fig. 3.10a). We observe slow dynamics over time following three significant earthquakes (i.e. 2005 Mw 7.2 Miyagi; 2008 Mw 6.9 Iwate; 2011 Mw 9.0 Tohoku; Fig. 3.10, R1, R2, R3, respectively). We analyzed the time scales of the recoveries of the backbones for the weakest events, which corresponded to the weakest loading, to remove conditioning effects. Assuming a time-logarithmic function (TenCate et al., 2010) (Fig. 3.10c), we observe that the recovery slopes increased with the loading amplitude and the damage state, as also seen previously in several laboratory-tested materials (Ostrovsky and Johnson, 2001; Johnson and Sutin, 2005; TenCate et al., 2010). The THU building was exposed to significantly higher levels of maximum acceleration at the top of the building, PTA and showed recovery slopes that were an order of magnitude larger than for the ANX building before 2011 (i.e. R1, R2). On the other hand, the recovery slope after the Tohoku earthquake (i.e. R3) was around 5-fold steeper for the THU building, which was severely damaged during this event (Okawa et al., 2013; see chapter 2). Although the log-time adjustment does not have any physical basis, we assume that the rate of recovery is linked in some way to the rate of coalescence within the cracked zones, so that an equilibrium state can be reached. Here, densely cracked media would show steep recovery slopes because there are more voids to be filled after strong excitation.



**Figure 3.10** Long-term recovery of the fundamental frequency of two Japanese buildings (ANX, THU). a) Co-seismic frequency computed over the years, showing slow dynamics after important earthquakes in 2005, 2008 and 2011. b) Zoom-in on the recovery after the 2011 event, showing the conditioning cycles (i.e. R3a, R3b and R3c). a), b) Each symbol corresponds to a single earthquake, and the colorscale is related to the maximum acceleration at the top of the building (PTA). The large symbols were used to monitor the backbone curve. c) Log-linear recovery of the normalized frequency variation ( $\Delta f/f_f = (f-f_f)/f_f$ , where  $f_f =$  maximum final frequency), which indicates the slopes computed from the log-linear function applied to the periods shown in a) and b).

To explore the conditioning effects on the recovery slope, the log–linear model was applied to the internal recovery cycles created during the aftershock sequence of the Tohoku earthquake (i.e. Fig. 3.10b, R3<sub>a</sub>, R3<sub>b</sub>, and eventually R3<sub>c</sub>). The recovery slope decreased progressively as the conditioning effects were lost: from 0.069 to 0.057 for ANX, and from 0.196 to 0.128 for THU (Fig. 3.10c). This suggests a gradual closing mechanism for the cracks that were activated by the contribution of the local stress-chain adjustments to the total recovery. Furthermore, for the ANX building (which was slightly damaged during the 2011 event, see chapter 2) the recovery slopes due to the conditioning cycles were steeper than the backbone slope, whereas the opposite was seen for the THU building. This reflects the sensitivity of the structural material to the opening/ closing processes of temporary cracks while the structure is still recovering from the main shock. In a densely damaged medium, much more energy would be necessary to perturbate the bond system and generate new stress states that can change the global response, which will be limited, however, by the ultimate collapse of the building.

Our results demonstrate that both loading amplitude and structural state control the structural response. However, in the presence of a densely cracked material (i.e. the THU building), loading effects are less important and the structural response mostly depends on the structural state.

#### **3.6.2.** Relaxation models applied to long-term structural recovery

Three theoretical models (Fig. 3.11a-c) were used to study the recovery processes shown in Figure 3.10. These models (Snieder et al., 2017; Shokouhi et al., 2017a; Ostrovsky et al., 2019) are based on physical concepts and were developed based on laboratory experiments carried out on the recovery of broken contacts in granular materials. The models proposed by Snieder et al., (2017) and Shokouhi et al., (2017a) are presented in Section 3.3. Ostrovsky et al., (2019) analyzed the relaxation of elastic properties of geomaterials throughout the shift of wavespeed,  $\Delta c/c$ , following a dynamic excitation. Analogous to Snieder et al., (2017), the authors proposed a long-term relaxation model based on the hypothesis that the recovery is defined by two main parameters: temperature and activation energy. Therefore, the rate of the recovery can be described by an Arrhenius-type equation similar to the Eq. 3.5. Moreover, the authors consider the macroscopic elastic modulus of the medium (i.e. Young modulus) as  $E = E_0(1 - \varphi) + \varphi E_d$ , where  $E_0$  is the initial unperturbed elastic modulus, and  $E_d$  represents the elastic modulus of the fraction of soft contacts (i.e.  $\varphi$ ) in the bond system. Given that  $E_d \ll E_0$ , the contribution of  $\varphi E_d$  is neglected and the decrease in modulus can be represented as  $E_s = E_0 - E = \varphi E_0$ .

The return to an equilibrium state depends on a force, necessary to overcome a potential barrier. This force is proportional to  $E_s$  and to the characteristic contact volume between particles, V. After several substitutions (see Ostrovsky et al., 2019 for details) the Arrhenius-type equation is written as:

$$E_s = \Lambda \ln \left( -\frac{AE_0}{\Lambda} t + e^{\frac{E_{s0}}{\Lambda}} \right)$$
(3.9)

with 
$$\Lambda = \frac{K_B T}{V}$$
; and A and  $K_B$  similar to Eq. 3.3

This implies that the metastable contacts return to equilibrium in a finite time interval  $\tau$  that depends on the loading parameter,  $\Lambda$ . For  $E_s$  (t =  $\tau$ ) = 0, the authors estimate  $\Delta c/c$  during recovery, as follows:

$$\frac{\Delta_c}{c} = \frac{\Lambda}{2E_0} \ln\left[\frac{t}{\tau} + e^{-\left|\frac{E_{s0}}{\Lambda}\right|} \left(1 - \frac{t}{\tau}\right)\right]$$
(3.10)

We can simplify the Eq. 3.10 as follows:

$$\frac{\Delta c}{c} = a \log[10^m + e^{-G}(1 - 10^m)]$$
(3.11)

where 
$$a = \frac{2.3\Lambda}{2E_o}$$
;  $G = \left|\frac{E_s}{\Lambda}\right|$ ; and  $m = \log\left(\frac{t}{\tau}\right)$ 

The authors tested the model in different examples of laboratory data and field observations, observing that the characteristic size of broken contacts responsible for slow dynamics is in the order of  $10^{-9}$  m, and predicts that their number increases with impact amplitude. To describe variations of elastic properties we used frequency variations  $\Delta f/f$  rather than  $\Delta c/c$ . In our data, parameters *a* and G were obtained by nonlinear regression of Eq. 3.11, assuming the constant  $\Lambda$  is related to loading amplitude, and  $E_0$  and  $E_s$  are proxies of the pre-seismic and co-seismic elastic modulus (i.e. frequency), respectively. In this section, we use the long-time relaxation model by Ostrovsky et al., 2019 to describe recovery effects during aftershock sequences and long-term frequency variations.

First, we computed the a and G parameters, as proxies for the elasticity before and during longterm recovery processes, respectively, shown in Figure 3.11a. The parameter a, which is inversely proportional to the pre-seismic elastic modulus, increased sharply during the post-Tohoku recovery. Parameter G, which is directly proportional to the co-seismic elasticity, decreased. This confirms the increase in the softening in both of these buildings. Secondly, the ratio  $\tau_{max}/\tau_{min}$  computed from the relaxation function in Figure 3.11b (Snieder et al., 2017) increased from ~6 to ~23 in the ANX building, and from ~9 to ~18 for THU. This ratio denotes the different time-scale mechanisms that act in the time–logarithmic segment of the recovery, and characterizes the diversity of the crack sizes (section 3.4). From this we can infer that after the 2011 event the variety of the cracks in the ANX and THU structures was quadrupled and doubled, respectively. Additionally, the gradual reduction in  $\tau_{max}/\tau_{min}$  during the re-loading cycles confirms the progressive crack-closing process during the aftershocks inferred from the recovery slopes. In addition, after the Tohoku earthquake we observe clear changes in the maximum frequency variation (i.e.,  $\Delta f/f$ ) for both models, which increased from ~3% to ~12% for ANX and from ~13% to ~33% for THU (Fig. 3.11a, b), which is consistent with the modulus softening and then the global change in the structural states.



**Figure 3.11** Relaxation models adapted to the frequency recovery of buildings for the different periods defined in figure 3.10. a) Normalized frequency variation over recovery time according to the model proposed by Ostrovsky et al., (2019). Here, *a* and G are proxies for elasticity.  $m = \log(\frac{t}{\tau})$ . b) Normalized frequency variation over the recovery time according to the relaxation function of Snieder et al., (2017). Here,  $\tau_{max}/\tau_{min}$  is the ratio between the final and initial relaxation times computed from the model. c) Relaxation spectra proposed by Shokouhi et al., (2017a), indicating the bandwidth defined using the  $1/\sqrt{2}$  of the maximum spectrum amplitude.

A complete signature of the recovery process is given by the relaxation spectrum (Shokouhi et al., 2017a) shown in Figure 3.11c. We detect mechanisms over five orders of magnitude in

time, from t  $\sim 0.1$  to  $\sim 1200$  seconds; i.e., extreme values that are not revealed by the previous models. The spectrum bandwidth represents the range of the dominant relaxation times, and as the ratio  $\tau_{max}/\tau_{min}$ , this serves as a hint of the diversity of the crack sizes that are closed over the time of the recovery. These data suggest that a large variety of crack sizes was activated following the 2008 event (i.e., R2). The post-Tohoku spectrum does not indicate new types of cracks; nevertheless, the maximum spectrum amplitude is ~3.5-fold that observed in the periods before Tohoku. This implies that the crack density increased around 3.5-fold after 2011 for both of these structures. It can also be noted that the spectra of the THU building are approximately 3-fold those of the ANX building, showing the different levels of damage between the buildings even before 2011. At the same time, conditioning effects might have been significant in the recovery process for the ANX building, which activated mechanisms with relaxation times in the order of  $10^{1}$ - $10^{2}$  seconds. In contrast, during the recovery of the THU structure, the conditioning cycles just contributed to the activation of inner small mechanisms, as shown by the narrow left-shifted spectra  $R3_{a,b,c}$  (Fig. 3.11c). Thus, theoretical models applied to earthquake data from real buildings fit the long-term recovery of the fundamental frequency after earthquakes. These data indicate that non-linear elastic processes within the structural bond system explain the transitory and permanent variations of structural dynamic responses to seismic events. In particular, the relaxation parameters reveal the internal material changes that are related to cracking and stiffness degradation; i.e., in relation to the structural health and safety of a building.

# 3.7. Conclusions

The main goals of this study were (1) to corroborate the manifestation of nonlinear elastic signatures at the building's scale, especially the slow dynamics behavior, (2) to test existent slow dynamics relaxation models developed at the laboratory scale to real earthquake data and real buildings and (3) to investigate the behavior of several relaxation parameters with respect to different loading amplitudes and structural states.

Astorga et al., (2018) detected clear signatures of nonlinear elastic behavior in the ANX building, similar to what is seen in laboratory and seismological scales. Fundamental frequency fluctuations at short (inter-events) and long term (intra-events) show evident both transitory and permanent variations of stiffness, controlled primarily by the bond system (i.e. cracks and other heterogeneities). The creation and growth of cracks along with the resultant constant frequency decrease entail significant energy expended in damaging the material, causing rearrangement of the internal structure, and resulting in variations of detectable physical

properties. Two periods of relatively stable dynamic response, where permanent frequency variations are detected in the ANX building are observed (1) between 2005 and March 2011, that is, after a constant softening occurred during the first 7 years of the structure and (2) between October 2011 and May 2018, that is, after the Tohoku earthquake in March 2011 and its immediate aftershocks. These permanent stiffness variations suggest an extension of the cracks system and are therefore linked to damage.

We focused our study in the recovery process, a time-dependent relaxation mechanism in which the fundamental frequency shifts back to higher values after the loading is finished. We observe such elastic response at any given loading amplitude. Even after very strong earthquakes, the fundamental frequency recovers: that is, during the earthquake of 23 October 2004 (Table 3.1), the frequency dropped significantly. However, the recovery was practically immediate (Figure 3.4). After the Mw 9 Tohoku earthquake in 2011, the frequency recovery is observed during approximately 6 months, although in this case, it is only partial. Seven years after this great event, the fundamental frequency seems to be stable in a value that represents around the 60% of the initial elastic frequencies obtained in 1998. Despite of that, the building is still operative, and no damage is apparently seen.

Parameters linked to the slow dynamics recovery also show evidence of the variable structural response. The level of heterogeneity in the material controls the behavior of relaxation times, recovery rates, and relaxation mechanisms amplitudes. This is manifested by clear variations of these parameters between periods (Figures 3.5, 3.7, 3.8, 3.11, and Table 3.2). Some of the relaxation parameters also seem to be sensitive to loading: that is, within a stable response period (P2), spikes in parameters  $\tau_{min}$ ,  $\tau_{max}$ , and Amax are more or less coincident with the occurrence of large events (Figure 3.5). Moreover, variations of frequency *Df*, characteristic relaxation times and proxies of the extension of the bond system (i.e. bandwidth and Amax) are well correlated to the strain amplitude (Figure 3.7). This dependence is consistent with laboratory observations that indicate that the nonlinear response is proportional to the level of dynamic strain (Shokouhi et al., 2017a; Guyer et al., 1999; Ostrovsky et al., 2000).

Laboratory experiments are performed under controlled conditions and strain amplitudes that do not damage the material. In practice, real buildings facing real earthquakes represent a much more complex problem. Multiple uncertainties coming from several sources (i.e. loading, environment, soil, instrumentation, material, connections, construction process, etc.), the possible mix of modal responses and the interaction between manifold elements, are all factors that make the structural response complex and unique. Despite all that, the relaxation models proposed by Snieder et al., 2017; Shokouhi et al., 2017a, and Ostrovsky et al., 2019, originally developed for laboratory experiments, are well adapted to the building data. Our results are analogous to observations of nonlinear elastic behavior at small (i.e. laboratory) and large (i.e. Earth's crust) scales; and they are satisfactory as a first attempt to apply the models to real data at intermediate-scale (i.e. buildings).

By applying seismic interferometry to borehole accelerometric data located right next to the building, Astorga et al., (2018) concluded that the contribution of the soil (i.e. soil-structure interaction) on the variation of the total response (i.e. soil-structure system) is less important in comparison to the contribution of the response of the structure itself. That study together with the results of this study allows us to confirm the structural origin of the slow dynamics, clearly seen in Figures 3.7, 3.8, 3.10 and 3.11, in relation to the degree of fracturing of the structural elements. A careful analysis of the slow dynamics recovery might allow us to understand the emergent unrevealed behavior related to cracks growth, friction, contact rates, healing, and so on. Understanding nonlinear elastic behavior might be helpful to improve our knowledge in dynamic response, allowing us to develop and calibrate models that are fundamental for predicting real structural behavior.

By analogy with relaxation studies in granular materials, we infer that under proper loading conditions, micro-to-nanoscale heterogeneities such as cracks, might coalesce until causing macroscopic alterations of the structural properties. Here, nonlinear elastic processes within the bond system might explain transitory and permanent variations of structural dynamic response to seismic events; where thermodynamical and mechanical factors might contribute to the internal organization of particles that allows for the closing of cracks. This, however, is an ongoing research that require a lot of effort to try to unveil the physics behind the slow dynamics effects.

The procedure followed to obtain the results presented in this chapter could be an easy way to detect changes in the structural response (i.e. damage). For example, the comparison of the structural response in terms of relaxation parameters before and after a specific event, for a same level of deformation, can provide us with important information about the extension and density of heterogeneities, i.e. cracks. The automatized computation of relaxation parameters applied to continuous and real-time instrumented buildings, would allow us to detect permanent variations in the response (increase of *bandwidth*,  $A_{max}$ , variations of *a* or *G*, etc. for equivalent loading), which is fundamental for making prompt and accurate decisions about structural health.

# 4

# NONLINEAR ELASTIC RESPONSE TO MONITOR STRUCTURAL DAMAGE IN BUILDINGS OF DIFFERENT TYPOLOGIES

This chapter describes similarities between different building typologies, providing important hints for structural health and earthquake engineering. The general tendencies of frequency variations and the behavior of relaxation parameters are analyzed as a function of time and loading features, and the structural response variabilities are given according to building material and structural state. We analyze the influence of loading rates on buildings with different levels of damage, showing the parameters that have the most influence on the structure's response during loading and recovery.

# 4.1. Introduction

Damage is understood as any change in the material of a system that negatively affects its current or future performance (i.e. Farrar and Worden, 2006). This can include changes to the boundary conditions, connectivity between system elements, variations in geometry or in the internal configuration of the material. Damage does not necessarily mean a complete loss of system operability or structural capacity, but rather a loss of its optimal and original design. To identify and characterize damage, it is necessary to compare two different conditions, for example, before and after an extreme event.

All damage begins at the scale of the material, usually as a small defect or an anomaly of variable degree. Under proper loading conditions, these micro heterogeneities might coalesce until macroscopic alterations are caused on the scale of the system. Moreover, damage can accumulate gradually over long time periods (i.e. aging effects, cycling loadings, etc.) or can result from unexpected extreme events, such as earthquakes (i.e. Farrar and Worden, 2007). Structural health monitoring methods are deployed to detect damage, either based on the continuous assessment of information on system performance in the case of long-term monitoring, or on the rapid provision of reliable information on system capacity in the case of

extreme events. In civil engineering structures, a structural health monitoring strategy consists in tracking variations in dynamic features related to structural capacity, such as stiffness or dissipation coefficients. The choice of the feature used for structural health monitoring is therefore crucial to detect, localize and quantify damage.

Structural damage generally causes variations in the resonance frequencies. However, frequency changes alone do not automatically imply damage, given that frequency changes also occur due to environmental and operating conditions (i.e. traffic, wind, temperature, etc.). Frequency variations due to this type of natural wandering are actually expected to be much smaller than those caused by damaging events. For example, in buildings, frequency fluctuations of about 0.5% - 2% are generally associated with ambient conditions (i.e. Clinton et al. 2006; Hua et al. 2007; Nayeri et al. 2008; Xia et al., 2011; Mikael et al. 2013; Guéguen and Tiganescu, 2018). Conversely, frequency changes due to structural damage in buildings can reach values of between 20% and 50%, characterizing different damage levels without systematically compromising structural safety (i.e. Dunand et al. 2004; Calvi et al. 2006; Todorovska et al., 2006; Trifunac et al., 2010; Vidal et al., 2014; Astorga et al., 2018). Structural damage caused by earthquakes produces permanent frequency changes related to a loss of stiffness (i.e. Clinton et al. 2006; Dunand et al. 2006; Celebi et al. 2016; Astorga et al., 2018; Di Sarno and Amiri, 2019); this is usually linked to the disconnection of structural and nonstructural elements, joint deformation, variations in the friction/border conditions between elements, and the opening of cracks. The recovery process observed after earthquakes is therefore the (partial or total) restoration of these effects, mostly due to the activation of relaxation processes on the scale of the material (Chapter 3).

Moreover, co-seismic variations of fundamental frequencies in buildings are also related to loading features. Astorga et al., (2018, 2019) observed the maximum frequency drop as being proportional to structural drift and peak accelerations. Guéguen et al., (2016) observed the dependency of the frequency variation on deformation, starting at very slight deformations (i.e. around  $10^{-7}$ ) related to nonlinear elastic behavior. Similar behavior is observed on Japanese data in our database (Fig. 4.1). This figure shows two different earthquakes (i.e. one small earthquake in 2009 and the 2011 Tohoku event) recorded in 36 buildings and the frequency variations are displayed as a function of structural drift. Nonlinearity starts at drift values between  $10^{-7}$  and  $10^{-6}$ , i.e. 3 orders of magnitude lower than the yield strain value given for typical buildings (around ~ $10^{-3}$ ). Furthermore, a given drift value does not imply the same frequency variation. Considering the same buildings for both earthquakes, a between-building

variability of the response can be observed, providing the same distribution of residue for both earthquakes with respect to the adjusted linear functions. Between-event variability is also observed, as illustrated by the function shifts between the smallest and the largest earthquakes. Both kinds of variability (between-event and between-building) suggest that (1) building response is not governed only by loading amplitude, and (2) building response is not governed only by structural typology. Considering structural drift as a strain proxy of the structures and resonance frequency as a proxy of their elastic properties, the observed behavior differs from the conventional representation of nonlinear models, and is more similar to the typical nonlinear elasticity model, as demonstrated by laboratory tests and building monitoring (i.e. Johnson and Jia, 2005; Guéguen et al., 2016).



**Figure 4.1** a) Variation of the normalized frequency  $(\Delta f/f = (f-f_i)/f_i)$  as a function of drift computed for 36 buildings with different characteristics, during two earthquakes with dissimilar amplitudes and maximum strains. Each marker represents the average of 20 data bins. Dashed lines correspond to the mean response, given by the 2-term exponential function of the form  $a^*\exp(b^*x)+c^*\exp(d^*x)$  b) Histograms of the errors between the observed response and the fitting curve.

In this chapter, we consider the results from 36 Japanese buildings of different typologies, monitored for approximately 17 years. First, we show the variation of fundamental frequency over time for reinforced concrete, steel, and mixed steel-reinforced concrete buildings under weak to strong seismic loadings. Next, we study the evolution of relaxation parameters after damaging earthquakes, showing the variability between building typologies. Finally, we evaluate the effects of loading and loading rate on the nonlinear elastic response of buildings during loading and recovery cycles. We conclude on the main factors controlling frequency variations and the importance of monitoring nonlinear elastic processes to detect structural damage.

#### 4.2. Frequency variations over time

In Chapter 2, we showed fundamental frequency variations over time for two buildings of similar typology: buildings ANX and THU. We observed that the variations are well correlated with loading amplitudes and structural degradation, and that they are notably modified after the largest earthquakes. Figure 4.2 shows the frequency variations of long-term monitored buildings grouped by structural material. A total of 9,343 earthquake recordings from the tops of 36 Japanese buildings were analyzed. The data correspond to:

- 3,076 recordings in 14 reinforced concrete structures (i.e. RC), typically with 1 to 4 floors,
- 4,313 recordings in 14 steel-reinforced concrete structures (i.e. SRC), typically with 4 to
   9 floors, and
- 1,954 recordings in 8 steel buildings (i.e. S), characterized by high-rise structures (≥ 10 floors).

The number of earthquakes recorded varies in each building, but each of the selected structures shows long-term fundamental frequency variations during 1998 and 2014, and all of the selected buildings recorded the great (Mw 9) Tohoku earthquake (March, 2011) and some of its aftershocks. Several buildings also recorded other significant damaging earthquakes (i.e. Nigata-Chetsu in 2004, Miyagi in 2005 and Iwate in 2008). For each recording, the frequency variation is computed according to the procedure explained in Chapter 1, corresponding to the soil-structure system.

Fig. 4.2 shows the frequency variations for the whole dataset, given as the difference between the co-seismic  $f_{min}$  and the pre-seismic  $f_i$  fundamental frequency values, normalized with respect to  $f_i$ . The color curves (i.e. blue, red and black) correspond to the smoothed functions (Savitzky-Golay, 3<sup>rd</sup> order) of the mean frequency variations according to building material (i.e. SRC, S and RC, respectively), with shaded areas representing one standard deviation. The total response,  $\mu \pm \sigma$  (i.e. *MeanALL*), is computed using all the buildings, and represented by yellow markers. *MeanALL* is computed considering different weights for each typology, according to the amount of data contributing to the overall analysis (0.46 for SRC, 0.33 for RC and 0.21 for S). Mean and standard deviation values are given Table 4.1.



**Figure 4.2** Fundamental frequency variations over time for Japanese buildings of different typologies: RC (black), SRC (blue) and Steel (red). The main curves correspond to the mean response per typology computed for each 10 time bins, and smoothed with a  $3^{rd}$  order Savitzky-Golay filter. The shaded areas represent inter-typology variability (i.e. one standard deviation). The mean response  $\pm$  standard deviation of all typologies is represented in yellow. a) Frequency variations between 1998 and 2014, with the arrows on the X-axis indicating the occurrence of known damaging earthquakes. b) Zoom on the frequency variations after the Tohoku earthquake in 2011.

Regardless of building typology, the tendency of the fundamental frequency variations over time shows surprisingly similarities to that observed in the ANX building (Chapters 2 and 3). The same three periods are distinguished in the frequency variations, notably marked by the occurrence of major earthquakes: before 2005, between 2005 and 2011, and after 2011. A systematic decrease in frequency is observed for the most serious earthquakes, followed by recovery during conditioning (i.e. loading) in the aftershock sequence.

Before 2005, frequency shift corresponds to an average reduction of  $9.3\pm6.5\%$  for S buildings,  $13.8\pm8.8\%$  for SRC, and  $15.0\pm9.5\%$  for RC buildings, respectively. For the Nigata-Chetsu earthquake in October 2004, a frequency shift is observed followed by recovery, which lasts until the Miyagi event in August 2005, causing a new frequency decrease and posterior recovery. In 2008, the same tendency is observed with the occurrence of the Iwate earthquake. The average frequency variations over this period (i.e. 2005-2011) are  $9.6\pm6.4\%$ ,  $17.8\pm8.9\%$ , and  $15.3\pm9.6\%$  for the S, SRC and RC structures, respectively. With the occurrence of the Tohoku event in 2011 and its aftershock sequence, variations increased sharply for all building typologies, especially for SRC structures. From 2011 to 2014, average reductions were  $13.9\pm5.5\%$ ,  $28.7\pm12.3\%$  and  $19.3\pm8.8\%$  for S, SRC and RC buildings, respectively (Table 4.1).

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	Before 2005	2005-2011	After 2011
SRC	$0.138 \pm 0.088$	0.178 ± 0.089	0.287 ± 0.123
RC	0.150 ± 0.095	0.153 ± 0.096	0.193 ± 0.088
S	0.093 ± 0.065	0.096 ± 0.064	0.139 ± 0.055
meanALL	0.132 + 0.031	0.158 ± 0.041	0.222 + 0.063

Table 4.1 Mean and standard deviation values associated with Fig. 4.2

Before 2005, RC structures show the largest frequency decrease as a consequence of the energydissipation mechanisms of cracking in concrete material (Chapters 2 and 3). The opening/closing of preexisting cracks leads to a co-seismic softening of the medium, and the cumulative effect over time induces a progressive softening of the system because some of the cracks remain open after excitation. After 2005, the mean frequency variation for all structures changes slightly (Tab. 4.1) after the 2004 Nigata-Chetsu event. This relatively strong earthquake suddenly increased the degree of cracking, particularly in RC and SRC buildings, thus making the formation of new cracks in these buildings without significant earthquakes less likely during the 2005-2011 period before the major Tohoku earthquake in 2011.

The most significant frequency drop after major earthquakes and their aftershocks is recorded in SRC buildings. This might be explained by the interaction between steel and reinforceconcrete structural elements, representing an important source of variability in overall structural behavior, particularly under strong shaking.

Steel structures have the smallest frequency variations during the whole period analyzed. In Chapters 2 and 3, we showed a direct correlation between frequency variations and the presence of heterogeneities. Nonlinear elastic signatures have been observed in metallic materials under laboratory tests (i.e. Johnson and Sutin, 2005; and Granato et al., 1956) proposed a model to describe nonlinear response based on dislocations in metals. In steel buildings, cracks can appear under certain conditions (i.e. fatigue, extreme temperatures or stress corrosion). In an earthquake situation, we assume that the frequency variations are related to joint deformations and other heterogeneities or discontinuities, caused, for instance, in the connections between structural and non-structural elements, which therefore have a greater effect on the frequency variations in RC and SRC structures.

Fig. 4.2 gives *MeanALL* values with deviation. The values per period are  $13.2\pm3.1\%$ ,  $15.8\pm4.1\%$  and  $22.2\pm6.3\%$ , before 2005, between 2005-2011, and after 2011, respectively. In Fig. 4.2 and Table 4.1, the *meanALL* values are predominantly influenced by RC and SRC structures, i.e. heterogenous materials control the mean frequency variations of the whole dataset.

The variability associated with each building-specific response is confirmed over the period 2005-2011 (Fig. 4.2), as a result of the variability of damage experienced by each building. The responses of RC and Steel structures show smaller standard deviations after the largest earthquake in 2011 (i.e.  $\sigma = 0.088$  and  $\sigma = 0.055$ , respectively) than SRC buildings (i.e.  $\sigma = 0.123$ ). The higher variability among SRC structures after the Tohoku earthquake is probably due to hybrid contribution of steel and RC behavior to the global response of a specific SRC building.

The variability between countries is also studied and shown in Fig. 4.3. This figure displays the histograms of frequency variations (i.e.  $\Delta f=f_{min}-f_i$ ) for buildings of the same typology (i.e. RC and S) located in Japan and US from our NDE1.0 database (Chapter 1). First, we observe a higher  $\Delta f$  in RC structures than in Steel structures located in the same country. US buildings show a mean frequency variation of  $0.445\pm0.285$  Hz and  $0.272\pm0.157$  Hz for US-RC and US-S buildings respectively: in the US, the coseismic frequency of RC structures decreases an average 1.6 times more than in S buildings. In Japan,  $\Delta f$  is higher for RC than for S buildings (i.e.  $0.345\pm0.227$  Hz and  $0.059\pm0.062$  Hz, respectively, representing a  $\Delta f$  almost 6 times greater for RC than for S). This comparison confirms the universality of the governance of heterogeneous materials over the nonlinear elastic response of structures, whatever the country. RC structures also show greater dispersion compared with Steel buildings in the same country, revealing high variability in the degree of cracking between RC structures.



**Figure 4.3** Histograms of the fundamental frequency variations ( $\Delta f = f_{min} - f_i$ ) in Japanese and US buildings with a) Reinforced Concrete, RC and b) Steel, S structures. Mean and standard deviation values are indicated in the legend.

In Fig. 4.3b, the difference in frequency variations between S structures is related to several reasons: the number of buildings considered (only 8 Japanese buildings compared with 48 buildings in US), the range of loading values experienced by US and Japanese buildings, and the higher intra-typology variability for US buildings. Some authors have also observed the effects on resonance frequency of weather conditions, such as temperature variations (i.e. Clinton et al., 2006; Nayeri et al., 2008; Xia et al., 2011; Mikael et al., 2013; Guéguen and Tiganescu, 2018) or strong winds and heavy rain (i.e. Clinton et al., 2006; Herak and Herak, 2009). Without additional information on the weather conditions, we cannot reach a valid conclusion, but these variations must be of a lesser order of magnitude compared with those produced by earthquakes.

### 4.3. Evolution of relaxation parameters

In Figure 4.2 the fundamental frequency recovers over several months after the largest earthquakes, regardless of the structural material, and shows hysteresis and discrete memory signatures as already seen in the ANX and THU buildings (i.e. Chapter 3). The relaxation models (i.e. Snieder et al., 2017; Shokouhi et al., 2017a and the log-linear function according to TenCate and Shankland, 1996) are applied to the earthquake data of the Japanese RC, SRC and S buildings, following a procedure similar to that described in Chapter 3 for the ANX building. The relaxation parameters (i.e.  $\tau_{min}$ ,  $\tau_{max}$ , p, A<sub>max</sub>,  $\tau_{c}$  and bw) are computed for each event and averaged by building typology. Fig. 4.4 shows the evolution of relaxation parameters over time. The mean response  $\pm$  standard deviation for all the buildings is also computed and represented by the yellow diamonds in Fig. 4.4 (i.e. meanALL). Generally, the relaxation parameters values increase over time, with a major shift occurring after the large damaging earthquakes (October 2004 and March 2011, denoted by vertical dashed lines). These significant earthquakes define time periods characterized by notable variations in the structural response, as seen in the behavior of the fundamental frequencies (i.e. Fig. 4.2). The mean values of the relaxation parameters are shown in Table 4.2. The increase in relaxation parameters over time is particularly noticeable for  $\tau_{\min}$ ,  $\tau_{\max}$ ,  $\tau_c$  and *bw* in Fig. 4.4 and Table 4.2, suggesting:

A general softening in all building typologies due to increasing heterogeneities (i.e. cracks in RC and SRC structures, joint deformations in Steel buildings, changes in friction conditions and connections between elements, etc.). The increase of τ<sub>min</sub> values indicates delays to the start of the recovery process, associated with the closing of the smallest cracks, which might occur in damaged materials that are less sensitive to changes of state.

- Longer relaxation processes and later relaxation mechanisms.  $\tau_{max}$  and  $\tau_c$  values are related to the final and typical relaxation times, respectively. The increments of  $\tau_{max}$  and  $\tau_c$  are associated with the increase in size of maximum and typical cracks, respectively. This might explain the results observed in granular materials, such as RC and SRC. In Steel buildings, however, the increase in  $\tau_{max}$  indicates a longer relaxation process, related to the time that the building takes to return to a state of equilibrium after the earthquake. As structural flexibility increases, the final recovery time (i.e.  $\tau_{max}$ ) might also increase. In turn, if the recovery process is longer, the characteristic relaxation time (i.e.  $\tau_c$ ) is most likely to increase, since the whole relaxation would be expected to be determined by longer mechanisms.
- Increment in the variety of heterogeneities. Bandwidth values, *bw*, are directly related to the duration of the entire relaxation process. In RC and SRC structures, the increase of *bw* over time can be translated by a greater variety of crack sizes. In Steel buildings, *bw* values are higher than in RC and SRC, indicating longer relaxation processes, as also inferred from the behavior of  $\tau_{max}$ . However, the main relaxation mechanisms in this typology are not linked to cracking, but to joint activation. Therefore, this suggests that the recovery associated with joints and connections takes longer than the process of closing cracks. Moreover, the increase of *bw* in S buildings over the different time periods (i.e. from <2005 to 2005-2011 and then to >2011) might indicate the transition from the elastic domain to the ductile domain, with a longer time required to recover initial shape.

Considering the slope p and the maximum spectrum amplitude  $A_{max}$ , the observations of Chapter 3 are confirmed: the rate of recovery tends to increase with damage, and  $A_{max}$  can be a relevant proxy of crack density. However, the relaxation parameters must be considered together to provide a better idea of the cracking phenomena. For example, although *bw* increases in RC buildings over the time periods (i.e. from (log) 1.355 to 1.443 and then to 1.632, Table 4.2), reflecting a greater variety of crack size, the mean values of p and  $A_{max}$  are more or less stable from one period to other (i.e. around 3-4%). This suggests equivalent crack density over time. On the other hand, SRC buildings show an increase in the variety of cracks after 2005 (i.e. *bw* increases from (log) 1.270 to 1.645) but a smaller number (i.e.  $A_{max}$  decreases from 3.2% to 2.4%). After the Tohoku event in 2011, not only the variety but also the number of cracks increases (i.e. *bw* changes from (log) 1.645 to 1.735 and  $A_{max}$  from 2.4% to 4.7%). These observations are reflected in the recovery rate, p, which progressively increases from



2.0% to 3.7% and then to 4.1%, during the <2005, 2005-2011 and >2011 periods, respectively (i.e. Table 4.2).

**Figure 4.4** Evolution of the different relaxation parameters as a function of time for Japanese SRC (triangles), RC (squares), and S (circles) buildings. a) Minimum relaxation time  $\tau_{min}$  b) Maximum relaxation time  $\tau_{max}$  c) Log-time slope *p* d) Characteristic relaxation time  $\tau_c$  e) Bandwidth *bw* and f) Maximum spectral amplitude *A<sub>max</sub>*. Each marker represents the averaged value of 10 consecutive events. The yellow diamonds correspond to the mean response, *MeanALL*, considering all typologies, and the vertical black lines are one standard deviation from *MeanALL*. The two vertical dashed lines correspond to the mean response. *MeanALL*. The two vertical dashed lines correspond to the mean response of the error ( $\epsilon$ ) for each building typology with respect to the mean response *MeanALL*, is shown at the bottom of each plot. The error is computed as:  $\epsilon = (X_{typology} - X_{meanALL}) / \sigma_{meanALL}$ ; where  $X_{typology}$  denotes each relaxation parameter (i.e.  $\tau_{min}$ ,  $\tau_{max}$ , *p*...) for SRC, RC and S, respectively;  $X_{meanALL}$  is the corresponding standard deviation.

Finally, granular materials show, on average, higher values of p and  $A_{max}$  for equivalent time periods, confirming that the slow dynamic processes are controlled by the heterogeneities. This is also confirmed by the error ( $\epsilon$ ) plots shown at the bottom of each plot in (Fig. 4.4). The errors  $\epsilon$  for RC and SRC buildings mostly fall within  $2\sigma$  throughout the time period. Errors for Steel structures are larger, and some points fall outside the  $2\sigma$  threshold, notably after 2011, reaching maximum errors of approximately  $5\sigma$ . From this, we infer that: 1) mean recovery in buildings is mainly governed by heterogeneous materials such as RC and SRC, 2) as the level of damage increases (i.e. after significant earthquakes), the mean response of "homogeneous" materials (i.e. Steel) is not as well described by the mean response *MeanALL*. This proves the relationship between the recovery of elastic processes and the degree of fracturing in granular materials, as seen in laboratory and seismological observations (i.e. TenCate et al., 2000; Ostrovsky and Johnson, 2001; Johnson and Sutin, 2005; Brenguier et al., 2008; Wu et al., 2009a and 2009b; Johnson et al., 2009).

Table 4.2 Mean and standard deviation values of the relaxation parameters for the periods before 2005,
between 2005 and 2011, and after 2011 (defined by the occurrence of the 2004 Nigata-Chetsu and the
2011 Tohoku events, respectively) and for each building typology (i.e. SCR, RC and S). A graphic
representation of these values is shown in Fig. 4.4.

		Before 2005	2005-2011	After 2011
		$\mu\pm\sigma$	$\mu \pm \sigma$	$\mu\pm\sigma$
	RC	$0.508\pm0.179$	$0.623\pm0.159$	$0.762\pm0.190$
$log  \tau_{min}$	SRC	$0.581\pm0.178$	$0.919\pm0.158$	$0.924\pm0.153$
_	S	$0.663\pm0.140$	$0.855\pm0.182$	$1.076\pm0.135$
	RC	$0.981\pm0.127$	$1.179\pm0.175$	$1.314 \pm 0.130$
$\log\tau_{max}$	SRC	$0.924\pm0.141$	$1.315\pm0.158$	$1.443\pm0.145$
_	S	$1.198\pm0.034$	$1.305 \pm 0.091$	$1.525 \pm 0.071$
	RC	$0.033\pm0.016$	$0.031\pm0.010$	$0.035\pm0.011$
р	SRC	$0.020\pm0.008$	$0.037\pm0.013$	$0.041\pm0.014$
	S	$0.018\pm0.003$	$0.019\pm0.011$	$0.023\pm0.009$
	RC	$0.903\pm0.126$	$1.014\pm0.127$	$1.136 \pm 0.116$
$\log \tau_c$	SRC	$0.809\pm0.133$	$1.138\pm0.124$	$1.251 \pm 0.106$
	S	$0.941\pm0.157$	$1.171 \pm 0.127$	$1.352\pm0.105$
	RC	$1.355\pm0.117$	$1.443 \pm 0.105$	$1.632 \pm 0.116$
log bw	SRC	$1.270\pm0.140$	$1.645 \pm 0.136$	$1.735\pm0.109$
	S	$1.366\pm0.137$	$1.586\pm0.095$	$1.807\pm0.121$
A <sub>max</sub>	RC	$0.045 \pm 0.015$	$0.041 \pm 0.014$	$0.042 \pm 0.012$
	SRC	$0.032\pm0.011$	$0.024\pm0.010$	$0.047\pm0.014$
	S	$0.021\pm0.007$	$0.022\pm0.007$	$0.025\pm0.007$

In Fig. 4.4, some relaxation parameters such as p and  $A_{max}$ , are highly dependent on the material, whereas others, like  $\tau_{min}$  and  $\tau_c$ , are material-dependent but to a lesser extent. Values of  $\tau_{max}$ , however, do not seem to be controlled by the building typology (i.e. the mean response of each material falls within  $2\sigma$  of the *meanALL* response). This is consistent with the findings of Snieder et al., 2017, which state that  $\tau_{max}$  is mainly controlled by the loading amplitude.

The maximum frequency variation shows to be related to loading and structural material, showing also a clear link between material and damage. In Figure 4.5, the model of Ostrovsky et al., (2019) is applied to fit our building data. This gives the mean frequency variations before and after the Tohoku earthquake, for each building typology and according to the level of structural deformation (i.e. proxy of loading, in this case). Two groups of data are displayed in Fig. 4.5 corresponding to structural drifts,  $\Delta_2$  ranging from  $5\times10^{-6}$  to  $1\times10^{-5}$ , and  $\Delta_5$  from  $1\times10^{-4}$  to  $5\times10^{-4}$ . The mean ( $\mu$ ) and standard deviation ( $\sigma$ ) values are shown in Table 4.3. For all typologies, the mean frequency variation and its variability increase as loading increases (i.e. from  $\Delta_2$  to  $\Delta_5$ ) and damage increases (i.e. from before to after Tohoku), corresponding to the observations in Chapter 3. RC is the typology with the highest values of  $\mu$  and  $\sigma$  (Table 4.3). For example, after Tohoku and for  $\Delta_5$ , the variations are  $12.357\pm7.076$ ,  $10.518\pm3.722$  and  $5.426\pm1.119$  for RC, SRC and S, respectively. Frequency degradations in RC structures are approximately twice those of S buildings. This proves the sensitivity of fundamental frequency fluctuations to the presence of heterogeneities in the material, confirming the conclusions of Fig. 4.2.



**Figure 4.5** Recovery of the fundamental frequency over time for RC, SRC and S buildings and different strain amplitudes a) before the Tohoku earthquake and b) after the Tohoku earthquake. The solid curves represent the mean frequency recovery,  $\Delta f$ , normalized to the final frequency,  $f_{fapp}$ , (see Chapter 3). The colors correspond to two drift ranges ( $\Delta_2 = 5 \times 10^{-6} - 1 \times 10^{-5}$  and  $\Delta_5 = 1 \times 10^{-4} - 5 \times 10^{-4}$ ).

	-	Before 2011	After 2011
		$\mu \pm \sigma$	$\mu \pm \sigma$
	RC	$7.155 \pm 2.928$	$9.380 \pm 3.667$
Δ2	SRC	$4.620 \pm 1.774$	$6.313 \pm 2.970$
	S	$3.389\pm0.324$	$4.468 \pm 1.443$
	RC	$9.691 \pm 2.832$	$12.357 \pm 7.076$
Δ5	SRC	$9.889 \pm 2.526$	$10.518\pm3.722$
	S	$5.105 \pm 1.316$	$5.426 \pm 1.119$

**Table 4.3** Mean and standard deviation of fundamental frequency variations of the curves shown in Fig. 4.5 for each period, material and strain level.

Todorovska et al., (2006) and Guéguen et al. (2016) attribute the recovery of the system frequency in buildings to changes in the soil or in the soil-structure boundary. In Chapter 2, we showed that soil effects cannot be ignored, particularly after major earthquakes causing high strain values in the uppermost soil layers. However, the slow dynamics observed in the ANX building were mainly controlled by its structural state. In order to develop this analysis, the relaxation spectrum (i.e. Shokouhi et al., 2017a) applied to one building of each typology (i.e. SRC, RC and S) is shown Fig 4.6 in order to analyze the frequency recovery after the Tohoku earthquake. The buildings were built during the same period (i.e. 1994, 1995 and 1998), they are located in the same area (i.e. around Tokyo) with foundations in soil with Vs<sub>30</sub> falling within the 220-300m/s range.

Clear distinctions are observed between the slow dynamics of the 3 buildings, with stronger evidence of relaxation effects in the RC building. The RC building (i.e. black curve) shows the maximum frequency variation  $\Delta f/f$ , reaching 5% degradation. The S building (i.e. red curve) shows the smallest  $\Delta f/f$  (i.e. 0.8%) and the SRC structure (i.e. blue curve) exhibits an intermediate response, with a maximum frequency variation of 1.8%. The recovery slope is also clearly steeper for the RC structure. The RC spectrum shows a value of A<sub>max</sub> that is 3.3 and 7.2 times higher than the values for the SRC and S buildings, respectively. The spectrum bandwidth is smaller in the RC building and larger in the S sample. This suggests that the RC building presents a smaller variety of cracks, with a smaller typical size (i.e. smaller  $\tau_c$ , spectrum shifted to the left) but a much larger number of cracks than the other two structures. The SRC building shows a response more similar to the S building, but with larger characteristic cracks (i.e. larger  $\tau_c$ , spectrum shifted to the right). Because the site conditions are the same, the effects of the soil on the system recovery should be equivalent for all three buildings. These results therefore confirm the conclusions drawn from the ANX building in Chapter 2 on the

predominance of structural response on total recovery. These results also support the observations from Figures 4.2-4.5 and Tables 4.1-4.3 on the stronger influence of structural heterogeneities on frequency variations and slow dynamics.



**Figure 4.6** (Top) Normalized fundamental frequency variation,  $\Delta f/f$ , as a function of relaxation time during recovery after the Tohoku earthquake in 3 buildings: RC (black dashed curve), SRC (blue dotted-dashed curve) and S (red solid curve). (Bottom) Relaxation spectra for the 3 analyzed buildings, which are located in the same area, founded on similar soil and built during the same period.

In this study we relate frequency variations to loading amplitude, building typology, and structural state. The relationship with loading features, however, is still not totally clear. The behavior observed in the frequency variation as a function of structural drift (Fig. 4.1) cannot be explained by either loading amplitude or building typology. In the next section, we will evaluate the influence of loading rate on the structural response, presenting two case studies: the ANX building and the THU building.

## 4.4. Influence of loading and loading rate

In field observations, velocity reductions and recovery rates due to earthquake-related processes are proportional to the degree of excitation (i.e. Richter el al., 2014; Gassenmeier et al., 2016; Rubinstein et al., 2007). Similarly, in rocks and granular media, modulus softening and successive recovery appear to be dependent on both effective pressure and wave duration (TenCate and Shankland, 1996; Johnson and Jia, 2005; Johnson and Sutin, 2005). Moreover, Johnson and Jia, (2005) detected a strain threshold below which the material behaves in a linear elastic manner. This threshold increases progressively with pressure, to approximately 10<sup>-6</sup>. The resonance frequency is therefore dependent on strain for strain values above this threshold. However, Guéguen et al., (2016) noticed significant nonlinear elastic behavior in the

dependency of the frequency variation on deformations in buildings under weak and strong excitation, starting at values as low as  $<10^{-5}$ .

#### 4.4.1. Case 1: the ANX building

Figure 4.7 shows the transient change in normalized frequency of the ANX building as a function of structural deformation, computed each 0.04s time step, during earthquakes occurring during the structurally stable period before the Tohoku event. (i.e. P2 in Fig. 3.4). The events are classified according to the maximum strain triggered in the building,  $\Delta_{max}$ , as a parameter consistent with loading amplitude (Fig. 3.2 and Chapter 1). Five ranges of  $\Delta_{max}$  were considered, varying from  $1 \times 10^{-6}$  to  $5 \times 10^{-4}$  (similar to the strain ranges defined in Chapter 3). The relationship between frequency reduction and deformation starts at very low values, around  $1 \times 10^{-8}$  for the smallest events, and at  $1 \times 10^{-6}$  in cases of stronger excitation (Fig. 4.7). This is analogous to the observations from Fig. 4.1 and those made by Johnson and Jia, (2005) and Guéguen et al., (2016), and represents a typical signature of nonlinear elasticity, as observed in several materials (i.e. Guyer and Johnson, 1999; Ostrovsky and Johnson, 2001; Guyer and Johnson, 2009 and references therein). Moreover, for the same deformation, the modulus softening diminishes progressively as loading increases, indicating that the structural response is not governed by strain amplitude.



**Figure 4.7** The change in normalized frequency,  $\Delta f/f = (f-f_i)/f_i$ , of the ANX building with respect to structural deformation, and drift, computed every 0.04s, within the time-histories of earthquakes occurring during the stable period between 2006 and March 2011.  $f_i$  represents the pre-seismic fundamental frequency. Five ranges of loading amplitude are considered, i.e.  $\Delta_{max}$ .

It is known that most structural materials used in civil engineering are highly sensitive to the process of loading. Properties such as strength, stiffness and ductility might be strain rate dependent (Bischoff and Perry, 1991; Houqun et al., 2016). The strain rate effect is a basic property in the dynamic response of heterogeneous materials, such as concrete. For example,

stiffness can be enhanced at high loading rates in the compressive behavior of concrete. Bischoff and Perry (1991) explained this enhancement by the decrease in internal microcracking, for a given level of stress, as strain rate increases. Furthermore, Johnson and Jia (2005) noted that, for equivalent strain amplitudes, the dynamically-induced reduction in elastic modulus was less pronounced than in resonance experiments, concluding that conditioning effects also control modulus softening; this is also seen in diverse systems, including damaged materials (TenCate and Shankland, 1996; Johnson and Sutin, 2005). Moreover, previous studies of concrete structures under earthquakes (Hougun et al., 2016) have indicated that the maximum strain rate observed in these cases ranges from 10<sup>-3</sup> s<sup>-1</sup> to 10<sup>-2</sup> s<sup>-1</sup>. The authors determined that the mechanisms controlling strain rate effects are mainly related to thermally-activated mechanisms and energy dissipation. Similar conclusions were made by Bischoff and Perry (1991) and Qi et al., (2009), who observed that the low-rates region is controlled by thermalactivated mechanisms. As strain rate increases, there is a transition between the mechanisms governing the material response: dissipation mechanisms emerge and gradually become dominant, as well as inertial effects. Crack density increases, and rate sensitivity is a result of the combined effects of ongoing energy dissipation and new thermally-activated mechanisms. In heterogenous materials, such as concrete, there might be a critical strain rate above which internal microcracks start to open/close progressively, controlling the overall response of the system.

Figures 4.8a and 4.8b show the effects of strain rate on the frequency variation analyzed during fast loading (i.e. co-seismic transient frequency drop) and recovery (i.e. after loading), respectively. Different strain amplitudes (i.e.  $\Delta_{max}$  in Fig. 4.7) are considered. Earthquakes occurring during stable periods (i.e. P2 and P4 in Chapter 3) are considered, grouped and averaged. Several comments can be made about Figure 4.8:

- The pattern of frequency variations with strain rate during loading is different to that during recovery regimes. Frequency reductions during the loading phase seem to be dependent on strain rate. Regardless of loading amplitude, frequency reductions are observed at values around  $10^{-7}s^{-1}$  (Fig. 4.8a-Left), which represent the activation point of nonlinearities related to the presence of cracks (i.e. Houqun et al., 2016; Bischoff and Perry, 1991; Qi et al., 2009). Above  $10^{-7}s^{-1}$ , as strain rate increases, the fundamental frequency decreases, following the same slope for different loading amplitudes (i.e. parallel slopes in the  $\Delta f/f$  versus strain rate relationship). This result suggests that the transition to nonlinear response at a strain rate of  $10^{-7}s^{-1}$  implies the progressive

activation of internal microcracks, causing constant frequency reductions at any loading amplitude.

- The activation strain rate value 10<sup>-7</sup>s<sup>-1</sup> does not change after the Tohoku earthquake (Fig. 4.8a-Right). Internal microcracking is activated for the same strain rate. However, the Δf/f loading amplitude slopes decrease after Tohoku, reaching smaller frequency reductions than before 2011 for the same strain rate. This effect is more obvious with stronger excitation and higher strain rates, implying that structural response is governed by secondary order factors rather than loading. For example, the distribution of crack size controls stiffness and the response is more dependent on structural state. As crack density increases, the energy necessary to activate new cracks (and therefore reduce frequency) increases, as illustrated by the slighter slope after Tohoku (Fig. 4.8a-Right). After 2011 the maximum frequency reduction seems to decrease, and this value is not clearly correlated with loading amplitude.
  - During the frequency recovery period before the Tohoku earthquake (Fig. 4.8b-Left), frequency variations with strain rate are guasi parallel for different loading amplitudes. This indicates gradual frequency recovery as conditioning effects decrease. For example, taking the stronger excitation levels (i.e. red or yellow curves in Fig. 4.8b-Left) and starting from a strain rate of 10<sup>-4</sup>s<sup>-1</sup>, the normalized frequency variation is around 0.10; at this strain rate, the structure is still under the effects of conditioning. As the excitation passes, strain rate decreases, as does the frequency variation. At 10<sup>-5</sup>s<sup>-1</sup>,  $\Delta f/f$  is around 0.02, and at 5x10<sup>-6</sup>s<sup>-1</sup>  $\Delta f/f$  is 0, denoting the end of recovery (i.e. relative to the final apparent value, f<sub>f</sub> of the earthquake recording). The same result is observed for all loading amplitudes, with recovery starting at lower strain rates, corresponding to the strength of the event. Recovery rates thus increase as the amplitude of excitation increases, as also observed in laboratory experiments on granular materials (i.e. Ostrovsky and Johnson 2001; Shokouhi et al., 2017a). At strain rates in the order of 10<sup>-</sup>  $^{7}$ s<sup>-1</sup>, corresponding to noise rate levels, recovery is complete for any excitation amplitude. This value matches the threshold that activates microcracking during loading (i.e. Fig. 4.8a).



**Figure 4.8** The change in normalized frequency,  $\Delta f/f$ , of the ANX building with respect to strain rate during a) loading and b) recovery phases. Strain rate is computed as  $(\Delta_{i+1}-\Delta_i)/t_s$ , with  $t_s = 0.04s$  and  $\Delta$ =structural drift. (Left) Structural response before the Tohoku earthquake (considering events from the stable period between 2006 and March 2011). (Right) Structural response after the Tohoku earthquake (considering events from the stable period in 2012). f<sub>i</sub> represents the pre-seismic fundamental frequency and f<sub>f</sub> corresponds to the final apparent fundamental frequency observed in the earthquake recordings. The colors represent different loading amplitude ranges, in terms of maximum strain,  $\Delta_{max}$ , indicated in the last plot. For b), the plots must be read from right to left, because we are in the recovery phase and loading effects are passing.

Effects of the 2011 Mw 9.1 earthquake are also evident in frequency recovery as a function of strain rates (Fig. 4.8b-Right). Firstly, behavior is less dependent on loading amplitude (i.e. equal behavior for different amplitude ranges, for example, the red and green curves). Secondly, the shape of the  $\Delta f/f$  versus strain rate curves has changed, except for the smallest loadings (i.e. black curve) and shows two regimes: at the beginning of the recovery period (i.e. highest strain rates), the  $\Delta f/f$  variation is very fast with no strain rate variation. Then, the frequency recoveries as strain rate decreases, until the structure is no longer conditioned by shaking and therefore  $\Delta f/f$  becomes 0 at around  $10^{-7}s^{-1}$ .

# 4.4.2. Case 2: the THU building

Elastic property variations in granular materials after dynamic excitation depend on loading and structural state (i.e. Van Den Abeele et al., 2000b; Ostrovsky and Johnson, 2001; Guyer and Johnson, 2009). Figures 4.7 and 4.8 show the effects of loading and loading rate on the reduction and recovery of fundamental frequency in the ANX building. However, Figure 4.8

also shows that structural response changes after damaging earthquakes, causing frequency variations less sensitive to loading features and more linked to the current state of cracking within the structural material. These observations correspond to the ANX building, which was considered as being slightly damaged by the 2011 Tohoku event (Kashima, 2014). The effects of greater damage are observed in Figure 4.9, which shows frequency variations as a function of strain rate during loading and recovery (similar to Fig. 4.8) for the THU building.

The lowest range of loading amplitude (i.e.  $\Delta_{max}$ ) in Figure 4.9 is equivalent to the  $\Delta_{3max}$  of Figure 4.8, denoting the high excitation levels experienced by the THU building over its lifespan. This is also shown by the lowest strain rate computed, around  $10^{-7}$  s<sup>-1</sup>. Frequency variations start to become dependent on strain rate at ~ $10^{-5}$  s<sup>-1</sup> (i.e. vertical dashed lines in Figure 4.9a) for any loading amplitude, confirming the existence of a threshold that determines the transition between linear and nonlinear behavior. However, the level of cracking in the THU building causes frequency variations to appear at a higher strain rate value  $10^{-5}$  s<sup>-1</sup>, compared with  $10^{-7}$  s<sup>-1</sup>), confirming that more energy is required to change the current state of a material that is already cracked.

The building response during loading changed after 2011, showing faster frequency reductions which might be linked to the substantial increase in the density of existing cracks. The energy required to propagate an existing crack is much less than the energy required to open a new crack. Therefore, the strain rate (i.e. energy) necessary to cause certain frequency reductions after 2011 decreases compared with that needed to cause the same frequency reduction before 2011. This implies that the already cracked structure (i.e. before 2011) suffered a significant increase in existing cracks, because of its high susceptibility to crack propagation at lower energy levels. This explains the steeper gradient of the slopes in Figure 4.9a-Right.

In the recovery phase figure 4.9b., high frequency recovery rates are observed, reaching  $\Delta f/f = 0$  at strain rates between 10<sup>-5</sup> s<sup>-1</sup> and 10<sup>-4</sup> s<sup>-1</sup>. Steep recovery slopes are observed in damaged materials (i.e. TenCate et al., 2000; Ostrovsky and Johnson, 2001; Chapter 3 and Table 4.2) because there are more cracks, and internal particles move quickly to fill them (See Chapter 3).


Figure 4.9 Similar to Figure 4.8 but for the damaged building, THU.

## 4.5. Conclusions

In this chapter, earthquake data recorded in several buildings were analyzed with the main objective of determining the parameters controlling nonlinear elastic processes, such as frequency drops and slow dynamics. Long-term monitored buildings of different typologies were studied, showing evidence of nonlinear elasticity similar to specific cases, such as the ANX and THU buildings, presented in previous chapters. Extending the behavior observed to buildings with different characteristics is an important step that might constitute the basis for a general method to estimate overall structural health, based on the monitoring of nonlinear elasticity and recovery effects after earthquakes.

The fundamental frequency of a structure varies proportionally to the square root of its stiffness. Detecting frequency changes might therefore represent stiffness changes, which is fundamental to the monitoring of structural degradation and damage. Damage assessment is usually based on visual inspections; however, civil infrastructures suffer from modifications of their internal properties that are not visible, but that can accumulate to the point of causing macroscopic changes and compromising the safety of the structure. One of the advantages of examining nonlinear elasticity is that we can infer internal degradation due to aging or external forces, which enables structural response variations to be tracked and damage to be detected at an early stage. Fundamental frequency, however, is just a global indicator of damage; it cannot detect

stiffness changes if the damage is localized. A complementary study is therefore necessary to include the analysis of higher modal frequencies.

Our results show a general tendency of frequency variations and relaxation parameters over time that is independent of building typology. However, the intensity with which nonlinear elastic processes are demonstrated is clearly dependent on the structural material as well as on the structural state. The parameters of frequency drops and slow dynamics are much more pronounced in granular, heterogeneous materials, such as RC or SCR. Steel buildings also show nonlinear elastic signatures, but to a lesser extent, and the mechanisms governing the response might be more related to heterogeneities in border conditions or joint deformations.

Response variability is also associated with building typology and structural state. Steel structures show an almost uniform response, with little inter-building and inter-event variation. Conversely, RC and SRC structures show higher inter-building variability, due to different levels of cracking and the contribution of responses from different structural -and nonstructural-elements. Variations in environmental conditions can also modify system response (Clinton et al., 2006; Todorovska and Al Rjoub, 2006). The characteristics of our data do not enable quantitative correlation between weather conditions and frequency variations. However, ambient effects are assumed to be small (i.e. around 1%) and compensatory during long-term monitoring. The degree of frequency variation observed is directly linked with the occurrence of strong motions. Soil effects are also known to contribute to system frequency variation, but in our case, we determined that the superstructure-response controls all the observed behavior.

Furthermore, we observed that frequency variations are also dependent on loading features (i.e. strain amplitude, strain rate):

- Softening (i.e. a decrease of frequency due to a decrease in the modulus) increases as loading values increase (Fig. 4.7). The observed variations in the elastic properties are mainly due to transient and permanent changes in structural stiffness, which is controlled by the bond system; in this case, it is the system of cracks and other heterogeneities that creates weaknesses in the medium. The structural state changes reported for different building typologies (Fig. 4.2) are apparent in the slow dynamic recovery process following earthquakes (Fig. 4.2-right) and the relaxation parameters over time (Fig. 4.4).
- As the density of heterogeneities (i.e. damage) increases, the structural response is less dependent on loading features and more governed by the level of heterogeneities

(Section 4.4). The structural response becomes more complex and more difficult to predict using loading parameters, which is typical in nonlinear behavior.

- Structural response seems to be more sensitive to strain rates than to strain amplitudes.
  Strain amplitudes determine the maximum frequency variation Δf, whereas strain rates control the behavior of frequency variations during loading and recovery periods (Section 4.4).
- Variations of frequency with strain rates show different behavior for loading and recovery periods: during loading, a threshold defines nonlinear behavior, i.e. the activation of heterogeneities such as cracks. During recovery, frequency variations decrease as conditioning effects decrease, and higher excitation amplitudes show higher recovery rates.
- The threshold defining strain rate dependence during loading is around 10<sup>-7</sup> s<sup>-1</sup> in slightly cracked media. This threshold increases to 10<sup>-5</sup> s<sup>-1</sup> in densely cracked materials, indicating the higher level of energy needed to cause structural response variations in damaged media.

In this study we observe a strong correlation between nonlinear elastic processes and structural damage. Analyzing the recovery of frequency drops and slow dynamics after earthquakes is therefore a potential means of monitoring structural health for several reasons. Firstly, because studies suggest that different building typologies behave in the same manner under certain degrees of variability. Secondly, because nonlinear elastic processes are produced at different levels of loading, starting at very small strain levels, and can be observed by short- and long-term monitoring. Thirdly, because the slow dynamics and relaxation parameters, which are relatively easy to compute, can describe internal properties linked to structural damage. Comprehensive new models to analyze structural response and monitor structural degradation should include: frequency variations over time, slow dynamic relaxation parameters, and loading features such as strain rate.

## **General Conclusions**

In this study we characterized the building response by analyzing nonlinear elastic processes during earthquakes. Previous studies have revealed the presence of these processes in buildings, but to our knowledge, this is the first time that they are analyzed in more detail, notably the slow dynamics recovery following seismic excitations. The physical origin of these processes was discussed presenting direct analogies with laboratory observations. The main concerns were to determine if it is possible to find clear relationships between nonlinear elastic signatures and structural state, and to reveal the parameters governing the observed structural response.

We first compiled real earthquake data recorded in buildings to create a new database including computed parameters that describe ground motion intensity and building response. This database will soon be available to the public in the form of a flat-file, providing invaluable information for the earthquake engineering community. Real earthquake data collected in buildings allow for the validation and calibration of existing models, and help the understanding of the physics controlling the structural behavior. Reduction of epistemic uncertainties is also possible with the development of data-driven damage prediction equations. The database might be also useful to determine efficiency and sufficiency of ground motion parameters in the prediction of structural response, besides other numerous applications with high potential use in performance-based earthquake engineering.

The database includes the ANX building, with 1630 events recorded in 20 years of measurements. We analyzed this building to confirm the presence of nonlinear elasticity at short- and long-term structural monitoring. The ANX building shows clear signatures of nonlinear elastic response, in the same manner as observed in laboratory experiments and in the Earth's crust. Variations of the fundamental frequency of the soil-structure system are observed even for very low levels of strain, and slow dynamics recovery is observed for ~6 months during the Mw 9.0 Tohoku aftershock sequence. We concluded that nonlinear elastic response in the ANX building is mostly controlled by nonlinearities in the superstructure, created due to the presence of cracks during earthquakes. The contribution of the nonlinear soil-response is non-negligible, particularly for higher strains (>10<sup>-5</sup>). An interesting observation is that, despite a permanent frequency, the building is still operative, contradicting some frequency-based decision criteria that would have catalogued the building as inoperative. This

evidences the complexity of nonlinear structural response, as well as the need for further datadriven analyzes to calibrate and improve decision-making methods for structural health monitoring after strong earthquakes.

The frequency recovery observed in buildings after earthquakes was analyzed by adapting models developed in laboratory to describe relaxation of granular materials. Despite the differences of scales, loading conditions and complexity levels, the relaxation models are able to characterize frequency recovery in buildings, showing clear connections with the degree of fracturing of the structural elements. The level of heterogeneities in the material controls the behavior of relaxation parameters, which are also sensitive to loading. We observed a good correlation with strain amplitudes, confirming that nonlinear response is proportional to the level of dynamic strain, as also seen in laboratory. We similarly observed that some relaxation parameters can serve as good proxies of the extension of the bond system (i.e. increase in the number of cracks, or extension of the cracks variety). On the other hand, the multiscale feature of slow dynamics was proved in buildings: recovery processes are observed within- and between earthquakes, showing relaxation processes lasting from seconds to years. Hysteresis and discrete memory effects during the recovery are also comparable to those observed in laboratory, confirming the universality of this phenomenon. By analogy with relaxation experiments, we infer that nonlinear elastic processes within the bond system might explain transitory and permanent variations of structural response to seismic events, where thermodynamical and mechanical factors might contribute to the internal organization of particles that allows for the closing of cracks, manifested as frequency recoveries.

The extension of the observed nonlinear elastic behavior to different typology buildings might constitute the fundamental basis for a general method to estimate global structural health, based on the monitoring of frequency shifts and recovery effects after earthquakes. Our results show that there is a general tendency of frequency variations and relaxation parameters that is independent on the building typology. However, variabilities in the nonlinear elastic processes are material-dependent. These processes are manifested with greater intensity in granular materials such as reinforced concrete, and mixed structures like steel-reinforced concrete buildings. Nonlinear elastic observations in steel structures are less forceful, and they might be linked to joints deformations or heterogeneities in boundary conditions. On the other hand, we observed that the behavior of fundamental frequency variations is dependent on strain-rates, which seem to control loading and recovery phases. During loading, there is a threshold value of strain-rate defining the nonlinear behavior, i.e. the activation of heterogeneities. This threshold is around 10<sup>-7</sup> s<sup>-1</sup> in slightly-cracked media and around 10<sup>-5</sup> s<sup>-1</sup> in densely-cracked materials, evidencing the increase of energy needed to cause variations in the structural response of damaged media. During recovery, damaged materials and strongest earthquakes show faster recovery rates, supporting the conclusions from the behavior of the relaxation slope. We clearly see that, as damage increases, the structural response becomes more complex and less sensitive to loading parameters. This is typical of nonlinear behavior. We believe that the analysis of nonlinear elastic response in buildings, notably slow dynamics, might become a simple and powerful means to monitor structural health.

## Perspectives

A lot of effort and further research are still necessary to fully understand the nonlinear elastic structural response, and to unveil the physical processes responsible for that. However, our results indicate that nonlinear elasticity and slow dynamics in buildings have a great potential as a proxy for structural damage. Nonlinear elastic behavior has revealed that fundamental frequencies are not static over time, on the contrary, they change along the life-span of a building. Structural models, therefore, must be updated according to the current structural state, to guarantee the correct structural performance during the next earthquake. In figure C1, we show the maximum structural fundamental period (i.e. coseismic fundamental frequency) of the ANX building as a function of maximum structural drift.



Figure C1. Fundamental period as a function of structural drift for the direction Y of the ANX building.

It is evident that the relationship between the two parameters has changed accordingly the timeperiods observed in the building (chapters 2 and 3). The difference in the behavior, especially noted from P2 to P3 (i.e. after the Tohoku event) indicates that the building does not respond in the same manner for equivalent loadings. We practically are in presence of a different structure. Novel models should therefore consider the possibility of structural degradation by including frequency variations over time.

A powerful contribution to the research would be the incorporation of machine learning for the prediction of structural responses based on the observed nonlinear elastic processes. We created a simple basic model to predict the observed frequency variations in the ANX building over time, shown in figure C2. The model was trained with all the Japanese data (except those of the ANX building) using magnitude, epicentral distance and pre-seismic fundamental frequency as predictor variables.



**Figure C2.** Real observed (red curve) versus predicted (gray curve) variations of coseismic fundamental frequency in the direction Y of the ANX building. Dotted lines correspond to the confidence interval of the model ( $\pm 1\sigma$ ).

Another example is displayed in figure C3, where the coseismic fundamental frequency of reinforced concrete US buildings is predicted as a function of magnitude, epicentral distance, pre-seismic frequency and structural material, using a model trained with all the Japanese data and all US building typologies, excluding reinforced concrete. Figures C2 and C3 represent just some preliminary examples of the potential uses of machine learning to analyze and forecast nonlinear elastic responses in buildings facing earthquakes. The understanding of these processes in buildings will help us to improve our knowledge of the dynamics of civil engineering structures; and machine learning techniques might represent a useful tool for automatized early damage detection and prediction of structural responses.



**Figure C3.** True versus predicted values of coseismic frequency  $f_{min}$  as a function of a) magnitude  $M_w$ , b) epicentral distance R, c) structural material, and d) pre-seismic fundamental frequency,  $f_i$ . The predicted versus true  $f_{min}$  values is shown in e), and the histogram of the model errors is shown in f).

## References

Akkar and Bommer, J. (2007). Empirical prediction equations for Peak Ground Velocity derived from strong-motion records from Europe and the Middle East. *Bulletin of the Seismological Society of America* (2007), 97 (2), pp. 511-530. https://doi.org/10.1785/0120060141

Al Atik, L., Abrahamson, N., Bommer, J. J., Scherbaum, F., Cotton, F., and Kuehn, N. (2010). The variability of ground-motion prediction models and its components. *Seismological Research Letters*, 81(5), pp. 794-801.

Ambraseys, N.N., Douglas, J., Sarma, S.K., and Smit, P.M. (2005). Equations for the estimation of strong ground motions from shallow crustal earthquakes using data from Europe and the Middle East: Horizontal peak ground acceleration and spectral acceleration. *Bulletin of Earthquake Engineering*, Springer Verlag, 2005, 3(1), pp.1-53.

Araya, R., and Saragoni, G.R. (1980). Capacidad de los moviemientos sismicos de producir daño estructural. Report No. SES I 7/80, *Division of Structural Engineering*, University of Chile.

Araya. R., and G.R. Saragoni. (1984). Earthquake accelerogram destructiveness potential factor. 8th *World Conference on Earthquake Engineering*. San Francisco, USA.

Arias, A. (1970). A measure of earthquake intensity. R.J. Hansen. *Seismic Design for Nuclear Power Plants*, MIT Press, Cambridge, pp. 438-469.

ASCE Standard, ASCE/SEI 7-05. (2006). Minimum design loads for buildings and other structures. *American Society of Civil Engineers*. ISBN: 0-7844-0831-9

Astorga A., Guéguen, P., and Kashima, T. (2018). Nonlinear elasticity observed in buildings during a long sequence of earthquakes. *Bulletin of the Seismological Society of America* (2018) 108(3A), pp. 1185-1198. DOI: https://doi.org/10.1785/0120170289.

Astorga A., Guéguen P., Rivière J., Kashima T., and Johnson P.A. (2019). Recovery of the resonance frequency of buildings following strong seismic deformation as a proxy for structural health. *Structural Health Monitoring*, DOI: 10.1177/1475921718820770

Auger, F., and Flandrin, P. (1995). Improving the readability of time-frequency and time-scale representations by the reassignment method. *IEEE Transactions on signal processing*, 43(5), pp. 1068-1089.

Averbakh, V.S., Lebedev, A.V., Maryshev, A.P., and Talanov, V.I. (2009). Observation of slow dynamics effects in nonconsolidated media under *in-situ* conditions. *Acoustical Physics*, 55, pp. 211-217.

Baisch, S., and Bokelmann, G. H. (2001). Seismic waveform attributes before and after the Loma Prieta earthquake: Scattering change near the earthquake and temporal recovery, *Journal Geophysics Research: Solid Earth* 106(B8), pp. 16323-16337, DOI: 10.1029/2001JB000151.

Baker, J., and Cornell, C.A. (2008). Uncertainty propagation in probabilistic seismic loss estimation. *Structural Safety* 30(3), pp. 236-252.

Barbosa, A.R., Ribeiro, F. L., and Neves, L.A. (2017). Influence of earthquake ground motion duration on damage estimation: application to steel moment resisting frames. *Earthquake Engineering and Structural Dynamics*, 46(1), pp. 27-49.

Bindi, D., Cotton, F., Spallarossa, D., Picozzi, M., and Rivalta, E. (2018). Temporal Variability of Ground Shaking and Stress Drop in Central Italy: A Hint for Fault Healing? *Bulletin of the Seismological Society of America*, 108, (4), pp. 1853-1863. DOI: http://doi.org/10.1785/0120180078.

Bischoff, P.H., and Perry, S.H. (1991). Compressive behaviour of concrete at high strain rates. *Materials and Structures*, 24, pp. 425-450.

Bodin, P., Vidale, J., Walsh, T., Çakir, R., and Çelebi, M. (2012). Transient and long-term changes in seismic response of the natural resources building, Olympia, Washington, due to earthquake shaking. *Journal of Earthquake Engineering*, 16(5), pp. 607-622.

Bommer, J., and Martínez-Pereira, A., (1999). The effective duration of earthquake strong motion. *Journal of Earthquake Engineering*, 3(2), pp. 127-172, DOI: 10.1080/1363246990.

Bommer, J., Magenes, G., Hancock, J., and Penazzo, P. (2004). The influence of strong-motion duration on the seismic response of masonry structures. *Bulletin of Earthquake Engineering*, 2(1), pp. 1-26.

Bommer, J., and Alarcon, J. (2008). The prediction and use of Peak Ground Velocity. *Journal of Earthquake Engineering*. DOI: https://doi.org/10.1080/13632460609350586

Bonilla, L.F., Tsuda, K., Pulido, N., Régnier, J., and Laurendeau, A. (2011). Nonlinear site response evidence of K-NET and KiK-net records from the 2011 off the Pacific coast of Tohoku Earthquake. *Earth, Planets and Space* (Online), 63(7), 785.

Boore, D.M. (2005). On pads and filters: processing strong-motion data. *Bulletin of the Seismological Society of America*, 95(2), pp. 745-750.

Brenguier, F., Campillo, M., Hadziioannou, C., Shapiro, N.M., Nadeau, R.M., and Larose, E. (2008). Postseismic relaxation along the San Andreas fault at Parkfield from continuous seismological observations. *Science*, 321(5895), pp. 1478-1481.

Brenguier, F., Campillo, M., Takeda, T., Aoki, Y., Shapiro, N.M., Briand, X., ... and Miyake, H. (2014). Mapping pressurized volcanic fluids from induced crustal seismic velocity drops. *Science*, 345(6192), pp. 80-82.

Building Research Institute, BRI Strong Motion Network. https://smo.kenken.go.jp/

Buratti, N. (2012). A comparison of the performances of various ground-motion intensity measures. *WCEE*, Lisboa.

California Strong Motion Instrumentation Program, CSMIP. https://www.conservation.ca.gov/cgs/smip

Calvi G.M, Pinho, R., and Crowley, H. (2006). State-of-the-knowledge on the period elongation of RC buildings during strong ground shaking. Proceedings of the *1<sup>st</sup> European conference of earthquake engineering and seismology*, Geneva, paper N° 1535.

Çelebi, M., Phan, L.T., and Marshall, R.D. (1993). Dynamic characteristics of five tall buildings during strong and low-amplitude motions. *The Structural Design of Tall Buildings*, 2(1), pp. 1-15.

Çelebi, M., Okawa, I., and Kashima, T. (2012). March 11, 2011 M=9.0 Great East Japan Earthquake and Aftershocks: The story of a 9-story building retrofitted, damaged during, and repaired after the mainshock. *15th World Conference on Earthquake Engineering*, Lisbon. Paper N° 0292.

Çelebi, M., Kashima, T., Ghahari, F., Koyama, S., and Taciroglu, E. (2016). Before and after retrofit behavior and performance of a 55-story tall building inferred from distant earthquake and ambient vibration data. *Earthquake Spectra*. 32(1), pp. 463-495. DOI: 10.1193/122216EQS249M

Center for Engineering Strong Motion Data, CESMD. https://strongmotioncenter.org/

Chandra, J., Guéguen, P., Steidl, J.H., and Bonilla, L.F. (2015). In Situ Assessment of the G– $\gamma$  Curve for Characterizing the Nonlinear Response of Soil: Application to the Garner Valley Downhole Array and the Wildlife Liquefaction Array. *Bulletin of the Seismological Society of America*, 105(2A), pp. 993-1010.

Chandra, J., Guéguen, P., and Bonilla, L.F. (2016). On the use of seismic interferometry technique for testing PGV/Vs as a proxy for predicting the nonlinear soil response. *Soil Dynamics and Earthquake Engineering*, *85*, pp. 146-160.

Chandramohan, R., Baker, J.W., and Deierlein, G.G. (2016). Quantifying the influence of ground motion duration on structural collapse capacity using spectrally equivalent records. *Earthquake Spectra*, 32(2), pp. 927-950.

Chiauzzi, L., Masi, A., Mucciarelli, M., Cassidy, J.F., Kutyn, K., Traber, J., Ventura, C., Yao, F. (2012). Estimate of fundamental period of reinforced concrete buildings: code provisions vs. experimental measures in Victoria and Vancouver (BC, Canada). *Proceedings of the 15th world conference on earthquake engineering*, Lisboa.

Clayton, R. W., and Wiggins, R. A. (1976). Source shape estimation and deconvolution of teleseismic bodywaves. *Geophysical Journal International*, 47(1), pp. 151-177.

Clinton, J., Bradford, C., Heaton, T., and Favela, J. (2006). The observed wander of the natural frequencies in a structure. *Bulletin of the Seismological Society of America* 96(1), pp. 237-257. https://doi.org/10.1785/0120050052

Clough, R.W., and Penzien, J. (1993). Dynamics of Structures. McGraw-Hill.

Coburn, A.W., and Spence, R.J.S. (2002). Earthquake protection, Second edition. John Wiley & Sons.

Cohen, L. (1989). Time-frequency distributions-a review. Proceedings of the IEEE, 77(7), pp. 941-981.

Daniels, K.E. & Hayman, N.W. (2008). Force chains in seismogenic faults visualized with photoelastic granular shear experiments. *Journal of Geophysical Research*, 113, B11411.

Delrue, S., Aleshin, V., Truyaert, K., Bou Matar, O., Van Den Abeele, K. (2018). Two dimensional modeling of elastic wave propagation in solids containing cracks with rough surfaces and friction – Part II: Numerical implementation. *Ultrasonics*, 82, pp. 19-30.

Di Sarno, L., and Amiri, S. (2019). Period elongation of deteriorating structures under mainshock-aftershock sequences. *Engineering Structures*. https://doi.org/10.1016/j.engstruct.2019.109341

Douglas, J. (2003). Earthquake ground motion estimation using strong-motion records: a review of equations for the estimation of peak ground acceleration and response spectral ordinates. *Earth-Science Reviews* 61 (2003), pp. 43-104.

Douglas, J. (2011). Ground motion prediction equations 1964-2010. Final report. *BRGM*/RP-59356-FR. 444 pages, 9 illustrations.

Dunand, F., Ait Meziane, Y., Guéguen, P., Chatelain, J. L., Guillier, B., Ben Salem, R., ... and Machane, D. (2004). Utilisation du bruit de fond pour l'analyse des dommages des bâtiments de Boumerdes suite au séisme du 21 Mai 2003. *Mémoires du Service Géologique de l'Algérie*, 12, pp. 177-191.

Dunand, F., Guéguen, P., Bard, P-Y., Rodgers, J., and Celebi, M. (2006). Comparison of the dynamic parameters extracted from weak, moderate and strong building motion. *Proceedings of the 1<sup>st</sup> European Conference of Earthquake Engineering and Seismology*. Geneva, Switzerland, paper 1021.

EPRI (1988). A Criterion for determining exceedance of the operating basis earthquake. In: *Electric Power Research Institute*, Palo Alto, CA, prepared by Jack R. Benjamin and Associates Inc, Report No: NP-5930

Farrar, C., and Worden, K. (2006). An introduction to structural health monitoring. *Philosophical Transactions of the Royal Society A: Mathematical, Physical and Engineering Sciences*, 365(1851), pp. 303-315.

Farrar, C., and Worden, K. (2007). Structural Health Monitoring - Preface. *Philosophical Transactions of the Royal Society A: Mathematical, Physical and Engineering Sciences*. 365(1851), pp. 299-301.

FEMA (1999). HAZUS Earthquake loss estimation methodology. *Federal Emergency Management Agency*, Washington D.C.

FEMA - Building Seismic Safety Council. (2004). NEHRP Recommended provisions for seismic regulations for new buildings and other structures (FEMA 450). *National Institute of Building Sciences*, Washington, D.C.

Field, E. H., Johnson, P. A., Beresnev, I. A., and Zeng, Y. (1997). Nonlinear ground-motion amplification by sediments during the 1994 Northridge earthquake, *Nature*, 390, pp. 599-601.

Gao, K., Guyer, R., Rougier, E. Ren, C.X., and Johnson, P.A. (2019). From stress chains to acoustic emission. *Physical Review Letters*. 123, 048003.

Garnier, V., Piwakowski, B., Abraham, O., Villain, G., Payan, C., and Chaix, J.-F. (2013). Acoustic techniques for concrete evaluation: Improvements, comparisons and consistency. *Construction and Building Materials*, 43, pp. 598-613.

Gassenmeier, M., Sens-Schönfelder, C., Eulenfeld, T., Bartsch, M., Victor, P., Tilmann, F., and Korn, M. (2016). Field observations of seismic velocity changes caused by shaking-induced damage and healing due to mesoscopic nonlinearity. Geophysical Journal International, 204(3), pp. 1490-1502, https://doi.org/10.1093/gji/ggv529

Gist, G.A. (1994). Fluid effects on velocity and attenuation in sandstones. *The Journal of the Acoustical Society of* America, 96, 1158.

Goel, R., and Chopra, A. (1996). Evaluation of code formulas for fundamental period of buildings. *Eleventh World Conference on Earthquake Engineering*, Paper N°. 1127.

Goueygou, M., Abraham, O., Lataste, J.-F. (2008). A comparative study of two non-destructive testing methods to assess near-surface mechanical damage in concrete structures. *NDT & E International*, 41(6), pp. 448-456.

Granato et al., (1956). Application of dislocation theory to internal friction phenomena at high frequencies. Journal of Applied Physics, 27, pp. 789–805.

Guéguen, P., Johnson, P., and Roux, P. (2016). Nonlinear dynamics induced in a structure by seismic and environmental loading. *The Journal of the Acoustical Society of America*, 140(1), pp. 582-590.

Guéguen, P., and Tiganescu, A. (2018). Consideration of the Effects of Air Temperature on Structural Health Monitoring through Traffic Light-Based Decision-Making Tools. *Shock and Vibration*, (2018), Article ID 9258675, 12 pages, https://doi.org/10.1155/2018/9258675

Guéguen, P., Brossault, M.-A., Roux, P. and Singaucho J.C. (2019). Slow dynamics process observed in civil engineering structures to detect structural heterogeneities. Accepted in *Engineering Structure*.

Guyer, R., McCall, K., and Boitnott, G. (1995). Hysteresis, discrete memory, and nonlinear wave propagation in rock: a new paradigm. *Physical Review* Letters, 74(17), pp. 3491-3494.

Guyer, R.A., and Johnson, P.A. (1999). Nonlinear mesoscopic elasticity: Evidence for a new class of materials. *Physics today*, 52, pp. 30-36.

Guyer, R.A., TenCate, J., and Johnson, P.A. (1999). Hysteresis and the dynamic elasticity of consolidated granular materials. *Physical Review Letters*, 82, 3280. https://doi.org/10.1103/PhysRevLett.82.3280

Guyer, R.A., and Johnson, P.A. (2009). Nonlinear mesoscopic elasticity: the complex behaviour of rocks, soil, concrete. *John Wiley & Sons*.

Herak, M., and Herak, D. (2009). Continuous monitoring of dynamic parameters of the DGFSM building (Zagreb, Croatia), *Bulletin of Earthquake Engineering*, 8(3), pp. 657-669.

Houqun, C., Shengxin, W., and Faning, D. (2016). Research progress on dynamic mechanical behavior of high arch dam concrete. Research progress on dynamic mechanical behavior of high arch dam concrete. In: *Seismic Safety of High Arch Dams*, pp. 239-256. https://doi.org/10.1016/B978-0-12-803628-0.00010-0

Housner, G.W. (1959). Behavior of structures during earthquakes. *Journal of the Engineering Mechanics Division*, 85(4), pp. 109-130.

Iervolino, I. (2017). Assessing uncertainty in estimation of seismic response for PBEE. *Earthquake Engineering & Structural Dynamics*, 46(10), pp. 1711-1723.

Jennings, P.C., and Kuroiwa, J.H. (1968). Vibration and soil-structure interaction tests of a nine-story reinforced concrete building. *Bulletin of the Seismological Society of America*, 58(3), pp. 891-916.

Jin, J., Rivière, J., Ohara, Y., and Shokouhi, P. (2018). Dynamic acousto-elastic response of single fatigue cracks with different microstructural features: An experimental investigation. *Journal of Applied Physics*, 124, 075303.

Johnson, P.A., and Rasolofosaon, P.N.J. (1996). Nonlinear elasticity and stress-induced anisotropy in rock. *Journal of Geophysical Research: Solid Earth*, 101(B2), pp. 3113-3124.

Johnson, P.A. and Jia, X. (2005). Nonlinear dynamics, granular media and dynamic earthquake triggering. *Nature*, 437, pp. 871-874.

Johnson, P.A., and Sutin, A. (2005). Slow dynamics and anomalous nonlinear fast dynamics in diverse solids. *The Journal of the Acoustical Society of America*, 117(1), pp. 124-130.

Johnson P.A. (2006). Nonequilibrium Nonlinear Dynamics in Solids: State of the Art. In: Delsanto P.P. (eds) *Universality of Nonclassical Nonlinearity. Springer*, New York, pp. 49-67.

Johnson, P.A., Bodin, P., Gomberg, J., Pearce, F., Lawrence, Z. and Menq, F. Y. (2009). Inducing in situ, nonlinear soil response applying an active source. *Journal of Geophysical Research: Solid Earth*, 114(B05) 304.

Kashima T. (2004). Dynamic Behavior of an 8th- storey SRC Building Examined from Strong Motion Records. *Proceedings of the 13th World Conference on Earthquake Engineering (13WCEE)*, Vancouver, 169.

Kashima, T., Koyama, S., Okawa, I. and Iiba, M. (2012). Strong Motion Records in Buildings from the 2011 Great East Japan Earthquake. *15th World Conference on Earthquake Engineering*, Lisbon. Paper N° 1768.

Kashima, T. (2014). Dynamic behavior of SRC buildings damaged by the 2011 great east Japan Earthquake based on strong motion records. *Second European Conference on Earthquake Engineering and Seismology*, Istanbul.

Kodera, K., De Villedary, C., and Gendrin, R. (1976). A new method for the numerical analysis of nonstationary signals. *Physics of the Earth and Planetary Interiors*, 12(2-3), pp. 142-150.

Kohler, M., Davis, P.M., and Safak, E. (2005). Earthquake and Ambient Vibration Monitoring of the Steel-Frame UCLA Factor Building. *Earthquake Spectra*, 21(3), DOI: 10.1193/1.1946707

Kohler, M.D., Heaton, T.H., and Bradford, S.C. (2007). Propagating waves in the steel, moment-frame factor building recorded during earthquakes. *Bulletin of the Seismological Society of America*, 97(4), pp. 1334-1345.

Korobov, A.I., Odina, N.I., and Mekhedov, D.M. (2013). Effect of slow dynamics on elastic properties of materials with residual and shear strains. *ISSN 1063 7710, Acoustical Physics,* 59(4), pp. 387-392.

Landau, L. and Lifshits, E. (1986). Theory of Elasticity. 3rd edition. Pergamon Press, Oxford.

Lebedev, A.V. and Ostrovsky, L.A. (2014). A unified model of hysteresis and long-time relaxation in heterogeneous materials. *Acoustical Physics*, 60, pp. 555-561.

Legland, J.-B., Zhang, Y., Abraham, O., Durand, O., and Tournat, V. (2017). Evaluation of crack status in a meter-size concrete structure using the ultrasonic nonlinear coda wave interferometry. *The Journal of the Acoustical Society of America*, 142, 2233, https://doi.org/10.1121/1.5007832.

Lesueur, C., Cara, M., Scotti, O., Schlupp, A., and Sira, C. (2013). Linking ground motion measurements and macroseismic observations in France: A case study based on accelerometric and macroseismic databases. *Journal of Seismology*, 17(2), pp. 313-333. DOI: https://doi.org/10.1007/s10950-012-9319-2

Limongelli, M.P., Dolce, M., Spona, D., Guéguen, P., Langlais, M., Wolinieck, D., Maufroy, E., Karakostas, C.Z., Lekidis, V.A., Morfidis, K., Salonikios, T., Rovithis, E., Makra, K., Masciotta, M.G., and Lourenço, P.B. (2019). S<sup>2</sup>HM in Some European Countries. In: Limongelli M., Çelebi M. (eds) Seismic Structural Health Monitoring. *Springer Tracts in Civil Engineering*. Springer, Cham, DOI 10.1007/978-3-030-13976-6\_13.

Lobkis, O.I., and Weaver, R.L. (2009). On the Larsen effect to monitor small fast changes in materials. *The Journal of the Acoustical Society of America* 125, pp. 1894-1905.

Luco, J.E., Trifunac, M.D., and Wong, H.L. (1987). On the apparent change in the dynamic behavior of a nine-story reinforced concrete building. *Bulletin of the Seismological Society of America*, 77(6), pp. 1961-1983.

Luco, N. (2002). Probabilistic seismic demand analysis, SMRF connection fractures, and near-source effects. PhD thesis. Stanford University.

Luco, N., and Cornell, C.A. (2007). Structure-specific scalar intensity measures for near-source and ordinary earthquake ground motions. *Earthquake Spectra*: May 2007, 23(2), pp. 357-392.

McCall, K.R, and Guyer, R.A. (1994). Equation of state and wave propagation in hysteretic nonlinear elastic material. *Journal of Geophysical Research*, 99, pp. 23887-23897.

Mehta, K., Snieder, R., and Graizer, V. (2007). Downhole receiver function: a case study. *Bulletin of the Seismological Society of America*, 97(5), pp. 1396-1403.

Michel, C., and Guéguen, P. (2010). Time-frequency analysis of small frequency variations in civil engineering structures under weak and strong motions using a reassignment method. *Structural health monitoring*, 9(2), pp. 159-171.

Michel, C., Guéguen, P., El Arem, S., Mazars, J., and Kotronis, P. (2010). Full-scale dynamic response of an RC building under weak seismic motions using earthquake recordings, ambient vibrations and modelling. *Earthquake Engineering & Structural Dynamics*, 39(4), pp. 419-441.

Michel, C., and Guéguen, P. (2018). Interpretation of the velocity measured in buildings by seismic interferometry based on Timoshenko beam theory under weak and moderate motion *Soil Dynamics and Earthquake Engineering*, 104, pp. 131-142.

Mikael, A., Gueguén, P., Bard, P.-Y., Roux, P., and Langlais, M. (2013). The Analysis of Long-Term Frequency and Damping Wandering in Buildings Using the Random Decrement Technique. *Bulletin of Seismological Society of America*, 103(1), pp. 236-246. DOI: 10.1785/0120120048

Moehle, J., and Deierlein, G. (2004). A framework methodology for Performance-Based Earthquake Engineering. *13<sup>th</sup> World Conference on Earthquake Engineering*, Vancouver. Paper N° 679.

Motosaka, M., Sato, T., Yamamoto, Y. (2004). The amplitude dependent dynamic characteristics of an existing building before and after seismic retrofit. *13th World Conference on Earthquake Engineering*, Vancouver. Paper N° 1023.

Motosaka, M., and Mitsuji, K. (2012). Building damage during the 2011 off the Pacific coast of Tohoku Earthquake. *Soils and Foundations*, 52(5), pp. 929-944. https://doi.org/10.1016/j.sandf.2012.11.012

Nakata, N., and Snieder, R. (2012). Estimating near-surface shear wave velocities in Japan by applying seismic interferometry to KiK-net data. *Journal of Geophysical Research: Solid Earth*, 117(B1).

Nayeri, R.D., Masri, S.F., Ghanem, R.G. and Nigbor, R.L. (2008). A novel approach for the structural identification and monitoring of a full-scale 17-story building based on ambient vibration measurements. *Smart Materials and Structures*, 17(2), paper N° 025006.

Neild, S.A., McFadden, P.D. and Williams, M.S. (2003). A review of time-frequency methods for structural vibration analysis. *Engineering Structures*, 25, pp. 713–728.

Newmark, N.M., and Hall, W.J., (1982). Earthquake spectra and design. *Monographs Series (EERI) Earthquake Engineering Research Institute*.

Okawa, I., Kashima, T., Koyama, S., and Iibaa, M. (2013). Recorded responses of building structures during the 2011 Tohoku-Oki earthquake with some implications for design practice. *Earthquake Spectra* 29(S1), pp. S245-S264.

Ostrovsky, L.A., Johnson, P.A., and Shankland, T. (2000). The mechanism of strong nonlinear elasticity in Earth solids. *CP524, Nonlinear Acoustics at the Turn of the Millennium: ISNA 15 American Institute of Physics* 1-56396-945-9/007

Ostrovsky, L.A., and Johnson, P.A. (2001). Dynamic nonlinear elasticity in geomaterials. *Rivista del nuovo cimento*, 24(7), pp. 1-46.

Ostrovsky, L., Lebedev, A.V., Rivière, J., Shokouhi, P., et al. (2019). Long-time relaxation induced by dynamic forcing in geomaterials. *Journal of Geophysical Research. B: Solid Earth* 124(5), pp. 5003-5013.

PacificEarthquakeEngineeringCenter(PEER).PBEEMethodology.https://peer.berkeley.edu/research/pbee-methodology

Papazafeiropoulos, G. (2015). Response Spectra. Dynamic response spectrum analysis, elastic and elastoplastic response spectra. *MathWorks* file exchange.

Payan, C., Ulrich, T.J., Le Bas, P.Y., Saleh, T.A., and Guimaraes, M. (2014). Quantitative linear and nonlinear Resonance Inspection Techniques and Analysis for material characterization: Application to concrete thermal damage. *Journal of the Acoustical Society of America*, 136(2), pp. 537-546. DOI: 10.1121/1.4887451. hal-01044669

Peng, Z., and Ben-Zion, Y. (2006). Temporal changes of shallow seismic velocity around the Karadere-Düzce branch of the North Anatolian fault and strong ground motion. *Pure Applied Geophysics*. 163, pp. 567-600. DOI 10.1007/s00024-005-0034-6.

Péquegnat, C., Guéguen, P., Hatzfeld, D., and Langalis, M. (2008). The French accelerometric network (RAP) and national data center (RAP-NDC). *Seismological Research Letters* 79(1), pp. 79-89. DOI: 10.1785/gssrl.79.1.79

Perrault, M., and Guéguen, P. (2015). Correlation between ground motion and building response using California earthquake records. *Earthquake Spectra*, 31(4), pp. 2027-2046.

Peters, J.F., Muthuswamy, M., Wibowo, J. and Tordesillas, A. (2005). Characterization of force chains in granular material. *Physical Review Letters E*, 72, pp. 041307.

Porter, K., (2003). An overview of PEER's performance-based earthquake engineering methodology. *Ninth International Conference on Applications of Statistics and Probability in Civil Engineering (ICASP9)* July 6-9, 2003, San Francisco, USA.

Porter, K., Kennedy, R., and Bachman, R. (2007). Creating fragility functions for Performance-Based Earthquake Engineering. *Earthquake Spectra*, 23(2), pp. 471-489. Earthquake Engineering Research Institute. DOI: 10.1193/1.2720892

Priestley, M.J.N. (2000). Performance based seismic design. *12th World Conference on Earthquake Engineering*, New Zealand, Paper N° 2831.

Qi, C., Wang, M., Qian, Q. (2009). Strain-rate effects on the strength and fragmentation size of rocks. *International Journal of Impact Engineering*. 36(12), pp. 1355-1364. https://doi.org/10.1016/j.ijimpeng.2009.04.008

Renaud, G., Rivière, J., Larmat, C., Rutledge, J.T., Lee, R.C., Guyer, R.A., ... and Johnson, P.A. (2014). In situ characterization of shallow elastic nonlinear parameters with dynamic acoustoelastic testing. *Journal of Geophysical Research: Solid Earth*, 119(9), pp. 6907-6923.

Richter, T., Sens-Schönfelder, C., Kind, R., Asch, G. (2014). Comprehensive observation and modeling of earthquake and temperature-related seismic velocity changes in northern Chile with passive image interferometry. *Journal of Geophysical Research: Solid Earth*, 119(6). https://doi.org/10.1002/2013JB010695

Rivière, J., Shokouhi, P., Guyer, R.A., and Johnson, P.A. (2015). A set of measures for the systematic classification of the nonlinear elastic behavior of disparate rocks. *Journal of Geophysical Research: Solid Earth*, 120(3), pp. 1587-1604.

Rivière, J., Shokouhi, P., Guyer, R.A., and Johnson, P.A. (2016). Fast and slow nonlinear elastic response of disparate rocks and the influence of moisture. *Journal of the Acoustical Society of America*. 140, 3326.

Rubinstein, J.L., and Beroza, G.C. (2004). Evidence for widespread nonlinear strong ground motion in the Mw 6.9 Loma Prieta earthquake. *Bulletin of the Seismological Society of America*, 94(5), pp. 1595-1608.

Rubinstein, J.L., Uchida, N., and Beroza, G.C. (2007). Seismic velocity reductions caused by the 2003 Tokachi-Oki earthquake. *Journal of Geophysical Research: Solid Earth.* 112, B5. https://doi.org/10.1029/2006JB004440

Sawazaki, K., Sato, H., Nakahara, H., and Nishimura, T. (2006). Temporal change in site response caused by earthquake strong motion as revealed from coda spectral ratio measurement. *Geophysical research letters*, 33(21).

Sawazaki, K., Sato, H., Nakahara, H., and Nishimura, T. (2009). Time-lapse changes of seismic velocity in the shallow ground caused by strong ground motion shock of the 2000 Western-Tottori earthquake, Japan, as revealed from coda deconvolution analysis. *Bulletin of the Seismological Society of America*, 99(1), pp. 352-366.

Sens-Schönfelder, C., Snieder, R., and Li, X. (2018). A model for nonlinear elasticity in rocks based on friction of internal interfaces and contact aging. *Geophysical Journal International*, 216(1), pp. 319-331.

Shokouhi, P., Rivière, J., Guyer, R. A., and Johnson, P.A. (2017a). Slow dynamics of consolidated granular systems: Multi-scale relaxation. *Applied Physics Letters*, 111, 251604. https://doi.org/10.1063/1.5010043

Shokouhi, P., Rivière, J., Lake, C., Le Bas, P.Y. and Ulrich, T.J. (2017b). Dynamic acousto-elastic testing of concrete with a coda-wave probe: comparison with standard linear and nonlinear ultrasonic techniques. *Ultrasonics*, 81, pp. 59-65.

Shome N., and Cornell C.A. (1999). Probabilistic seismic demand analysis of nonlinear structures. *Reliability of Marine Structures Program* Report No. RMS-35, Department of Civil and Environmental Engineering, Stanford University, California.

Snieder, R., and Şafak, E. (2006). Extracting the building response using seismic interferometry: Theory and application to the Millikan Library in Pasadena, California. *Bulletin of the Seismological Society of America*, 96(2), pp. 586-598.

Snieder, R., Sens-Schönfelder, C., and Wu R. (2017). The time dependence of rock healing as a universal relaxation process, a tutorial. *Geophysical Journal International*, 208, pp. 1-9 doi: 10.1093/gji/ggw377

Takagi, R., and Okada, T. (2012). Temporal change in shear velocity and polarization anisotropy related to the 2011 M9.0 Tohoku-Oki earthquake examined using KiK-net vertical array data. *Geophysical Research Letters*, 39(9).

TenCate, J.A., and Shankland, T.J. (1996). Slow dynamics in the nonlinear elastic response of Berea sandstone. *Geophysical Research Letters*, 23(21), pp. 3019-3022.

TenCate, J.A., Smith, D.E., and Guyer, R. (2000). Universal slow dynamics in granular solids. *Physical Review Letters*, 85, pp. 1020-1023.

TenCate, J.A. (2011). Slow dynamics of earth materials: An experimental overview. *Pure and Applied Geophysics*, 168(12), pp. 2211-2219.

TenCate, J., and Johnson, P.A. (2019). Nonlinear Resonant Ultrasound Spectroscopy: Assessing Global Damage. *Nonlinear Ultrasonic and vibro-acoustical techniques for nondestructive evaluation. Springer*, CHAP. 2.

Todorovska, M., Hao, T.-Y., and Trifunac, M. (2006). Variations of apparent building frequencies – lessons from full-scale earthquake observations. *First European Conference on Earthquake Engineering and Seismology*, Geneva, paper N°. 1547.

Todorovska M.I., and Al Rjoub, Y. (2006). Effects of rainfall on soil-structure system frequency: example based on poroelasticity and a comparison with full-scale measurements. *Soil Dynamics and Earthquake Engineering*, 26 (6-7), pp. 708-717.

Todorovska, M., and Trifunac, M. (2007). Earthquake damage detection in the Imperial County Services Building I: The data and time-frequency analysis. *Soil Dynamics and Earthquake Engineering*, 27(6), pp. 564-576.

Todorovska, M. (2009). Seismic interferometry of a soil-structure interaction model with coupled horizontal and rocking response. *Bulletin of the Seismological Society of America*, 99(2A), pp. 611-625.

Tremblay, N., Larose, E., and Rossetto, V. (2010). Probing slow dynamics of consolidated granular multicomposite materials by diffuse acoustic wave spectroscopy. *The Journal of the Acoustical Society of America* 127, 1239. DOI: 10.1121/1.3294553.

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

Trifunac, M.D., Ivanovic, S.S., and Todorovska, M.I. (2001). Apparent periods of a building. II: time-frequency analysis. *Journal of Structural Engineering*, 527.

Trifunac, M.D., Todorovska, M. I., Manic, M.I., and Bulajic B. (2010). Variability of the fixed-base and soil–structure system frequencies of a building - The case of Borik-2 building. *Structural Control Health Monitoring*, 2010; 17:120-151. DOI: 10.1002/stc.277

Vakhnenko, O. O., Vakhnenko, V. O., Shankland, T., and TenCate, J. (2004). Strain- induced kinetics of intergrain defects as the mechanism of slow dynamics in the nonlinear resonant response of humid sandstone bars. *Physical Review E*, 70, pp. 1560215604.

Van Den Abeele, K.E.-A., Johnson, P. A., and Sutin, A. (2000a). Nonlinear Elastic Wave Spectroscopy (NEWS) techniques to discern material damage. Part I: Nonlinear Wave Modulation Spectroscopy (NWMS). *Research on NonDestructive Evaluation*, 12, pp. 17-30.

Van Den Abeele, K.E.-A., Carmeliet, J., TenCate, J.A., and Johnson, P.A. (2000b). Nonlinear Elastic Wave Spectroscopy (NEWS) techniques to discern material damage. Part II: Single mode nonlinear resonance acoustic spectroscopy. *Research on NonDestructive Evaluation*, 12, pp. 31-42.

Van Den Abeele, K.E.-A., Sutin, A., Carmeliet, J., and Johnson, P. (2001). Micro-damage diagnostics using nonlinear elastic wave spectroscopy (NEWS). *NDT & E International*, 34(4), pp. 239-248.

Vidal, F., Navarro, M., Aranda, C., and Enomoto, T. (2014). Changes in dynamic characteristics of Lorca RC buildings from pre-and post-earthquake ambient vibration data. *Bulletin of Earthquake Engineering*, 12(5), 2095.

Wald, D., Quitoriano, V., Heaton, T., and Kanamori, H. (1999). Relationships between Peak Ground Acceleration, Peak Ground Velocity, and Modified Mercalli Intensity in California. *Earthquake Spectra* 15(3):557-564. DOI: 10.1193/1.1586058

Wu, C., Peng, Z., and Assimaki, D. (2009a). Temporal changes in site response associated with the strong ground motion of the 2004 Mw 6.6 Mid-Niigata earthquake sequences in Japan. *Bulletin of the Seismological Society of America*, 99(6), pp. 3487-3495.

Wu, C., Peng, Z., and Ben-Zion, Y. (2009b). Non-linearity and temporal changes of fault zone site response associated with strong ground motion. *Geophysical Journal International* 176, pp. 265-278. DOI: 10.1111/j.1365-246X.2008.04005.x

Xia, Y., Xu, Y.L., Wei, Z.L., Zhu, H.P., and Zhou, W.Q. (2011). Variation of structural vibration characteristics versus nonuniform temperature distribution. Engineering Structures, 33 (1), pp. 146-153.

Zhou, Y. and Xie, L.L. (1984). A new definition of strong ground motion duration. *Earthquake* engineering and engineering vibration, 4(2).

<b>Table A1.</b> Variability (i.e. standard deviation, $\sigma$ ) associated to the correlation between drift ratio and
different ground motion intensity measures (indicated in the first column). Variability is given by the
mean residual of the functional form $\log \Delta = a + b \log IM + \epsilon$ . Smallest $\sigma$ per building class is indicated
in bold, and the number of data considered per category is indicated between parenthesis.

	US-data					Japan-data			
	All	RC	Steel	Masonry	Wood	All	RC	SRC	Steel
	(684)	(224)	(302)	(134)	(24)	(7438)	(1422)	(4924)	(1092)
PGA	0.518	0.523	0.522	0.486	0.299	0.518	0.487	0.369	0.579
PGV	0.437	0.407	0.406	0.469	0.323	0.346	0.453	0.324	0.330
PGD	0.426	0.409	0.394	0.442	0.364	0.367	0.445	0.331	0.184
Ia	0.440	0.426	0.421	0.447	0.322	0.359	0.452	0.227	0.379
DP	0.469	0.446	0.439	0.483	0.349	0.357	0.443	0.295	0.258
CAV	0.420	0.402	0.389	0.439	0.338	0.352	0.444	0.289	0.294
Sal	0.497	0.520	0.494	0.486	0.304	0.371	0.439	0.300	0.202
Sv1	0.448	0.430	0.414	0.478	0.312	0.304	0.451	0.254	0.237
Sd1	0.470	0.397	0.417	0.490	0.323	0.322	0.457	0.274	0.158
Sa2	0.512	0.526	0.520	0.497	0.316	0.352	0.441	0.259	0.150
Sv2	0.441	0.421	0.403	0.484	0.308	0.302	0.421	0.250	0.206
Sd2	0.472	0.390	0.422	0.497	0.317	0.306	0.437	0.264	0.143
Avg_Sa	0.503	0.522	0.506	0.491	0.303	0.352	0.427	0.261	0.169
Avg_Sv	0.442	0.423	0.404	0.483	0.307	0.293	0.426	0.240	0.215
Avg_Sd	0.471	0.391	0.421	0.496	0.319	0.301	0.435	0.255	0.139
D <sub>Sa2</sub>	0.551	0.541	0.543	0.533	0.469	0.628	0.478	0.675	0.581

**Table A2.** Variability (i.e. standard deviation,  $\sigma$ ) associated to the correlation  $f_{\min}$  and ordinary ground motion intensity measures (indicated in the first column). Variability is given by the mean residual of the functional form  $\log(f_{\min}) = a + b \log(IM) + \varepsilon$ . Smallest  $\sigma$  per building class is indicated in bold, and the number of data considered per category is indicated between parenthesis.

	US-data					Japan-da	Japan-data			
	All	RC	Steel	Masonry	Wood	All	RC	SRC	Steel	
	(684)	(224)	(302)	(134)	(24)	(7438)	(1422)	(4924)	(1092)	
PGA	0.379	0.284	0.423	0.304	0.158	0.212	0.222	0.086	0.048	
PGV	0.356	0.274	0.378	0.277	0.151	0.212	0.221	0.085	0.045	
PGD	0.327	0.253	0.318	0.269	0.163	0.211	0.221	0.085	0.045	
Ia	0.369	0.278	0.400	0.293	0.152	0.212	0.221	0.083	0.045	
DP	0.333	0.268	0.337	0.262	0.144	0.212	0.222	0.083	0.045	
CAV	0.353	0.266	0.361	0.291	0.157	0.212	0.221	0.082	0.045	
D <sub>Sa2</sub>	0.351	0.259	0.371	0.297	0.192	0.214	0.222	0.090	0.046	