Experimentally supported modelling of an existing masonry building by measuring ambient and forced vibrations

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SUMMARY:
Seismic performance assessment of existing masonry buildings is affected by many uncertainties, which are difficult to evaluate, especially, if destructive tests of masonry are not available. In such cases, the effect of uncertainties can be reduced by the use of non-destructive experimental techniques. However, their efficiency is not well defined. Thus we performed measurement of ambient and forced vibrations on an old two-storey masonry building. We investigated if such experiments can contribute to greater accuracy of the results. Herein the experimental setup is briefly described. The estimated vibration periods based on the two types of measurement are compared and seismic performance of the building is assessed for the near-collapse limit state. We found that accuracy of the seismic performance assessment of the building can to some extent be improved if structural model is calibrated to the estimated vibration periods.

Keywords: ambient and forced vibrations, modelling uncertainties, seismic assessment, N2 method, existing masonry building

1. INTRODUCTION

Seismic performance of an existing masonry building is a challenging task, especially in the case, when it is not allowed to perform destructive tests of masonry. Consequently, adopted input parameters of the structural models are highly uncertain and their effect to seismic response parameters is not negligible, which applies for different types of buildings (Dolsek 2009; Rota et al. 2010). One of the possibilities to reduce modelling uncertainty is to measure ambient or forced vibrations for the purpose of estimating the natural vibration periods of the building. The pioneering work in this field of research was done by Crawford and Ward (1964). They used random wind excitations to find the first three modes of vibration of a nineteen storey building. The most common technique is to measure ambient vibrations due to human activities like traffic or due to natural forces like wind excitations (Crawford and Ward 1964; Hans et. al 2005; Michel et al. 2008; Gallipoli et al. 2009).

Techniques for estimation of natural frequencies are based on time domain or frequency domain, which has recently been more often used. Technique called Horizontal-to-vertical-spectral ratio (HVSR) (Nakamura 2000) is often used and enables the estimation of natural frequencies of the building in frequency domain, by calculating the ratio between the amplitudes of the Fourier spectra corresponded to the horizontal and the vertical component of the same measured signal at the highest level of the structure. Herein the natural vibration frequencies were estimated by using basic technique, so called peak picking (PP) technique (Crawford 1964; Trifunac 1972), which involves the Fourier transforms of short time windows of a signal and picking the value of the frequency peaks of the Fourier spectrum.

Many studies were done in order to calibrate elastic structural models of the existing buildings (Michel et al. 2008; Cunha et al. 2006) based on the estimated vibration periods and mode shapes obtained from the ambient or forced vibration measurements. However, it is not well understood to which extent it is possible to limit the effects of uncertainty on the seismic performance assessment of buildings if elastic dynamic characteristic of the building are available based on in-situ tests. Herein we present results of a case study of a two-storey masonry building. We estimated the vibration periods of the building based on measurement of ambient and forced vibrations. The forced vibrations were induced into the ground with a large machine called Vibroscan. In the second part of the paper, the natural vibration periods of the elastic structural model were validated based on the experiment and the seismic performance of the building was assessed utilizing simplified nonlinear method (Fajfar 2000). Based on the parametric study we discuss how sensitive are results compared to the uncertain input parameters of the structural model.

2. EMPIRICAL ESTIMATE OF NATURAL FREQUENCIES FOR AN OLD MASONRY BUILDING

2.1. Tested structure and experimental setup

Two different techniques were used in order to estimate natural frequencies of an old two-storey masonry building (Fig. 1a). The first technique involves measurement of ambient vibrations, whereas the second is based on forced vibrations induced with Vibroscan (Fig. 1b), whose original purpose is to simulate and evaluate vibrations of buildings caused by trains (iC Consulenten 2010). The 13 t vehicle has a hydraulic vibration generator, which is able to generate vertical sinus waves over the frequency interval from 5 to 120 Hz with a force capacity of 68 kN. The vibrations of the reaction mass are transmitted into the ground through a steel base plate. Machine generates sweep signal, which is defined as a sine signal with constant amplitude and linearly increasing frequency over a specific period of time. Three different types of sweep signals, which were defined in the frequency intervals, respectively, from 5 to 100 Hz, from 5 to 30 Hz and from 5 to 20 Hz, were used. Forced vibrations were induced close to the building, since it was not allowed to damage adjacent buildings.

Vibrations were measured with 3D velocity sensors, which were located in the centre and the three corners of the roof storey (Fig. 1c). They were wired to the computers, where analog signal from sensors was transformed into digital signal and saved. Two main axes of the sensors were oriented in the main directions of the building (EW - longitudinal X direction and NS – transverse Y direction). The sensors were positioned close to the bearing walls in order to minimize undesired effects of the flexible wooden floors. Measurement of ambient vibrations was performed during the night, when the building was unoccupied. The data from ambient vibrations was acquired by using a sampling frequency of 500 Hz and stored into separate files every 4 minutes.

Figure 1. a) Old masonry building in Ljubljana b) Vibroscan machine located 1 m from the tested building c) The position of the sensors – roof storey

2.2. Technique for determination of natural frequencies

The first natural frequencies of translational modes in X and Y direction and the first torsional mode were evaluated by utilizing peak picking method. The procedure is illustrated in Fig. 2. and based on the selected signal from measurement of ambient vibrations (e.g. Fig. 2a). For each such signal a Fast Fourier Transform (FFT) was performed. Fourier spectra corresponding to a selected 4 min interval of X component of velocity time histories measured at a central and corner location of building are presented in Figs. 2b and 2c. A typical bell shape curves with the peaks corresponding to the location of natural frequencies can be observed for these Fourier spectra. In Figs. 2b and 2c, the first bell-shaped peak at f_x = 6.3 Hz corresponds to the natural translational frequency in longitudinal X direction (EW) since the amplitudes in the Fourier spectra for corner and central sensor are of the same magnitude. The second peak at f_t = 10.1 Hz, represents the first torsional natural frequency, since the amplitude in the Fourier spectrum corresponding to the SW corner of the buildings is significantly larger than that from the central sensor.

Similar procedure for determination of building's natural frequencies was also used in the case if velocity time histories were recorded by utilizing Vibroscan. The main difference between the Fourier spectra based on signals corresponding to the ambient or forced vibrations can be observed in high frequency range (Figs. 2b and 2d). Naturally, due to stronger vibrations and wide frequency range of forced vibrations, it is possible to estimate natural frequencies which correspond to higher modes (Fig. 2d). The first two peaks of the bell-shaped curve clearly correspond to the first translational and torsional natural frequencies.

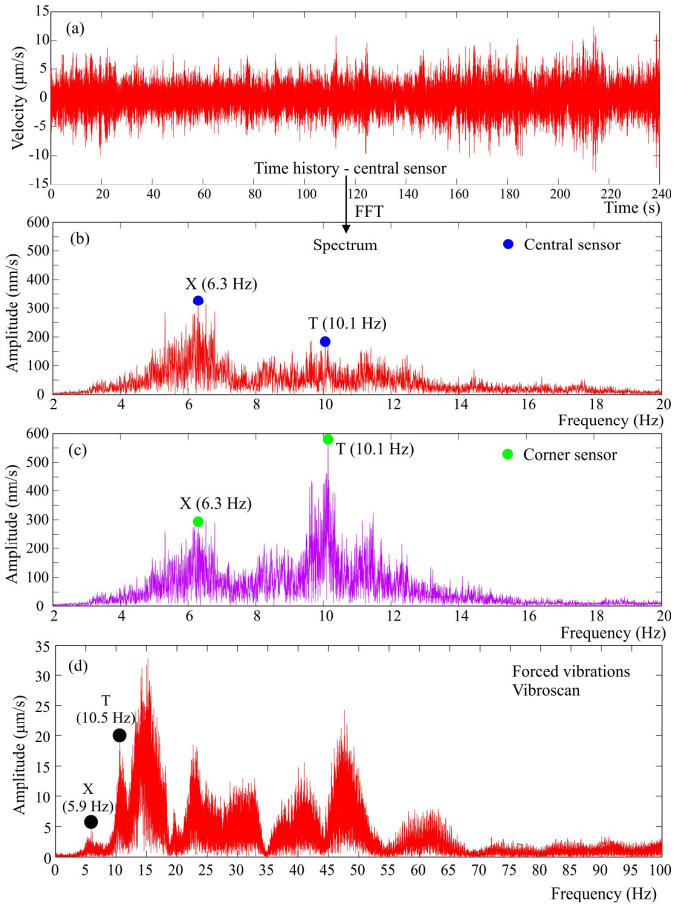

Figure 2. Illustration of the procedure for estimation of the first natural frequency in EW direction (X) and the first torsional frequency (T) a) the recorded 4 min time interval of velocities at the center of the building (ambient vibrations), b) corresponding Fourier spectrum of measured velocities in the center of the building (ambient vibrations), c) Fourier spectrum corresponding to location of a SW corner sensor (ambient vibrations) and d) Fourier spectrum of the forced vibrations and the determination of first natural frequencies

2.3. Results

During the experiment we recorded more than 140 signals of velocity time histories based on measurement of ambient vibrations and 53 signals due to forced vibrations induced by Vibroscan for each positioned geophone. Natural frequencies were estimated according to the procedure described above for each independent signal. Based on these results we computed the probability density function of the estimated translational and torsional natural frequencies (see Fig. 3). The difference between the mean natural frequencies if estimated based on the ambient or forced vibrations is minor, only around 6 %, which confirms similar conclusions of some other authors (e.g. Trifunac 1972; Hans et al. 2005). In addition, we found that coefficient of variation of estimated natural frequencies is low (0.03-0.11), which indicates that measurements are quite reliable.

The experimental procedure utilizing Vibroscan is more complicated than measurement of the ambient vibrations. Firstly, we had to be very careful not only due to the transportation of the heavy Vibroscan, but also due to a selection of the level of forced vibration, since we did not want to cause any damage on the tested building as well as on the buildings in its vicinity. Even the smallest intensity of Vibroscan force (3 % of the capacity) produced very intense vibrations of the building. For example, horizontal velocity (4 mm/s) measured at the top floor of the building in the case of forced vibration was about 400 times larger than that we observed from the ambient vibrations (10 μm/s). In addition, Vibroscan can induce only a sine signal with changing frequency. Wider frequency range of forced vibrations offers a possibility for estimation of higher natural frequencies of buildings (e.g. peaks at high frequencies in Fig. 2d), but considering the small amount of time, when the building oscillates near its first frequency in the beginning of the sweep signal, the determination of the first natural frequency is not as clear as it could be.

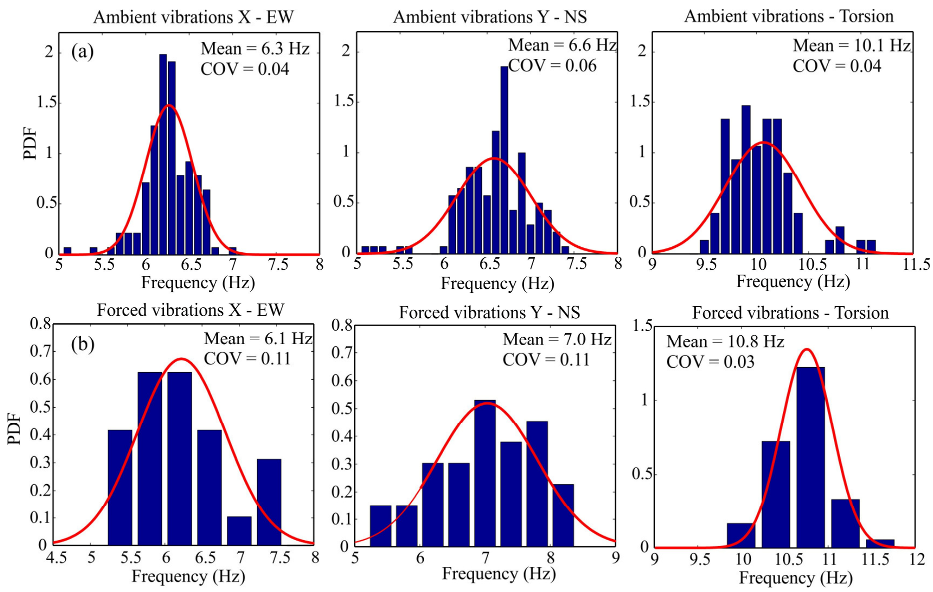

Figure 3. Estimated probability density function of the first translational natural frequencies in X and Y direction and torsional natural frequency based on the measurement of a) ambient and b) forced vibrations. Mean value and the coefficient of variation were estimated based on the maximum likelihood method.

For brevity, we will not distinguish between the results based on measurement of ambient and forced vibrations and we will refer to the experimental results, which are determined as the average between the mean results from both measurements. Therefore the estimated translational vibration periods of this masonry building in X (EW) direction and Y (NS) direction are, respectively, 0.16 s and 0.15 s, whereas the first torsional vibration period is 0.10 s. Low vibration periods of the building were expected since residential masonry houses with thick walls (0.45 m), which represent large percentage of the building's plan (around 10%), are very stiff. Note that we preferred to report natural vibration periods rather than natural frequencies since this is more common for structural engineers.

3. NUMERICAL INVESTIGATION OF NATURAL VIBRATION PARAMETERS

In this Section we firstly describe the base-case structural model, which was defined according to our best judgment, and compare computed natural vibration periods with those reported in the previous Section. Later on we present the results of simple parametric study aiming to identify which input parameters of the structural models have an important impact on the natural vibration periods of the building. Finally, we discuss the influence of uncertain input parameters on the seismic assessment of the building according to the N2 method (Fajfar 2000).

3.1. Description of the structure and base-case structural model

Two-storey unreinforced masonry structure was built of solid clay brick. It has wooden floor in the bottom storey and wooden floor with 6 cm concrete topping in the top storey. Wooden beams (14/24 cm) are oriented along X direction and distributed evenly every 90 cm. The building has 8.5 % shear walls in X direction and 10.8 % shear walls in Y direction. The storey height and the thickness of walls amounted 3.4 m and 0.45 m, respectively. The plans of the bottom and top storey are shown in Figs. 4a and 4b. The self weight and the dead load of the building were considered to be as realistic as possible at the time of the experiment. We calculated that the self weight of the wooden floor above the bottom and top storey is, respectively, 1 kN/m^2 and 1.7 kN/m^2, whereas additional dead load due to the roof amounted 1.25 kN/m^2. The dead load of the equipment was estimated 0.5 kN/m^2 and 0.3 kN/m^2, respectively, for the floor above the bottom and top storey. The specific weight of the walls was assumed 16 kN/m^3. Note that we did not detect any visible structural damage, therefore all structural elements were modelled as being undamaged. The floors of the base-case model were assumed rigid in their plan.

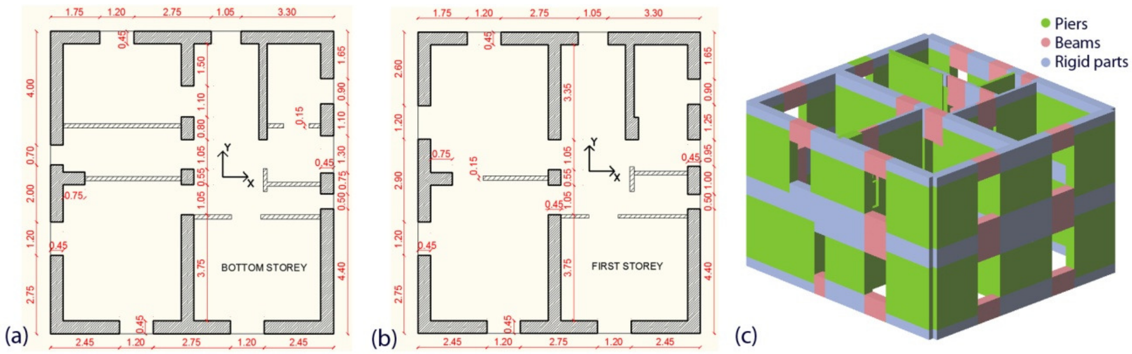

Figure 4. The building's plan of the a) bottom, b) top storey and c) structural pseudo-3D model in 3Muri.

The pseudo-3D non-linear structural model (see Fig. 4c) of the building was made by using program 3Muri, which is a specialized program for seismic analysis and performance assessment of masonry structures, whereas the analysis was performed by using the research version of the same program, Tremuri (Galasco et al. 2009). Tremuri is based on the effective macro-element approach, thus enables sufficiently accurate modelling of whole masonry building for the purpose of nonlinear pushover and time history analysis (Penna 2002). The non-linear model consisted of elastic beam/column elements with flexural and shear hinges, defined by an elasto-plastic moment-rotation and force-displacement relationship, respectively. Note, that the horizontal resistance of the element was assumed 0, if drift demand exceeded ultimate drift, which was defined according to Eurocode 8-3 requirements (CEN 2005a) and amounted δ_s = 0.4 % and δ_f = 0.8 % of the wall height, respectively, for the case of shear-sliding mechanism (CEN 2005b) and flexural collapse mechanism (e.g. Tomaževič 2009; CEN 2005a).

In the absence of destructive in-situ tests of walls, which we were not allowed to perform, we decided to assess mechanical characteristics of masonry based on the experimental results provided by Tomaževič (2009) for the solid brick specimen, whose compressive strength of bricks and the mortar were 15 and 2.5 MPa, respectively. Therefore, the assumed compressive and initial shear strength of masonry for the examined building are f_k = 2.3 MPa and f_{vk0} = 0.20 MPa, respectively. Moduli E and G reported in the literature (Tomaževič 2009) were increased for 25% according to engineering judgment, since experimentally determined moduli are based on the secant stiffness, which usually corresponds to one third of the maximum compression strength (CEN 1998). Their final values were E =1000 MPa and G = 250 MPa. By using these material characteristics, we obtained the natural vibration periods T_X = 0.159 s, T_Y = 0.145 s and T_{tor} = 0.122 s for the X direction, Y direction and torsion, respectively. The difference between the computed and experimentally obtained mean natural vibration periods (Fig. 3), is less than 2 %, except in the case of torsion where the numerically computed vibration period is around 25 % larger than the vibration period from the experiment. This is the consequence of the smaller torsional stiffness of the pseudo-3D model.

3.2 The effect of input parameters variations on the vibration period of the structural model

Base-case structural model was defined based on the engineering judgment. It is therefore very likely that other analyst would assume different input parameters. For this reason we performed a simple parametric study based on 16 additional structural models, which were defined in order to identify the sensitivity of natural vibration periods to the model input parameters. These structural models were classified in four different groups. In the group 1 (Models 01 – 06) the elastic modulus E and shear modulus G, were varied simultaneously for ± 20 % 50 % and 100 %, which represents possible choices of these parameters by other engineers. In this case the G/E ratio remained constant, whereas for group 2 (Models 07 – 11) the ratio G/E was varied in the range between 0.15 and 0.40 with an increment of 0.05. The highest value of this ratio represents the recommendation from Eurocode 6, but according to Tomaževič (2009), it is very unlikely, that the ratio G/E will reach such high values in reality. The uncertainty in mass was investigated with the variation of the specific weight of the walls (Group 3: Models 12 - 15), which contributes almost 90 % to the total mass of the model. These models were defined based on the specific weight of the walls in the interval from 14 kN/m^3 to 18 kN/m^3 with an increment of 1 kN/m^3. In addition two more models were defined (Group 4) in order to check the effect of the flexible floor due to the wooden beams and the effect of partition walls, which could be neglected in the analysis by some engineers.

As expected, results revealed that uncertain modelling parameters may have large impact on the computed vibration periods of the building (Fig. 5). Probable material characteristics affected natural vibration periods with respect to those estimated from the experiment even for more than 40 %. The largest impact on the vibration period has the simultaneous change of elastic and shear modulus (Group 1), which results in the range of vibration periods between 0.11 s and 0.23 s for translational mode shape in X direction. Possible variations of the ratio G/E have a slightly smaller effect on the elastic vibration periods (between 0.14 s and 0.19 s). For this particular building, the variation of material density and consideration of flexible floor have minor impact on vibration periods that corresponds to translational mode shapes (5 % compared to the experimental results). The vibration period are mainly governed by the change of the stiffness, which is also the case with the model without partition walls, where the reduced stiffness results in 10 % larger vibration period than in the base-case model.

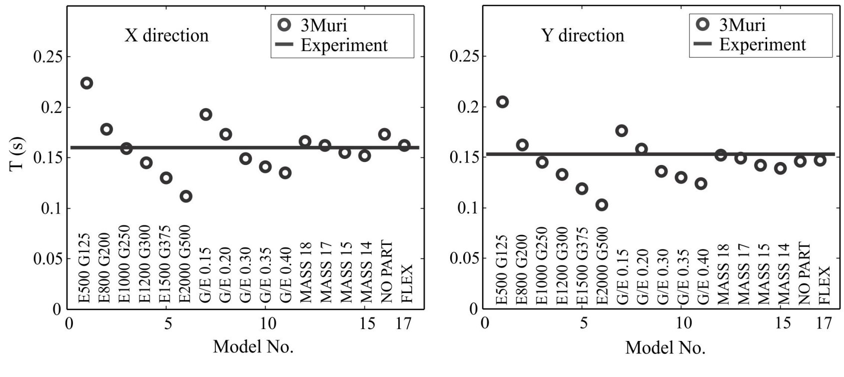

Figure 5. The effect of possible input parameter variations on the vibration periods in X and Y direction.

3.3 The effect of input parameters variations on seismic assessment of the existing building

Our final objective is to assess whether uncertain modelling parameters have a significant impact on the seismic performance assessment of the building for ultimate limit state, and to show to which extent it is possible to limit the effects of uncertainty if elastic dynamic characteristic of the building are available from in-situ tests.

Seismic performance assessment of the building was performed for the pseudo-3D model (Tremuri) using the N2 method (Fajfar 2000) which involves non-linear static analysis. In order to compute the peak ground acceleration that causes near collapse (NC) limit state of the building $a_{g,NC}$, pushover curves were idealized with elasto-plastic force-displacement relationship (Fig. 6). The initial stiffness of the equivalent SDOF model is defined based on 70 % of the maximum base shear F_{max}, whereas the yield force F_y of the idealized force-displacement relationship is calculated by assuming equal area under the pushover curve and idealized force-displacement relationship, if measured in the interval of displacements from origin to near-collapse displacement d_{NC}, which was defined in the softening range of pushover curves and corresponds to 80% of base shear resistance ((d_{NC}, F_{NC}) in Fig. 6).

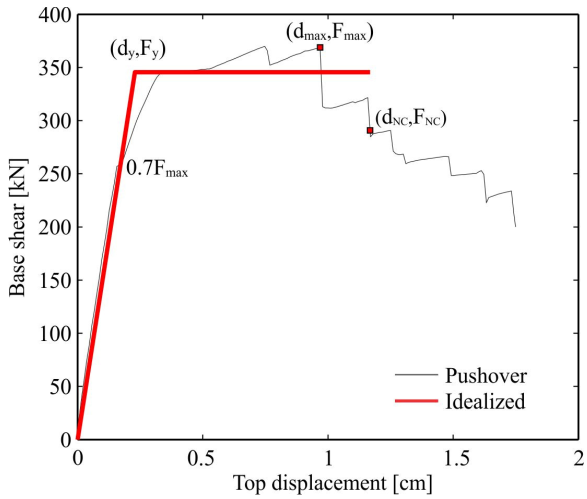

Figure 6. The pushover curve of the base-case model and idealized force-displacement relationship

The lateral loads used in the pushover analysis corresponded to the product of mass matrix $\boldsymbol{M}$ and the displacement vector $\boldsymbol{\varphi}$, which had value 1 at the location corresponding to the top displacement (φ_n = 1), and was assumed uniform over the height or proportional to the first mode shape. According to Eurocode 8 we considered four directions of seismic forces (+X, -X, +Y, -Y), two vertical distributions of horizontal forces (‘uniform’ and ‘modal’) and 5% accidental eccentricity ($\pm e_{ai}$), which resulted in 24 pushover analyses for each model, whereas the analysis with the smallest $a_{g,NC}$ was considered as critical.

The force-displacement relationship of the equivalent SDOF model (F_y^*, d_y^* and d_{NC}^*) was determined by dividing displacements and forces of the idealized pushover curve by the transformation factor Γ, which was defined as follows

$$\Gamma = \frac{m^*}{\sum_{i=1}^{n} m_i \varphi_i^2}, \quad m^* = \sum_{i=1}^{n} m_i \varphi_i \tag{3.1}$$

where m_i and φ_i are masses and normalized displacements at the location of the i-th storey, and m^* is the mass of the equivalent SDOF model. The corresponding initial stiffness (F_y^*/d_y^*) and the mass of the equivalent SDOF model defined its period

$$T^* = 2\pi\sqrt{\frac{m^* d_y^*}{F_y^*}} \tag{3.2}$$

where F_y^* and d_y^* are, respectively, the yield force and the yield displacement of the equivalent SDOF model. The available ductility at NC limit state of the building was defined as $\mu_{NC} = d_{NC}/d_y$, where d_{NC} and d_y corresponded to ultimate displacement and yielding of the idealized force-displacement relationship (Fig. 6), respectively.

The simple approach to calculate the elastic spectral acceleration associated with the near collapse limit state $S_{ae,NC}$ involves R-μ-T^* relationship, which is according to the N2 method defined as

$$R_{\mu,NC} = \frac{S_{ae,NC}}{S_{ay}} = \left(\mu_{NC} - 1\right)\frac{T^*}{T_C} + 1 \tag{3.3}$$

where T_C is the corner period between the constant acceleration and constant velocity range of the acceleration spectrum and $R_{\mu,NC}$ is the so called reduction factor, which is defined as the ratio between the spectral acceleration $S_{ae,NC}$ and the spectral acceleration capacity of the equivalent SDOF model $S_{ay} = F_y^*/m^*$. The plateau of the elastic acceleration spectrum according to Eurocode 8 (CEN 2004) was used for defining the seismic action, since the period of the equivalent SDOF models were always in the range between T_B and T_C. According to above definitions, the peak ground acceleration $a_{g,NC}$ taking into account the soil factor S was for the building under consideration calculated as

$$a_{g,NC} = a_{g,A} \cdot S = \frac{F_y^*}{2.5 \cdot m^*}\left(\left(\frac{d_{NC}^*}{d_y^*} - 1\right)\cdot\frac{T^*}{T_C} + 1\right) \tag{3.4}$$

Results of the pushover analyses have shown that the maximum base shear of the building is much smaller in X direction due to significantly smaller area of masonry walls with respect to that in Y direction. In most cases the minimum $a_{g,NC}$ corresponded to the pushover analysis in +X direction with consideration of accidental eccentricity and the modal lateral forces pattern (Fig. 6). The peak ground acceleration, which causes NC limit state $a_{g,NC} = 0.17$ g, was significantly smaller than the design peak ground acceleration, which amounted $a_{g,design} = 1.2 \cdot 0.25\text{g} = 0.30$ g (Ljubljana, soil type B). Hence we concluded that the building does not fulfil safety requirements according to Eurocode 8.

A parametric study was performed based on the group of models defined in Section 3.2 in order to investigate which input parameters can significantly affect results of the seismic performance assessment of the building. The effect of variation of modelling parameters on the yield displacement d_y, yield force F_y, the ultimate displacement at near collapse limit state d_{NC} and on the peak ground acceleration, which causes NC limit state, is presented in Fig. 7.

The effect of E and G on the yield displacement of idealized critical pushover curve d_y is significant (Fig. 7a), since this parameter is very well correlated with stiffness and vibration period (correlation coefficient $\rho_{dy,T} = 0.95$). For example, the simultaneous increase of E and G results in the variation of d_y in a very wide range between 0.14 cm and 0.4 cm, whereas the variation of G/E ratio caused that d_y varied only from 0.19 cm to 0.27 cm. The probable variation of mass does not have such a high impact on d_y, since its variation did not exceed 10% with respect to d_y of the base-case model. The effect of variation of input parameters on the displacement at near-collapse limit state d_{NC} is shown in Fig. 7b. The d_{NC} is strongly affected by the formation of various plastic mechanisms and damage propagation and it varies in the range between 0.85 cm and 1.3 cm for all models considered in the parametric study. In specific case of this building, we did not observe many different plastic mechanisms due to the low number of elements. However, in cases of more complex buildings, the formation of plastic mechanisms could have even greater impact on d_{NC}.

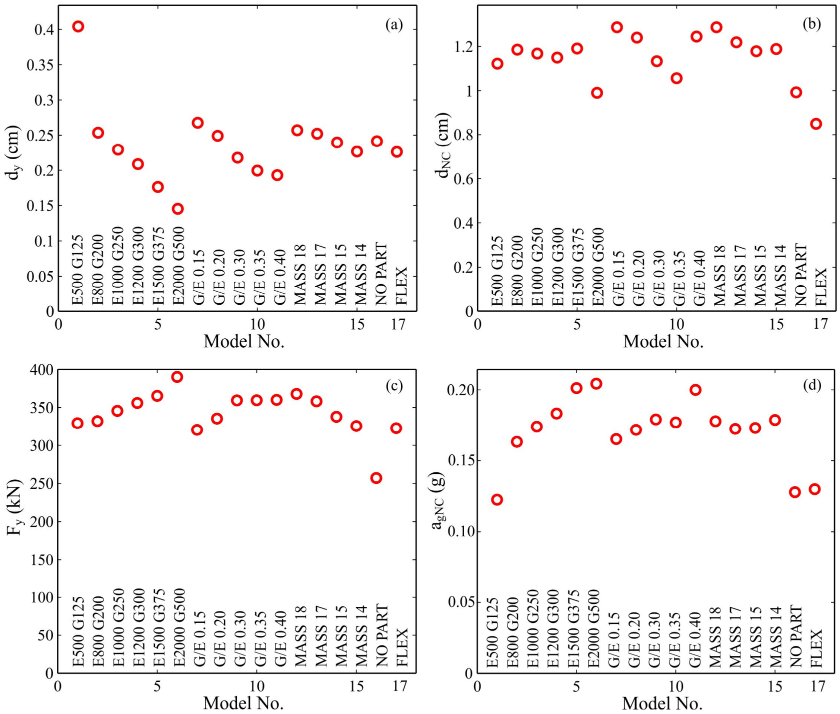

Figure 7. The effect of the variation of the four groups of input parameters on a) the yield displacement d_y b) the near-collapse limit state displacement d_{NC} c) the force F_y of the idealized pushover curve and d) the limit-state peak ground acceleration $a_{g,NC}$

The F_y of the idealized pushover curve is significantly affected by the partition walls (model No. 16, Fig. 7c) since the flexural and shear resistance of the elements strongly depend on the axial force. Hence, quite large impact on F_y was observed due to small variation of the wall density, since lower density resulted in lower axial forces in walls, which significantly affected the resistance of the walls according to adopted failure models of the masonry walls. The variation of E, G also affected the yield force F_y (Fig. 7c). Based on this parametric study it is likely that different analysts would determine F_y in the range between 320 kN and 390 kN, i.e. in the range within the ±11 % compared to the F_y of the base-case model.

Finally, the global behavior of an existing building was evaluated based on the minimum peak ground acceleration $a_{g,NC}$ (Eq.(4)), as described in Section 3.3. In Fig. 7d, it is shown that the simultaneous increase of E and G and also the increase of G/E ratio resulted in the increase of $a_{g,NC}$, but the opposite trend was observed for the case of variation of the wall density. Combination of low base shear resistance (Fig. 7c) and relatively low ductility (large d_y and small d_u in Fig. 7d) were the main cause for small $a_{g,NC}$ calculated for the cases of models 1, 16 and 17. The variation of uncertain input parameters resulted in a relatively wide range of possible $a_{g,NC}$ (between 0.12 g and 0.20 g).

5. CONCLUSIONS

Vibration periods of an old masonry building were estimated in this research based on the measurement of ambient and forced vibrations. Both techniques provided very similar results. However, in the case of stiff masonry residential house, which is dominated by the first mode shape associated with the translations in X or Y direction, it is probably more appropriate to utilize technique based on ambient vibrations, since it provides reliable results for a reasonable price. However, in case of higher or heavier buildings (dams, bridges) the use of more complicated and more expensive forced vibrations, which enable clearer view on the higher modes, is justified.

In the second part of the paper, the effect of modelling parameters (E, G, G/E ratio, mass, flexible floors and partition walls) on the vibration period was investigated. We found that different engineers could misjudge the vibration period even for 40 % due to possible values of E and G. The variation of mass and other modelling parameters did not have a significant impact on the vibration period of the elastic model. However, calibration of the model to the estimated vibration periods could increase accuracy of the seismic performance assessment of the building, since the vibration period of the elastic model is highly correlated with the vibration period of the equivalent single-degree-of-freedom model, which was used for determination of $a_{g,NC}$. Further, the impact of uncertain parameters and the importance of calibration of the model to the estimated vibration periods reduce with respect to the parameters which are less and less correlated with the vibration periods of the buildings. Without adequate calibration of structural model, different engineers could estimate $a_{g,NC}$ in a range between 0.12 g and 0.20 g even for the case of this simple old masonry building. This is in the interval from -30% to 20% in comparison to $a_{g,NC}$ = 0.17 g associated with the base-case structural model, whose natural vibration periods were practically the same as those estimated based on ambient or forced vibrations.

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