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**FULL SCALE DYNAMIC BEHAVIOUR OF AN RC-BUILDING UNDER LOW TO
MODERATE SEISMIC MOTIONS:
1-YEAR RECORDINGS IN GRENOBLE CITY HALL (FRANCE)**

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Abstract

In countries with a moderate seismic hazard, the classical method developed for countries with strong earthquakes to estimate the behaviour and subsequent vulnerability of buildings during earthquakes are often inadequate and not financially realistic. An alternative method is proposed whereby the structural characteristics of the building are obtained by using experimental values of the modal parameters. This article describes the application of an advanced modal analysis technique (Frequency Domain Decomposition) to process ambient vibration recordings taken at the Grenoble City Hall building (France). The frequencies of ambient vibrations are compared with those of low-to-moderate earthquakes recorded by the permanent accelerometric network that was installed to monitor the building. The frequency variations of the building under moderate earthquakes are shown to be slight and therefore ambient vibration frequencies are relevant over the entire elastic domain of the building. The modal parameters extracted from ambient vibrations are then used to determine the lumped-mass model in order to reproduce the building motion under moderate earthquakes.

Introduction

Since the 30s, earthquake engineers have recorded and studied the ambient vibrations of buildings (Carder, 1936). They were especially interested in the resonance frequencies for design code and engineering purposes (e.g. Housner and Brady, 1963). In the 60s and 70s, new forcing methods (explosion, harmonic forcing, etc.) were proposed to reach higher amplitudes of motion. Nevertheless, Crawford and Ward (1964) and Trifunac (1972) showed that ambient vibration-based techniques were as accurate as active methods for determining vibration modes and much easier to implement for a large set of buildings. More recently, Hans et al. (2005) showed that the vibration modes extracted from ambient vibrations and active methods were quite similar in the 10^{-5} to $10^{-2}g$ range of loading. Dunand et al. (2006) used Californian accelerometric data to determine the decrease in frequency during earthquakes and found variations of less than 20% under moderate (7 cm/s^2) to strong (525 cm/s^2) earthquakes without apparent damage. Clinton et al. (2006) also studied the resonance frequencies of the Millikan Library building since its construction and found only minor variations except for the strongest damaging earthquakes. These findings confirm the interest of using ambient vibration studies for post-seismic assessment of building integrity.

Simultaneously, in the last 20 years, modal analysis techniques in civil engineering applications have been considerably improved thanks to technical (instrumentation, computers) and theoretical developments in modal analysis in the electrical and mechanical engineering fields (Peeters and De Roeck, 2001). These techniques provide a means of understanding the behaviour of complex structures and fixing their elastic properties by means of their modal parameters (frequency, damping and modal shape). These are the main parameters controlling building response and vulnerability since two structures with the same mass distribution and the same fundamental period may experience shear forces of

appreciably different magnitudes if the internal structures (mode shapes) are different (Housner and Brady, 1963).

The major difficulty in the seismic vulnerability assessment of existing buildings is the lack of available data such as quality of material, structural plans, ageing and damage. In such cases, the classical tools in earthquake engineering may turn out to be very expensive or lead to over-simplistic hypotheses to overcome these difficulties. In countries with a moderate hazard, like France, for the excessive cost does not justify making such assessments on large sets of buildings even though the hazard described in the seismic regulations would be a motive for the vulnerability assessment. The experimental behaviour of buildings using modal analysis of ambient vibrations (called Operational Modal Analysis, OMA) can be formalized in a complete and detailed model (Brincker et al., 2003). Such a model could become an essential step in the vulnerability assessment. OMA is now widely used in civil engineering applications (e.g., Cunha and Caetano, 2005) to understand the linear behaviour of structures in terms of vibration modes. Models based on these experimental values, from the simplest analytical model to the most comprehensive finite-element model, may be of use in evaluating the deformation that occurs in buildings during moderate earthquakes. These simulations could be the linear starting point of a more extensive vulnerability assessment.

This paper studies the response of Grenoble City Hall (France), a 13-storey reinforced concrete building, using ambient vibration tests and the network of permanent accelerometric monitoring stations installed by the French Permanent Accelerometric Network. After briefly describing the structural design of the building and the experimental networks used (ambient vibration survey and accelerometric network), the results of the modal analysis of the building using low-to-moderate earthquakes recorded in the structure are compared with those of the

ambient vibration survey. The accelerometric data observed at the top of the building are then compared to those predicted using a lumped-mass model adjusted using the modal analysis results obtained from ambient vibration recordings.

The Grenoble RC City Hall building

The city of Grenoble is located in the northern French Alps (Fig. 1), one of the most seismic-prone areas in France ($a_N=1.5 \text{ m/s}^2$ for the national seismic code PS92). Several strong historical events have occurred in the surrounding area and the regional seismic network (Sismalp, <http://sismalp.obs.ujf-grenoble.fr>) indicates an active fault along the Belledonne range, 15km from the city (Thouvenot et al., 2003). Furthermore, the city is founded on a very deep sedimentary basin which and this gives rise to strong site effects (Lebrun et al., 2001; ESG, 2006; Guéguen et al., 2007). The moderate seismicity and sedimentary contrast, coupled with the high vulnerability due to the number of inhabitants, hi-tech and nuclear facilities, make Grenoble a national case study for seismic risk analysis.

The Grenoble City Hall is an isolated RC structure completed in 1967 (Fig. 1). It is divided into two parts: a 3-level horizontal building and an independent 13-story tower that is the subject of this study. The tower has a 44 m by 13 m plan section and rises 52 m above the ground. The inter-storey height is regular between the 3rd and 12th floors (3.2 m) and higher for the 1st (4.68 m) and 2nd storey (8 m), above which there is a precast slab of 23 m span supported by two inner cores. These cores, consisting of RC shear walls, enclose the stair wells and lift shafts and are located at two opposite sides of the building. The structural strength system combines these shear walls with RC frames with longitudinal beams bearing the full RC floors. The glass frontage is fastened to a light steel framework placed on the external perimeter of the structure. The foundation system consists of deep piles, anchored in an underlying stiff layer of sand and gravel.

Since November 2004, the building has been monitored by six accelerometric stations, three on the ground floor called OGH1, OGH2 and OGH3 and three on the 13th floor called OGH4, OGH5 and OGH6 (Fig. 2). This instrumentation is part of the French Permanent Accelerometric Network (RAP) (<http://www-rap.obs.ujf-grenoble.fr>), which is in charge of recording, collecting and disseminating accelerometric data in France. The scientific board of RAP subsequently decided to develop the building monitoring system in order to contribute to assessing its seismic behaviour and establishing the French vulnerability assessment through an experimental approach. The City Hall network is managed by the Geophysical Laboratory of Grenoble University (LGIT). Each station consists of one 3C Episensor (Kinematics) accelerometer connected to a MiniTitan 24-bit digital acquisition system (Agecodagis). The horizontal components are oriented along the longitudinal and transverse directions of the building, with the longitudinal direction having an azimuth of 327°N. The sampling rate is 125 Hz and the recordings are divided into files of 2 minutes in length. Time is controlled by a GPS receiver located on top of the building. The stations are connected via an Ethernet hub allowing data transfer from each station to the computer located in the basement of the building. This computer is permanently online for remote data control and station management. The dial-up data retrieval system at the LGIT extracts the data from the continuous recordings in accordance with a list of epicentres provided by the national seismological survey (RéNaSS). All the data are integrated in the online database of the RAP and may be retrieved in ASCII, SAC and SEED format (<http://www-rap.obs.ujf-grenoble.fr>).

Within the context of this study, a temporary network was also installed to determine the full-scale behaviour of the structure under ambient vibration. A Cityshark II station (Chatelain et al., 2000) was used for the simultaneous recording of 18 channels. Six Lennartz 3D 5s velocimeters were used for this purpose, having a flat response between 0.2 and 50 Hz. Eight

datasets were recorded, corresponding to 36 different points in the building, i.e., at least two points per floor. One sensor was installed on top of the building to serve as reference instrument for all the datasets. This reference point is necessary in order to normalize and combine all the components of the modal shape. The first frequency was estimated to be close to 1 Hz, so a 15 min recording time was selected for each set, corresponding to more than 1000 periods, at a 200 Hz sampling rate.

Earthquake recordings

Since 2004, more than 25 earthquake events have been recorded in the building. Because of the moderate seismic level in France, nine earthquakes were selected with a signal to noise ratio greater than 3 in the 0.6-5 Hz frequency band corresponding to a PGA from 0.6 to 23 mm/s² (Tab. 1, Fig. 3 and 4). The French Seismic Network (RéNaSS) located these earthquakes in the most active zones of the western part of the Alps, corresponding to the Northern (events #1, 4, 5, 7) and Southern (events #6, 9) French Alps, the Italian Alps (event #2) and the Rhine Graben (events #3, 8). Event #4 is located on the Belledonne Border Fault (Thouvenot et al., 2003). Table 1 summarises the Horizontal Peak Ground Accelerations (PGA), Velocities (PGV) and the Arias Intensity (I_{ag}), i.e. the energy of the accelerogram $a(t)$, defined as follows (Kramer, 1996):

$$I_{ag} = \frac{\pi}{2g} \int_0^{\infty} [a(t)]^2 dt$$

This table also shows the usual parameters for describing building motion: the Peak Top Accelerations (PTA), Velocities (PTV), the Arias Intensity at the top (I_{at}) and the maximum drift D_m between the top and the ground floor. This last parameter, calculated from the difference between top and base displacement divided by the building height, is related to the stresses imposed on the building. The values of parameters on the ground and at the top

displayed in Tab. 1 correspond to the maximum values given by the three stations on the ground floor and on top of the building, respectively (unfiltered data). The earthquakes are sorted with respect to D_m in decreasing order. Their values range from 10^{-6} to 3×10^{-5} . According to the French design code (PS92, 1995), the regulation drift in this case is $H/250 \approx 0.2$ which means that the building can be considered to be undamaged. The peak acceleration and velocity at the top are 1.8 to 10.2 times and 4.1 to 14.2 times the peak acceleration and velocity at the base, respectively. The response spectra of these events computed for the OGH1 station (Fig. 5) are lower than the design spectrum in the Grenoble area (PS92, 1995).

One remarkable event is the Vallorcine (Haute-Savoie, France) $M_L=4.6$, September 8th 2005 earthquake (#1), which is the strongest event recorded in the Grenoble City Hall since data monitoring started. Only minor damage and rock falls occurred in the epicentre zone, but it was strongly felt in the Alps and especially in the Grenoble basin, due to the strong site effects (Lebrun et al., 2001, Guéguen et al., 2007). Although no damage was observed in Grenoble, more than 120 km from the epicentre, people working above the third level (i.e. above the precast slab) spontaneously evacuated the City Hall. However, the drift observed for this event (Tab. 1) is 3×10^{-5} , i.e. four orders of magnitude lower than the regulation drift.

Ambient vibration recordings processing using Frequency Domain Decomposition

The ambient vibration recordings at the top and on the ground floor of the building (Fig. 6) show large motion amplifications in the building. The power spectral density (PSD) computed at the base and at the top shows that this amplification occurs only at the resonance frequencies of the building. There are two clear peaks in each direction, the second having the same frequency (1.45 Hz) regardless of the direction. All these peaks have very close

frequencies. The peak observed at the base suggests a coupling effect due to soil-structure interaction.

In order to extract the modal parameters of the structure from the ambient vibration recordings, the Frequency Domain Decomposition (FDD) method (Brincker et al., 2001) was used. This method is able to decompose modes, even if they are very close as is probably the case for the buildings in question. The first step of this method is to calculate the Power Spectral Density (PSD) matrices for each dataset. The Welch method was used for this purpose, i.e. the modified smoothed periodogram for which Fourier Transforms of the correlation matrices on overlapping Hamming windows are averaged over the recordings. Given that 18 channels are recorded simultaneously, the size of these matrices is 18x18 for each frequency. Only a limited number of modes (frequencies λ_k , mode shape vectors $\{\phi_k\}$) have energy at one particular angular frequency ω noted $\text{Sub}(\omega)$. On the one hand, it can be shown (Brincker et al., 2001) that the PSD matrices of the sensors $[Y](\omega)$ using the pole/residue decomposition take the following form:

$$[Y](\omega) = \sum_{k \in \text{Sub}(\omega)} \frac{d_k \{\Phi_k\} \{\Phi_k\}^T}{j\omega - \lambda_k} + \frac{\overline{d_k} \{\overline{\Phi_k}\} \{\overline{\Phi_k}\}^T}{j\omega - \overline{\lambda_k}} \quad (1)$$

with d_k a constant and $j^2 = -1$.

Moreover, a singular value decomposition of the estimated PSD matrices at each frequency can be performed:

$$[\hat{Y}](\omega_i) = [U_i][S_i][\overline{U}_i]^T \quad (2)$$

Identification of Eq. 1 and 2 show that the modulus of the first singular value gives a peak for an ω value corresponding to a resonance frequency ω_k linked to the continuous-time eigenvalues $\lambda_k = -\xi_k \omega_k \pm j\omega_k \sqrt{1 - \xi_k^2}$ (Fig. 7a). Furthermore, if $\text{Sub}(\omega)$ has only one or two geometrically orthogonal elements, the first or the first two singular vectors are proportional to the modal shapes. In practice, French buildings are often equally stiff in both longitudinal

and transverse directions so that the first modes in each direction are very close each other. The FDD method is capable of decomposing these modes contrary to the traditional “Peak Picking” method. Moreover, this method can be enhanced (Brincker et al., 2001b) to select the complete mode “bell”, and consequently its damping ratio, by comparing the mode shape at the peak to the mode shapes of the surrounding frequency values. The Modal Assurance Criterion (MAC) (Allemang and Brown, 1982) is used for this purpose. This compares two modal shapes Φ_1 and Φ_2 through the following expression:

$$MAC(\Phi_1, \Phi_2) = \frac{|\Phi_1^H \Phi_2|^2}{|\Phi_1^H \Phi_1| |\Phi_2^H \Phi_2|}, \text{ where } ^H \text{ denotes the complex conjugate and transpose.}$$

For a MAC value greater than 80%, it is considered that the point still belongs to the mode “bell”, even on the second singular value. The bell then represents the Fourier Transform of the mode auto-correlation so that an inverse Fourier Transform leads to the Impulse Response Function (IRF) of the mode. The logarithmic decrement of the IRF gives the damping ratio and a linear regression of the zero-crossing times gives the enhanced frequency. A decision as to whether or not a peak is a structural mode can be taken by considering the extent of the mode “bell”, the damping ratio and the shape. The proposed evaluation of the uncertainties on the peak position in the spectrum does not include epistemic errors.

Only 3 modes have been accurately determined (Fig. 7): the first longitudinal mode at 1.157 ± 0.006 Hz, with a damping of about 0.9%, the first transverse mode at 1.217 ± 0.006 Hz with a damping of about 1.1% and the first torsion mode at 1.45 ± 0.01 Hz with a damping of about 0.9%. The first longitudinal mode is not pure but has a slight torsion component that is not present in the first transverse mode. Following the aforementioned decision process using MAC, the second longitudinal mode may be distinguished at 4.5 ± 0.2 Hz and a mode that

looks like the second torsion mode may be found at 5.7 ± 0.2 Hz. In addition, the first vertical mode can be determined at 9.3 ± 0.2 Hz.

The values of the first bending frequencies in each direction are very close to each other, which means that the structural system has roughly the same stiffness in both directions. The longitudinal direction is even “softer” than the transverse direction. Moreover, from Fig. 7b, it can be seen that the storeys under the precast slab are very stiff in the modal shapes. The frequency ratio f_2/f_1 is equal to 3.9 in the longitudinal direction and 4.0 for the torsion modes. Although the modal shapes look like cantilever beams, these ratios do not correspond to pure cantilever beam behaviour (theoretical ratio 6.3) nor to shear beam behaviour (theoretical ratio 3) (Clough and Penzien, 1993) but may correspond to a composite model integrating both behaviours (e.g. Timoshenko beam as described in Hans et al., 2005).

Frequencies under earthquake recordings

To demonstrate the relevancy of the modes determined under ambient vibrations, these modes were compared to the resonance frequencies using earthquake recordings. For this purpose, Auto-Regressive (AR) modelling of the structure was used (Dunand et al., 2006). Each couple of base/top sensors (OGH1-OGH4, OGH2-OGH5 and OGH3-OGH6) is modelled by an AR filter obtained using the Linear Prediction method on the Matlab software. The top motion is first deconvolved by the base motion with a water-level method (Clayton and Wiggins, 1976) and the resulting spectrum is approximated by the best AR filter. A stabilisation diagram with several numbers of poles in the AR filter is used to estimate the confidence in the frequency and damping obtained for the first resonance frequency in each direction. The results are approximately the same for the three couples of sensors so that only the median value is kept for each earthquake (Fig. 8). A slight decrease (less than 2%) in the frequencies is observed with increasing drift up to 10^{-5} . This trend seems

to follow a logarithmic scale and would mean that the frequency decreases logarithmically with respect to the drift amplitude. This decrease may be due to the aperture of micro-cracks in the concrete that temporarily decreases the stiffness of the structure and therefore the frequencies, as already mentioned by Dunand et al. (2006) using Californian strong-motion data collected in buildings.

The frequency during the Vallorcine earthquake is approximately 2% lower than the frequency during the weakest earthquakes. The values obtained at low drifts are higher (2 to 3%) than the values obtained by the FDD method using ambient vibrations. This slight difference may be due to the system studied with FDD and AR methods: in the first case, the flexible-base building is considered including the soil-structure interaction while, in the second case, the system considered is the fixed-base building. This is confirmed by the large proportion of the energy recorded on the ground floor at the building frequency as shown with the PSD computed on this floor using ambient vibrations (Fig. 6).

Although the basic assumption of white noise is required for the FDD method, it was also used to determine the structure modes during Vallorcine earthquake. The FDD is robust enough to allow this process (Ventura et al., 2003a). In order to consider the same structural system (i.e. fixed- or flexible-base system) for ambient vibrations and earthquakes using an AR filter, consideration is given to the weakest ground motion having the same order of magnitude as ambient vibrations and generated by the Meribel earthquake (#7, Tab. 1). Here again, a slight decrease in the first frequencies from 1.5% to 4% is found between ambient vibration and earthquake (Tab. 2).

In conclusion, regardless of whether or not the soil-structure interaction is included in the FDD method or the AR modelling, a slight decrease (2-4%) in the first frequencies is found during the Vallorcine earthquake compared to the weakest motions (Tab. 2). This decrease

has already been mentioned with reference to other buildings (Celebi, 1993, 1996, 1998, Ventura et al., 2003, Dunand et al., 2006) but in this particular case two different and precise methods are used to quantify this decrease for weak-to-moderate ground motion. Care should be taken when extrapolating these results to higher drifts but this logarithmic decrease with increasing drift may be valid in the elastic domain. This means that the frequency values obtained under ambient vibrations are relevant in a building model for moderate earthquakes and that no dramatic decrease occurs between ambient vibrations and moderate earthquakes.

Modal Modelling

The modal parameters obtained under ambient vibrations are unscaled (Brincker et al., 2003), i.e. it is not possible to deduce the amplitude of the building motion with only modal parameters. A physical model integrating the modal parameters is therefore required. As the masses are mostly concentrated in the floors of a building, a lumped-mass model was assumed for this structure. In this case, the Duhamel integral (Clough and Penzien, 1993) gives the elastic motion $\{U(t)\}$ of each floor of the structure assuming a constant mass along the storeys $[M]$, and knowing the vibration modes ($[\Phi]$ the modal shapes, $\{\omega\}$ the frequencies and $\{\xi\}$ the damping ratios) and the ground motion $U_s(t)$:

$$\{U(t)\} = [\Phi]\{y(t)\} + U_s(t)$$

$$\text{with } \forall j \in [1, N] \quad y_j(t) = \frac{-p_j}{\omega'} \int_0^t U_s''(\tau) e^{-\xi_j \omega_j (t-\tau)} \sin(\omega' (t-\tau)) d\tau,$$

$$\omega_j'^2 = \omega_j^2 (1 - \xi_j^2) \text{ and } p_j = \frac{\{\Phi_j\}^T [M] \{1\}}{\{\Phi_j\}^T [M] \{\Phi_j\}} = \frac{\sum_{i=1}^N \Phi_{ij}}{\sum_{i=1}^N \Phi_{ij}^2} \text{ the participation factor of mode } j.$$

Only the first bending modes are considered to provide energy, with the torsion mode being neglected for the sake of simplicity. A 1D model is then assumed so that the experimental modal shapes are averaged at each floor. The motion at each floor can be computed for any

deterministic earthquake scenario. This is of course a linear model, which is suitable only for moderate motions. Nevertheless, as mentioned in Boutin et al. (2005), elastic modelling can be used to detect whether the building reaches the post-elastic state or not. The uncertainties of this model are only epistemic because the errors on the parameters used are quite low. The experimental values could be used to adjust a more complicated model, e.g. a 3D finite element model (Ventura et al., 2003b, Pan et al., 2004), but only a few parameters of such a model can be accurately determined with the result that the model is still based on a large amount of *a priori* knowledge and does not provide much more information than the 1D lumped-mass model.

In order to test the relevancy of this model, the motion obtained at the top of the structure during the recorded earthquakes was compared with the corresponding modelling results. The input motion is an average of the recorded motion at the ground floor of the structure. Even though there are usually four independent motions in a structure (Guéguen et al., 2005) (relative motion of the foundation, base rocking, torsion and structural drift), it was observed for this building that the motion is essentially structural drift fitted by the modal model used in this study. The four previously mentioned parameters describing the motion (PTA, PTV, I_{at} , D_m) are considered, together with the duration of the building motion. The duration is defined here as the time between 5% and 95% of the Arias Intensity (Kramer, 1996). The accelerations often tend to be underestimated when determined from the torsion mode that is not taken into account in the lumped-mass model considered here (Fig 9). Conversely, the durations are sometimes overestimated in the model possibly as a result of the damping ratio that may be higher and therefore may decrease the resonance duration. Most of the errors on PTV, D_m and duration are less than 20%. The Arias Intensity at the top of the building is well reproduced except for the smallest earthquakes. Neglecting the torsion mode is certainly the

biggest approximation in the model. The overall results are nevertheless satisfactory and they validate the simple modal model results obtained with ambient vibrations and used to reproduce the building motion under moderate earthquakes.

The Vallorcine recordings (Fig. 4) show significant anisotropy in the building: the amplitude in the transverse direction (East) is twice the amplitude of the longitudinal direction at the top, despite a greater PGA in the longitudinal direction. Considering the recording at the base floor as input, the modelled motion is compared with the motion recorded at the top (Fig. 10). The correlation coefficients between observed and modelled accelerations are 88% and 76% in the longitudinal and the transverse directions respectively. This anisotropy is not induced by complex behaviour of the building because the model fits the data well, but by the ground motion itself. This shows how complex ground motion can be, compared to the simplicity of a building but only if the building is modelled using good parameters based on experimental work. Thanks to the lumped-mass model, it has also been shown that the maximum inter-storey drift (Fig. 11) is greater in the transverse than in the longitudinal direction only above the precast slab. This maximum drift is approximately the same in the longitudinal direction from the 3rd to the 12th floor (2×10^{-5}). In the transverse direction, it is also constant from the 5th to the 12th floor at 4×10^{-5} , i.e. twice the longitudinal value. This maximum drift along the storeys is significantly lower (two orders of magnitude) than the minimum strain able to damage the building (4×10^{-3} for immediate occupancy according to FEMA (2000) for concrete shear walls).

Conclusions

This paper shows how the dynamic response of existing buildings in the elastic domain is obtained from ambient vibrations. Thanks to the past development of new

Operational Modal Analysis methods based on ambient vibrations, the precise modal response of buildings can be understood and obtained in order to predict building behaviour under moderate earthquakes. The study is focused on the Grenoble City-Hall building that has the advantage of being permanently monitored. The first year of permanent accelerometric recording in the Grenoble City Hall, supplemented with full-scale ambient vibration measurements have enabled a better understanding of the dynamic behaviour of the structure to be obtained. This behaviour is largely dominated by the first bending mode in each direction, including nevertheless a slight torsion mode. During recorded earthquakes, the frequencies of the structure decreased by 3% with respect to the ambient vibration values. The decrease in frequency follows a logarithmic decay with respect to the drift of the structure. This decrease is sufficiently small to consider that the modal properties obtained from ambient vibrations are relevant in a wide range of amplitudes, in the elastic behaviour domain of the structure.

Assuming a 1D lumped and constant mass model, the experimental modal parameters were used to reproduce the motion of the building for moderate earthquakes, without any hypothesis on the structural design and materials. Such building motion parameters as acceleration or velocity amplitude, duration, drift and energy are reproduced relatively well with this simple model. Therefore, the response of a structure to moderate earthquakes can be easily predicted as soon as the intrinsic behaviour of the building under ambient vibrations has been accurately determined using experimental techniques. This model can be used to calculate the inter-storey drift for any moderate ground motion and a subsequent preliminary assessment of the integrity of the building. For example, in the case of Grenoble City Hall, the inter-storey drift is maximum for the last floors of the building (above the precast slab) and especially in the transverse direction. Assuming an inter-storey drift threshold for a given building performance (immediate occupancy in this case), as proposed by the FEMA, it is

possible to predict whether or not the building will be damaged considering a deterministic earthquake scenario.

Obtaining the dynamic elastic properties of existing buildings is thus crucial for estimating building motion under earthquake conditions. Fixing the elastic domain behaviour of existing buildings may be assumed to be the first step in an exhaustive vulnerability analysis that generally explores the anelastic domain.

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Legend

Table 1. Characteristics of the earthquakes and their recordings in Grenoble City Hall used in this study (T,t indexes correspond to the top of the building; G,g indexes correspond to the ground)

Table 2. Comparison between resonance frequencies of the structure under weak motion (ambient vibrations and Meribel earthquake) and Vallorcine earthquake using the FDD method and AR modelling.

Figure 1: a) Location of Grenoble in France b) Grenoble City Hall, viewed from the southeast; c) Plan view of a current storey of the tower

Figure 2. Location of the accelerometers of the French Accelerometric Network (RAP) in the City Hall

Figure 3. Epicentre map of the earthquakes used in this study and located by the French National Seismological Survey (RéNaSS)

Figure 4. Examples of accelerometric time history of the nine earthquakes recorded in Grenoble City Hall at the OGH6 roof station and at the OGH1 ground station in the longitudinal L and transverse T directions. All waveforms are plotted in relative mode and scaled by the maximum amplitude of each station/component pair.

Figure 5. Response spectra (5% damping) of the earthquakes in longitudinal/North (left) and transverse/East (right) directions at station OGH1 compared to the French Seismic Code (PS92, 1995; $a_N=1.5 \text{ m/s}^2$).

Figure 6. Time histories (top) and Power Spectral Densities (bottom) of ambient vibrations at the top and on the ground floor of the building expressed in dB ($PSD_{dB} = 10 \log \left(\frac{PSD}{PSD_0} \right)$) for longitudinal (left) and transverse (right) components. PSD_0 corresponds here to a velocity of 10^{-5} m/s .

Figure 7. a) Spectrum (mean value of the 8 datasets of the first 6 singular values of the PSD matrices) of the structure under ambient vibrations computed using Frequency Domain Decomposition (FDD) (Brincker et al.,

2001a). b) first 3 structural modes of the structure obtained using FDD (from left to right: longitudinal bending, transverse bending and torsion)

Figure 8. Resonance frequencies of the building in longitudinal and transverse directions for the nine earthquakes using AR modelling and plotted as a function of the structure drift Dm . The solid line represents the frequency value obtained by Frequency Domain Decomposition (FDD) under ambient vibrations (+/- uncertainties shown by dashed lines)

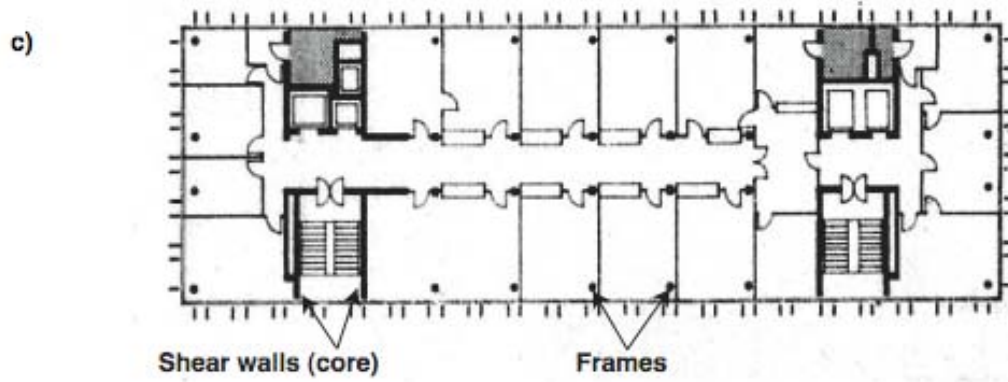
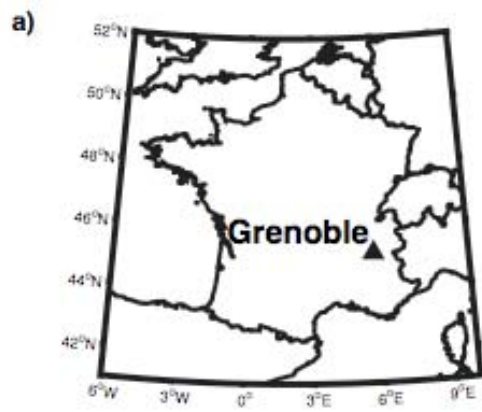
Figure 9. Comparison of the parameters (PTA, PTV, maximum drift, Arias intensity and duration) computed at the building top using the lumped-mass model and the accelerometric recordings of the City Hall

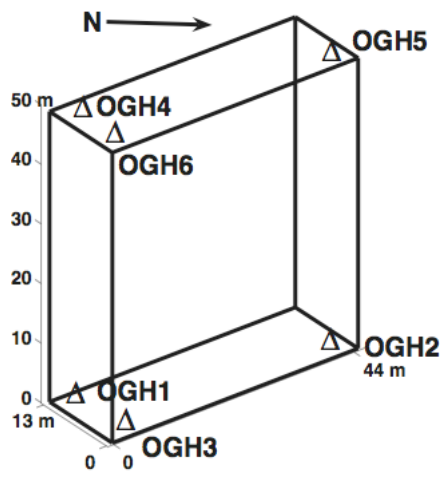
Figure 10. Comparison between recordings (dashed line) and modelling (solid line) of the Vallorcine earthquake at the roof of the structure in the longitudinal/North (top) and transverse/East (bottom) directions

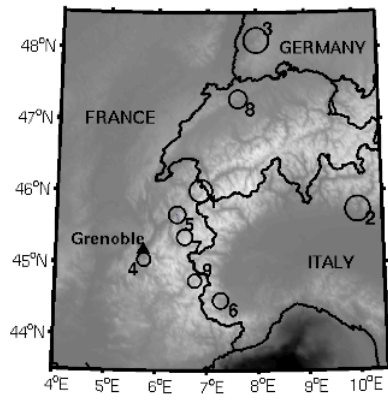
Figure 11. Modelling of maximum drift along the storeys of the structure during the Vallorcine earthquake using the base motion as input and the lumped-mass model extracted from ambient vibrations.

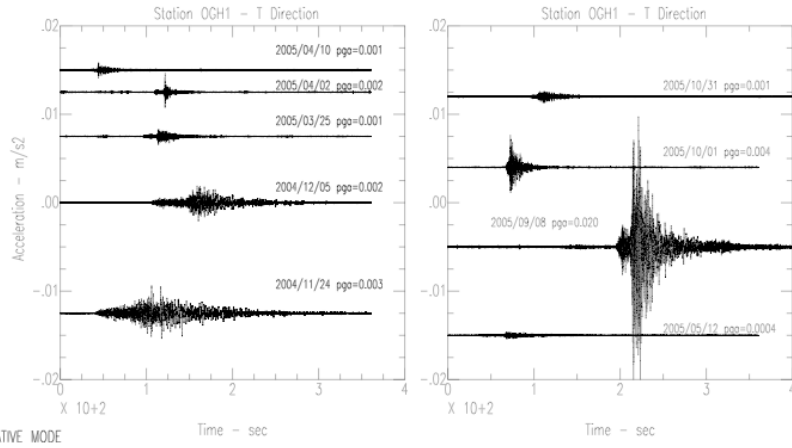
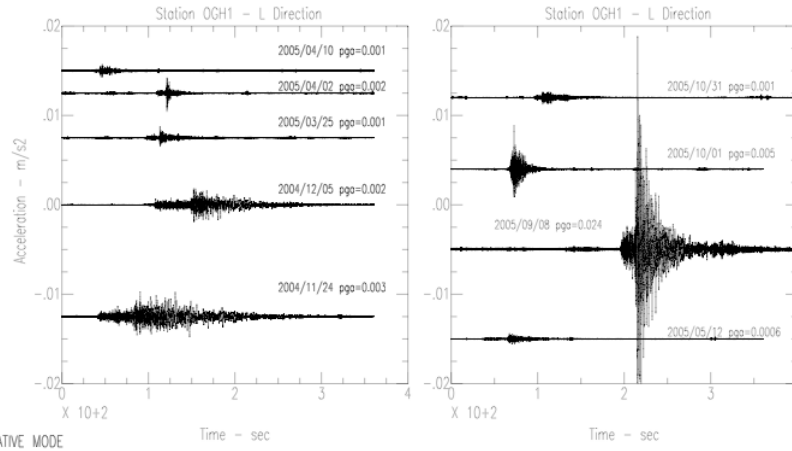
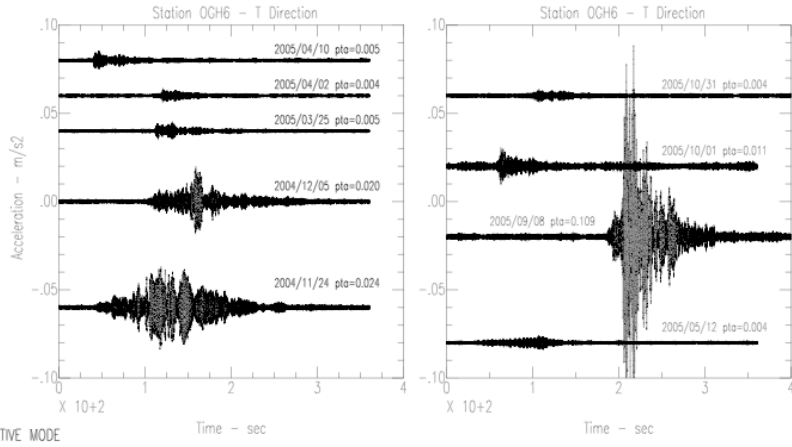
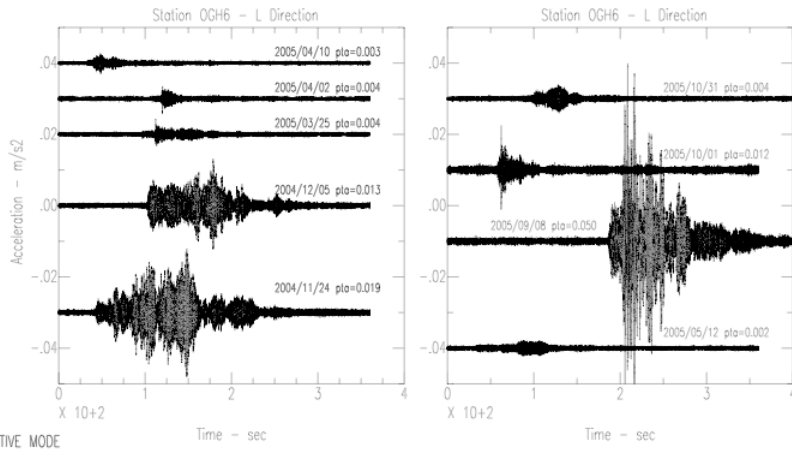
#	Event characteristics (RéNaSS)					Epicentral Distance (km)	Maximum Acceleration (mm/s ²)		Maximum Velocity (mm/s)		Maximum Arias Intensity (μm/s)		Max. Drift (10 ⁻⁶)
	Location	Long.	Lat.	M _L	Date		PGA	PTA	PGV	PTV	I _{ag}	I _{at}	
1	Vallorcine	6.87	46.01	4.9	2005/09/08	127.3	22.94	107.42	1.376	11.80	106.71	7943.1	30.59
2	Lago di Garda (Italy)	10.01	45.74	5.5	2004/11/24	339.8	3.10	22.89	0.695	3.10	9.39	1229.3	8.30
3	Freiburg (Germany)	8.00	48.11	5.3	2004/12/05	368.3	1.88	19.17	0.294	2.21	3.46	301.1	5.74
4	Laffrey	5.75	45.05	3.1	2005/10/01	15.2	4.70	11.62	0.159	0.94	2.08	50.2	2.13
5	Albertville	6.40	45.68	3.6	2005/10/31	75.6	0.78	3.73	0.053	0.53	0.18	17.4	1.40
6	Dronero (Italy)	7.27	44.48	3.5	2005/03/25	144.5	1.31	4.70	0.084	0.52	0.33	17.5	1.38
7	Meribel	6.56	45.36	3.4	2005/04/10	67.5	0.88	5.10	0.040	0.56	0.14	18.1	1.25
8	Balstahl (Switzerland)	7.63	47.29	3.9	2005/05/12	275.7	0.56	3.73	0.033	0.47	0.13	20.6	1.21
9	Arvieux	6.76	44.75	3.1	2005/04/02	94.2	1.98	3.47	0.084	0.35	0.31	9.2	0.85

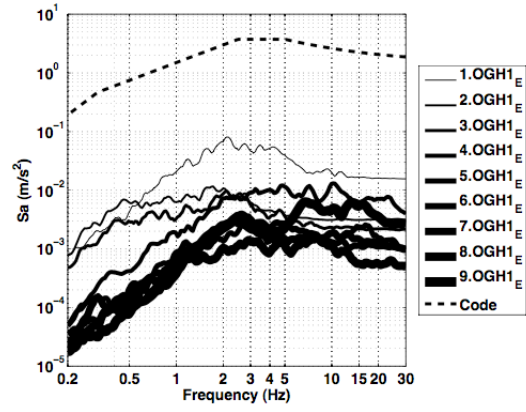
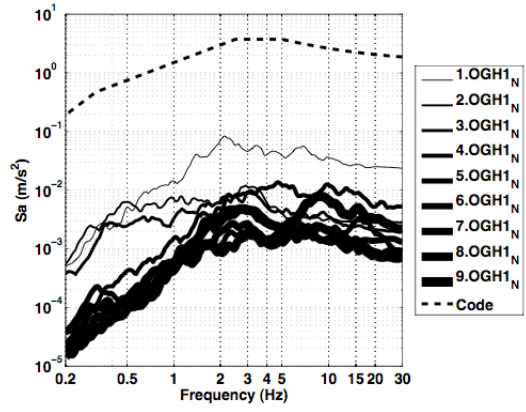
Resonance frequencies	FDD method			AR modelling		
	Ambient vibrations	Vallorcine earthquake	Decrease	Meribel earthquake	Vallorcine earthquake	Decrease
1 st longitudinal (Hz)	1.16	1.13	2.6%	1.180	1.152	2.4%
1 st transverse (Hz)	1.22	1.17	4.1%	1.242	1.220	1.8%
1 st torsion (Hz)	1.44	1.42	1.4%	1.442	1.414	2.0%

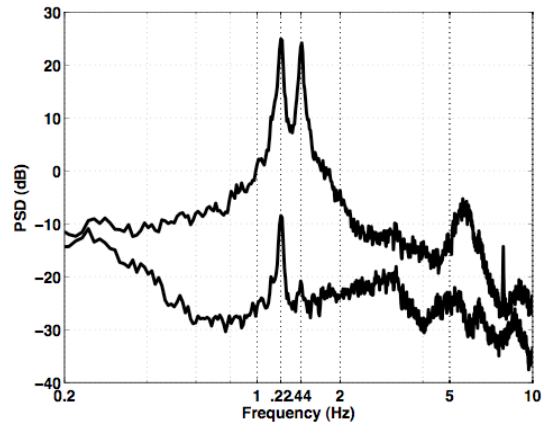
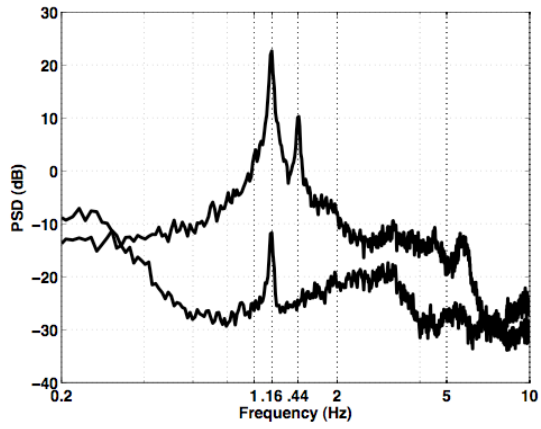
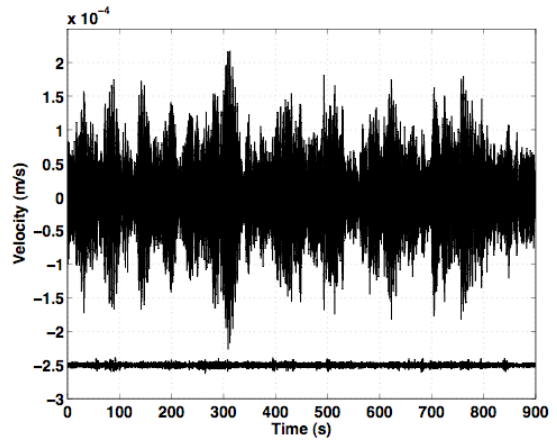
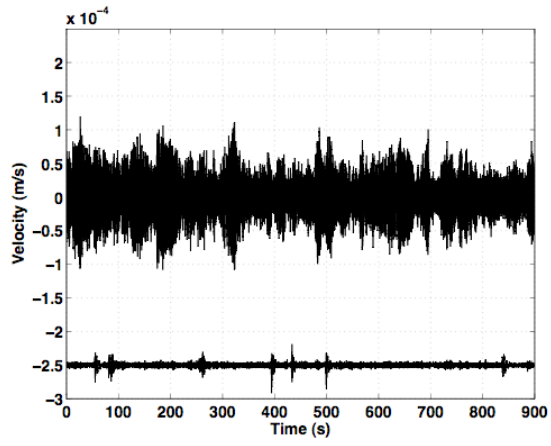


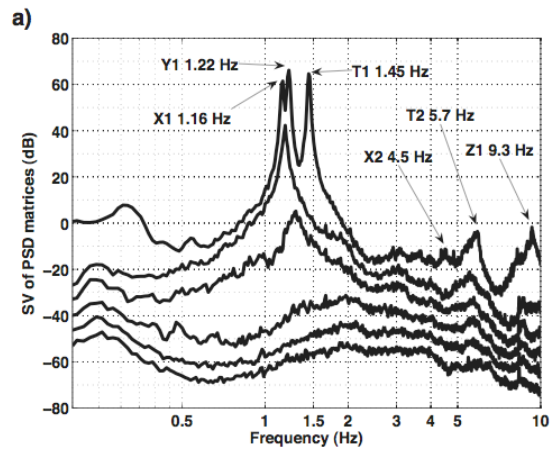












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