Seismic retrofit of cultural heritage buildings – when less is more

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ABSTRACT: In 1356, Basel was subjected to an earthquake with an estimated magnitude of 6.6; it is the largest known historical earthquake in central northern Europe. During this earthquake and the fire that followed large parts of Basel were destroyed. Such an event can occur again at any time in the future. To be prepared, it is essential to evaluate the seismic resistance of the existing building stock. The character of the city centre of Basel results largely from its naturally grown fabric of stone masonry buildings from different epochs. Stone masonry buildings belong to the most vulnerable structures under seismic excitation. At the same time, they are part of the Swiss cultural heritage and not only their appearance but also the structure as such should be maintained.

This paper reports on a collaborative project between EPFL and the University of Pavia, which aims on the one hand at developing realistic models for the seismic response of stone masonry buildings and on the other hand at developing minimum intervention strategies that are reversible and as little intrusive as possible and therefore suitable for cultural heritage buildings. This paper describes first findings from the different experimental campaigns that are part of the project and explores how stone masonry typologies can be analysed using image analysis procedures.

1 INTRODUCTION

Earthquakes present not only in countries of high seismicity, such as New Zealand, Japan or California, a significant natural hazard, but also in countries with low or moderate seismicity like Switzerland the risk resulting from earthquakes can be significant. The Swiss Federal Office for the Environment estimates that in Switzerland earthquakes account for about half of the risk caused by natural hazards (Federal Office for Civil Protection 2008). Within Switzerland, the city of Basel is one of the most vulnerable regions (SIA 2014a) whose earthquake history is characterized in particular by the tremors in 1356. During these earthquakes, of which the largest one reached a magnitude of approximately 6.6 (Fäh et al. 2009), and the subsequent fire Basel was largely destroyed.

An earthquake of such intensity can occur again at any point in the future. To be prepared for such an event, new buildings are designed for earthquake loading (SIA 2014a) and the expected earthquake performance of existing buildings is assessed (SIA 2014b). While the design of new buildings can often be done with very simple, conservative models, it is far more difficult to predict the expected earthquake behavior of an existing building that was not designed for earthquakes. This is also reflected in the design and assessment procedures. For the seismic design of new buildings, linear models are used and the buildings are designed using a so-called force-based methods, which are approaches that use load models similar to other types of load cases.

Such linear models are naturally relatively strong approximations of the actual seismic response of a building when it is subjected to a ground motion for which it reaches the limit states Significant Damage or Near Collapse, which are the reference limit states of most assessment codes. In order to compensate for the discrepancy between actual and predicted behaviour, these linear models have been calibrated to lead to conservative results. Such an approach is acceptable for new buildings, where conservatism in the design approach causes typically only marginal extra costs. When applied to existing buildings, the conservative approach can cause, however, large additional retrofit or replacement costs (Figure 1). Applying more realistic, nonlinear models allows the identification of the reserve capacity, and buildings that are truly in need of retrofit or replacement can be separated from those buildings that do not satisfy code requirements when classical linear analysis approaches are determined.

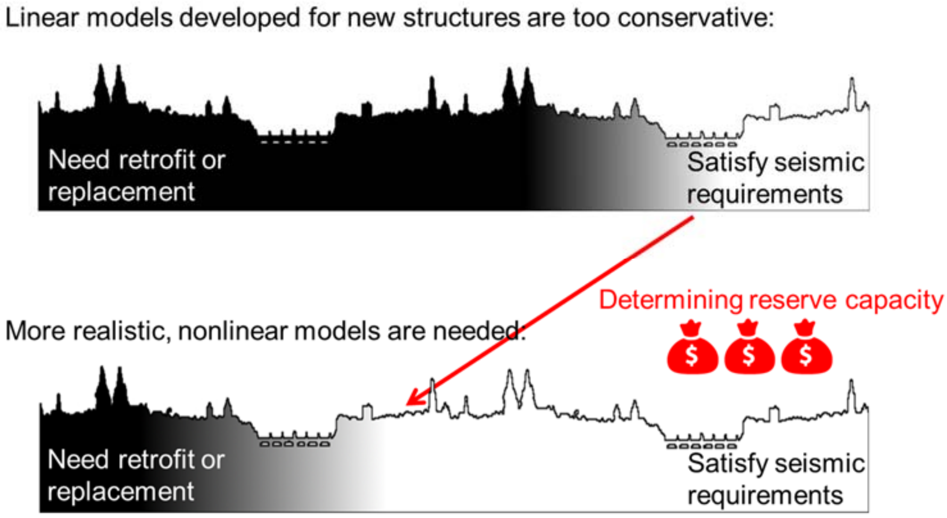

Figure 1. Benefit of determining the reserve capacity by means of more realistic, nonlinear models

It is argued here that the impact of advanced nonlinear models is particular important when assessing masonry buildings in regions of moderate seismicity, such as the seismicity of the city in Basel, which is the reference site of the described project. Masonry buildings are among the most vulnerable buildings under earthquake loading (Grünthal 1993). When today's simple methods are applied to masonry buildings in low seismicity regions, they allow to demonstrate—despite their conservatism—that most masonry buildings satisfy the design code. In regions of higher seismicity, even sophisticated methods would not allow to demonstrate that masonry buildings are sufficiently earthquake resistant. In regions of moderate seismicity, however, the method of analysis makes often the decisive difference: customary, simplistic methods often show insufficient earthquake resistance and, as a consequence, lead to unnecessary reinforcement measures, which often interfere very strongly with the appearance or even lead to the demolition of the building. Using more refined methods of analysis, such as nonlinear simulations, typically allows to reduce the strengthening measures to a minimum or even to show that no strengthening measures are necessary.

Figure 2. Historical masonry buildings in Basel: Building aggregate from the late Middle Ages (left); a representative building from the early modern period in the baroque style (centre); a school building from the industrialization (right).

The cityscape of the center of Basel is determined by masonry buildings from different epochs. Many buildings that shape the fabric of the inner centre were constructed after the earthquake in 1356 and stem from the 15th and the 16th century (Figure 2). In this type of construction, buildings form aggregates and neighbouring buildings share fire walls. Further typologies of masonry buildings are representative buildings in the baroque style and buildings from the period of the industrialization (Figure 2). All of these buildings are typically stone masonry buildings with timber slabs (in the period of the industrialization also other slab types exist, too).

The challenges of the seismic assessment of stone masonry buildings are well known and partially related to their flexible ceilings (wooden ceilings), which largely prevent the transfer of forces between walls and favour local out-of-plane failure mechanisms (D'Ayala 2005; Penna 2015b), the largely unknown deformation capacity of stone masonry walls under in-plane loading (Vanin et al. 2017), and the interaction between adjacent buildings in building aggregates (Formisano et al. 2010). Furthermore, many of the natural stone masonry buildings in Basel are of cultural-historical interest and are protected as a monument. When planning the cultural value of these buildings must therefore be considered and possible interventions should be reduced to a minimum and be reversible in order to preserve the fabric and the appearance of these buildings (Lagomarsino and Cattari 2015). To address the research needs a collaborative project between the Earthquake Engineering and Structural Dynamics (EESD) laboratory at EPFL, Switzerland, and the Masonry Research Group of the University of Pavia, Italy, were set up. The key objectives of this research program are the following:

- Developing pushover analysis methods for analyzing buildings with flexible floors by means of the substructuring method;
- Investigating the influence of adjacent buildings in aggregates by means of a shake table test that tests for the first time a conglomerate of two buildings;
- Investigating the static and kinematic boundary conditions that result from the flexible slabs for out-of-plane loaded walls;
- Improving existing and developing new models for predicting the stiffness, strength and deformation capacity of in-plane loaded walls and spandrels.
- Developing strengthening interventions that are ideally reversible and have a minimum impact of the fabric and appearance of the structures and are therefore suitable for buildings that are part of the cultural heritage;
- Investigating the damage potential of plaster and stucco of such cultural heritage buildings under low intensity natural and induced records.

In Switzerland, a further challenge when assessing the seismic performance of stone masonry buildings relates to the fact that this topic does not feature in the civil engineering curriculum. Therefore, engineers often lack the tools to capture the expected earthquake behavior of an existing building. As a result, conservative assessments of the earthquake behavior are often carried out and too extensive reinforcement concepts and interventions in the building fabric are proposed. To guarantee a direct transfer of the research findings to engineering practice, continuing education courses are offered in parallel.

This paper outlines the research objectives of this project, gives an overview on the ongoing experimental campaigns and introduces new ideas with regard to the drift capacity stone masonry walls.

2 OBJECTIVES OF THE RESEARCH PROJECT

The research project aims at addressing several open questions with regard to the seismic assessment of stone masonry buildings. For this purpose, numerical and experimental methods will be combined. A central part of the research project is a large-scale shake table test on two stone masonry buildings that are representative of the buildings from the Medieval Ages in the city centre of Basel. The buildings have been constructed and the shake table test will be conducted in the next month. The shake table test has been designed under lead of the research group from Pavia. The buildings are constructed at half scale and the scaling law applied required reduced material strength values, which were obtained by adding special admixtures including polystyrene balls to the mortar (Senaldi et al. 2017). The buildings will be subjected to uni-directional excitation orthogonal to the gable walls. The excitation records will be representative for natural and induced seismicity in Switzerland, spanning limit states from no damage to significant damage.

Figure 3. Photos of the two buildings to be tested on the shake table at the Eucentre, Pavia, Italy. View of the entire structure and one structural detail showing a spandrel (Photos: I. Senaldi).

2.1 Global response of buildings with flexible floors and buildings that are part of aggregates

In engineering practice and research, masonry buildings are commonly analysed by means of an equivalent frame analysis (Magenes 2000; Lagomarsino et al. 2013; Penna 2015a). In buildings with flexible floors, the force redistribution between walls is limited. To capture this effect in dynamic analysis, the timber slab is typically modelled as elastic orthotropic diaphragm (Lagomarsino et al. 2013). For engineering practice, it is however common to perform nonlinear static rather than nonlinear dynamic analysis (Fajfar 2000). When computing pushover curves, the model needs to be sub-structured into near-planar façade elements (Penna 2015b) as classical pushover methods with a single control-node will fail for structures with limited slab stiffness. The objective of this project is to develop a tool that allows the automatic computation of pushover curves of buildings with flexible floor diaphragms.

A further challenge lies in the interaction of adjacent buildings that are part of building aggregates. In historical centres, such as the centre in Basel, adjacent building units were constructed sequentially and densification of previous fabric or the growth of a building conglomerate leads to aggregates, in which adjacent buildings often share structural walls (Figure 2). The joints between buildings are therefore dry joints, which were typically roughened before the new wall was connected to the existing wall. The numerical modelling of such building aggregates is challenging (Formisano et al. 2010) and relies on simplifying assumptions with regard to the force-transfer across the joint. Experimental testing of building aggregates is limited by the size of the required structures and the limitations, with which masonry structures can be scaled (Petry and Beyer 2014a). As a result, to our knowledge, building aggregates have not yet been tested experimentally. The two buildings tested on the shake table are connected by a dry joint and one connecting stone per storey and wall.

2.2 Behaviour of masonry elements under out-of-plane loading

Out-of-plane failure modes are promoted by the large mass of stone masonry walls, the small restraint provided by timber floors and the poor interlock between the stones (D'Ayala and Speranza 2003). Significant experimental, numerical and analytical research has been devoted to the prediction of out-of-plane failure modes by (Ferreira et al. 2015; Sorrentino et al. 2016). To assess the out-of-plane vulnerability of a building, possible kinematic mechanisms are identified. Using force-based assessment procedures the excitation triggering the out-of-plane mechanism can be identified, while more advanced displacement-based or energy-based methods can yield better estimates of the excitation that results in overturning of the wall (Sorrentino et al. 2016). The results of both approaches depend, however, strongly on the assumed static and kinematic boundary conditions (Derakhshan et al. 2013; Beyer and Lucca 2016). For stone masonry buildings, an open question relates to the interaction of the flexible timber slab with regard to the out-of-plane response of the masonry wall. Unlike reinforced concrete slabs, timber slabs span primarily in one direction. The walls parallel to the beams will be hardly influenced by the timber slabs. Walls that support the beams can be affected in two ways by the slabs: on one hand, part of the mass of the slab might contribute to the out-of-plane mechanism and therefore reduce the excitation level that the wall can with stand. On the other hand, the slab provides a support to the wall and therefore increases the capacity of the wall. The interaction of these two counter-acting effects is not yet solved. Field observations indicate that the second mechanism tends to prevail, as walls with larger vertical spans are more vulnerable to out-of-plane excitation than walls that support at midheight a timber slab (see Figure 3a).

Figure 4. Stone masonry buildings that failed due to an out-of-plane mechanism (a, Photo: A. Paparo) and an in-plane mechanisms (b, Photo: A. Penna).

The objective of this project is to investigate the effect of flexible slabs on the out-of-plane response. For this purpose, the beam orientation in the shake table test varies between the floors two and three: While the floor beams of the first and second storey are supported by the gable walls, the beams of the third storey span in the opposite direction. Next to the experimental investigation, static and kinematic boundary conditions of walls in natural stone masonry buildings are investigated numerically.

2.3 *Behaviour of masonry elements under in-plane loading*

In-plane failure modes include the failure of walls and spandrels. The in-plane response of masonry elements depends on their stiffness, strength and deformation capacity. Although there is certainly still significant scope for research on strength of masonry walls and spandrels, the existing literature is scarcest when it comes to the stiffness and the deformation capacity. The effective stiffness of masonry elements is a key input parameter both for force-based and displacement-based seismic assessment methods. Present codes propose to estimate the effective stiffness as a ratio of the gross sectional stiffness; typically the effective stiffness of masonry elements is estimated as 30% (SIA 2014b) or 50% (CEN 2004) of the gross sectional stiffness. The origin of these values is not known to the authors and it is likely that these values were derived for concrete elements or modern masonry elements, which both have been much more extensively researched than stone masonry elements.

While the failure of spandrels causes local failures and a global decrease in stiffness and strength, the failure of walls can lead to the collapse of the building (Beyer and Mangalathu 2012). The deformation capacity of the walls is therefore essential when assessing the ultimate limit state of stone masonry buildings. At present, codes provide only drift capacity models intended for modern brick masonry, which are—due to the absence of specific models for stone masonry—also used when assessing stone masonry buildings. Eurocode 8, Part 3 (EN1998-3 2005), for example, assigns the drift capacity based on the failure mode (shear vs flexure) and the shear span ratio H_0/L where H_0 is the height of zero moment and L the wall length. The drift capacity at the near collapse limit state, which corresponds to the point at which the strength has dropped by 20%, is assumed to be for walls failing in shear:

$$\delta_{SD} = 0.4\%, \tag{1}$$

and for walls failing in flexure:

$$\delta_{SD} = 0.8\% \cdot \frac{H_0}{L}. \tag{2}$$

In order to evaluate and refine existing models for stiffness and strength and to develop drift capacity models that address specifically stone masonry walls, a database on shear-compression tests on stone masonry walls was set up (Vanin et al. 2017). The database contains 123 shear-compression tests on stone masonry walls from 16 test campaigns. The paper evaluates existing formulas for the effective stiffness and strength and proposes new adaptation were required. It further proposes the first drift capacity equations for stone masonry that are derived from a large number of tests.

A key challenge when treating stone masonry is how to account for the large variety of properties of stone masonry when deriving empirical models for stiffness, strength and deformation capacity. Traditionally, stone masonry has been divided into five discrete classes (MIT 2009), which have been described by sketches. Section **Error! Reference source not found.** investigates alternative parameters for the classification of stone masonry and whether such parameters are also suitable for predicting the drift capacity of stone masonry walls.

2.4 Strengthening interventions that are suitable for cultural heritage buildings

The objective of this project is to investigate the performance of strengthening interventions that are suitable for the use in cultural heritage buildings, i.e., interventions that are ideally reversible, do not damage the fabric of the structure or compromise the appearance of this building. The shake table test will focus on interventions that improve the connections between slabs and walls. These interventions will not be activated at the beginning of the test but activated once the first limit states have been reached.

2.5 Response of plaster and stucco in low intensity earthquakes

Plaster and stucco are brittle components that might be damaged already in low intensity earthquakes. Such ground motions can be caused by natural and induced seismicity. For Switzerland, earthquakes due to natural earthquakes and induced earthquakes due to geothermal activities are of interest. In Switzerland, two pilot projects were conducted. The first project in Basel was stopped when an earthquake of magnitude 3.4 was triggered (Mignan et al. 2015), which led to damage claims in the order of 9 Million Swiss Francs (Badische Zeitung 2009). The second project in St. Gallen proved uneconomically and was stopped after first investigations. Earthquakes of such small magnitudes are typically not critical regarding the structural safety of buildings. However, as the earthquake in Basel showed, they can still lead to considerable damage of structural and non-structural components. Non-structural components are in-particular roof tiles, plaster and stucco, which can have a financial as well as a cultural value.

The Swiss government aims to make geothermal energy an important energy source for Switzerland. According to its 2050 energy vision, 4.4 Terawatt hours of electricity should be produced annually by geothermal power plants, which would require approximately 100 stations of the size of the pilot study in Basel (NZZ 2006; Verband der Geothermie 2016). Next to natural earthquakes, it is therefore expected that also induced earthquakes might lead to non-structural damage of buildings. Research on the damage to plaster and stucco on masonry components is rather scarce and has so far been limited to quasi-static monotonic and cyclic tests (Calderini et al. 2015; Didier et al. 2017). To investigate the performance of these elements under dynamic loading, part of the building surface will be covered with lime-based plaster and one room will be equipped with various stucco elements that are fixed to the walls and the ceiling.

3 STONE MASONRY CLASSIFICATION – COMMON SYSTEM AND ALTERNATIVE APPROACHES

Traditional classification systems such as the one in the Italian code (MIT 2009) distinguish discrete typologies of stone masonry by describing the shape of the units and the fabric that is created in the wall plane and through the thickness of the wall (Vanin et al. 2017):

- Class A: irregular stone masonry, with pebbles, erratic and irregular stone units;
- Class B: uncut stone masonry, with external leaves of limited thickness and infill core (three-leaf stone masonry)
- Class C: cut stone masonry with good bond
- Class D: soft stone regular masonry (built with tuff or sandstone blocks)
- Class E: Ashlar masonry, built with sufficiently resistant blocks (i.e. blocks with higher resistance than those of class D). This class was further subdivided into regular squared block masonry with mortar joints (E) and dry-joint ashlar masonry (E1).

Typical cross-sections of these masonry typologies are shown in Figure 5. Such classification systems can be readily described by text and sketches. However, they bear several disadvantages:

- For each class, appropriate values for stiffness, strength and deformation capacity need to be tabulated. To derive robust values, requires a sufficient number of tests for each class.
- Assigning a masonry typology to a class, is to a certain degree subjective. Two engineers might therefore assign one wall to different classes.
- Masonry typologies might fall in between two classes. Deriving suitable parameters for such typologies by averaging parameters might be questionable.
- With the rise of imaging techniques, the classification of masonry typologies by means of image analysis seems desirable but requires parameters that can be readily derived from such analysis.

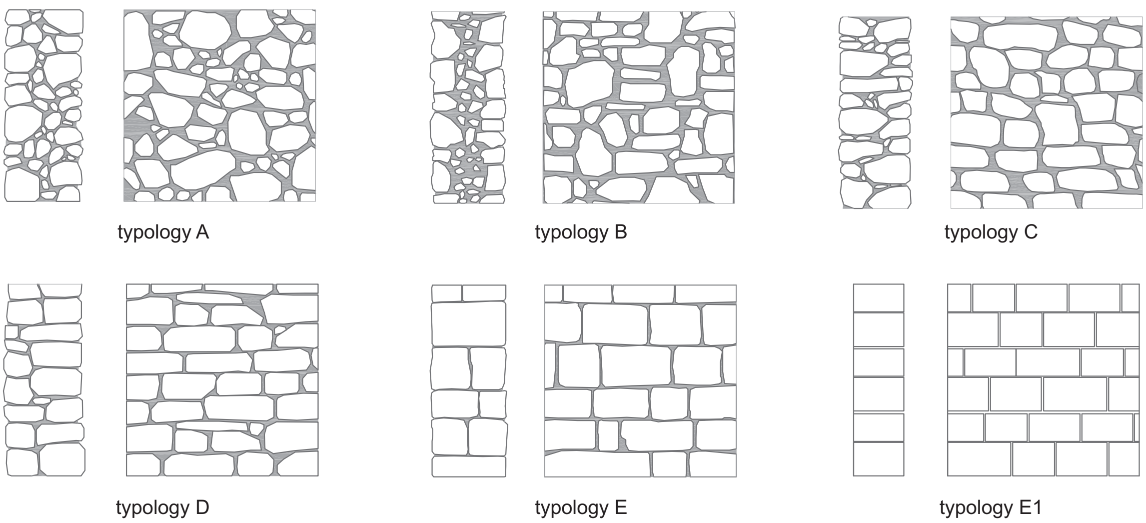

Figure 5 Stone masonry typologies: sketches of typical textures and cross sections (Vanin et al. 2017).

A first step towards a non-discrete classification system is the Masonry Quality Index (MQI), which has been developed by Borri et al. (Borri and Maria 2009; Borri et al. 2015) based on a procedure by Binda et al. for assessing the quality of the stone masonry and its compliance to the "rules of the art" (Binda et al. 2009; Cardani and Binda 2015). The procedure is based on a visual inspection and the evaluation of local geometric parameters. Borri et al. summarized these through different weighting factors in a single Masonry Quality Index, which has been correlated to the masonry strength (Borri et al. 2011). It accounts for the texture of the masonry by considering the following criteria: i) mechanical properties and conservation state of the stone units, ii) the dimensions of the stones, iii) the shape of the stones, iv) the characteristics of the wall section, including the connection of leaves, v) the horizontality of the bed-joints, vi) the staggering of the vertical joints, vii) the quality and conservation of the mortar joints. These characteristics are evaluated largely qualitatively, according to criteria specified in Borri et al. (2015).

One parameter that can be determined quantitatively is the interlock of the units, both in the in-plane and out-of-plane directions, which can be described using the concept of the length of the minimum trace (LMT), as proposed by Doglioni et al. (2009). It is defined as the minimum length of a line passing only through mortar joints, between two points that are vertically aligned and at a distance h_v, generally taken equal to 100 cm:

$$LMT = \frac{Min.\ trace\ through\ joints}{h_v} \quad (4)$$

For the test units in the data base, the length of the minimum trace has been determined for those 90 test units for which photos were available (Vanin et al. 2017). Figure 6 shows the correlation between the masonry class and the Masonry Quality Index as well as the line of minimum trace. In these plots only cyclic tests on unstrengthened walls were included that passed the quality check and for which the experimental data was available so that the drift capacity could be determined (Vanin et al. 2017). Masonry typology, Masonry Quality Index and line of minimum trace are of course by definition correlated. However, there is not a unique relationship between these three parameters and therefore their influence on the drift capacity of the walls will be investigated in the next section.

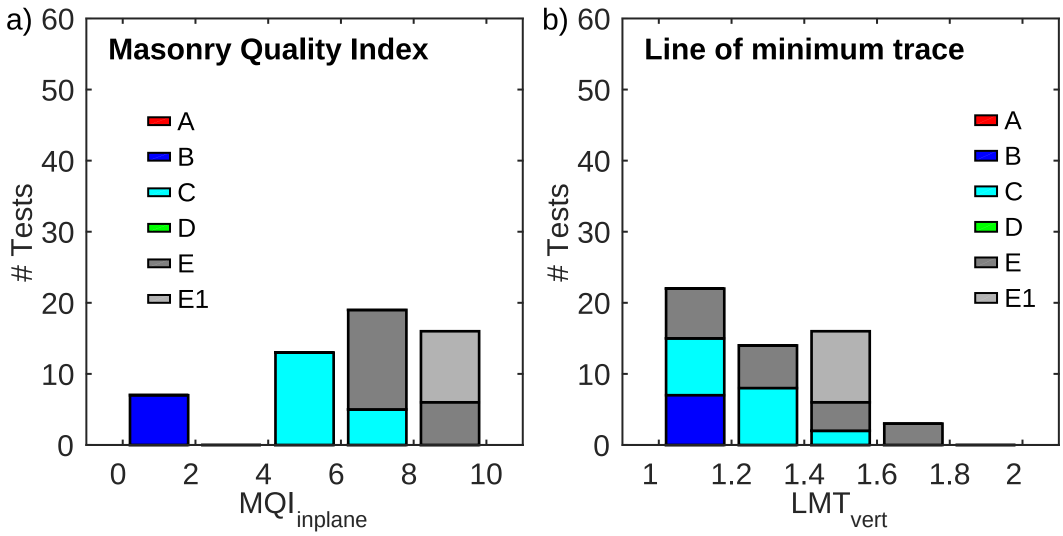

Figure 6 Correlation between the masonry typology and the Masonry Quality Index (a) and the line of minimum trace (b) (Vanin et al. 2017).

4 IN-PLANE DEFORMATION CAPACITY OF STONE MASONRY PIERS

Traditional drift capacity models (Section 2.3) assign drift capacities based on the failure mode and in the case of flexural failure also on the shear span ratio. For modern brick masonry, it has already been shown that empirical models that relate the drift capacity to the axial load ratio and the normalized shear span H_0/H or the shear span ratio H_0/L yield equally good or better predictions of the drift capacity (Pfyl-Lang et al. 2011; Petry and Beyer 2014b), where H_0 is the shear span, H the height of the wall, L the length of the wall and δ_u the drift capacity at 20% drop in strength. The analysis of the stone masonry data base confirmed that such empirical models can also be derived for stone masonry walls and Figure 7 shows the normalized drift capacity as a function of the axial load ratio (Vanin et al. 2017). In average, the drift capacities obtained for walls of typologies E and E1 yielded larger drift capacities than walls of typologies A-D. Considering the limited amount of data that is available, a further differentiation between masonry typologies did not seem warranted and the following two drift capacity equations were derived (Vanin et al. 2017):

Group 1: $\delta_u = max(1.5\% - 4\% \cdot \frac{\sigma_{0,tot}}{f_c}, 0.3\%) \cdot \frac{H_0}{\min(H,L)}$ (4)

Group 2: $\delta_u = max(2.25\% - 6\% \cdot \frac{\sigma_{0,tot}}{f_c}, 0.45\%) \cdot \frac{H_0}{\min(H,L)}$ (5)

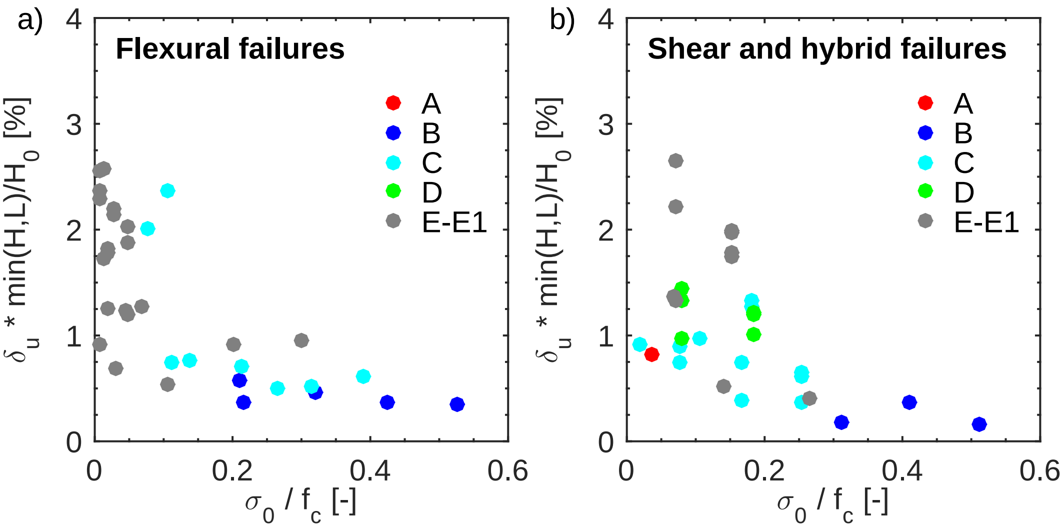

Figure 7. Drift capacity as a function of the axial load ratio (Vanin et al. 2017).

On one hand, it seems desirable to propose for stone masonry drift capacity equations that are of similar form as those for brick masonry fits (Pfyl-Lang et al. 2011; Petry and Beyer 2014b). However, the large variety of shapes of the units and the generated pattern of stone masonries calls for investigating whether parameters such as the Masonry Quality Index or the line of minimum trace are better correlated with the ultimate drift capacity equation than the axial load ratio and the shear span. Figure 8 and Figure 9 show the normalized drift capacity as a function of these two parameters. Both parameters seem to be suitable for predicting the drift capacity of walls failing in a shear or hybrid mode, while they are less correlated to the drift capacity of walls failing in flexure. This applies in particular for the line of minimum trace, for which this observation is also rather intuitive: As the critical cracks for walls failing in flexure are horizontal cracks, which do not pass through the body of the wall, the drift capacity of walls failing in flexure should not be related to the vertical line of minimum trace.

For walls failing in shear, the line of minimum trace seems, however, to show the best correlation with the drift capacity and this geometric measure, which can in the future be derived from image analysis of stone masonry walls, could be a suitable parameter to estimate the drift capacity of walls failing in flexure. Of course, the available data set is limited and because experimental tests of this kind are expensive and time consuming, it is expected that the data base remains limited in the years to come. Furthermore, it is difficult to vary all parameters systematically in experimental tests. To investigate further the influence of these parameters on the drift capacity, it is necessary to perform advanced nonlinear simulations of stone masonry. For this purpose, simplified micro-models that model stones and mortar explicitly seem best suited. First models of this kind have been developed (Zhang et al. 2017).

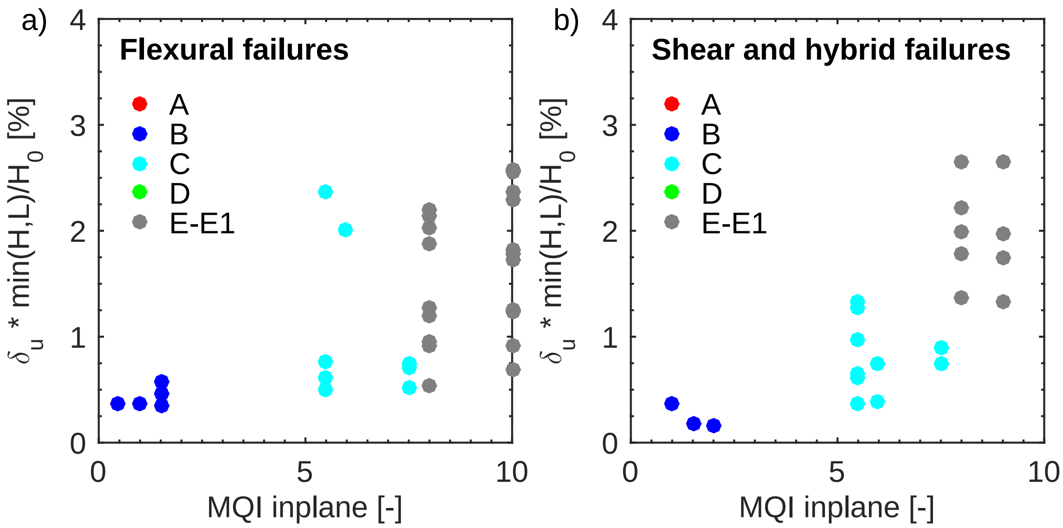

Figure 8. Drift capacity as a function of the Masonry Quality Index.

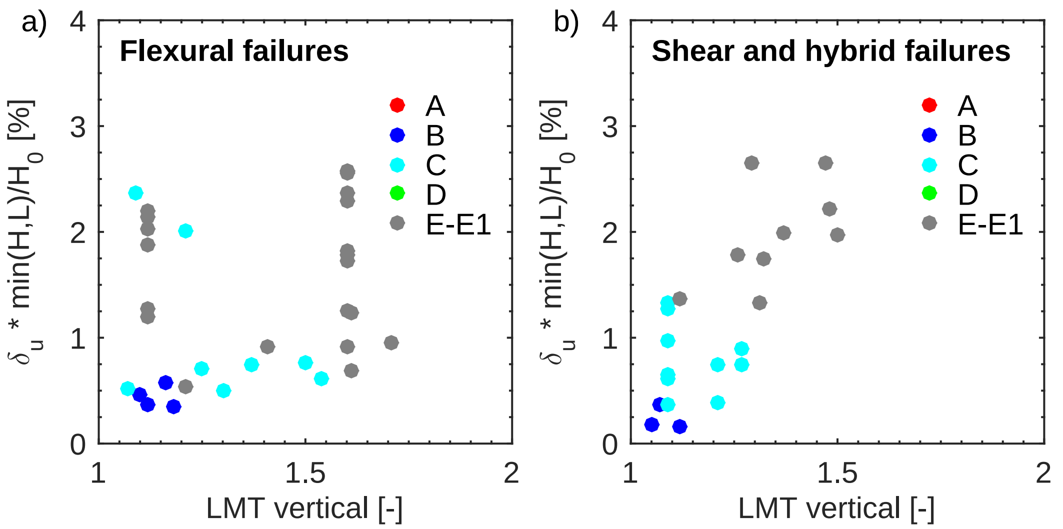

Figure 9. Drift capacity as a function of the normalized length of minimum trace.

5 CONCLUSIONS

In regions of moderate seismicity, the choice between a simple, conservative assessment method or an advanced nonlinear assessment method leads often to very different intervention schemes. Stone masonry buildings that are constructed with stone masonry walls and timber slabs are among the most vulnerable buildings under earthquake loading. It was argued here that in particular in regions of moderate seismicity the application of advanced nonlinear numerical simulations might make a key difference in the assessment. The project focuses on the city of Basel, which was largely destroyed in the 1356 earthquake, which had a magnitude of approximately 6.6.

If simple linear analysis techniques are used, engineering practice in Basel has shown that often extensive strengthening interventions become necessary. The aim of the presented research project is to show that with advanced analysis tools it might be possible to demonstrate that the capacity of most stone masonry buildings is sufficient, if minor interventions that aim at a good connections between the slabs and walls are performed. Such analysis tools are still under development and this project aims at making contributions to these efforts. The paper gave an overview of the research objectives and the experimental program. One objective is to develop drift capacity models specifically for stone masonry walls. First empirical models that consider the parameters, which have already been used for empirical models of brick masonry walls, have been recently proposed (Vanin et al. 2017).

If out-of-plane failures are prevented, uncertainties related to drift capacity models control the epistemic uncertainty when predicting the seismic response of a stone masonry building to a given seismic hazard (Vanin and Beyer 2017). Future research work should therefore strive to investigate whether the uncertainties related to drift capacity models of stone masonry walls can be further reduced. As a first step, this paper investigates whether new parameters, which can be derived from image analysis, could be used as predictors for the drift capacity. It is suggested that the length of the line of minimum trace through mortar joints could be a suitable candidate for predicting the drift capacity of walls failing in shear or a hybrid mode. As expected, this parameter is, however, not correlated to the drift capacity of walls failing in flexure. Identifying such parameters will also help to develop mechanical models for the drift capacity of stone masonry walls. Such models have been recently put forward for modern brick masonry walls (Benedetti and Steli 2008; Petry and Beyer 2015; Wilding and Beyer 2016) and have shown to outperform empirical models.

6 ACKNOWLEDGMENTS

The Basel-Project is supported by the Swiss Federal Office of the Environment and the Construction Department of the Canton Basel-Stadt.

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