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COMPUTATIONAL MODELLING OF DAMAGE ACCUMULATION IN UNREINFORCED MASONRY DUTCH CONSTRUCTIONS SUBJECTED TO INDUCED SEISMICITY

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Abstract: This paper aims to quantify the cumulative damage of unreinforced masonry (URM) subjected to induced seismicity. A numerical model based on discrete element method (DEM) has been develop and was able to represented masonry wall panels with and without openings; which are common typologies of domestic houses in the Groningen gas field in the Netherlands. Within DEM, masonry units were represented as a series of discrete blocks bonded together with zero-thickness interfaces, representing mortar, which can open and close according to the stresses applied on them. Initially, the numerical model has been validated against the experimental data reported in the literature. It was assumed that the bricks would exhibit linear stress-strain behaviour and that opening and slip along the mortar joints would be the predominant failure mechanism. Then, accumulated damage within the seismic response of the masonry walls investigated by means of harmonic load excitations representative of the acceleration time histories recorded during induced seismicity events that occurred in Groningen, the Netherlands.

Introduction

In the last years, induced seismicity in the northern part of Europe has considerably increased. At the same time, the existing building stock was not designed for seismic loading and therefore there are events in which damages in buildings have been observed. For example, in Switzerland magnitudes of up to 3.5 Richter have been reported resulting in hairline cracks (associated with joints sliding and opening) on walls alongside with nonstructural damages (Abbiati et al., 2018). Such induced seismicity events resulted in damage claims with an approximated cost of 7-20 million Swiss Francs.

Unreinforced masonry (URM) is brittle anisotropic material that responds to cyclic load reversals in a non-ductile way. Evidence from past earthquakes demonstrates that large amplitude loads can cause partial or total collapse. The response of URM to recursive, frequent but low-amplitude seismic loads, is a relatively new topic that needs experimental and analytical validation to further understand how masonry structures behave under such conditions. The URM buildings in Groningen, the Netherlands, have been subjected to low-amplitude load reversals in the past years, especially in the very last decade, due to the induced-seismicity earthquakes caused by gas extraction. These earthquakes have a significant impact on the URM buildings that were not designed according to any seismic design criteria and are characterized by very slender cavity walls, absence of reinforcement, and little cooperation between walls and floors. Accumulation of damage under small and recursive earthquake motions becomes an important issue in case of induced seismicity, mainly because after the strongest shakings a damage-claim procedure is triggered, in which engineers are asked to judge whether the present cracks and damages are caused by the specific earthquake or not (Dais et al., 2019). Already inherently difficult, the evaluation becomes cumbersome due to the ambiguity of the relevance to the previously existing damages.

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In more detail, the seismicity of Groningen is characterized by low in acceleration repetitive earthquakes that are associated with the gas extraction that is taking place in the region. The Huizinge event of 2012 exhibited the maximum magnitude (3.6 ML) while the maximum peak ground acceleration (PGA) recorded was 0.11g during the Zeerijp event of 2018 (3.4 ML) (Bal, Dais, et al., 2018; Bal, Smyrou, et al., 2018). Even though these magnitudes are characterized as low to medium, they have raised an extensive number of damage claims. The seismic activity of the region is characterised by approximately 15 records per year of magnitude greater than 1.5. These frequent, but mainly low in amplitude, seismic excitations cannot be overlooked.

Therefore, in the past years, attention has been pointed to experimentally investigate the impact of the seismic activity on the structures of the region. Godio et al., (2018) and Korswagen et al., (2019) showed that the load history plays a role in the response of URM structures under recursive load and the key parameters which characterise damage are the drift ratio and the extent of cracks appearing in the structure. In particular, from experimental studies on full scale masonry wall panels carried out by Godio et al., (2018), it was shown that the ultimate drift of walls subjected to monotonic loading is approximately twice as large as the ultimate drift of walls subjected to cyclic loading. Graziotti et al., (2017) performed shaking table tests on a two-storey full scale URM structure, typical example of the building stock found in Groningen. The specimen was subjected to a sequence of incremental dynamic tests. Damage limits were defined based on the experimental findings. In particular, the first damage limit, which corresponds to no damage, was associated with a maximum inter-storey drift of 0.07% at the first floor level. The second damage limit, which accounts for minor damage/slight structural damage, was set for the maximum inter-storey drift at 0.12% at the first floor level. Even though the residual inter-storey drifts remain zero until the second damage limit is reached, cracks have already been formed along the tested structure. Consequently, quantification of damage solely based on the residual drift would oversee the sustained damage (Sarhosis et al., 2019). Therefore, a robust methodology for the quantification of the damage in URM structures needs to be developed which takes into consideration the initiation and propagation of damage during the earthquake excitations.

Since induced seismicity is characterized by high recurrence of seismic events, it is highly possible that numerous properties need to be inspected multiple times to trace damage and its development along the various events in limited time between different earthquakes. Researchers have been working on the development of new vision-based techniques that can rapidly inspect structures and track any signs of degradation by detecting cracks (Baqersad et al., 2017; Cha et al., 2017). Nevertheless, these techniques are not yet ready to offer automatic assessment of a structure; still highly skilled personnel is required to investigate the detected damage and classify it to different damage levels. Consequently, there is a necessity of an automatic procedure that will quantify the detected damage from the new-developed technologies; thus, expanding further their usability and efficacy.

During the inspection of the structure only the residual damage can be observed and assessed. For the majority of the structures, which are not covered by a monitoring scheme, the structural response during the seismic excitation in terms of maximum drift, maximum crack length and width, can be estimated only after numerical analysis and a decision for the condition of the investigated building can be based on residual damage indicators.

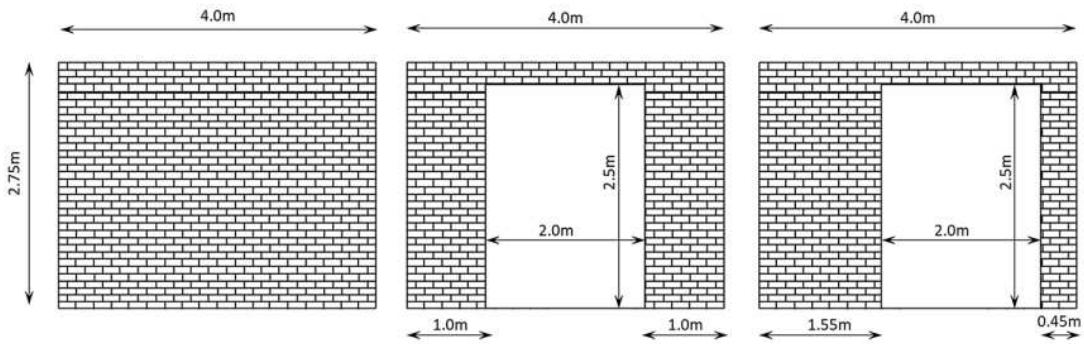
Different studies have focused on the quantification of the development of damage caused by small to moderate seismicity. In particular, Abbiati et al. (2018) undertook an experimental campaign attempting to produce a probabilistic model for quantifying plaster cracks on URM structures due to induced seismicity. The von Mises strain fields of plaster were obtained by implementing a Digital Image Correlation (DIC) algorithm. In the output pictures of the DIC technique, the regions of higher brightness were labeled as cracked regions and then two different damage scores were defined based on the width and the length of the regions labeled as cracked. The damage scores proposed by Abbiati et al. (2018) seem to capture well the accumulated damage. Nonetheless, the method suggested to obtain the crack regions requires a cumbersome procedure (surface preparation, lighting conditions, DIC algorithms etc.) which limits its application to laboratory conditions. Furthermore, it is hard to correlate the experimental damage score which is based on von Mises strains to findings from numerical models.

Furthermore, Korswagen et al., (2019) carried out an experimental campaign in order to quantify light damage on URM structures due to repeated horizontal load. DIC methods were utilized to precisely capture the initiation and propagation of cracks during the experiments. A dimensionless damage parameter was proposed based on the number of cracks, their width and length. According to Korswagen et al., (2019), the produced equation defining the damage parameter is applicable for the limited cases treated in the study but its potential to serve as a damage indicator for a wider range of cases needs to be investigated further.

The aim of this paper is to understand the accumulation of damage in URM structures subjected to induced seismicity. A numerical model has been developed to simulate masonry wall panels with and without openings in representative common typologies of house façades of buildings in the Groningen gas field, the Netherlands. The model was based on DEM. DEM was initially developed by Cundall (1971) to model rock engineering problems in which continuity between separate blocks of rock did not exist. The approach was later applied to model the mechanical behaviour of masonry structures in which the failure mechanism is governed primarily by the masonry unit to mortar interface characteristics (Sarhosis & Sheng, 2014). In the present study, numerical results were validated based on experimental findings obtained from the literature (Graziotti et al., 2017). A damage equation was proposed herein, where the drift, the length of joints opened and the length of joints at shear limit were included to evaluate the extent of damage in the structure subjected to harmonic load excitations. The damage quantification was applicable for low bond strength masonry and in particular for masonry structures, in which opening and slip along the mortar joints is the predominant failure mechanism.

Development of the numerical model based on the DEM

A two dimensional numerical model based on DEM has been developed to estimate and understand the extent of damage accumulation in masonry walls with and without openings subjected to induced seismicity events. All walls had dimensions equal to 4 m width and 2.75 m height, typical in the Dutch construction practice. An opening was allowed to be either symmetric or asymmetric (Figure 1b & c). The size of the opening was set to 2 m by 2.5 m. The vertical sides of the wall were free. In addition, the top of the wall was also free to rotate creating a cantilever condition. A vertical pre-compression equal to 0.3 MPa was applied in the wall during the loading. Figure 1 shows the geometry of the models developed. Each brick of the masonry wall panel was represented by deformable block separated by zero thickness interfaces at each mortar bed and perpendicular joint. To allow for the 10 mm thick mortar joints in the real wall panels, each deformable block was based on the nominal brick size increased by 5 mm in each face direction.



(a) (b) (c)

Figure 1: The geometry of the models developed: (a) masonry wall panel with no opening/solid wall; (b) wall panel with symmetric opening; (c) wall panel with asymmetric opening.

Masonry units represented as deformable blocks behaving in a linear elastic manner. Deformable blocks were internally discretised into finite difference triangular zones and each element responds according to a prescribed linear or non-linear stress–strain law. These zones are continuum elements as defined in the finite element method (FEM). Material properties assigned for the masonry units are: i) the density (d), ii) the shear modulus (G), and iii) the bulk modulus (K). The mortar joints were represented by zero-thickness interfaces between the masonry units. These interfaces can be viewed as interactions between the blocks and are governed by appropriate stress-displacement constitutive laws. The interaction between the blocks is represented either by sets of point contacts or by sets of edge-to-edge contacts, with no attempt to obtain a continuous stress distribution through the contact surface (Sarhosis et al., 2015). The mechanical interaction between the blocks was simulated at the contacts by spring like joints with normal and shear stiffness. In particular, mortar joints were assumed to behave in an elastic-perfectly plastic Coulomb slip-joint area contact option. This provides a linear representation of the mortar joint stiffness and a yield limit based upon elastic normal (JKn) and shear (JKs) stiffness, frictional (Jfric), cohesive (Jcoh) and tensile (Jten) strengths, characteristics of the mortar joints. In addition, if the bond tensile strength or shear strength is exceeded in the numerical calculation, then the tensile strength and cohesion are reduced to zero. In the developed computational model, the material properties for the interface were calibrated based on the experimental test results obtained from Graziotti et al. (2017) as shown in Table 1. For the masonry units, the bulk modulus was 2.9 x 10^9 N/m^2 and the shear modulus 2.16 x 10^9 N/m^2. For further information regarding the development of the computational model, the reader is referred to Sarhosis et al., (2019).

Joint normal stiffness JKn (N/m^3)	Joint shear stiffness JKs (N/m^3)	Joint friction angle Jfric (deg)	Joint tensile strength Jten (N/m^2)	Joint cohesive strength Jcoh (N/m^2)
12 x 10^{11}	4 x 10^{11}	32	0.21 x 10^6	0.21 x 10^6

Table 1: Material properties of the zero-thickness interface which was used in the numerical model.

Damage index equation and results obtained

A damage index (DI) equation (eq. 1) has been developed to quantify damage accumulation on URM structures subjected to earthquake excitations. The DI equation is a function of three key damage parameters (eq. 2 to eq. 4) including: a) the extent of drift (DI_{drift}); b) length of joints opened (DI_{open}); and c) length of joints at slipping/shear failure (DI_{slip}).

$$DI = 1 – (1 - DI_{drift}) * (1 - DI_{open}) * (1 - DI_{slip}) \quad (1)$$

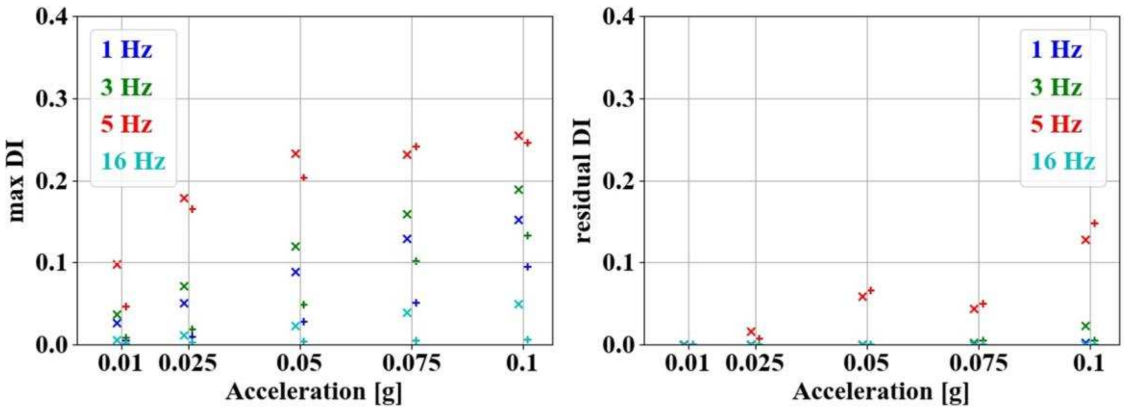
$$DI_{drift} = \frac{\delta}{\delta_{limit}} \quad (2)$$

$$DI_{open} = \frac{\text{Length of joints opened}}{\text{Total length of joints}} \quad (3)$$

$$DI_{slip} = \frac{\text{Length of joints slipped}}{\text{Total length of joints}} \quad (4)$$

where, DI_{drift} accounts for the damage associated to the attained value of drift (δ) divided by a limit value (δ_{limit}) which corresponds to the near collapse state of the modeled structures and was assumed as 2% to denote complete failure of the wall (as per EC8 or NTC 2008). DI_{open} and DI_{slip} account for the length of joints that opened and the length of joints slipped (or at shear limit) respectively, divided by the total length of joints in each masonry wall panel. Thus, DI_{open} and DI_{slip} are normalized metrics of the cracks along the surface of the wall and are independent of the dimensions of the wall. According to Giardina (2013) cracks greater than 0.1mm are visible to the naked eye. Therefore, in the numerical model, cracks which are greater or equal to 0.1mm have been recorded. The value of the suggested DI and the individual components DI_{drift}, DI_{open}, and DI_{slip} ranges from 0 (indicating no damage) to 1 (indicating total damage/catastrophic collapse in which each joint fails).

Using the DI (eq. 1), estimates of the damage occurred in the masonry wall panels subjected to different in amplitude and frequency harmonic loading, were estimated. From the results analysis, it was shown that, there is very small damage in the solid wall when subjected to low amplitude harmonic loadings and thus these results were not presented in this paper. Also, Figure 2 presents the maximum and residual DI values as obtained from the numerical analyses of the masonry walls subjected to harmonic loading with varying acceleration amplitude (0.01, 0.025, 0.05, 0.075 and 0.1g) and frequency content (1, 3, 5, 16Hz). From 2a, when the applied to the wall frequency is equal to 5Hz which is the same as the natural frequency of the wall (i.e. resonance condition), the DI reaches substantially higher values in comparison to the cases when there is no resonance. Furthermore, from Figure 2b, the residual DI is negligible when the frequency of the harmonic load diverges from the natural frequency of the wall.



(a) (b)

Figure 2: Numerical results of the maximum (a) and residual (b) DI in the masonry walls with respect to the applied amplitude of acceleration and frequency. The results for the symmetric and asymmetric models are shown as *'+' and 'x'* accordingly.

Figure 3 shows the maximum and residual DI_{drift}, DI_{open}, DI_{slip} and total DI for the case of the symmetric wall. The results derived from the asymmetric wall yielded similar behaviour as the symmetric wall and are not presented herein. From Figure 3, when the wall was subjected to a harmonic load with frequency equal to 1Hz and low amplitude of acceleration (i.e. 0.010g, 0.025g and 0.050g), the maximum DI was moderate. However, when the wall was subjected to a harmonic load with frequency content equal to 1 Hz and corresponding high acceleration values (i.e. 0.075 and 0.1g) a high DI observed. Also, from Figure 3, when the wall subjected to a harmonic load with frequency equal to 3Hz, the estimated maximum DI found to be similar to what was observed when the wall was subjected to a frequency equal to 1Hz.

Moreover, when the wall subjected to a harmonic load with frequency equal to 5Hz, resonance was attained between the natural frequency of the walls and the imposed signal. In this particular case and as shown from Figure 3a, substantially higher values of DI were recorded with respect to the DI obtained when the wall subjected to harmonic load with frequencies equal to 1, 3 and 16Hz.

When the wall subjected to a harmonic load with acceleration amplitude equal to 0.01g, the components DI_{drift} and DI_{slip} seem to equally contribute to the DI, while the inflicted damage seems to be cancelled out at the end of the analysis, i.e. residual DI. Also, from Figure 3a, it was observed that the DI increases linearly with an increase of the amplitude of acceleration applied to the wall. Nevertheless, from Figure 3b, the residual DI does not follow a similar trend. In particular, when the wall was subjected to accelerations equal to 0.025g, residual damage was very small. The residual DI was mainly attributed to joints slipping (DI_{slip}) while the drift component seems to be cancelled out when there is no imposed load. Similar results were also presented by Graziotti et al. (2017) where after applying excitations of low amplitude, the residual drift reaches zero but cracks could still be observed in the wall.

Moreover, from Figure 3a it is observed that the DI fluctuates. This is realized by the fact that during earthquake excitations, cracks can open and close. Therefore, when quantifying damage in masonry walls, it is important to consider not only the residual damage index but the maximum damage index which the structure sustained during the seismic excitation. This effect is also illustrated in the experiments carried out by (Abbiati et al., 2018). Particularly, a difference is found between the DI corresponding to the end of each cycle (residual) and the DI at the epoch that the maximum displacement of each cycle is attained. In more detail, for the first cycles with low amplitude of drift even though some damage could be observed during the peak drift, no residual damage could be recorded at the end of the cycles and the residual DI was zero. In particular, from Figure 3a, when the wall subjected to a harmonic load with frequency equal to 5Hz and acceleration amplitudes equal to 0.05g and 0.075g, the ratio of the residual to the maximum DI was about 20% to 30% accordingly. However, when the harmonic load acceleration applied to the wall was equal to 0.1g, this ratio exceeds 50%.

Furthermore, from Figure 3 it is highlighted that the maximum DI mostly derives from the DI_{drift}, and DI_{slip} components while the contribution of DI_{open} is rather limited. This behaviour can be attributed to the expected failure modes. In more detail, limited rocking is expected at the base of the modeled walls, given the assumed boundary conditions, that is cantilever. Therefore, the length of joints that opened is limited. On the other hand, the cracks attributed to joints slipping follow a zigzag pattern that activates the onset of failure of a greater length of joints.

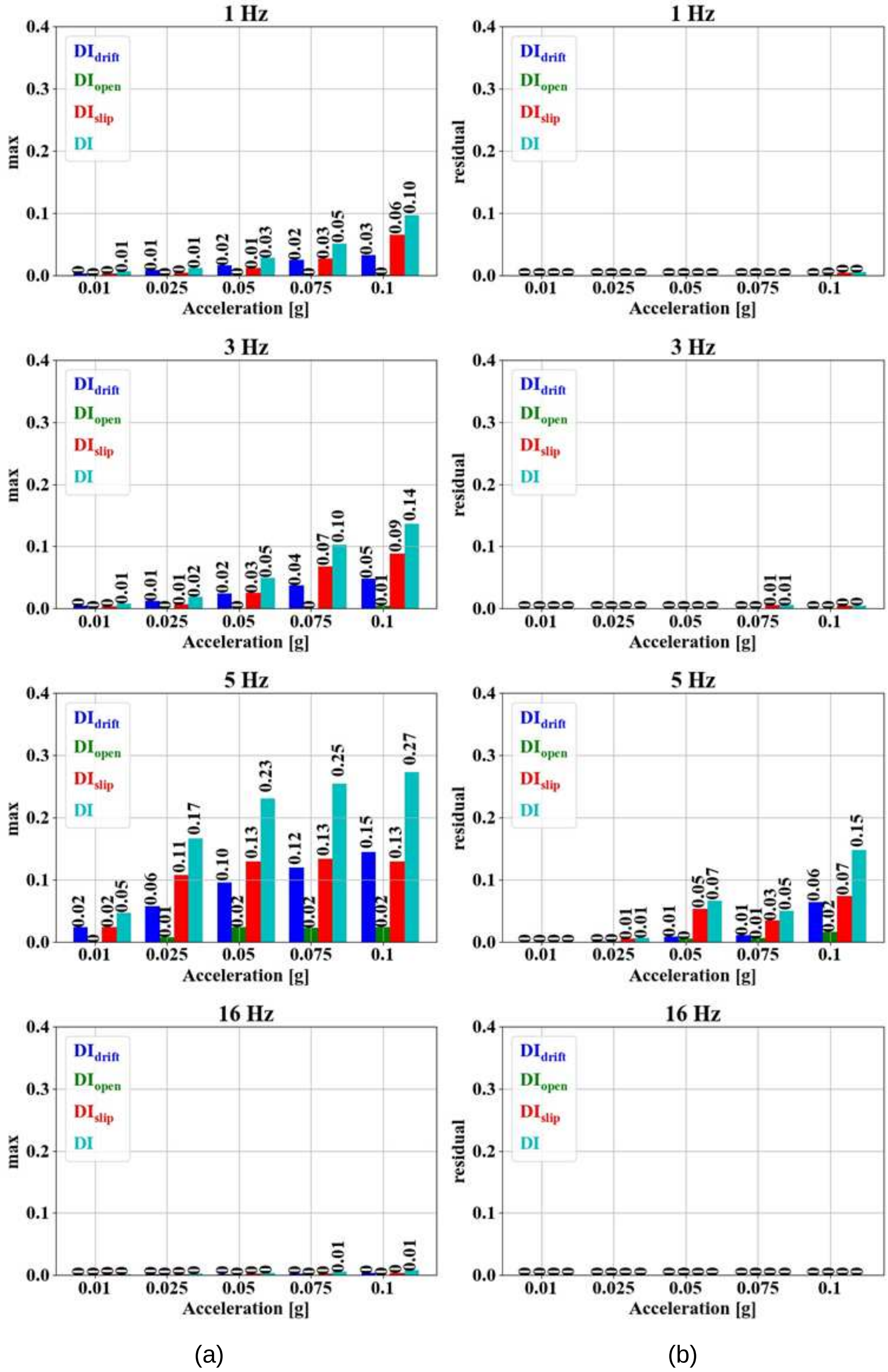
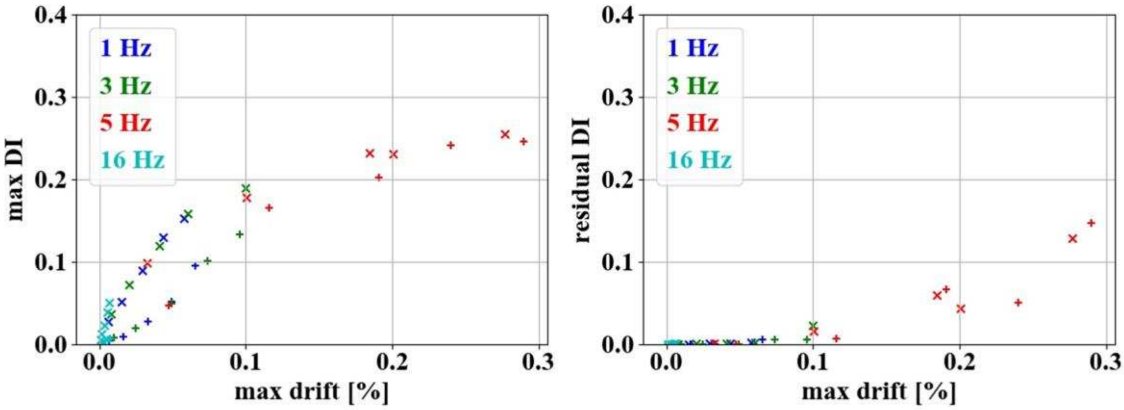


Figure 3: Maximum (a) and residual (b) D_{drift}, D_{open}, D_{slip} and DI presented for the case of the wall with the symmetric opening. Each graph corresponds to a different value of frequency (1, 3, 5 and 16Hz).

Figure 4 presents the maximum and residual DI attained during the numerical analyses with respect to the maximum drift obtained after the application of the harmonic load. A linear relationship between the maximum drift and the maximum DI was observed. On the other hand, the residual DI is zero until a minimum value of drift is reached. This drift threshold can be assumed to be 0.05% for the walls modeled herein; for drift above this value some residual damage can be expected. Similar findings observed by Abbiati et al. (2018) where a linear relationship was extracted between the maximum top displacement and the developed damage scores. Nevertheless, a closer look in the obtained results of Abbiati et al. (2018) shows that for a displacement up to approximately 0.08% the damage scores are zero which stands for “no crack regions”. After the threshold value of horizontal displacement exceeded, an abrupt increase of the damage scores was observed, which relates to the brittle behaviour of the URM walls. This behaviour can be correlated with the results shown in Figure 4b where the residual DI is expressed with respect to the maximum obtained drift.



(a) (b)

Figure 4: Maximum (a) and residual (b) DI attained during the numerical analyses are diplayed with respect to the maximum drift. The results for the walls with symmetric and asymmetric openings are noted with *‘+’* and *‘x’* markers accordingly.

Figure 5 shows the evolution of DI over time for the URM wall with a symmetric opening when subjected to acceleration amplitudes equal to 0.025g and 0.1g and frequencies 3Hz and 5Hz. Such accelerations correspond to low and moderate acceleration amplitude accordingly. As explained above, a frequency of 5Hz matches the natural frequency of the wall while the frequency 3Hz is slightly lower than the resonance frequency. The comparison of the response for the different frequencies explains why even slight difference between the frequency of the signal and the natural frequency can significantly alter the response of the structure. More specifically, when the wall is subjected to a frequency equal to 5Hz, the achieved maximum DI increases for every cycle of the harmonic signal for both acceleration amplitudes (see Figure 5b). On the contrary, when the wall is subjected to frequency equal to 3Hz and acceleration amplitude equal to 0.025g, the inflicted damage is negligible. When the wall is subjected to 3Hz and acceleration amplitude equal to 0.1g, the maximum value of DI is attained during the first cycle of the excitation and in the upcoming cycles, the same value of DI is reached but it is not exceeded.

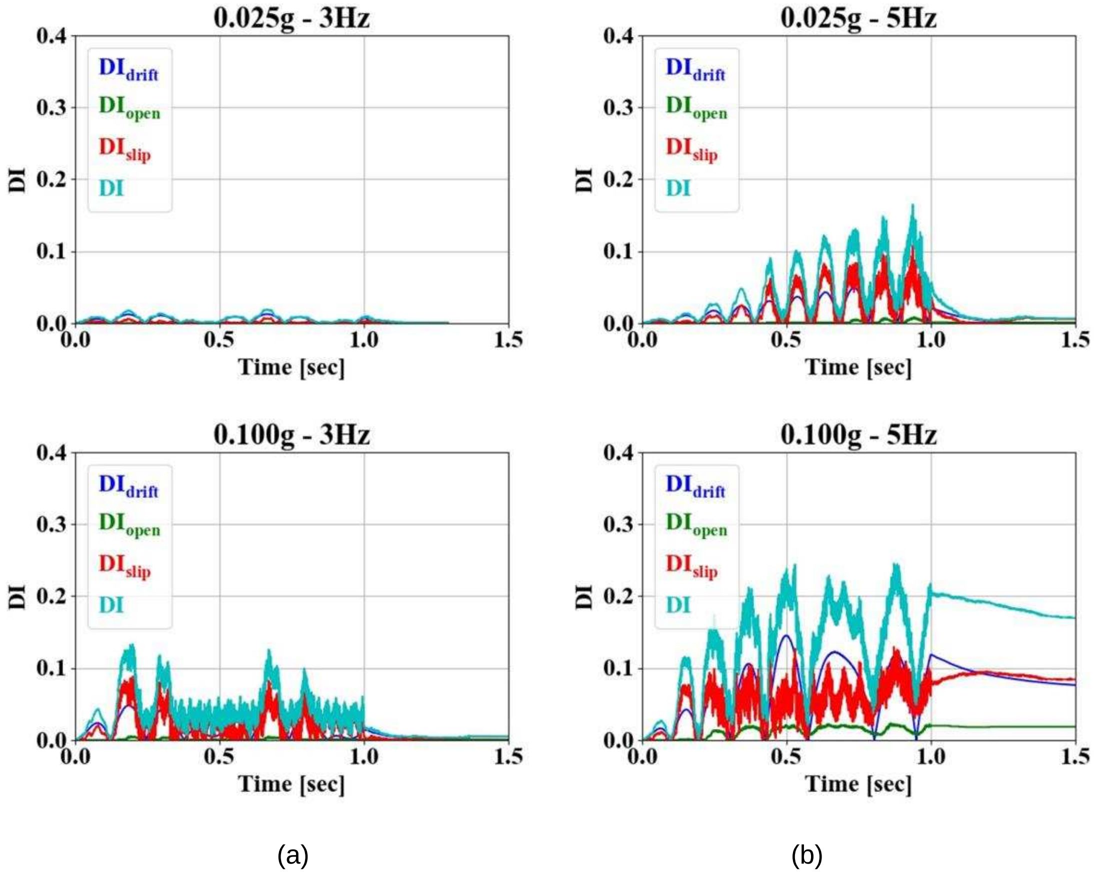


Figure 5: The evolution of DI and its components for the case of the symmetric model for acceleration amplitudes 0.025 and 0.1g and frequencies 3 (a) and 5Hz (b).

Conclusions

Induced seismicity in north Europe and in particular in Groningen, the Netherlands, has significantly increased over the last few years, despite the recent reduction of the annually extracted gas. The existing urban infrastructure was not designed for such seismic loads and needs to be assessed in terms of its vulnerability to cope with such loading regimes. In particular, the Dutch building stock is characterised by slender walls with limited connection between the floors and the restraining walls. Numerical models are required to be developed to assess Dutch unreinforced masonry buildings and evaluate the damage accumulation to the induced seismicity. The aim of this paper was to quantify the cumulative damage on unreinforced masonry subjected to induced seismicity. A numerical model, based on the DEM of analysis, has been developed to simulate masonry wall panels with and without openings to represent common typologies of house façade of buildings in Groningen region. The numerical model has been validated against experimental data reported in the literature. A DI equation has been proposed which includes as damage parameters the following: a) the length of joints opened; b) the length of joints at shear limit; and c) the relative drift of the wall. Different in geometry masonry wall panels with openings were considered. The walls were subjected to harmonic loadings with different acceleration amplitudes and frequency contents. From the implementation of the proposed methodology it was shown:

- The suitability of the DI equation to realistically represent the level of damage and its sensitivity to low amplitude loading was highlighted using the results of the developed using the DEM numerical model.
- The frequency content of a harmonic excitation was found to be critical for the extent of damage in the wall panel. When no resonance between the wall panel and the harmonic signal takes place, the expected damage is limited even for excitations of moderate acceleration amplitude. On the other hand, when the wall panel was harmonically excited with its natural frequency, there is potential for damage even for low acceleration

amplitude. The residual DI obtained was relatively small when the frequency of the harmonic load diverges from the natural period of the wall.

- Even for zero residual drift, the estimated residual DI of the wall may indicate damage due to the existence of cracks in the wall.
- The maximum DI mostly derives from the DI_{drift}, and DI_{slip} components while the contribution of DI_{open} is rather limited. On the other hand, at the onset of residual damage, the residual DI is mainly attributed to the joints sliding (DI_{slip}) while the drift component seems to relatively small in the case that there is no imposed load applied on the wall.
- When the wall subjected to harmonic loading, cracks opened and closed. Therefore, for the quantification of damage in the masonry wall panels, it is suggested to evaluate both the residual and the maximum damage index.

In the future, further research will be carried out to establish a correlation between the maximum damage during a seismic event and the residual damage that the URM can sustain.

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