Seismic behaviour of brickwork chimneys in buildings

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Abstract. The construction of chimneys of solid bricks in buildings with sloped roofs was commonplace in Bulgaria for almost a century. The collapse of a chimney during an earthquake could potentially lead to damages greatly exceeding the loss of the chimney itself, e.g. partial damage to the roof tiling and leaks, as well as material damage, injury or loss of life due to debris fall. A FEM model was created, in which the storeys of the building are represented in a generalised way, while the chimney is modelled explicitly as a cantilever supported at roof level. The internal forces in chimneys with heights ranging from 0.5 m to 2.0 m, belonging to buildings with height ranging from two to seven storeys were computed. Acceleration records from real earthquakes acting at the base of the building with varying peak ground acceleration and predominant period were used for input loading. The maximum tensile stresses at the bed joints were computed and were compared to the typical tensile strength of the mortars used for chimney construction, to assess the possibility of collapse. A simple, low-tech method for upgrading of existing chimneys by applying a coat of cement-based plaster with embedded fiberglass mesh is proposed.

1. Introduction

The widespread construction of chimneys made by solid brick units and lime or lime-cement mortar in buildings with sloped roofs began after Bulgaria became an independent country in 1878, and continued until industrialized construction became commonplace. During the 2012 earthquake with magnitude $M = 5.8$ and epicentre in the vicinity of the city of Pernik, a large number of masonry chimneys in the city collapsed, figure 1. Debris of masonry chimneys were also observed on the pavements in the capital city, Sofia, although virtually no structural damage occurred there. Such behaviour could potentially lead to damages greatly exceeding the loss of the chimney itself, e.g. partial damage to the roof tiling and leaks, as well as material damage, injury or loss of life due to debris falling down from the roof. The problem is further aggravated by the location of buildings with masonry chimneys within a city. Given their relatively old age, they are concentrated in or close to the city centres, where streets are generally narrower and the number of pedestrians and parked vehicles larger, thus increasing the probability of secondary damage.

Research on the seismic behaviour of chimneys is focused mostly on industrial ones, [1], [2], [3] and reference therein, which pose a greater threat, are structurally more challenging and possess some heritage value, which is rarely the case for chimneys of residential or office buildings. No past research on the seismic response of ‘small’ chimneys is known to the author. The objective of this study is to
clarify their behaviour when subjected to ground shaking, and to assess the likelihood of failure for different levels of ground motion and mortar tensile strength.

![Figure 1. Damage to chimneys of residential buildings in the city of Pernik.](image)

2. Analysis model

This study only considers a scenario whereby a chimney collapses before significant damage has occurred to the building it belongs to. Indeed, if the building collapses or suffers heavy damage, a damage to the chimney will be of little or no consequence. In other words, cases where collapse of the chimney is caused by damage or collapse of the building are not considered. This assumption allows us to use elastic material behaviour, considering dissipative mechanisms through increased damping at larger ground motion levels, instead of explicitly specifying nonlinear material behaviour. Also, since tensile failure at the bed joints being the reason for the onset of collapse, elastic analysis is justified as far as the behaviour of the chimney is concerned, considering the low, sometimes negligible strength of mortars used for chimney construction. The modelling and the analyses were carried out by the computer program SeismoStruct (https://seismosoft.com).

2.1. Model parameters

The analysis model is shown in figure 2. It is a finite element model, in which the storeys of the building are represented in a generalised way by their intrinsic stiffness and mass, while the chimney is modelled explicitly as a cantilever supported at roof level. The lumped mass value of 258 t at each floor level is calculated assuming it represents a floor in a building with floor area of 200 m², 0.25 m thick external walls, 0.12 m thick internal walls and standard finishes. A single frame element is specified between two floors. Its bending stiffness is adjusted so as a seven story building has a natural period of 0.7 sec, considering that as rule of thumb, buildings like the ones being studied have a natural period of 0.1 sec per story. The rotational degrees of freedom at each floor level are restrained, and the base node at which the ground motion is input is fully restrained. The resulting model is thus of a symmetrical shear building. The range of building heights investigated is between 2 stories and 7 stories. Eigenvalue analysis produced fundamental periods of 0.237, 0.334, 0.423, 0.512, 0.605 sec for the 2, 3, 4, 5 and 6 story buildings respectively, which agrees well with the 0.1 sec per story assumption. Single-story buildings were excluded from the study for two reasons: a) they are very rare (at least in Bulgaria), and b) their natural periods are so short that they are not expected to significantly amplify the base ground motion. On the other hand, buildings with brick chimneys higher than 7 stories are extremely rare. A separate model was created for each of the six individual building heights.

A chimney is fixed to the top floor node and is modelled by four frame elements of equal length. The Young modulus was specified as 10,000 MPa, a typical value for brick masonry, while the specific weight was specified as 20 kN/m³, a little higher than the usual value of 18 kN/m³ to allow for additional loads such as plaster and concrete crowns on top of the chimney. Also due to limitations of the program,
the wall between the two flues of the stack is not included in the model, so the increased specific weight caters for this too. For the chimney the mass is automatically lumped at the nodes by the program. The range of chimney heights investigated is between 0.5 m and 2.00 m at 0.25 m increments, a total of seven chimney heights. This range was decided upon by visual inspection of the skyline of the city of Sofia and other Bulgarian cities. In order to reduce the number of runs all seven chimneys were included in a model of a particular building height. This would not alter the response significantly, as the total weight of the seven chimneys is 3 t compared to the floor mass of 258 t. The cross section of the chimney used in the analyses represents a double-flue stack. This choice was done for two reasons: a) single-flue stacks which would undoubtedly produce higher stresses are very rare; b) the contribution of the bending moment about the strong axis to the total normal stress is small enough as to assume that the behaviour of double-flue stack is not overly conservative, and may be used to represent the behaviour of stacks with more flues.

The resulting model is deemed simple, yet sufficient for the purpose of this study. The building is modelled so that its most important modes of vibration can be activated, and the ground motion filtered and modified accordingly while it reaches the top of the building and becomes input motion for the chimney.

2.2. Input ground motion and damping
The 2012 Pernik earthquake was recorded at three locations in Sofia, with the ground motions described and analysed in detail in [4]. The shapes of these three ground motions were used as reference, while the peak ground acceleration (PGA) of the stronger horizontal component was scaled to levels of 0.05 g, 0.10 g, 0.15 g and 0.20 g in order to investigate the behaviour with increasing strength of the ground shaking. The buildings studied here are of different structural typology given the time span of almost a century during which they were built. They can be masonry buildings with deformable or RC floors, with walls unframed or framed by RC elements. In terms of seismic design, they can be pre-code, low-code or sometimes moderate-code buildings. The upper bound of 0.20 g was chosen because according to the results of vulnerability studies presented in [5], [6] and [7], the investigated building types are likely to develop significant damage for higher PGA, and thus may collapse together with the chimneys.

To account for the different degree of energy dissipation from all sources within the framework of linear elastic analysis, a different damping ratio was adopted for each PGA level listed above, namely 2%, 3%, 4% and 5% of the critical respectively. The damping was specified as Rayleigh type, whereby for the fundamental period of the building it has the values given above, according to the PGA, and for the
fundamental period of the 2.00 m high chimney it is a constant of 2%, considering the low levels of stress the chimney is exposed to. Note that the use of varying damping ratios precludes the simple scaling of response values which could be done if a constant damping were adopted throughout the analyses. All three components of the ground motion were applied simultaneously at the base node of the model and time-history analysis carried out with duration of 25 sec., sufficient to include the strongest part of each record.

3. Results and discussion

Three tri-axial ground motion records, four levels of PGA, seven chimney heights and six building heights were used in the analyses to produce a total of 504 data sets. For each of them the maximum tensile stress perpendicular to the bed joint at the node where the chimney stack is attached to the building is computed by the formula,

$$\sigma_z = \frac{|M_y|}{W_y} + \frac{|M_x|}{W_x} - \frac{|N|}{A}$$  \hspace{1cm} (1)

where $M_y, M_x,$ and $N$ are the bending moments about the principle axes of the cross section and the axial force, while $W_y, W_x,$ and $A$ are the section moduli about the principle axes of the cross section and the cross-sectional area. The resulting stresses are compared to the tensile strength of the bed-joint mortar. If the stress is higher than the strength, the stack is assumed to have failed in tension, and therefore in high risk of collapsing. Whether collapse would actually occur cannot be established with the analysis implemented here, and is beyond the scope of this study.

3.1. Common features of the response

A number of important trends were observed for all data sets as follows:

The bending moments about the principle axes of the chimney stack obtained from the analyses have the same values as the theoretical bending moments in a cantilever subjected to an uniformly distributed load $q = ma$, where $m$ is the mass per unit length of the stack, and $a$ is the acceleration at the top of the building/base of stack.

The influence of the fluctuation of the axial force due to the vertical vibrations is negligible when obtaining the maximum of the time history of stress values computed by equation (1). Practically the same maximum is obtained when the contribution of the axial force is computed using the constant self-weight.

The reason for the above is that the chimney stack itself is practically rigid, having a fundamental period of 0.027 sec. Thus no dynamic amplification occurs within the chimney stack. Therefore, for practical purposes, the bending moments in a chimney stack can be obtained quickly and reliably by knowing the acceleration at their base in the direction of the two principle axes. It is not necessary to include the chimney stack explicitly in the computational model.

It was confirmed that the chimney stack can be regarded as an independent (secondary) structure fixed to the building it belongs to instead of to the ground. Therefore, the way the building amplifies the ground motion is crucial to the response of the chimney stack. Indeed, it was observed that due to matching of the response spectrum peaks of the ground motion and the fundamental period of the model, the response of a particular stack height to a particular ground motion strongly depends on the height of the building (in terms of stories) it is attached to.

3.2. Vulnerability of the chimneys

The tensile strength $f_t$ of lime or lime-cement mortars used for masonry varies, with values ranging from 0.04 MPa to above 0.20 MPa been reported by technical recommendations of professional organizations [8], or experimental studies [9]. Given the old age and low maintenance of the buildings under consideration, it is assumed that 0.20 is the highest tensile strength that can be realistically expected. A number of lower strength levels were also used in the vulnerability study, namely 0.10 MPa,
0.05 MPa and 0.005 MPa. The last value was meant to represent extreme cases, e.g. very poor maintenance, substandard quality of the mortar, poor workmanship or the use of mud instead of mortar.

The results of the vulnerability study are summarized in figure 3. Chimney heights missing from the figures either have a failure rate of 0% (no failure) or 100% (total failure) for the whole range of PGAs. Many valuable insights can be gained from the results, which may be used to shape disaster mitigation strategies. If a chimney is constructed by good quality mortar ($f_t = 0.20$ MPa), it is likely to fail only if its height is above 1.50 m. At the same time, even chimneys with height 2.00 m would most likely survive a shaking with PGA of 0.10 g.

At the other end, with poor tensile strength ($f_t = 0.005$ MPa), only chimneys with height of up to 0.75 m may be considered safe, with this being only for very low levels of ground shaking – PGA of up to 0.50 g. It is interesting to note, that for PGA = 0.05 g, the stress at the base of the 0.50 m chimney remains compressive, thus precluding collapse. For PGA = 0.20 g, chimneys of all heights would fail. If we assume that low tensile strength is prevalent, these results are in good agreement with field observations from the 2012 Pernik earthquake. In the city of Pernik, which experienced PGA of around 0.20 g, the majority of the chimneys in the block of houses show in figure 1, collapsed. These houses are very old pre-war buildings, showing no signs of maintenance. In the city of Sofia, which experienced PGA between 0.05 g in the city centre and 0.10 g at locations closer to the epicentre [4], brick debris possibly from chimneys, were observed on the pavements.

4. Upgrading method for collapse prevention

Considering the demonstrated high vulnerability of brick chimneys to relatively low levels of ground shaking it is desirable to propose a strengthening method which at best should be cost effective and low-tech to make it implementable by an average bricklayer or plasterer. A design methodology for improving the flexural capacity of masonry walls using CFRP sheets is proposed in [10]. Using FRP technology however, requires somewhat specialized skills, and the sheets are not cheap either. In the following it will be demonstrated that adequate seismic retrofit of the chimneys may be achieved by using ordinary glass fibre mesh for plastering.
Although glass fibre mesh is not meant for structural use, its strength is occasionally quoted by manufacturers, e.g. 1.88 kN/5cm strip is reported at https://terazid.com/shop/produkti-za-toploizolaciq/stuklofiburna-mreja-terazid/, which is equivalent to 37.6 kN/m. Next, we compute the flexural capacity about the weak axis of the cross section shown in figure 2, assuming it is cracked. The mesh will provide a tensile force \( N_t = 0.64 \times 37.6 = 24.1 \) kN. Assuming very conservatively the compression zone to spread over the width of the whole brick, 0.12 m, will result in lever arm of 0.32 m, and capacity of the section \( M = 24.1 \times 0.32 = 7.71 \) kNm. This is well over the largest bending moment obtained from the analyses \( M_{y,\text{max}} = 6.85 \) kNm for \( \text{PGA} = 0.20 \) g and chimney height 2.00 m. Hence a single layer of glass fibre mesh embedded in suitable plaster will be sufficient to prevent failure of the chimneys considered in this study.

5. Summary and conclusions

The seismic response of chimneys in buildings was investigated. It was shown that for a given ground motion, the internal forces in the chimneys depend entirely on the dynamic properties of the building.

Fragility curves for chimneys of heights ranging from 0.50 m to 2.00 m and tensile strengths of mortar ranging from 0.005 MPa to 0.20 MPa were developed. The high strength fragility curves indicate that if good quality materials and workmanship are applied, most chimneys will be safe against weak to medium ground shaking. On the other hand, low strength fragility curves which agree well with field observations from the 2012 Pernik earthquake, indicated that most chimneys will be damaged even for weak ground shaking.

A simple low-tech retrofit method using glass fibre mesh for plaster was proposed and shown to be adequate for the range of structural and loading parameters of the current study. It is recommended that this method is used whenever large scale roof repairs are done. During such repairs chimneys are often plastered, so embedding a glass fibre mesh in the process can be done at very little extra cost.

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