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Data-driven model updating for seismic assessment of existing buildings

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ABSTRACT: The seismic vulnerability assessment of the existing building stock poses major challenges with tremendous economic and societal impact. While advanced and well-enforced building codes remain the backbone of risk mitigation, large parts of the European building stock have been built prior to modern building codes and thus, do not comply with the current seismic standards. Furthermore, the seismic assessment of individual structures suffers from large uncertainties pertaining to unknown material properties, soil-structure interaction and unpredictable effects of ageing, while the current state of practice for urban-scale vulnerability assessment relies on heavily simplified physical models. To this end, Structural Health Monitoring (SHM) provides tools for the interpretation of structural-response measurements in order to gather information on the structural condition. Measurement data can further be utilized to update computational models with the goal of refining seismic-performance estimations. In this contribution, the impact of amplitude-depended model updating on the expected seismic performance of an existing masonry building is assessed. Dynamic recordings under ambient excitation are analyzed and compared to the response to higher amplitude vibrations, which are generated during demolition works. Special consideration is given to the analysis of amplitude-dependent properties that are shown to substantially affect the response in the nonlinear range. Overall, this contribution aims to highlight the significance of SHM-based model update for the reduction of uncertainties in seismic risk assessment of individual structures that are representative of common building typologies in places of moderate seismicity.

KEY WORDS: Seismic Assessment; Dynamic Testing of Real Structures; Measurements during Demolition; Output-only Modal Identification; Amplitude-dependent Stiffness; Existing Masonry Buildings,

1 INTRODUCTION

The destructive impact that earthquakes have on structures has been triggering research and advances in multiple engineering domains throughout centuries. While new structures can arguably be considered earthquake-safe, existing buildings often fail to comply with modern seismic code prescriptions. Unknown material properties, boundary conditions and geometrical traits undermine accuracy of engineering models, leading to unrealistic assumptions and, ultimately, to unnecessary and/or inappropriate interventions or possible negligence of required retrofitting. Improving knowledge of the behavior of existing structures may extend their lifespan without compromising the resilience of communities with respect to natural disasters.

Structural health monitoring (SHM) provides the tools to expand our knowledge concerning seismic performance of existing buildings, as a supplement to laboratory-based experimentation and possibly alleviating destructive testing. The use of SHM for dynamic characterization and monitoring of buildings typically exploits vibration measurements, often in ambient conditions, to infer the modal characteristics either as a proxy of structural health [1] or for inverse parameter updating of computational models [2].

Shaking events of real structures are scarce: a limited number of forced excitation tests for buildings can be found in existing literature, including the work of Steiger et al. and Soyoz et al. ([3], [4]), who employed an eccentric mass shaker to mobilize a reinforced concrete building prior to and after retrofitting, with the aim of assessing the efficacy of the retrofitting solution. Various complications undermine such controlled shaking experiments for testing of real structures, with perhaps a primary obstacle lying in the need to suspend the function of the tested infrastructure, or building. As a result, output-only modal identification algorithms ([5], [6]) for structures that are subjected to ambient excitation present the most common approach for dynamic testing. However, methodologies that rely exclusively on ambient vibrations are inevitably limited to the structural response in the commonly assumed linear elastic range, which may not be representative of structural behavior under high-amplitude loads [7]. This limitation becomes even more prominent for masonry structures, where non-linear behavior is present even for response amplitudes that one order of magnitude lower than the theoretical yielding point [8].

Furthermore, the effect of the soil-foundation-structure interaction, which is proven to substantially affect the dynamic response of low-rise buildings [9], cannot be captured by considering only ambient excitation. To this end, Song et al. [10] applied a hierarchical Bayesian model updating approach to demonstrate the influence of the response amplitude onto the estimated structural parameters of a two-story concrete building. Ceravolo et al. [11] attempted a rigorous derivation of the amount of nonlinearity that can be attributed to the soil. Yet a model-updating approach for both structure and soil, considering uncertainties, has not been accomplished. In this paper, excitations from planned demolition activities are used to derive the influence of response amplitude on the dynamic properties of masonry buildings. Particularly for countries with low-to-moderate seismic hazard, such measurements could offer valuable information for seismic assessment and health monitoring of characteristic building typologies, as recordings of building response to past earthquakes is scarce.

This paper starts with a description of the building and the measurement configuration, as well as the demolition process, which allowed to measure vibrations that exceed ambient levels. Section 3 delivers a description of the system identification framework based on ambient vibrations as well as on response due to higher levels of excitation. Finally, the parameter values of an equivalent-frame model are updated using a Bayesian approach and the influence of the variability in vibration amplitudes on the model-parameter identification and finally on the seismic performance are assessed.

2 CASE STUDY AND INSTRUMENTATION

The present case study is a two-storey masonry building, constructed in Switzerland in the beginning of the 20th century. The outer dimensions are 10.2 m in length and 8.0 m in width. The façade of the building at the beginning of the demolition and the geometry of a typical floor are provided in Figures 1 and 2 respectively. The walls consist of clay masonry, while the floors are formed by timber beams, aligned parallel to the short direction of the building. The building is regular in plane and in elevation and almost symmetric in both directions. The geometrical and material characteristics are representative for central European buildings of this age and compose a substantial part of the residential building stock in Switzerland and central Europe ([12], [13]).

The studied building was instrumented prior to demolition with ten triaxial accelerometers (ADXL 354) placed in similar positions at levels 1 and 2, as illustrated in Figures 1 and 2. The data acquisition was conducted by means of a National Instruments cDAQ-9188 at a sampling rate of 1720 Hz.

The demolition of masonry buildings is performed progressively from top to bottom with the shovel of an excavator. Non-structural elements, including furniture, opening frames, glass windows, floor coverings etc. are removed beforehand. During demolition, buildings are subjected to hits and pulls of arbitrary direction and intensity, applied at random positions of the structure, resulting in a rich variety of impulse responses. In order to classify the impulse responses in terms of amplitude, the Root Mean Square (RMS) acceleration of the impulse responses was classified into four classes with upper limits: 1, 2, 4 and 40 mg respectively. As an example, the highest intensity bin (#4) comprises impulse responses with amplitude: 4 mg < α_{rms} < 40 mg. Figure 3 demonstrates a characteristic response of bin #4, captured at different senor positions and directions. The recorded signals indicate higher response amplitude in the direction of the hit (Z-axis). As expected, the amplitude is stronger in the vicinity of the sensor placed closer to the impact location. In the longitudinal direction (Y-axis), the time-delay of the impact between sensor 9 and sensor 10 is evident. Additionally, sensor 10, for which the Y-component of the acceleration is oriented along the out-of-wall plane, exhibits higher acceleration in this direction than the sensor closest to the impact, indicating the possible absence of diaphragmatic behavior of the floor slabs for higher amplitudes of excitation.

3 DATA-DRIVEN SYSTEM IDENTIFICATION

Initially, a baseline identification is conducted, considering the ambient recordings prior to the beginning of demolition activities. The signals recorded during the demolition are segmented into separate impulse responses that are further analyzed in time domain with the Eigensystem Realization Algorithm (ERA) ([14], [15]), which provides identification of modal properties (frequency, mode shape and damping) for each impulse response.

Figure 1. Studied building at the beginning of demolition.

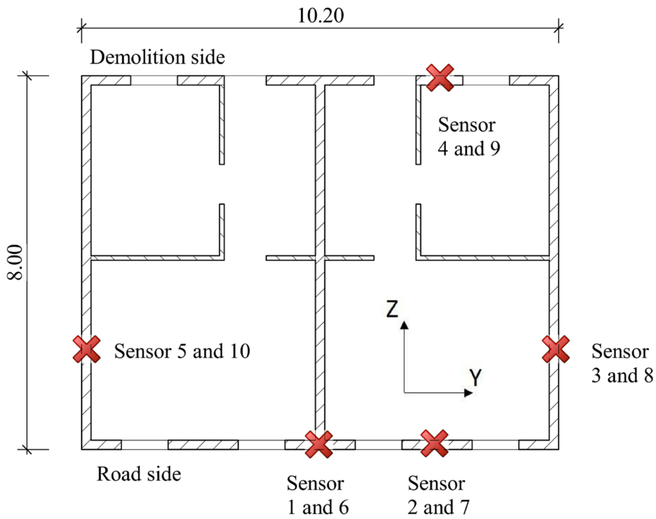

Figure 2. Dimensions of typical floor and sensor positions (1st and 2nd level respectively).

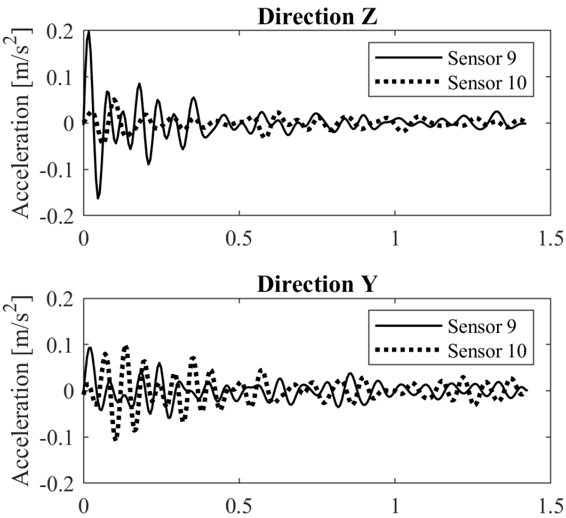

Figure 3. Impulse responses at different sensor positions (amplitude bin #4).

3.1 Baseline identification

The baseline identification of the modal characteristics is conducted by implementing the Stochastic Subspace Identification algorithm [6] on the ambient recordings prior to demolition. The data are subjected to a standard pre-processing, consisting of bandpass filtering between 1 and 40 Hz, exclusion of linear trends and down-sampling from 1720 to 172 Hz. The identified modal characteristics of the first four stable modes are summarized in Table 1. The characteristic frequencies of the structure are found to lie between 6 and 20 Hz. The first two modal shapes correspond to the main translational degrees of freedom of the building (first mode in the longer direction, Y, and second mode in the shorter direction, Z). The third modal shape seems associated to a torsional mode and the fourth modal shape reflects a combination of rotational and translational degrees of freedom. It is mentioned that the building is softer in the longitudinal direction (axis Y), which is attributed to the large openings of the corresponding facade. This baseline identification serves as reference for the subsequent time-domain analysis at the beginning of the deconstruction process, before any visible structural damage to the first two floors occurred and before any inelastic changes to the modal properties have been noted.

Table 1. Baseline Identification results

Mode #	Frequency [Hz]	Modal shape
1	6.4	Translation Y-axis
2	7.3	Translation Z-axis
3	9.3	Rotational
4	17.6	Combined

3.2 Operational Modal Analysis during demolition

During demolition, the response amplitude exceeds typical levels of ambient vibrations. Although it is impossible to quantify the input directly, response impulses of various amplitudes are utilized to identify the modal characteristics at different amplitude levels. The range of the response amplitude in terms of RMS acceleration during each impulse goes up to 0.04 g, which is over 2 orders of magnitude higher than ambient vibration levels. The signal measured by sensor 9, placed at level 2 on the demolition side, is illustrated in Figure 4 under hits of varying excitation level. Although the calculation of displacements based on numerical integration of acceleration recordings is not precise, it can be used to provide an estimate for the amplitude reached by the total displacements. As the maximum computed displacement is lower than 0.05 mm and the equivalent linear range, according to subsequent analysis (Section 4.3), covers the displacement range up to 1 mm, it can be assumed that the structure responds in the elastic range during demolition and no damage due to excessive lateral loading is expected.

In order to highlight the systematic influence of vibration amplitude on the global dynamic response, a statistical approach is performed. As the hits can be assimilated to impulse-like responses, the ERA method is used to derive the natural frequencies, the equivalent viscous damping and the modal displacements for each detected hit. Due to the short duration of impulses, their arbitrary input (location, direction and amplitude) and the inherent measurement noise, the operational modal analysis procedure is only able to identify the excited modes during the studied impulse. In order to ensure consistency of the identified modal properties and to limit identification to the global modal shapes, criteria that compare the baseline identification with the ERA results, as well as the goodness of fit in time domain between ERA predictions and real response are deployed. These criteria aim to balance two competing goals: exclude erroneous identification results, which would artificially increase the uncertainty in identified modal properties and yet include with high probability the changes in modal properties that originate from increasing amplitudes of excitation. All detected hits during the first 90 minutes of the deconstruction process (before the demolition activities reached structural parts of the two main floors and before any residual change in modal properties has been observed) are clustered into groups of similar response amplitudes. The first three modes are detected in the majority of the impulses, while the 4th mode is identified in nearly 40% of the hits.

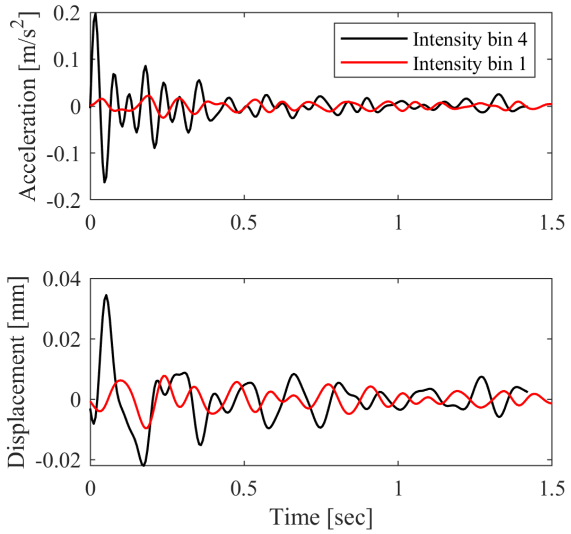

Figure 4. Impulse responses for different amplitude levels (sensor #9).

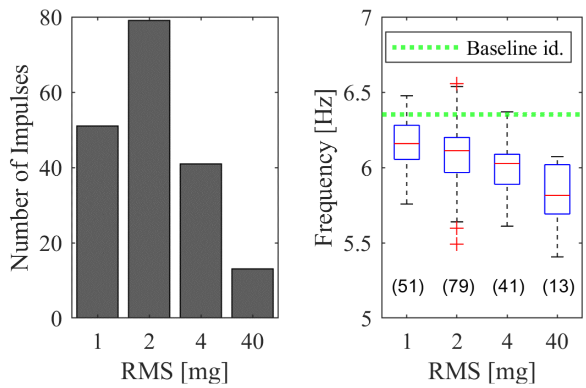

Figure 5. Left: Amount of impulses extracted from the response, classified to amplitude related bins. The acceleration notation indicates the upper bound of each intensity bin. Right: Descriptive statistics of the identified frequency of the 1st mode for different amplitude levels.

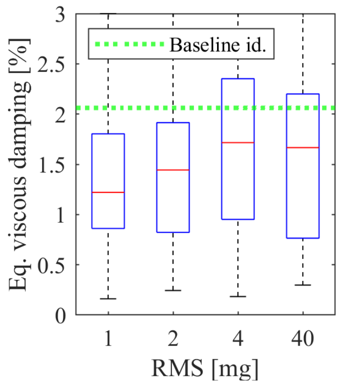

Figure 6. Descriptive statistics of the identified eq. viscous damping of the 1st mode for different amplitude levels.

The baseline identification tends to overestimate the frequency, as it lays above the 75th percentile for all amplitude bins. This shows that even for small-amplitude hits, the global dynamic response of the building demonstrates a softening behavior. Furthermore, it can be observed that the identified frequency shows a clear decreasing trend for increasing RMS amplitude of the analyzed impulses. The reduction of median frequency between the first and the fourth amplitude bin is 5%. Given the approximate value of inter-storey drifts of 0.5-1 10^{-5} m, these values of reduction in stiffness are in line with previous studies [8]. It is mentioned that similar trends are observed for the higher modes, with almost constant variability (coefficient of variation approximately 5%), with an increase in variability for the third (rotational) mode, which could possibly be linked to the uni-directional nature of hits. The observed amplitude-dependent softening of the system is not attributed to permanent structural damage, as the frequency drops appear to be transient. Increasing response amplitude is accompanied by an increase in the identified equivalent viscous damping, as shown in Figure 6. As expected, damping estimates are characterized by significant uncertainty and the values derived from ambient vibrations give unrealistic estimates. The damping estimates for the first three modes are similar, with median values around 1.5 % for low-amplitude hits and roughly 2 % for hits with higher amplitudes. This increase in damping may be explained by energy absorption from non-linear behavior due to opening and closure of micro-cracks or due to hysteretic behavior of soil, which is shown to exist even for very low strains, in the theoretical linear elastic regime [9]. Nevertheless, further studies would be necessary to explore energy dissipation at relatively low vibration amplitudes, but they fall outside the scope of this paper.

4 STRUCTURAL MODEL AND DATA-DRIVEN PARAMETER UPDATING

4.1 *Model description*

The studied structure is modelled using the commercial structural analysis software SAP2000, (version 22) as a three-dimensional equivalent frame model. The structural walls are discretized into piers and spandrels with cross sections in accordance with the real dimensions of the structure. The concrete walls in the basement are considered elastic above and rigid beneath the ground level horizon. The foundation impedance is simulated with three translational and three rotational springs, placed at the geometrical center of the foundation, according to the analytical formulations proposed in [16]. To account for the nonlinear behavior of masonry, plastic hinges have been adopted at the edges and the middle of all spandrels and piers, following the assumptions originally described in [17].

Uncertain parameters pertaining to material properties and boundary conditions are considered random variables with uniform distributions in predefined ranges that are chosen according to Swiss building codes [18]–[20] and engineering judgment. The uncertain parameters and the prior ranges are summarized in Table 2. For the walls, apart from the modulus of elasticity, the length of the overlapping parts between spandrels and piers, which defines rigid regions, is considered uncertain. The timber beams of the slabs are aligned parallel to the short direction of the building, causing an orthotropic behavior. In order to account for the stiffness and mass distribution of timber floors, equivalent orthotropic shell elements are adopted. The ranges for the elastic properties are calculated in accordance to the timber floor dimensions (square beams with 25 x 25 cm section at a distance of 70 cm). The equivalent elastic properties of the soil are considered unknown and thus, wide prior parameter ranges are defined according to [21]. Finally, the effective foundation embedment, accounting for loose contact with soil, according to [16], is bounded between 30 % and 100% of the total embedment.

Table 2. Prior ranges and point estimates of the uncertain parameters for different response amplitude bins.

	Prior range	Amp. Bin #1	Amp. Bin #4
E_{walls} [GPa]	0.6 – 3	1.64	**1.34**
$r_{spandr/beams}$ [%]	30 – 100	95	97
$E_{slabs\,//y}$ [GPa]	1 – 5	1.6	1.2
$E_{slabs\,//z}$ [GPa]	1 – 2	1.0	1.1
G_{soil} [MPa]	50 – 100	67.8	67.7
$r_{foundation}$ [%]	30 – 100	30	30

4.2 *Bayesian Model updating*

In order to determine the model parameters that best fit the identified modal properties, a Bayesian inference framework [22] is implemented through the UQLab toolbox [23]. In this framework, uncertain model parameters are considered as random variables with uniform prior distributions (per Table 2). These prior distributions are updated upon availability of measurement data, so that the posterior distributions are informed with the modal identification data. For the Bayesian model updating, the modal displacements and frequencies of the first two modes are considered. The posterior distributions are computed via Markov-Chain Monte-Carlo sampling [24], by considering 20 Markov chains of 5’000 samples. The computational model provides modal response predictions for a given set of input parameters. To account for measurement errors and model inaccuracy, a discrepancy term is considered, as a Gaussian random variable with zero mean and diagonal covariance matrix with constant variance equal to 0.0025. This assumption does not compromise the purposes of this analysis, which focuses on the comparison of the inferred model parameters for different response amplitude levels. The Bayesian Inverse update is conducted for the amplitude-related bins defined in Figure 5 (left). The posterior distributions for all uncertain parameters result narrow, centered on the maximum a posteriori (MAP) values, which are further considered as point estimates for the updated models. The point estimates for the amplitude bins 1 and 4 are summarized in Table 2.

The identified modal frequencies tend to drop with increasing response amplitudes, indicating nonlinear behavior in the equivalent elastic range of the response, before any damage occurs. To this end, the results from Bayesian model updating for increasing response amplitudes provide valuable insights into the properties that cause these shifts. In this case study, the governing parameter for the system softening due to increasing response amplitudes is the elastic modulus of masonry. The comparison of the inferred values of elastic modulus for increasing amplitude exposes a stiffness reduction of 18 % between amplitude bins 1 and 4. The corresponding shift of the predicted frequencies is significantly lower (below 5 %) and less evident. Hence, Bayesian model updating, based on identified modal properties, provides robust estimates of the equivalent elastic properties that are sensitive to changes in stiffness, due to increasing response amplitude.

The goodness of fit between the baseline identification and the modal predictions of the updated models for amplitude bins #1 and #4 are summarized in Table 3. While the frequencies for the amplitude bin #1 overestimate the frequencies compared to the baseline identification, the predictions for amplitude bin #4 are lower. The Modal Assurance Criterion (MAC) comparison between the baseline identification and the amplitude-dependent models is suboptimal, which can be attributed to the effect of response amplitude on the modal shapes.

Table 3. Baseline identification results and model predictions for amplitude bins #1 and #4. The MAC compares the baseline identification with the updated models.

	Baseline identification	Amp. Bin #1	Amp. Bin #4
F_1 [Hz]	6.35	6.52	6.37
F_2 [Hz]	7.24	7.40	7.04
MAC_1 [%]	1	0.70	0.71
MAC_2 [%]	1	0.80	0.81

4.3 *Impact of parameter updating on seismic assessment*

Understanding the influence of the response amplitude on the properties of masonry buildings may enable a more refined prediction of the seismic displacement demand and capacity. Since regional risk assessment models broadly rely on bi-linearized capacity curves, for which assumptions regarding reduced stiffness are taken, monitoring-driven knowledge of amplitude-dependent stiffness can provide valuable insights.

The pushover curves, obtained via imposed displacements that are proportional to the translational modal shapes, are reported in Figure 7. As it can be observed, the elastic parameters, namely the elastic modulus of masonry, bear an important influence on the overall nonlinear capacity curve. In order to demonstrate the effect of the uncertainty in the elastic stiffness on the calculated seismic performance, the lower and upper bound for the elastic modulus, according to relevant literature and existing building standards [20], have been considered. The estimated shear capacity varies between 136 kN, with practically no ductility, and 280 kN, with ductility over 3. These extreme cases expose the uncertainty pertaining to seismic assessment without prior information on the equivalent elastic stiffness. To this end, model updating is necessary in order to obtain robust estimates of the stiffness and reduce the uncertainty of the expected seismic performance.
Comparing the predictions resulting from model updating for amplitude bins 1 and 4, the E modulus drops by 18 % (Table 2). This significant reduction also impacts the predicted global seismic performance. According to Swiss standards [20], the ultimate displacement of the capacity curve is defined as failure of the first load-bearing element. While the ultimate displacement and the maximum shear strength do not vary much, the reduction in stiffness increases the yield displacement and thus, reduces the post-yield displacement capacity. In addition, the displacement demand depends directly on the elastic branch of the equivalent bilinear capacity curve. Therefore, lower stiffness values (for an overall stiff building typology, such as the low-rise shear building under study) translate to higher displacement demands.

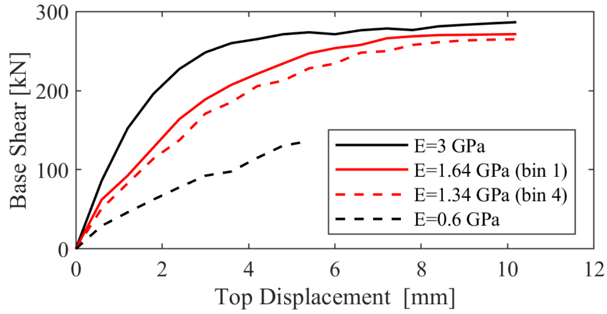

Figure 7. Pushover curves for varying E-modulus of masonry.

It is noted that both nominal and monitoring extracted values of the E-modulus are further reduced by 50% for the pushover analysis, to account for cracked material. The value of 50% can be considered to be arbitrary and the extent to which it covers stiffness changes under low amplitudes is debatable. Stronger hits and measurements under earthquake loads would be required to quantify the stiffness drop at the equivalent yield point. Local collapse mechanisms (such as out-of-plane failure of walls) are not examined, as they do not belong to the scope of this analysis.

Overall, while the pushover analysis is highly simplified, and application to additional buildings is required to formalize the findings, it is concluded that systematic measurement of buildings that are being demolished can provide a better understanding of the dynamic properties, and by extension of the seismic safety, of existing structures that are representative of building typologies that cover significant part of the existing building inventory.

5 CONCLUSIONS

This paper studies the influence of response amplitude on the dynamic properties of a masonry structure and consequently on its expected seismic performance. Based on the presented monitoring campaign on a real masonry building, equipped with sensors during planned demolition, the following conclusions are drawn: Transient nonlinear behavior is observed in what is commonly assumed as the linear elastic regime. Low-cost sensors prove adequate for this task, which carries the potential for a systematic analysis of this behavior, particularly in countries with low-to-moderate seismicity, where little data from earthquakes is available. Increasing levels of shaking are linked to a reduction in stiffness of the masonry, and not to transient changes in the foundation properties. The stiffness reduction at low excitation levels, which is modelled to be linear in classical engineering assessment approaches, may substantially affect the global seismic performance of the structure.

With the feasibility of such approaches established, future work is planned in order to systematically study the phenomenon on multiple buildings. Such an analysis reveals the influence of building types on the sensitivity of model parameters with respect to the response amplitude, thus providing a valuable starting point for many applications in structural health monitoring.

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REFERENCES

[1] V. Gattulli, E. Antonacci, and F. Vestroni, "Field observations and failure analysis of the Basilica S. Maria di Collemaggio after the 2009 L'Aquila earthquake," *Eng. Fail. Anal.*, vol. 34, pp. 715–734, 2013.

[2] Y. Reuland, P. Lestuzzi, and I. F. C. Smith, "Measurement-based support for post-earthquake assessment of buildings," *Struct. Infrastruct. Eng.*, vol. 15, no. 5, pp. 647–662, 2019.

[3] R. Steiger, G. Feltrin, F. Weber, S. Nerbano, and M. Motavalli, "Experimental modal analysis of a multi-storey light-frame timber building," *Bull. Earthq. Eng.*, vol. 15, no. 8, pp. 3265–3291, Aug. 2017.

[4] S. Soyoz *et al.*, "Ambient and Forced Vibration Testing of a Reinforced Concrete Building before and after Its Seismic Retrofitting," *J. Struct. Eng.*, vol. 139, no. 10, pp. 1741–1752, Oct. 2013.

[5] R. Brincker, L. Zhang, and P. Andersen, "Modal identification of output-only systems using frequency domain decomposition," *Smart Mater. Struct.*, vol. 10, no. 3, pp. 441–445, 2001.

[6] P. Van Overschee and B. De Moor, "Continuous-time frequency domain subspace system identification," *Signal Processing*, vol. 52, no. 2, pp. 179–194, Jul. 1996.

[7] A. Astorga, P. Guéguen, and T. Kashima, "Nonlinear Elasticity Observed in Buildings during a Long Sequence of Earthquakes," *Bull. Seismol. Soc. Am.*, vol. 108, no. 3A, pp. 1185–1198, Jun. 2018.

[8] C. Michel, B. Zapico, P. Lestuzzi, F. J. Molina, and F. Weber, "Quantification of fundamental frequency drop for unreinforced masonry buildings from dynamic tests," *Earthq. Eng. Struct. Dyn.*, vol. 40, no. 11, pp. 1283–1296, Sep. 2011.

[9] P. Martakis, D. Taeseri, E. Chatzi, and J. Laue, "A centrifuge-based experimental verification of Soil-Structure Interaction effects," *Soil Dyn. Earthq. Eng.*, vol. 103, no. April, pp. 1–14, 2017.

[10] M. Song, B. Moaveni, C. Papadimitriou, and A. Stavridis, "Accounting for amplitude of excitation in model updating through a hierarchical Bayesian approach: Application to a two-story reinforced concrete building," *Mech. Syst. Signal Process.*, vol. 123, pp. 68–83, 2019.

[11] R. Ceravolo, E. Matta, A. Quattrone, and L. Zanotti Fragonara, "Amplitude dependence of equivalent modal parameters in monitored buildings during earthquake swarms," *Earthq. Eng. Struct. Dyn.*, vol. 46, no. 14, pp. 2399–2417, Nov. 2017.

[12] H. Crowley *et al.*, "Exposure model for European seismic risk assessment," *Earthq. Spectra*, vol. 36, no. 1_suppl, pp. 252–273, Oct. 2020.

[13] L. Diana, J. Thiriot, Y. Reuland, and P. Lestuzzi, "Application of association rules to determine building typological classes for seismic damage predictions at regional scale: The case study of basel," *Front. Built Environ.*, vol. 5, no. April, pp. 1–17, 2019.

[14] L. D. Peterson, "Efficient computation of the eigensystem realization algorithm," *J. Guid. Control. Dyn.*, vol. 18, no. 3, pp. 395–403, May 1995.

[15] J.-N. Juang and R. S. Pappa, "An eigensystem realization algorithm for modal parameter identification and model reduction," *J. Guid. Control. Dyn.*, vol. 8, no. 5, pp. 620–627, Sep. 1985.

[16] G. Gazetas, "Formulas and Charts for Impedances of Surface and Embedded Foundations," *J. Geotech. Eng.*, vol. 117, no. 9, 1991.

[17] S. Lagomarsino, A. Penna, A. Galasco, and S. Cattari, "TREMURI program: An equivalent frame model for the nonlinear seismic analysis of masonry buildings," *Eng. Struct.*, vol. 56, pp. 1787–1799, 2013.

[18] Society of Engineers and Architects (SIA), *SIA 261: Einwirkungen auf Tragwerke*. Zurich, Switzerland, 2014.

[19] Swiss Society of Engineers and Architects, *SIA 266: Mauerwerk*. Zurich, Switzerland, 2015.

[20] Swiss Society of Engineers and Architects, *SIA 269: Erhaltung von Tragwerken*. Zurich, Switzerland, 2017.

[21] J. Studer, J. Laue, and M. Koller, *Bodendynamik*. 2007.

[22] J. L. Beck and L. S. Katafygiotis, "Updating Models and Their Uncertainties. I: Bayesian Statistical Framework," *J. Eng. Mech.*, vol. 124, no. 4, pp. 455–461, Apr. 1998.

[23] S. Marelli and B. Sudret, "UQLab: A Framework for Uncertainty Quantification in Matlab," in *Vulnerability, Uncertainty, and Risk*, 2014, pp. 2554–2563.

[24] P. Hewson, "Bayesian Data Analysis 3rd edn A.Gelman, J. B.Carlin, H. S.Stern, D. B.Dunson, A.Vehtari and D. B.Rubin, 2013 Boca Raton, Chapman and Hall-CRC 676 pp., £44.99 ISBN 1-439-84095-4," *J. R. Stat. Soc. Ser. A (Statistics Soc.*, vol. 178, no. 1, pp. 301–301, Jan. 2015.