A Multi-Scale Analysis Based on a Map-Oriented Database for the Out-of-Plane Seismic Behaviour of Masonry Structures

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A multi-scale analysis based on a map-oriented database for the out-of-plane seismic behaviour of masonry structures

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Abstract Since 1986, earthquakes have occurred in East Groningen, but most houses, schools and are made of unreinforced masonry, which must now withstand magnitude 4 earthquakes. This has created an urgent need to assess large amounts of buildings in a fast but reliable manner.

The out-of-plane behaviour is important for seismic assessments of unreinforced masonry buildings. Although the most accurate analysis method to determine the out-of-plane response of such walls is non-linear time-history analysis (NLTH), non-linear kinematic analysis (NLKA) provides a simple, fast but still reliable solution due to the computational difficulties of NLTH for structures constructed of unreinforced masonry.

In this paper, the out-of-plane behaviours of masonry structures are up-scaled from a component scale to a provincial scale in a multi-scale manner. A map-oriented database is established to describe both local behaviours of walls and global behaviours of a province. The out-of-plane assessment by non-linear kinematic analysis (NLKA) is automated via the database without further calculations after the static analysis. The database provides a solid guidance to determine which detailed assessment methods will be adopted with limited data before a FEM model is built.

Keywords Structural mechanics · Data science · Masonry · NLKA · Multi-scale

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1 Introduction

Until recently, the computation of unreinforced masonry response was not an urgent problem because unreinforced masonry or unreinforced concrete were not used for structures that need to carry large loads. However, since 1986, earthquakes have occurred in East Groningen, the Netherlands, due to gas extraction from deep, porous rocks. Gas extraction began in 1959 and considerable subsidence of the surface has occurred as a result. It was not expected that parts of the porous rock would suddenly compact. Most houses, schools and churches in East Groningen are made of unreinforced masonry, which must now withstand magnitude 4 earthquakes. This has created an urgent need for non-linear analysis to compute the response of unreinforced masonry and unreinforced concrete, not just in the linear design stage but up to failure and near-collapse Messali et al. (2018) Esposito et al. (2019).

The out-of-plane behaviour is important for seismic assessments of unreinforced masonry buildings. Since masonry buildings are not high, normally masonry buildings fail out-of-plane instead of in-plane. In The Netherlands, for the assessment of the near collapse (NC) limit state of unreinforced masonry walls under seismic loading, guidance on the out-of-plane displacement limit of such walls is needed if the assessment is performed by non-linear time-history analysis (NLTH), particularly when using implicit time marching schemes He et al. (2020). This has been recognized in NPR (2020), where section F.6.3 stipulates that criteria for the out-of-plane displacement of unreinforced masonry walls shall be based on the criteria set forth in section H. That section H of NPR (2020) describes analytical methods for the evaluation of seismically loaded unreinforced masonry wall, in particular non-linear kinematic analysis (NLKA). It is also a significant part of both seismic assessment guidelines of New Zealand NZSEE (2016) and Italian NTC (2008) Circolare (2009).

Although in section H.1 of NPR (2020) it is stated that the most accurate analysis method to determine the out-of-plane response of such walls is non-linear time-history analysis rather than non-linear kinematic analysis, the computation of response is very difficult for structures constructed of unreinforced masonry, unreinforced concrete or other quasi-brittle materials. The classical constitutive models for quasi-brittle materials are plasticity models, damage models and coupled damage-plastic models Houlsby (2005). The classical algorithm for solving non-linear mathematical equations is the Newton-Raphson method. This method is robust if the load-displacement curve is continuous and smooth and if all derivatives of the constitutive models are fully defined. However, for quasi-brittle materials, many cracks occur and grow at every load increment, which creates many small, sharp peaks in the load-displacement curve Yu et al. (2018). The stress-strain equations are well-defined, and the size effect is included. The problem is that algorithms for solving these equations often fail to find a solution. The negative tangent stiffness causes an ill-conditioned stiffness matrix when brittle material softens De Borst and Nauta (1985) Rots and De Borst (1987) Rots and Blaauwendraad (1989). In addition, two physical phenomena sometimes occur in non-linear analysis, namely that the solution can lack uniqueness and can be split into equally possible realities. This split is called a bifurcation. The solution to an increment can be a reduction in load and displacement, which is called snap-back. These phenomena need to be simulated correctly. The engineer

responsible for such an analysis must steer the software to obtain accurate results. Computation is slow when cracks propagate exponentially, snap-back and bifurcation arise, and the results typically do not continue beyond the onset of failure. Moreover, the results are even more unreliable and unstable when geometrical non-linearity, but it is important for the out-of-plane behaviour Yu et al. (2021). Therefore, it is impossible to carry out NLTH assessments for all buildings in Groningen. Non-linear kinematic analysis (NLKA) is still valuable for the assessments at a provincial scale due to its efficiency. This is particularly significant to determine which methods will be used with limited data before an assessment in the preliminary phase. In addition, A tri-linear model is proposed to assess the out-of-plane seismic behaviours of vertically spanning unreinforced masonry walls Godio and Beyer (2019).

The work presented in this paper up-scales the out-of-plane behaviours of masonry structures from a component scale to a provincial scale in a multi-scale manner. A map-oriented database is proposed which describes both local behaviours of walls and global behaviours of a province. The database provides a solid guidance to determine which assessment methods will be adopted with limited data before a FEM model is built Hermens et al. (2018). The out-of-plane assessment by non-linear kinematic analysis (NLKA) is automated via the database without further calculations after the static analysis.

2 NLKA assessment on a component scale

A nonlinear kinematic analysis (NLKA) Robinson and Priestley (1986)Blaikie and Spurr (1992)Blaikie and Davey (2005) is a displacement based inelastic method for assessing face-loaded unreinforced masonry walls to determine the out-of-plane (OOP) structural performance of unreinforced masonry walls based on the geometry of the walls. The NLKA method is based on the principle of virtual work to take into account both the elastic and nonlinear behaviour of the failure mechanisms. The failure mechanism (crack pattern) is related to the support conditions and geometry. It is improved from extensive experimental investigations Blaikie and Davey (2002)Derakhshan et al. (2014a)Derakhshan et al. (2014b)Derakhshan et al. (2013b). Recently, Further improvements are carried out due to the earthquakes in the Netherlands Graziotti et al. (2019).

For a wall with weight W and an effective thickness t_{eff} (in m), it holds that:

$$W_b = W_t = W/2, y_b = y_t = h/4, t_{eff} = t \left(0.975 - 0.025 \frac{F}{W} \right) \quad (1)$$

the angle A (in rad) NZSEE (2016)NPR (2020) at which masonry walls with weight W (in N) and height h (in m) become unstable is derived using a virtual work equation as:

$$A = \frac{b}{a} \quad (2)$$

where:

$$a = W_b y_b + W_t (h - y_t) + Fh$$

The maximum deflection Δ_i (in m) at mid-height of the wall is then:

$$\Delta_i = \frac{bh}{2a} \quad (3)$$

According to Derakhshan et al. (2013a)NPR (2020) the deflection of the wall must be limited to:

$$\Delta_m = 0.6\Delta_i \quad (4)$$

and for cantilever walls to:

$$\Delta_m = 0.3\Delta_i \quad (5)$$

The mass participation factor γ (dimensionless) to convert a single-degree-of-freedom (SDOF) system to the kinematic wall system is defined as:

$$\gamma = \frac{W_b y_b + W_t y_t}{2Jg} h \quad (6)$$

with g (in m/s^2) as the gravity acceleration and J (in kgm^2) as the mass moment of inertia of the total kinematic wall system:

$$J = \frac{1}{12} \frac{W}{g} \left(t^2 + \left(\frac{h}{2} \right)^2 \right) + \frac{W_b (e_b^2 + y_b^2) + W_t \left((e_o + e_b + e_t)^2 + y_t^2 \right) + F (e_o + e_b + e_t + e_p)^2}{g} + J_{anc} \quad (7)$$

The seismic resistance capacity of the masonry wall in terms of acceleration R_d (in g) is then:

$$R_d = \left(\frac{2\pi}{T_a} \right)^2 \frac{\Delta_m}{\gamma g} \quad (8)$$

with the fundamental vibration period of the wall or element T_a (in s) as:

$$T_a = \beta \sqrt{\frac{J}{a}} \quad (9)$$

where $\beta = 4.07$ for vertically spanning walls and $\beta = 3.1$ for cantilevers.

The design value of the horizontal spectral acceleration $S_{Ea;d}$ NPR (2020)NZSEE (2016)is derived depending on the properties of the structure as:

1.

$$S_{Ea;d} = a_{g;d} \times \frac{1}{q_a} \times \left[\frac{3 \left(1 + \frac{z}{h_n} \right)}{\left(1 + \left(1 - \frac{T_a}{T_{eff}} \right)^2 \right)} - 0,5 \right] \text{ and } S_{Ea;d} \geq S_{E;d} \quad (10)$$

named as H5a in NPR (2020) if the structure is elastic or if the fundamental vibration period of the element period (T_a) is longer than the effective vibration period of the inelastic structure (T_{eff});

2.

$$S_{Ea;d} = a_{g;d} \times \frac{1}{q_a} \left(2,5 + 3 \frac{z}{h_n} \right) \text{ and } S_{Ea;d} \geq S_{E;d} \quad (11)$$

named as H5b in NPR (2020) if the effective period of the inelastic structure (T_{eff}) is longer than the fundamental vibration period of the element (T_a), or if interaction can be expected in another way, or if connecting forces are determined,

where $S_{e;d}$ in g is the design value of the horizontal spectral acceleration to be derived from surface ground spectrum given the webtool, $a_{g;d}$ in g is the design value of the peak ground acceleration, T_{eff} is the effective vibration period, determined on the basis of the effective stiffness of the building in the direction considered and q_a is the element behaviour factor, which is 2.0 for horizontal spans, vertical spans, gables and cantilevers. $\frac{z}{h_n}$ is named as a height ratio.

36% of the instability displacement is used to determine the effective period for one-way spanning walls according to Derakhshan et al. (2013a). The fundamental vibration period of the element T_a is:

$$T_a = \sqrt{\frac{0,28h}{\left(1 + \frac{2F}{W}\right)}} \text{ for one-way vertically spanning walls and gables} \quad (12)$$

$$T_a = \sqrt{0,65h \left[1 + \left(\frac{t}{h}\right)^2\right]} \text{ for cantilevers}$$

The assessment is to perform a unity check of the seismic demand ($S_{Ea;d}$) versus the seismic resistance (R_d). If the unity check ($S_{Ea;d}/R_d$) is smaller than 1, it is concluded that the wall has sufficient out-of plane seismic capacity.

3 Seismic resistance and demand at a structural scale

The out-of-plane behaviour assessment contains the seismic resistance and the seismic demand. According to NPR (2020), the out-of-plane (OOP) behaviour of a wall is determined by multiple parameters, namely its geometry, its boundary condition and the overburden, the Uniform Hazard Spectrum (UHS) and the fundamental frequency of the building. Those parameters vary a lot and highly influence the OOP assessment.

In general, the fundamental frequency of a typical building in the Groningen province of the Netherlands ranges from 1 Hz to 6 Hz. For one-way spanning, the determining geometry of a wall includes a thickness of 100 mm or 210 mm and a height of approximate 2.7 m Messali et al. (2017). The boundary condition depends on the types of floors, which can be assumed as a rigid one for concrete floors and a flexible one for timber floors.

The Uniform Hazard Spectrum is determined by the location of a building, which contains $a_{g;d}$, T_b , T_c , T_d and p , where $a_{g;d}$ is the design value of the peak ground acceleration at surface level, T_b and T_c are the numerical values of the lower and upper limit of the vibration periods for which the spectral acceleration is constant, T_d is the

Fig. 3 Overburden versus the seismic resistance for the one-way spanning NLKA of a masonry wall with different thickness. (rigid boundary conditions on the left, flexible boundary conditions on the right)

period that indicates the start of the constant displacement response of the spectrum and p is the ratio between the peak ground acceleration and the plateau value of the elastic response spectrum. According to the time period reference T5 NPR (2020), $a_{g,d}$, T_b , T_c , T_d and p range from 0.26 m/s^2 , 0.09 s , 0.314 s , 0.559 s and 1.576 to 2.31 m/s^2 , 0.384 s , 0.738 s , 0.874 s and 2.74 respectively.

The seismic resistance of a wall is independent of its location, the database of which can be easily built and categorized by geometry, the boundary conditions and overburden. For the geometry, since the height of a wall is approximately 2.7 m , only wall thickness matters. The boundary conditions can be also easily categorized by a rigid one (1-4) or a flexible one (2-5) as shown in Figure 2. Hereby, a capacity database can be established in a relation of overburden and out-of-plane capacity as shown in Figure 3.

However, the seismic demand of a wall is dependent on its location, which determines UHS. The sensitivity study of other variables is based on an example of the UHS in which $a_{g,d}$, T_b , T_c , T_d and p are 0.1845 , 0.247 , 0.514 , 0.862 and 2.138 . Figure 4 depicts the relation between the fundamental frequency of the building versus the seismic demand when the overburden is zero and z/hn is 0.5 for the rigid and flexible boundary conditions. It can be seen no effect of boundary conditions on seismic demands since the fundamental period of element is irrelevant based on Equations (10), (11) and (12). The seismic demand is determined by Equation h5b (11) until 1 Hz , then follows by Equation h5a (10) until 2 Hz and finally depends on the lower bound S_{ed} . Although the seismic demand is determined by multiple equations and variables, it can be concluded that the seismic demand decreases along with the increase of the fundamental frequency of the building. The frequency of 2 Hz is the transition point, at which the sharp descent trend switches to a plateau in this case. The relation between the seismic demand and the overburden are shown for distinct building frequencies ranging from 1 Hz to 6 Hz in Figure 5. In this case, the seismic demand increases along with the overburden until the plateau of UHS, governed by H5a, H5b and S_{ed} based on the building frequency. The seismic demand does not follow UHS after the plateau. Instead, it is determined by H5a and H5b. Note that the changing trend of the seismic demand could be quite different from each other for

Fig. 4 Fundamental frequency of the building versus the seismic demand for the one-way spanning NLKA of a masonry wall (100mm thick and 2.7 thick) when the overburden is zero and z/h_n is 0.5.

varying UHS. The seismic demand is governed by UHS when $a_{g;d}$ and p are large, whereas H5a and H5b are dominant for small values of $a_{g;d}$ and p .

Fig. 5 Overburden versus the seismic demand for the one-way spanning NLKA of a masonry wall (100mm thick and 2.7 thick) for different fundamental frequencies of the building when z/h_n is 0.5.

4 Up-scaling seismic behaviour from a structural scale to a provincial scale

According to the previous section, overburden highly influences the seismic capacity and demand. Also, the value of overburden varies a lot, and it is impossible to define default overburdens. On the other hand, other parameters, like thickness and height of a wall, can be categorized easily based on the reality. Hereby, overburden is chosen as a variable. In order to up-scale the out-of-plane seismic behaviour from a structural scale to a provincial scale, the minimum overburden is needed for the Uniform Hazard Spectrum (UHS) of all locations based on various structural and component information including the component geometry, the boundary condition, the height ratio and the building frequency. Therefore, multiple buildings can be assessed based on the overburden from a static analysis or the code or the practical experience. A rou-

Fig. 6 Program structure diagram of NLKA check

tine with the name “NLKA check” is defined in Figure 6 to calculate the unity check μ . The program structure diagram is shown in Figure 7 to generate the minimum overburdens of all locations. If the overburden of a wall is larger than the required minimum overburden, it can be concluded that the wall has sufficient out-of-plane resistance. Note that this statement is reliable if the relation between the overburden and unity check is a decreasing function and the critical overburden is zero.

Fig. 7 Program structure diagram of the map-oriented database generation

For the given inputs mentioned in Table 1 and for a given overburden, the seismic demand, seismic resistance and consequently, the NLKA Check is calculated. A plot is then created between the unity Check μ and the Overburden (F) as shown in Figure 8. The critical overburden is defined as a value when μ is the largest. The overburden that corresponds to the unity check of 1, would be the minimum overburden to have sufficient out-of-plane capacity for the inputs mentioned in Table 1. This is done for all the different locations in the Groningen province. It can be seen that the largest unity check occurs at an overburden value of zero for a wall height of 1.5 m or 2.7 m while the unity check meets the peak at an overburden of 1 kN/m for a wall height of 4 m. This is due to the fact that the seismic demand increases along with the overburden until the plateau of UHS. Also, the fundamental vibration period of the element is influenced by the overburden, which determines that either Equation H5a or Equation h5b are adopted to calculate the seismic demand. However, the difference between the values from the critical overburden and zero overburden is small. In addition, for a normal height of wall (2.7 m), the critical overburden is zero for this example.

Table 1 Different inputs for the example of minimum overburden load

Parameters	Value	Unit
ag,d (PGA)	0.2534	in g (9.8 m/s ²)
TB	0.195	seconds (s)
TC	0.375	seconds (s)
TD	0.697	seconds (s)
p	2.324	-
Teff	0.5	seconds (s)
Ratio (z/hn)	0.5	-
Thickness of wall (t)	100	mm
Height of wall (h)	1.5, 2.7, 4	m
Boundary Condition (BC)	1-4	-
Interstorey drift (θ)	0.015	-

Fig. 8 Plot between the unity check (μ) versus overburden (F) for the inputs mentioned in Table 1. The dead weights of 2.87, 5.16 and 7.65 kN/m are used to convert between overburden load (kN/m, left figure) and overburden ratio (overburden load relative to dead weight, right figure) for wall heights of 1.5, 2.7 and 4.0 m respectively.

According to the example above, it matters to verify all locations to check whether the critical overburden is zero or not for all situations. If the unity check value reaches the peak after zero overburden, a value of overburden, larger than the minimum required one, could have larger unity check value than that of the minimum overburden. As a consequence, a wall could collapse if the difference of the unity check between the critical and zero overburden is large. Figure 9 indicates the program structure diagram to find the critical overburden until the value of the overburden is 100 kN/m. Figure 9 shows the critical overburden for the building frequencies ranging from 1 Hz to 5.5 Hz, rigid or flexible boundary conditions and height ratios of 0.5 and 0.25. Only when the building frequency is 2.5 Hz, the boundary condition is rigid and the height ratio ($\frac{z}{h_n}$) is 0.5 can the critical overburden be 1 kN/m according to Figure 10. Among the total 6236 locations, 438 locations have a critical overburden of 1 kN/m instead of zero shown in Figure 11. Nevertheless, the unity check values of critical overburden are 1 kN/m and zero almost overlap each other, the maximum error of which are 6.6%. Also, none of them exceed 1, the maximum value of which is 0.89. In addition, according to Figures 4 and 5, the seismic resistance keeps increasing while the seismic demand does not exceed the plateau of UHS and mostly does not increase after the plateau. As a consequence, the unity value cannot increase a lot after the first peak. In addition, it is time-saving to set the critical overburden zero. If the minimum overburden is zero, no any additional calculations are needed to satisfy the assessments. Therefore, the minimum overburden is reliable enough to set as a criteria to assess the out-of-plane seismic behaviour. If the overburden of a wall is larger than the required minimum overburden, it can be concluded that the wall has sufficient out-of-plane resistance.

Fig. 9 Program structure diagram to find the critical overburden

Fig. 10 Building frequency versus the critical overburden value for the one-way spanning NLKA of a masonry wall with different boundary conditions and building heights.

Fig. 11 UC value difference comparison of zero overburden and critical overburden for the one-way spanning NLKA of a masonry wall with a flexible boundary condition and a 2.5 Hz of a building fundamental frequency for 438 locations which have a critical overburden of 1 kN/m.

Following the PSD in Figure 7, overburden and seismic parameters are considered as a variable and the rest of the inputs are kept as constant in a loop to find the minimum overburden. This is done for all the different locations in the Groningen province and a contour plot is obtained for $T_{eff} = 0.5s$, ratio = 0.5, boundary condition of 1 at the top and 4 at the bottom, a wall height of 2.7 m and 100 mm thick wall, as shown in Figure 12, which is overlayed on the Groningen map.

Fig. 12 Plot of the minimum overburden of the wall for the Groningen Province, with $T_{eff} = 0.5s$, height ratio = 0.5, boundary condition of 1 at the top and 4 at the bottom, with a wall height of 2.7 m and a wall thickness of 100 mm.

In this way, a map of minimum overburden of the wall to have sufficient out-of-plane capacity is created for the given constants. However, to take into account the practicality and consider all the different possibilities, the following numbers are considered for the inputs.

1. Height ratio (z/h_n) = [0.25, 0.5, 0.75, 0.9]
2. Effective Time Period of the building (T_{eff} , in *seconds*) = [0.3, 0.5, 0.6]
3. Thickness of wall (t , in *m*) = [0.1, 0.21]
4. Height of wall (h , in *m*) = [1.5, 2.7, 4]
5. Boundary Condition = [1-4 (rigid), 2-5 (flexible)]
6. Interstorey drift (θ , in %) = [1.5 for the BC of 1-4, 2.5 for the BC of 2-5]

The reason to consider four different z/h_n ratios, is to take into account all the different cases in a building. A ratio of 0.25 includes a wall on the ground floor of a two-story building, whereas a ratio of 0.5 considers a wall on the ground floor of a one-story building. A ratio 0.75 considers a wall on the first floor of a 2-story building and a ratio of 0.9 considers a wall on the attic floor. The three different effective periods are taken into account to consider the general time period for one- and two-story building. The constants for the thickness of the wall are 100 mm and 210 mm. Finally, two boundary condition are considered – one where the top and bottom floor is made of concrete whereas the other where the floors are made of timber. If the conditions are different, interpolation between maps is possible to receive the desired output. For the angle theta, 1.5% is for rigid floors (boundary condition 1-4) and 2.5% is for flexible floors (boundary condition 2-5) according to Figure 2.

Figures from 13 to 17 compare the influences of different variables on the the minimum overburden map including boundary condition, wall thickness, wall height, effective time period, height ratio. A rigid boundary condition requires less overburden to satisfy the NLKA check due to its higher resistance compared to a flexible boundary condition in Figure 13. Similarly, a thicker or lower wall has higher resistance to pass the NLKA check with less overburden in Figure 14 and Figure 15. According to Eq. (10), a smaller effective time period results in a smaller seismic demand in Figure 16. In other words, a more rigid building with high natural frequency requires less seismic demand. Also, a larger height ratio brings about a larger seismic demand in Figure 17 based on Eq. (10).

Fig. 13 Boundary condition comparison of the minimum overburden map plot of the wall for the Groningen Province, with $T_{eff} = 0.5s$, height ratio = 0.5, with a wall height of 2.7 m and a wall thickness of 100 mm.

Fig. 14 Wall thickness comparison of the minimum overburden map plot of the wall for the Groningen Province, with $T_{eff} = 0.5s$, height ratio = 0.5, boundary condition of 2 at the top and 5 at the bottom, with a wall height of 2.7 m.

Fig. 15 Wall height comparison of the minimum overburden map plot of the wall for the Groningen Province, with $T_{eff} = 0.5s$, height ratio = 0.5, boundary condition of 2 at the top and 5 at the bottom, with a wall thickness of 100 mm.

Fig. 16 Effective time period comparison of the minimum overburden map plot of the wall for the Groningen Province, with height ratio = 0.5, boundary condition of 2 at the top and 5 at the bottom, with a wall height of 2.7 m and a wall thickness of 100 mm.

Fig. 17 Height ratio comparison of the minimum overburden map plot of the wall for the Groningen Province, with $T_{eff} = 0.5s$, boundary condition of 2 at the top and 5 at the bottom, with a wall height of 2.7 m and a wall thickness of 100 mm.

5 Conclusions

In this paper, the out-of-plane behaviours of masonry structures are up-scaled from a component scale to a provincial scale in a multi-scale manner. The algorithm is proposed to generate a map-oriented database which describes both local behaviours of walls and global behaviours of a province. The database provides a solid guidance to determine which assessment methods will be adopted with limited data before a FEM model is built. The out-of-plane assessment by non-linear kinematic analysis (NLKA) is automated via the database without further calculations after the static analysis.

By systematic elaboration of the seismic demand and resistance of one-way vertically spanning unreinforced masonry walls based on appendix H of NPR 9998:2020 NPR (2020) for the entire Groningen province, it is found that for compliance with that code and for practical height ratios, effective building periods and boundary conditions: the minimum overburden ratio on 100 mm thick walls of regular height (2.7 m) is at least 2.0 kN/m in a large part of the Groningen province when the boundary condition is 2-5; the minimum overburden ratio on 100 mm thick walls of regular height (2.7 m) is mostly 0.0 kN/m in a large part of the Groningen province when the boundary condition is 1-4 meaning that such walls mostly do not fail out of plane in case of T5 seismic loading; the minimum overburden ratio on 210 mm thick walls of regular height (2.7 m) is nearly 0.0 kN/m in a large part of the Groningen province for both boundary conditions 1-4 and 2-5 meaning that also these walls mostly do not fail out of plane in case of T5 seismic loading.

The wall geometry, building information and Uniform Hazard Spectrum (UHS) of the location all determine the required minimum overburden: a thinner or higher wall requires larger overburden due to its lower resistance; a larger overburden is needed for a flexible boundary condition compared to a rigid one owing to its lower resistance as well; A larger effective time period or height ratio results in larger minimum overburden because of its larger seismic demand; UHS influences the seismic demand based on the building location.

Conflict of interest

The authors declare that they have no conflict of interest.

References

- Blaikie, E., Davey, R., 2002. Methodology for assessing the seismic performance of unreinforced masonry single storey walls, parapets and free standing walls. Opus International Consultants.
- Blaikie, E., Davey, R., 2005. Methodology for the seismic assessment of face-loaded unreinforced masonry walls and parapets, in: 2005 New Zealand Society for Earthquake Engineering Conference.

- Blaikie, E., Spurr, D.D., 1992. Earthquake vulnerability of existing unreinforced masonry buildings. Works Consultancy Services.
- Circolare, 2009. Istruzioni per l' applicazione delle nuove norme tecniche per le costruzioni, di cui al D.M. 14/01/2008, G.U. n. 47 del 26/2/2009 (in Italian). Circolare n. 617 del 2/02/2009.
- De Borst, R., Nauta, P., 1985. Non-orthogonal cracks in a smeared finite element model. *Engineering Computations* 2, 35–46.
- Derakhshan, H., Dizhur, D., Griffith, M., Ingham, J., 2014a. Seismic assessment of out-of-plane loaded unreinforced masonry walls in multi-storey buildings. *Bulletin of the New Zealand Society for Earthquake Engineering* 47, 119–138.
- Derakhshan, H., Dizhur, D., Griffith, M.C., Ingham, J.M., 2014b. In situ out-of-plane testing of as-built and retrofitted unreinforced masonry walls. *Journal of Structural Engineering* 140, 04014022.
- Derakhshan, H., Griffith, M.C., Ingham, J.M., 2013a. Airbag testing of multi-leaf unreinforced masonry walls subjected to one-way bending. *Engineering structures* 57, 512–522.
- Derakhshan, H., Griffith, M.C., Ingham, J.M., 2013b. Out-of-plane behavior of one-way spanning unreinforced masonry walls. *Journal of Engineering Mechanics* 139, 409–417.
- Esposito, R., Messali, F., Ravenshorst, G.J., Schipper, H.R., Rots, J.G., 2019. Seismic assessment of a lab-tested two-storey unreinforced masonry dutch terraced house. *Bulletin of Earthquake Engineering* , 1–23.
- Godio, M., Beyer, K., 2019. Trilinear model for the out-of-plane seismic assessment of vertically spanning unreinforced masonry walls. *Journal of Structural Engineering* 145, 04019159.
- Graziotti, F., Tomassetti, U., Sharma, S., Grottoli, L., Magenes, G., 2019. Experimental response of urm single leaf and cavity walls in out-of-plane two-way bending generated by seismic excitation. *Construction and Building Materials* 195, 650–670.
- He, H., Meijers, S., Middelkoop, F., Vonk, R., 2020. Numerical study on the seismic out-of-plane performance of urm walls, in: *Brick and Block Masonry-From Historical to Sustainable Masonry*. CRC Press, pp. 916–923.
- Hermens, M.J., Kraaijenbrink, H.G., Meijers, S.J., 2018. Advances in non-linear time history and modal response spectrum analyses for the seismic assessment of buildings in groningen (netherlands), in: *High Tech Concrete: Where Technology and Engineering Meet*. Springer, pp. 1505–1514.
- Houlsby, G., 2005. A thermodynamic approach to constitutive modelling of concrete using damage mechanics and plasticity theory. Ph.D. thesis. University of Oxford.
- Messali, F., Esposito, R., Jafari, S., Ravenshorst, G., Korswagen, P., Rots, J.G., 2018. A multiscale experimental characterisation of dutch unreinforced masonry buildings, in: *Proceedings of 16th European Conference on Earthquake Engineering (ECEE)*, pp. 18–21.
- Messali, F., Ravenshorst, G., Esposito, R., Rots, J., 2017. Large-scale testing program for the seismic characterization of dutch masonry walls, in: *Proceedings of the 16th European Conference on Earthquake Engineering, 16WCEE, Santiago, CL*.

- NPR, 2020. Beoordeling van de constructieve veiligheid van een gebouw bij nieuwbouw, verbouw en afkeuren – Grondslagen voor aardbevingsbelastingen: geïnduceerde aardbevingen, Nederlandse praktijkrichtlijn, NPR 9998 (in Dutch). volume H. Nederlands Normalisatie Instituut.
- NTC, 2008. Norme tecniche per le costruzioni, Ministry of Infrastructures and Transportations, G.U. S.O. n. 30 on 4/2/2008 (in Italian). NTC 2008 – Decreto Ministeriale 14/1/2008.
- NZSEE, 2016. The seismic assessment of existing buildings, Part C8: Seismic assessment of unreinforced masonry buildings. volume C8. New Zealand Society for Earthquake Engineering.
- Robinson, L., Priestley, M., 1986. Seismic behaviour of unreinforced masonry walls. Bulletin of the New Zealand Society for Earthquake Engineering 19, 65–75.
- Rots, J.G., Blaauwendraad, J., 1989. Crack models for concrete, discrete or smeared? fixed, multi-directional or rotating? HERON, 34 (1), 1989 .
- Rots, J.G., De Borst, R., 1987. Analysis of mixed-mode fracture in concrete. Journal of Engineering Mechanics 113, 1739–1758.
- Yu, C., Hoogenboom, P., Rots, J., 2018. Incremental sequentially linear analysis to control failure for quasi-brittle materials and structures including non-proportional loading. Engineering Fracture Mechanics 202, 332–349. doi:<https://doi.org/10.1016/j.engfracmech.2018.07.036>.
- Yu, C., Hoogenboom, P., Rots, J., 2021. Extension of incremental sequentially linear analysis to geometrical non-linearity with indirect displacement control. Engineering structures 229, 111562. doi:<https://doi.org/10.1016/j.engstruct.2020.111562>.