Dynamic behaviour of a concrete building under a mainshock—aftershock seismic sequence with a concrete damage plasticity material model

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ABSTRACT
The aim of this paper was to investigate the dynamic response of a concrete structure subjected to a mainshock—aftershock seismic sequence. In the dynamic analysis, three components of the registered mainshock and aftershock were taken into account. The peak ground accelerations of about 0.5 g were assumed for both shocks. A one-storey shed was modelled with the ABAQUS software to represent a large concrete structure under the repeated earthquakes. For proper characterization of concrete structure behaviour under the sequence of shocks, a concrete damage plasticity model was assumed as a constitutive model of concrete. The obtained results indicate that aftershocks can have considerable effect on dynamic behaviour of concrete structures in terms of enlarging zones affected by irreversible strains or additional damage evolution. The analysis revealed that aftershocks, which are usually not as strong as mainshocks, may result even in total loss of concrete material strength while performing in mainshock—aftershock seismic sequences.

1. Introduction
Large seismic mainshocks are usually followed by numerous aftershocks within a short period of time. Aftershocks can have a significant effect on the dynamic behaviour of a structure in terms of irreversible plastic strains and accumulated damage, as they affect a structure already weakened during a mainshock.

The dynamic response of structures subjected to a mainshock—aftershock seismic sequence has been extensively studied recently. Representative examples of damages of buildings, observed under multiple earthquakes, can be found in current studies (Abdelnaby 2012; Abdelnaby & Elnashai 2012). Especially, concrete structures have the probability of severe damage while exposed to a sequence of seismic events (Massone et al. 2014; Zhai et al. 2015).

Structures may go plastic or even collapse during aftershocks, since they are already degraded and cracked during mainshocks. Only advanced constitutive models of a concrete material, which include damage and failure phenomena, are advisable for examining the dynamic behaviour of concrete structures under mainshock—aftershock sequences.

In the paper, the dynamic responses of a concrete building subjected to a mainshock—aftershock seismic sequence are investigated. In order to compare damages under repeated earthquakes and to
assess the impact of both shocks on the analyzed structure, the concrete damage plasticity (CDP) model, which describes multi-hardening plasticity and damage (cracking), was implemented.

2. A numerical model of the analyzed concrete structure

The dynamic analysis of a concrete structure under multiple earthquakes was carried out for a one-story engine shed located in an industrial area. A three-dimensional numerical model, created with the ABAQUS package (Simulia 2013), is presented in figure 1 along with points chosen for further dynamic analysis.

The main dimensions of the structure are: the length — 19 m, the width — 14 m and the height — 15 m. The structural walls (20 cm thick) of the shed are built of a concrete material. The windows of the object are strengthened by concrete heads. The structural roof system consists of steel trusses. The roof is covered by a steel sheeting.

3. Constitutive parameters for the concrete damaged plasticity model

To represent the elastic—plastic behaviour of the concrete engine shed under a sequence of seismic shocks, the CDP model was assumed as a constitutive model of concrete (Lubliner et al. 1989; Lee & Fenves 1998; Simulia 2013). The model is specially recommended for calculations of concrete structures subjected to dynamic loadings, like earthquakes (Simulia 2013, Dulinska & Jasinska 2014).

The CDP model consists of the combination of nonassociated multi-hardening plasticity and scalar damaged elasticity to describe the irreversible damage that occurs during the fracturing process. The yield surface is controlled by two variables representing equivalent plastic strains: $\bar{\varepsilon}_{pl}^t$ and $\bar{\varepsilon}_{pl}^c$, associated with failure mechanisms under tension and compression loading, respectively. The concrete damage is characterized by two different damage variables: $d_t$ (for tension) and $d_c$ (for compression). These variables are functions of equivalent plastic strains. They can take values from zero, which represents undamaged material, to one, which denotes total loss of strength. The CDP model assumes that the reduction of the elastic modulus $E$ is given in terms of a scalar degradation variable.
$d$ (function of the stress state and the uniaxial damage variables $d_t$ and $d_c$) as:

$$E = (1 - d) E_0$$  \hspace{1cm} (1)$$

where $E_0$ is the initial elasticity modulus of the undamaged material.

The essential constitutive parameters of the CDP model are summarized in table 1 (Schlangen 1993; Jankowiak & Lodygowski 2005). These properties were obtained through laboratory tests.

Other parameters of the concrete material were taken as follows: the elasticity modulus 34.0 GPa, the Poisson’s ratio 0.19, the dilation angle 38° and the mass density 2400 kg/m$^3$.

4. Data of the seismic mainshock and aftershock

In the dynamic analysis of the concrete structure, two earthquakes registered in Nocera Umbra (central Italy) were applied as kinematic excitations of the structure (ITACA 2015). The magnitude of the mainshock was 6.1, whereas the magnitude of the aftershock equalled 5.7. The phase of the strong ground motion of both shocks approximately lasted for 8 s. Time histories of accelerations of the mainshock and the aftershock in three directions are presented in figures 2 and 3, respectively. The peak ground accelerations (PGAs) of both shocks are summarized in table 2.

5. The dynamic response of the structure to the mainshock–aftershock sequence

The dynamic response of the concrete structure to both shocks was calculated using full time history analysis. It was carried out with the Hilber-Hughes-Taylor time integration algorithm provided by ABAQUS for a direct step-by-step solution. As the damage and failure model of concrete implements strong material nonlinearity, a step of numerical integration was not fixed. The step varied from $10^{-5}$ to $10^{-2}$ s, according to convergence requirements. The geometric nonlinearity of the problem was also taken into account. The time shift between the mainshock and the aftershock equalled 10 s, that allowed the complete unloading of the structure.

For the dynamic analysis, the Rayleigh model of mass and stiffness proportional damping was applied. The damping coefficients $\alpha = 0.466$ (referring to mass proportional damping) and $\beta = 0.005$ (referring to stiffness proportional damping) were determined for damping ratios of 3.5% for the first (4.3 Hz) and the second (5.1 Hz) natural frequency. Damping ratio values adopted for reinforced concrete buildings, currently recommended by Standards in different countries, are located in the range of 2%–5%. The damping ratio of 3.5% is located in the middle of this range. Such
adoption leads to more conservative results (larger displacements, plastic measures) than applying commonly adopted higher value of 5%.

The dynamic analysis was performed for the selected mainshock—aftershock sequence. The observed evolution of plastic and damage measures, incorporated into the CDP model of the material, allowed to assess the impact of both shocks on the structure.

The distribution of plastic zones in the concrete wall resulted from the mainshock and the aftershock is presented in figures 4(a) and (b), whereas the evolution of the tensile damage is shown in figures 5(a) and (b), respectively.

![Figure 2. Time histories of the mainshock accelerations: (a) west–east direction, (b) north–south direction and (c) vertical direction.](image)

|          | PGA (cm/s²) |
|----------|-------------|
|          | WE direction | NS direction | Vertical direction |
| Mainshock| 415         | 492          | 398               |
| Aftershock| 263         | 486          | 143               |

Table 2. The peak ground accelerations (PGAs) of the mainshock and the aftershock.
It can be noticed that plasticization of the upper parts of pillars between windows was reported already after the mainshock (figure 4(a)). The aftershock resulted only in a slight growth of the area affected by the plastic strains (see points B and D). The level of plastic strains at some points increased considerably (see point A) after the second seismic event. However, at the vast majority of analyzed points, the level of plastic strains did not undergo noticeable changes (see point C).

It also occurred that the tensile damage due to the mainshock affected mainly the upper parts of pillars between windows (figure 5(a)). The subsequent shock caused additional cracking in zones which did not undergo any damages during the first shock. The value of tensile damage variable reached 0.9 that indicates almost total loss of strength.

To present the plastic behaviour and the damage of the concrete material of the structure under the mainshock and the aftershock, a detailed analysis of the dynamic behaviour of the concrete material at particular zones of the structure is presented in figures 6–9.

Figure 3. Time histories of the aftershock accelerations: (a) west–east direction, (b) north–south direction and (c) vertical direction.
Figure 4. Distribution of plastic strain magnitude (PEMAG) on the front wall resulted from: (a) the mainshock, (b) the aftershock.

Figure 5. Distribution of tensile damage parameter $d_t$ (DAMAGET) of the front wall due to: (a) the mainshock, (b) the aftershock.
The following time histories of plastic and damage measures were examined: logarithmic maximal principal strains ($\varepsilon_{\text{max}}^{\text{princ}}$), equivalent plastic strains ($\varepsilon_{\text{c}}^{\text{pl}}$) and tensile damage parameter ($d_t$). Logarithmic strain shows the dependence of total strain on time. For geometrically nonlinear analysis, logarithmic strain ($\varepsilon$) is the default strain measure provided by ABAQUS. It takes into account the continuous variation of length. Equivalent plastic strain ($\varepsilon_{\text{c}}^{\text{pl}}$), in case of the CDP model, is a measure of plastic behaviour under compression. This parameter controls the evolution of the crushing surface, linked to failure mechanism under compression loading. Finally, tensile damage variable ($d_t$)
Figure 8. Time history of plastic ($\varepsilon_{\text{max}}^{\text{princ}}$, $\varepsilon_{\text{pl}}$) and tensile damage ($d_t$) measures of the concrete material at point D due to the main-shock and the aftershock.

Figure 9. Time history of plastic ($\varepsilon_{\text{max}}^{\text{princ}}$, $\varepsilon_{\text{pl}}$) and tensile damage ($d_t$) measures of the concrete material at point B due to the main-shock and the aftershock.
can be used as the indicator of the loss of strength: it can take values from zero, representing the undamaged material, to one, which represents total loss of strength.

On the basis of the presented time histories, some remarks concerning plastic behaviour, damage and failure in different zones of the structure can be formulated. Four main scenarios of the dynamic behaviour may be distinguished, depending on the locations of the analysed points.

The first scenario of the dynamic behaviour could be observed on the time histories of plastic and damage measures obtained at point C (figure 6). This point was located in the area between two windows. It occurred that the concrete material at point C went plastic after approximately 3 s of the mainshock (see figure 6, $\varepsilon_{\text{max}}^{\text{pl}}$ and $\ddot{e}_c^{\text{pl}}$ charts). The process of plasticization at this point was accompanied by tensile damage of the concrete material (see figure 6, $d_t$ chart). The aftershock did not influence plastic and failure measures at point C. The level of $\ddot{e}_c^{\text{pl}}$ and $d_t$ variables was reported constant, whereas the $\varepsilon_{\text{max}}^{\text{princ}}$ values oscillated around the value it had reached in the final phase of the mainshock.

The second scenario of dynamic behaviour may be detected from the time histories of plastic and damage measures obtained at point A (figure 7). Point A was placed just beneath the head of the window. It can easily be noticed that both the mainshock and the aftershock resulted in plastic behaviour of that part of the wall. The increase of the plastic strains (see figure 7, $\varepsilon_{\text{max}}^{\text{princ}}$ and $\ddot{e}_c^{\text{pl}}$ charts) appeared at the beginning of the strong phase of both shocks. However, damage (cracking) of the concrete material can be observed during the main event only (see figure 7, $d_t$ chart).

The third possible situation can be observed on the time histories of plastic and damage measures obtained at point D (figure 8). This point, like point A, was located just beneath the head of the window. It turned out that the increase of the plastic strains (see figure 8, $\varepsilon_{\text{max}}^{\text{princ}}$ and $\ddot{e}_c^{\text{pl}}$ charts) appeared at the beginning of the strong phase of both shocks. Also, the tensile damage time history indicated cracking at the same time (see figure 8, $d_t$ chart).

The last analyzed scenario of the dynamic behaviour may be clearly recognized from the time histories of plastic and damage measures obtained at point B (figure 9). This point was situated beneath the upper window (with no window head). It turned out that the mainshock resulted only in the elastic behaviour of that part of the front wall. However, this zone underwent plasticization during the aftershock, which was applied to the structure already weakened by the mainshock (see figure 9, $\varepsilon_{\text{max}}^{\text{princ}}$ and $\ddot{e}_c^{\text{pl}}$ charts). In addition, new cracks appeared near point B (see figure 9, $d_t$ chart), which was located on the trajectory of maximal principal stresses.

It must be pointed out that the presented distribution of plastic strains and damage areas of the building’s walls, which were obtained by the numerical analysis due to both shocks, bear resemblance to real damages of structures observed in consequence of replicate earthquakes (Abdelnaby 2012, Abdelnaby & Elnashai 2012). It validates the numerical results which is an important stage of investigation.

6. Conclusions

The following conclusions can be formulated on the basis of the dynamic analysis of the concrete engine shed subjected to the mainshock— aftershock seismic sequence:

The comparison of plastic and damage measures, increasing after subsequent seismic shocks, revealed that several scenarios can happen due to repeatability of seismic events. Both, the first stronger or the second lighter shock, can have crucial influence on plastic behaviour and progressive damage of concrete structures.

The obtained results indicate that aftershocks can have considerable effect on the dynamic behaviour of concrete structures in terms of enlarging zones affected with plastic irreversible strains. The building may also exhibit additional progress in structural damage evolution such as shear wall cracking.

Aftershocks are usually not so strong as mainshocks. Seismic shocks of similar size and magnitude would not cause any fracturing or failure if they strike an undamaged structure. But even being
not large events, they may result in total loss of concrete material strength while performing in mainshock—aftershock seismic sequences.

**Disclosure statement**

No potential conflict of interest was reported by the authors

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