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Publication Date:
2010

Permanent Link:
https://doi.org/10.3929/ethz-b-000029213 →

Originally published in:
Bulletin of Earthquake Engineering 8(6), http://doi.org/10.1007/s10518-010-9185-7 →

Bull Earthquake Eng (2010) 8:1295–1307
DOI 10.1007/s10518-010-9185-7

ORIGINAL RESEARCH PAPER

Comparison between seismic vulnerability models and experimental dynamic properties of existing buildings in France

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Received: 4 March 2009 / Accepted: 2 May 2010 / Published online: 20 May 2010

Abstract Elastic fundamental frequency is a key-parameter of simplified seismic design and vulnerability assessment methods. Empirical relationships exist in codes to estimate this frequency but they miss experimental data to validate them accounting for national feature of building design and, above all, corresponding uncertainties. Even if resonance frequency extracted from ambient vibrations may be larger than the elastic frequency (at yield) generally used in earthquake engineering, ambient vibration recordings may provide a large set of data for statistical analysis of periods versus building characteristics relationships. We recorded ambient vibrations and estimated the fundamental frequency of about 60 buildings of various types (RC and masonry) in Grenoble City (France). These data complete the set existing yet, made of 26 RC-buildings of Grenoble (Farsi and Bard 2004) and 28 buildings in Nice (France) (Dunand 2005). Statistical analysis of these experimental data was performed for fundamental frequencies of RC shear wall structures and the results are compared with existing relationships. Only building height or number of stories has a statistical relevancy to estimate the resonance frequency but the variability associated to the proposed relationships is large. Moreover, we compared the elastic part of capacity curves of RC and masonry buildings used in the European Risk-UE method for vulnerability assessment with the experimental frequencies. The variability is also large and the curves may not be consistent with French existing buildings.

Keywords Resonance frequency · Ambient vibrations · Frequency relationships · Capacity curve · Variability

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1 Introduction

The seismic coefficient in most seismic codes is used to quantify the design shear forces. It depends on design ground acceleration, site conditions, class of the building, and above all elastic period of the structure. For new buildings, the period can be evaluated using simplified empirical relationships found in codes, either based on in situ or earthquake recordings in existing buildings, numerical or analytical computations or laboratory tests. There is no doubt that the most reliable estimate of building response are those provided from seismic regions having experienced strong earthquakes, where seismic building arrays have been deployed. However, such data are rather limited, in quantity and quality, especially when they are analysed distinguishing materials (steel, concrete etc.), structural systems (reinforced concrete -RC- frames, shear walls etc.) and amplitude of shaking. Then, the dataset is too small to control the relevancy of the empirical relationships and their associated uncertainties.

This is above all the case for moderate seismic prone regions that do not often experience strong earthquakes, but for which seismic code must be applied since past strong earthquakes produced damages. For these countries, the practice is usually to import these empirical relationships from high seismic hazard regions. The main problem concerns the difference between design and actual structural systems that may be large. This is particularly the case in France where specific design and techniques of construction were used in the 60's for a large number of housings and the estimate of their period in the framework of their vulnerability assessment is not easy. In this country, as well as neighbouring countries like Switzerland, the main structural systems used for dwelling constructions are unreinforced masonry (URM) and RC shear walls. For this reason, the derivation of region-specific relationships is necessary for both design of new buildings and assessment of existing structures.

The observed structural period is a global parameter influenced by the structural design, the shaking amplitude, the soil-structure interaction, non-structural elements, etc. It is therefore necessary to catch the importance of all these parameters to provide reliable relationships. This objective should be reached by using experimental data and modelling in a complementary way. Crowley and Pinho (2010) investigated how modelling could constrain the frequency relationships and proposed enhancements for Eurocode 8. In the present paper, the use of ambient vibrations in civil engineering structures that allows collecting a large amount of experimental data is investigated. It studies especially the fundamental resonance frequencies of common RC French buildings. The values obtained by in situ ambient vibration recordings, called f_0 here and corresponding to the low amplitude domain is carefully compared with the resonance frequency derived from simplified formulas appearing in the seismic design codes of all countries, called f_1 here, and generally referred as "elastic fundamental frequency" in the literature. This frequency value is crucial to estimate the amplitude of the design motion. It should represent the building motion at the operational limit state, i.e. already slightly damaged. Moreover, this frequency is usually supposed to be overestimated by the design codes in order to be conservative in conventional design (Crowley and Pinho 2010) but the requirements for seismic assessment may be different. The f_0 value cannot really be compared with the theoretical "uncracked" frequency value since real structures always include previous cracking. Lestuzzi (2002) shows on laboratory tests on a RC wall how much this uncracked value is greater than the observed one, even with few damages.

Moreover, in large-scale vulnerability assessment methods like HAZUS (FEMA 1999) or Risk-UE (Milutinovic and Trendafiloski 2003) in Europe, buildings are modelled by standard bilinear capacity curves. The slope of the elastic part of these curves is theoretically the square angular frequency of the building f_1. Corresponding to the yield period. According to the equal displacement rule or any other linearization process, this period is crucial for the displacement demand estimation. Even if f_0 and f_1 are different for a given building, it is clear that $f_0 > f_1$ and the ratio f_1/f_0 can be constrained by laboratory tests and earthquake recordings (Dunand et al. 2006; Calvi et al. 2006). The goal of this paper is to compare f_0 resonance frequencies with f_1 values given by empirical relationships and f_1 values used as elastic part of capacity curves in the Risk-UE method. The aim of this study is also to warn the users of these formulas and curves to take their limitations into account.

2 Fundamental resonance frequencies

A permanent oscillatory motion is affecting civil engineering structures caused by natural (ocean, atmosphere…) and anthropogenic (traffic, industries…) vibrations of the ground, called ambient seismic noise, wind and internal dynamic loads. The recording of ambient vibrations (peak ground acceleration PGA between 10^{-5} and 10^{-3} m/s^2) at the top floor of a building, where the amplitude is the largest, allows estimating its resonance frequencies that mostly depends on the mass and stiffness distribution in the structure.

Other parameters may influence the observed resonance frequency such as soil-structure interaction and non-structural elements. Under ambient vibrations, the effect of the first may be limited and may be part of the observed variability, an argument for that is given in Sect. 3. However, its effect on strong motion recordings may be greater as suggested for example by Todorovska (2009). The relevance of the fixed-base assumption should still be discussed for ambient vibrations. The effect of non-structural elements in case of shear wall buildings has been shown to be of minor importance compared to the stiffness of the structure itself (including heavy non-structural elements like precast facades) (Hans et al. 2005).

As shown by experimental data, the perfectly linear behaviour of RC buildings ranges over several orders of magnitudes from ambient vibrations to small earthquakes (e.g. Hans et al. 2005; Michel et al. 2008, 2010). When the amplitude increases (moderate earthquake –PGA between 0.1 and 1 m/s^2), still in the elastic domain, the opening of cracks induces non-linearities, that produce a frequency decrease reaching 35% (Dunand et al. 2006). Rigorously speaking, such a non-linearity probably implies inelastic phenomena. However, in the formalism of a simple elasto-plastic model, what is generally used for both design and assessment, such a frequency drop is still considered as elastic. For stronger earthquakes (PGA greater than 1 m/s^2), the building starts to be damaged and a quick frequency drop occurs (Calvi et al. 2006). Values of frequency drops with amplitude and damage for different types of buildings are the purpose of further work, but it seems that a value of 60% drop in frequency is a limit before the collapse according to data compiled by Calvi et al. (2006). We should note that the frequency drop is no more correlated with damage for high damage grades because the buildings do not collapse thanks to their ductility, no more their stiffness. Therefore, the fundamental frequency seems to vary in between the value obtained under ambient vibrations and 40% of this value. This ambient vibration value f_0, taken with care, is therefore relevant for a comparison with elastic design of structures and the vulnerability study for low damage grades. Moreover, the frequency variations around yield makes the "elastic frequency" of codes f_1 difficult to determine directly by in situ earthquake recordings. On the contrary, the ambient vibration value f_0 is a stable and accurate experimental value that just needs more research to be turned relevantly into the value f_1, needed for engineering purposes.

The experimental data used in this paper have three different sources (Fig. 1). Farsi and Bard (2004) made a first survey at LGIT Grenoble. They recorded 30 min vibrations of 39

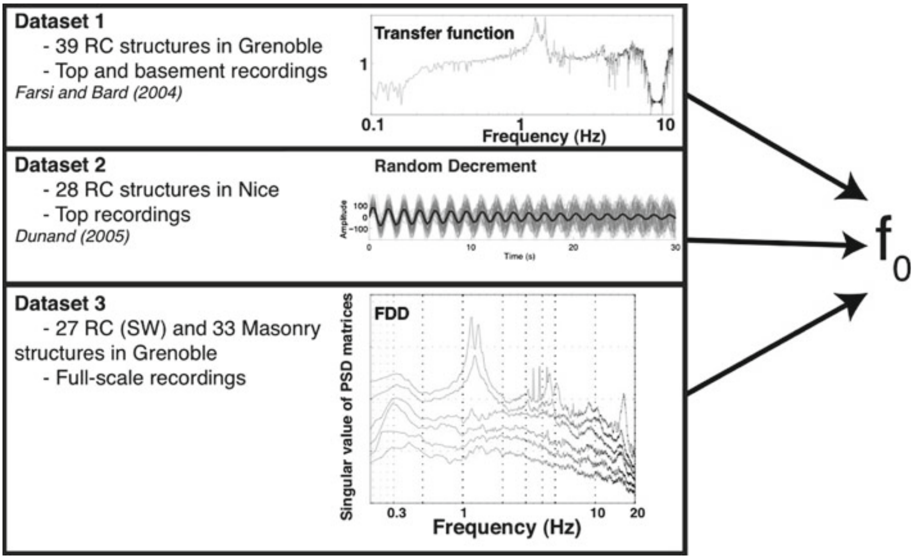

Fig. 1 Datasets and methods used in this paper to estimate resonance frequencies under ambient vibrations f_0

RC buildings in Grenoble, at the base and at the top. They used 6C RefTek digitizers with Guralp CMG5 sensors (flat response in acceleration between 0.1 and 50 Hz) and Mark Products L22 sensors (flat response in velocity between 2 and 50 Hz). The resonance frequencies of each building in its main directions have been obtained by peak picking on the transfer functions. These functions were estimated by averaging Fourier Transform of stationary windows. In 2002, in the frame of the GEMGEP project (Bard et al. 2005), ambient vibrations were recorded at the top of 28 RC buildings in Nice. They used Lennartz and Hathor 24 bits digitizers and Lennartz 3D-5s sensors (flat response in velocity between 0.2 and 50 Hz) for 15 min of recordings. The first frequencies were estimated by the random decrement method that is suited for frequency and damping estimation (Dunand 2005). Damping variations will not be discussed here, this paper being only focused on the frequency values.

Finally, the authors made 3 surveys in 2004, 2005 and 2006 in Grenoble. The use of the Cityshark II (Châtelain et al. 2000) digitizer allowed the simultaneous recording of 18 components (6 Lennartz 3D-5s sensors) for 15 min. At least one recording at each story was performed to better understand the building behavior. A total amount of 60 buildings has been studied including 33 URM buildings and 27 RC buildings. The resonance frequencies have been estimated using the Frequency Domain Decomposition method (FDD, Brincker et al. 2001) that allows a more precise and robust estimation of modes even if they are close.

3 Analytical frequency formulas for RC buildings

3.1 Existing formulas in design codes

In the design codes, the elastic resonance period of the building $T_1 = 1/f_1$ is crucial to calculate the design shear force that should be taken into account. This period T_1 should be representative of the building at the operational limit state, which includes already a moderate damage. The non-linearity that affects the periods in this so-called "elastic domain" remains lower than 35% (Hans et al. 2005; Dunand et al. 2006). However, a greater period increase occurs when the damage begins. The engineer should be aware of that in order to take conservative values (design) or best estimate values (assessment) of loading.

In order to help the designer, simplified formula give the elastic period T_1 in the codes. These formulas aim at underestimating periods T_1 of 10 to 20% in order to obtain conservative base shears for a conventional design (Goel and Chopra 1998; Crowley and Pinho 2010). We can note that in a displacement-based design, conservative values would be overestimated periods.

The simplest relationship, now only used in earthquake engineering lectures, is $T_1 = N/10$, with N the number of stories. It was allowed by former US buildings codes for moment resisting frame structures, but is not adapted to shear wall structures (Housner and Brady 1963).

Assuming a cantilever beam with some simplifications (Crowley and Pinho 2010), US codes proposed for shear wall buildings equations with the shape: $T_1 = C\, H/\sqrt{L}$, with C a constant, H the height of the building and L the length of the building parallel to the considered direction. The C values were first derived in the 1950s using the ambient vibrations recording survey of Carder (1936) after the Long Beach earthquake of 1933 and completed with Japanese data (Housner and Brady 1963). Carder studied 212 buildings, mainly masonry and steel structures built before 1940. The resulting formulas have been discussed in the 1960s (Housner and Brady 1963) using especially additional Japanese data. However, this type of formula has been also adopted widely in the world, for example by the Algerian (RPA88 1988), Korean (Lee et al. 2000) and partially in the French (PS92 1995) codes. Calibrations using recordings in buildings of the San Fernando earthquakes in 1971 thanks to the California Strong Motion Instrumentation Program (CSMIP) lead to updated C values in the ATC3-06 (ATC 1978). Crowley and Pinho (2010) review the work that was done at that time.

Housner and Brady (1963), Hong and Hwang (2000) as well as Farsi and Bard (2004) criticized the use of the lateral dimension L in the formula. For these authors, only the height of the building has a statistical influence on its period, if one wants to use empirical relationships, basing their conclusions on ambient vibration data. Carder has been a pioneer of in situ recordings in buildings, but the material and the frequency estimation technique he used seems very rough today (analogical recordings, zero-crossing times for frequency estimation…) and the building stock itself changed a lot. However, no other survey of this scale has been carried out since then for design codes and these data were used during many years. Moreover, the direct use of ambient vibration data for design code relationships is not straightforward. Considering they provide an overestimation of the frequencies, they lead however to conservative estimates in conventional design, that is not necessary the case for data based on strong motion recordings.

Originally designed for Moment Resisting Frames (MRF) structures in ATC3-06 (ATC 1978) as reviewed by Crowley and Pinho (2010), another shape of relationship spread worldwide in the design code at that time. It follows the expression:

$$T_1 = C_1 H^{\beta} \tag{3.1}$$

This relationship is also used in Eurocode 8 (CEN 2004) and in the SIA (2003) Swiss Codes. The UBC (1997), EC8 and SIA codes use $\beta = 0.75$, based on theoretical considerations on MRF, regardless of the structural type (Crowley and Pinho 2010) the EC8 limiting the use of this formula to buildings lower than 40 m. Various C_t coefficients were obtained using especially calibrations from the San Fernando earthquake depending on the building type (Crowley and Pinho 2010). In EC8, for example, $C_t = 0.05$ for RC shear wall structures.

In the Risk-UE project, Lagomarsino and Giovinazzi (2006) used $C_t = 0.065$ and $\beta = 0.9$ in order to estimate the elastic part of capacity curves for all RC buildings typologies without seismic prescriptions. However, these values are based on Chopra and Goel (2000) for MRF structures including an over-estimation of the periods for displacement-based analysis. This assumption may be justified for Italian building stock mainly based on MRF structures but not for all Europe. Crowley and Pinho (2010) remarked that a more simple but more relevant assumption leads to a linear relationship, i.e. $\beta = 1$, that is only valid between T_C and T_D as defined in the design codes (around 0.5 s, i.e. 2 Hz and 2 s, i.e. 0.5 Hz, respectively).

Goel and Chopra (1998) also criticize the UBC97 formula because, firstly, it does not underestimate the periods and secondly, it is not appropriate for shear walls structures. They suggest the following formula, based on the wall dimensions:

$$T_1 = C_t \frac{H}{\sqrt{\bar{A}_e}}$$

with

$$\bar{A}_e = \frac{100}{L \cdot l} \sum_{i=1}^{Nm} \left(\frac{H}{H_i}\right)^2 \frac{A_i}{1 + 0.83 \left(\frac{H_i}{L_i}\right)^2} \tag{3.2}$$

with A_i, H_i and L_i the area, the height and the length of wall i, respectively and l the width of the building. It is obtained by theoretical analysis and validated on 17 buildings in California that experienced earthquakes between 1971 (San Fernando) and 1994 (Northridge). $\bar{A}_e$ is the equivalent shear area expressed as a percentage of the building area. This method requires a detailed analysis of the building and particularly to have access to its plans that can be difficult, especially for existing buildings.

Since Carder (1936), all the suggested formulas appearing in design codes have been validated on a low number of real structures (typically 20) using strong motion recordings. On the contrary, a large amount of periods of buildings obtained using ambient vibrations now exist across the world. Even if these values are known to be underestimated compared to the periods of structures needed by engineering (c.f. Sect. 2), they offer the opportunity to validate or reject relationships found using analytical methods.

3.2 Multiple Linear regression on the data

Considering the large number of experimental data that are available in Grenoble and Nice for RC structures, three items seems interesting to be studied. Firstly, is the horizontal dimension of the building L of any help for the estimation of the period for these buildings? Secondly, which superscript should be affected to the H parameter (1, 0.9, 0.75…)? Finally, is the number of stories, easier to obtain, as relevant as the height to estimate the period in a simplified formula? A comparison with more complicated formulas such as the formula suggested by Goel and Chopra (1998) was not possible because the plans of all the study-buildings were not available, that is generally the case for seismic analysis of existing buildings.

The RC buildings investigated in this study (Fig. 2) are of various style and building periods: some are isolated, other organised in blocks. The load bearing system of all the buildings was also not always clear, even if the major part is made of RC shear walls. Buildings with infilled RC frames in Metropolitan France are indeed often mixed with shear walls (Guéguen and Vassail 2004; Guéguen et al. 2007a). Tests to discriminate these buildings with pure shear walls failed indicating that the shear wall system imposes its dynamic behaviour. Moreover,

Fig. 2 Examples of considered RC shear walls buildings in Grenoble constructed from left to right in the 1930s, 1950s, 1960s and the years 2000

the discussion in Crowley and Pinho (2010) on the new designed buildings that are stiffer than the pre-code structures is less relevant for France. Indeed, most of the existing buildings were constructed before the introduction of codes and shear wall structures generally satisfy the low demand in such a moderate seismicity area because of their high stiffness.

Longitudinal and transverse values, that did not show a statistically significant difference, are available so that finally 173 periods are used in this paper, based on all the RC study-buildings in Grenoble and Nice considered as shear walls structures. The uncertainties on the period values, due to the various recording material and processing methods are low compared to the observed variability due to the investigated parameters and they will not be considered in this paper.

We made a multiple linear regression of the logarithm of the period as a function of the logarithm of the height H, the number N of stories (including the ground floor) and the length L of the building. The height and length of buildings were either obtained from structural plans, in situ measurement of inter-story heights or Accurate Digital Elevation Model (DEM) from very high-resolution optical data. We calculated the correlation matrix of the data (i.e. the covariance matrix of the standardized data) that is inverted. The coefficients of the linear regression and the correlation coefficients (total and partials) are then calculated using this inverted matrix.

The regressions using the parameters H and L or N and L gave a low partial correlation coefficient to L (13%), which means that L is not correlated with the periods. Moreover, the total correlation coefficient is not significantly increased when L is taken into account. The regression coefficient would lead to a superscript of 0.05, which does not match the -0.5 of the preceding formulas. As other authors, we can conclude that the length of the building L is not relevant in a statistical point of view in the variance analysis of the resonance periods of the buildings considered herein. It is probably due to the fact that shear walls are especially distributed around the staircases whose lengths are not really correlated with the length of the building itself.

The height or the number of stories explains alone between 85 and 90% of the period variance. A relationship using the height or using the number of levels differs of some percents, the height being the most relevant. In the frame of a simplified relationship, the quality of these two parameters should therefore be considered as roughly equal.

The results for the superscript associated to the height H and the number of levels N are 0.98 and 0.92, respectively. In a physical point of view, they should be equal so that a superscript of 1 should be affected to the height (or the number of levels), as suggested by Goel and Chopra (1998), but not the 0.75 of UBC97 or Eurocode 8. We turned the problem into a simple linear regression:

$$T_0 = 0.013H = \frac{H}{75} = 0.039N = \frac{N}{25} \tag{3.3}$$

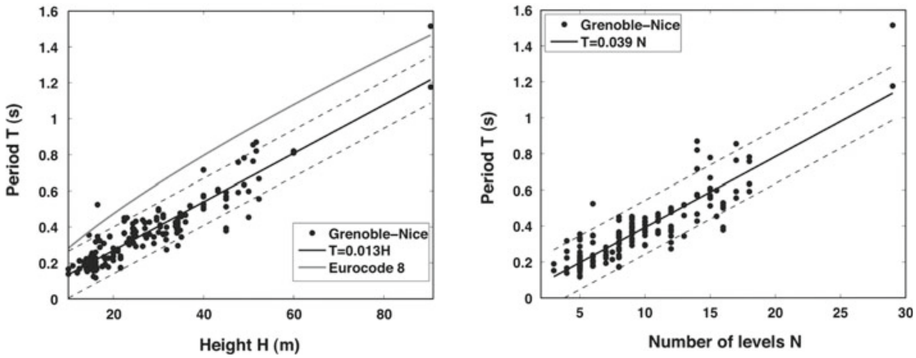

Fig. 3 First resonance periods of RC shear wall buildings in Grenoble and Nice (France) (*black dots*) versus their height (*left*) and their number of levels (*right*), the linear regression (*solid black line*) and the 95% confidence interval (*dashed black lines*). The solid grey line is the Eurocode 8 formula with $C_t = 0.05$

for RC shear walls, $\sigma^* = \{0.08; 0.09\}$ (in s)

$$\sigma^* = \sqrt{\frac{1}{n-2}\sum_{i=1}^{n}\left(T_i - T_i^{est}\right)^2} \tag{3.4}$$

with T_i^{est} the period estimated by the model

This regression and its 95% confidence interval are represented Fig. 3.

3.3 Comparison with other relationships

Figure 3 shows that the Eurocode 8 ($C_t = 0.05$ for shear walls) exceeds the experimental periods of 60 to 100% (40 to 50% less in frequency). More investigations should tell if these values really overestimates the f_1 value as expected. This is not obvious considering the fact that frequency drops of 35% maximum (Dunand et al. 2006) occur during non-damaging earthquakes. In any case, relationships found under ambient vibrations are underestimating the periods under earthquake so that they can be applied directly in the frame of a design code for conventional design.

Moreover, on the contrary to the regression here, the design code formula like in Eurocode 8 are not associated with standard deviations, which does not allow estimating the associated uncertainties that could make the user more careful with its results.

In the case of relationships used for vulnerability assessment, like the Lagomarsino and Giovinazzi (2006) formula ($C_t = 0.065$), the period should represent a good estimation for the stiffness until the yield point of the buildings capacity curve. The comparison with ambient vibrations data would correspond to a period elongation of 240 to 280% (i.e. a frequency decrease of 60 to 65%), which corresponds to the period of much damaged buildings. In that case, this relationship is therefore not adapted to French RC buildings, first because it is based on a formula suited for MRF structure and not shear walls that are stiffer and second because the used formula intends to overestimates the periods whereas it should provide best estimates.

Farsi and Bard (2004) already compared their recordings that are part of the dataset used here with the American (UBC88), French (PS92) and Algerian (RPA88) codes. They found a similar regression and showed the relationship in French codes (PS92 1995) was close to their results under ambient vibrations whereas the UBC97 and the RPA88 gave much higher periods. This either shows that French structures were known to be stiffer or that in the French codes more conservative periods were computed.

Oliveira and Navarro (2010) reviewed existing studies under ambient vibrations in Mexico, Chile, Spain, Venezuela, Ethiopia, Japan and Italy and developed frequency-height relationships for RC Portuguese buildings. Their review shows that the great majority of surveyed buildings across the world is made of MRF, with masonry infill or not. All the studies use a linear relationship, i.e. $\beta = 1$. They found C_t values between 0.012 and 0.020 except for Mexico City (0.035). In the present paper, results for shear walls structure with $C_t = 0.013$ fall in the lower bound of the results for infill wall structures.

The fact that similar relationships are found under ambient vibrations by very different studies from various countries indicates they should not be much biased due to soil conditions and therefore soil-structure interaction in these formulas, except for the very particular case of Mexico City (Kobayashi et al. 1987; Oliveira and Navarro 2010). The structures of Oliveira and Navarro (2010) study are mostly on firm soil, whereas in Grenoble the soil is generally made of soft sediments (Guéguen et al. 2007b).

Guler et al. (2008) also studied infilled MRF structures under ambient vibrations in Turkey. They found $C_t = 0.026$ and $\beta = 0.9$ by regression on 6 buildings. They suggest that $T_1 = 1.75T_0 (f_1 = 0.57f_0)$ by fitting the UBC97. This approach is however questionable since the curves of UBC97 do not pretend to represent the real fundamental periods.

4 Elastic part of capacity curves in large-scale vulnerability assessment

4.1 Large-scale vulnerability assessment methods

In large-scale vulnerability assessment methods, the basic object is no more the building but the structural type. The first step is therefore to build a typology for the study-area. For HAZUS (FEMA 1999) and Risk-UE (Milutinovic and Trendafiloski 2003) methods, the buildings are separated using their structural system. The types are rather simple and cover a large range of building properties (material properties, know-how of the different countries…).

Each type is then modelled using a capacity curve that links the force to the displacements of the structure. For HAZUS as for Risk-UE, these curves are bilinear. The first line represents the elastic behaviour until the yield point, the second one, with a flatter slope represents the ductile behaviour (with hardening) until collapse. Three curves are given for each type depending on the height of the structure: low (until 2 stories for URM, 3 for RC), medium (3–5 stories for URM, 4–7 stories for RC) and high (6 stories and more for URM, 8 and more for RC) buildings.

4.2 Comparison with in situ data

For a 1 degree-of-freedom system, the force F is written as a function of the displacement U as follows:

$$F = KU = \omega^2 MU \tag{4.1}$$

with K the stiffness, M the mass and ω the angular frequency. For multiple degrees-of-freedom systems assimilated to their first mode only, a participation factor can be included in this

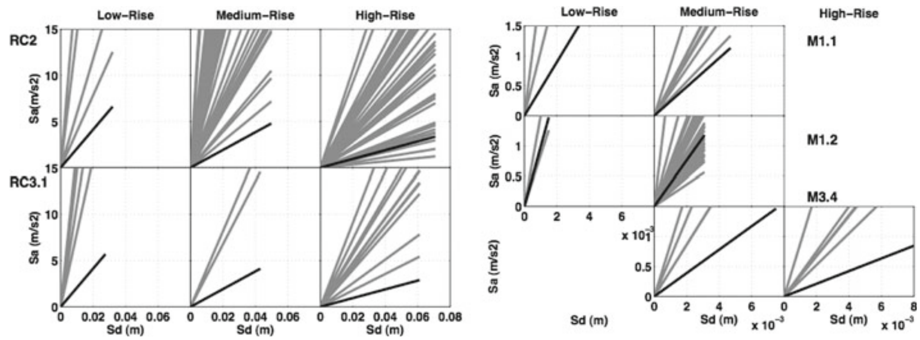

Fig. 4 Elastic part of the experimental capacity curves obtained from all the structures in Grenoble (*grey*) compared to Risk-UE curves (Lagomarsino and Giovinazzi 2006) (*black*) for different height classes. **a** RC buildings (RC2 = shear walls, RC3.1 = infilled frames). **b** Masonry buildings (M1.1 = rubble stone, M1.2 = simple stone, M3.4 = simple stone with concrete slabs)

formula but it changes only slightly the results. The capacity curve is usually represented in acceleration A:

$$A = \frac{F}{M} = \omega^2 U \tag{4.2}$$

The slope of the first line that stands for the elastic part is $\omega^2 = 4\pi^2 f_1^2$, with f_1 the "average" frequency until the yield point. It is therefore possible to compare the theoretical slopes with the experimental ones f_0, taking into account the fact that $f_0 > f_1 > 0.4f_0$ as shown by laboratory evidences (Calvi et al. 2006). We draw the experimental slopes and the Risk-UE curves given in Lagomarsino and Giovinazzi (2006) until the yield displacement Dy. The theoretical curves should therefore lie under the experimental ones. Since the assumptions for pre-code buildings in Lagomarsino and Giovinazzi (2006) seemed not to be adapted to French structures as showed in the previous sections, the low-code capacity curves are used here for comparison.

4.2.1 RC buildings

Among the Grenoble buildings, two Risk-UE types exist: RC shear walls (RC2) and RC infilled frames (RC3.1). The results (Fig. 4) show a very large scatter where only a single theoretical curve is given, without uncertainties. However, between a building with 7 stories and a building of 4 stories of the same type, the frequency can be doubled (Fig. 3), without taking the uncertainties into account (it can then be multiply by 4). The same variations apply to the yield strength.

For the RC2 type, the number of experimental data is enough. As expected, the theoretical curves lie under the data. The uncertainties on the yield strength, however, stand within a factor of 5. For the RC3.1 type, there are not enough data to conclude, but the trend is similar. The theoretical curves for this class are somehow similar to the RC2 type in Lagomarsino and Giovinazzi (2006) whereas the curves given by the University of Thessaloniki (Kappos et al. 2006) are much different. It may be due to the large scatter in the construction practice through Europe.

4.2.2 Masonry buildings

The masonry buildings studied in Grenoble are mainly part of the Risk-UE M1.1 (rubble stone) and M1.2 (simple stone) types (Guéguen and Vassail 2004; Guéguen et al. 2007a). The M3.4 (URM with RC floors) type is also interesting to study because its behaviour is still not well understood. The Risk-UE capacity curves were proposed by Lagomarsino and Giovinazzi (2006) using a displacement-based method independent from the formula studied in part 3.

They show the rubble stone buildings (M1.1) are more flexible (or more heavy) than the simple stone buildings (M1.2), which seems not so clear for buildings in Grenoble. The M1.2 curve is close to the average of the experimental curve so that it is no more a correct value for the stiffness at yield. Finally, the theoretical curve for M3.4 type assumes more flexible (or more heavy) structures, as also stated by Bal et al. (2008) for Turkish buildings, whereas the experimental data show they are at least as stiff as type M1.2 (Fig. 4). In conclusion, Grenoble experimental data show more homogeneous periods on average over the different masonry types, still with a large variability, compared to what is found in analytical approaches. This statement assumes however that the frequency drop up to yield of structures is the same whatever the construction material.

5 Conclusions

The study of 173 resonance periods of RC buildings and 66 resonance periods of URM buildings collected experimentally in Grenoble and Nice (France) allowed to point out the variability of this key parameter for earthquake engineering. Moreover, this paper tried to contribute to the meanings of "elastic fundamental resonance frequency" (f_0 and f_1) in earthquake engineering, especially evaluated using ambient vibrations. The values of frequencies relevant for design purposes or rough vulnerability assessment (f_1) are often supposed overestimated to be conservative in conventional design so that it is difficult to compare them. This study shows however that other parameters but the height of the building in the frequency relationships are statistically irrelevant. The experimental dataset used herein let us assume that for French existing RC buildings, a superscript of 1 should be affected to the period parameters, given the following relationship:

$$T_0 = 0.013H = 0.039N$$

with standard deviations σ^* of 0.08 and 0.09 s, respectively. The number of levels N and the height H are correlated with the period T_0 with roughly the same standard deviation. This relationship underestimates the real periods under earthquake so that it can be directly used for conventional design. A higher value T_1 may be more appropriate for vulnerability assessment. However, further research is needed to find a relationship between these two values.

Such empirical relationship provides only a rough estimate of the periods with a large standard deviation. In some cases like vulnerability assessment, no better estimate can be easily found. However, for design, relationships based on the wall lengths (Goel and Chopra 1998; Crowley and Pinho 2010) may be more adapted.

The capacity curves in the Risk-UE method are also linked to the resonance frequency f_1 and therefore f_0. Very large uncertainties on these curves have been shown by the experimental data used in this paper. A large part of this uncertainty may be removed by giving one curve for each number of stories using the proposed regressions. In addition, the Risk-UE capacity curves are not adapted for the typical French buildings found in Grenoble and region-specific curves must be proposed to take local design practice into account. For example, in the French Caribbean Islands, the construction practice uses extensively MRF structures that were not investigated here. Then, the period got from ambient vibrations could be more often used as a validation of the elastic part of capacity curves used for displacement demand estimation.

Finally, these relationships are necessary for a rough evaluation of ambient vibration results in a particular building. They provide a first insight of its stiffness compared to other existing structures.

Acknowledgments This work has been supported by French Research National Agency (ANR) through RGCU program (project ARVISE n°ANR-06-PRGCU-007-01), by LCPC laboratory and by the VULNERALP project of the Rhône-Alpes regional council. The authors thanks the two anonymous reviewers for their constructive and valuable remarks that greatly improved the quality of the paper.

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