Monitoring systems for damage detection of RC buildings in seismic area.

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ABSTRACT
The damage detection using a monitoring system based on the dynamic identification is a very promising procedure because of the possibility of correlating the modal parameters with the severity and the location of the damage.
In particular, the comparison between the dynamic experimental response of a structure and a reliable model can be an efficient method of analysis to assess the safety, integrity and performance of a structure identifying damage after an earthquake.
In this paper a sensitivity analysis of the main vibration-based damage features is proposed in order to establish their ability to detect damage of reinforced concrete framed structures due to earthquakes. The behaviour of a progressively damaged structure is numerically simulated through nonlinear analyses adopting a lumped plasticity model. The procedure is applied to two different RC frames: one designed only for vertical loads and another designed for seismic actions according to Eurocode 8 and Italian Code (NTC2018).The variation of period and frequency of vibration, the mode shapes and the interstorey drift are discussed as damage parameters.

1 INTRODUCTION

In the last decades the structural health monitoring (SHM) systems based on the dynamic identification consolidated their high potentiality for providing information on the state of a structure during the time or after an earthquake. However the efficiency of the dynamic identification requires the support of suitable assessed numerical models especially when damage detection has to be carried out. The aim of this paper is to evaluate the differences in the dynamic responses of framed buildings damaged by seismic actions, considering, in particular, frames designed to withstand only gravity loads or to resist seismic actions according to provisions of Italian (NTC2018) and Eurocode 8 (EC8, EN 1998-1, 2004).

The results can allow to understand if it is possible to calibrate a monitoring system to individuate the critical condition of damage of the structure using dynamic parameters that can be easily measured, if a reference and reliable model has been already implemented.

2 DYNAMIC IDENTIFICATION FOR DAMAGE DETECTION

The dynamic behaviour of framed buildings damaged by earthquakes has been widely experimentally analysed by different authors.

Several shaking table tests on scaled RC frame structure (Li et al. 2016) and full-scale RC buildings (Mousseau and Paultre 2008a and 2008b) were carried out to analyse the collapse mechanism of the structure. In particular, at various steps of the tests, when damage occurred, ambient or forced vibration tests were performed to monitor the variation of the natural periods of vibration.

The experimental results are usually used to update a numerical finite element model and assess a procedure to accurately predict the increasing damage in the building, like in (Paultre et al. 2016).

In this context, the procedure of damage detection requires a reliable numerical model, which correctly reproduces the damage to be detected (Farrar et al. 2001), as well as the choice of suitable damage features (Doebling et al. 1996) provides a review of the main damage parameters.

The most common damage features are related to modal parameters, such as frequencies and mode shapes.

Shifts in resonant frequencies has been widely investigated in literature as damage detection parameters (Rytter 1993; Yang et al. 2009). They have some practical limitations, such as the low sensitivity of frequency shifts to damage, which requires very precise measurements or large levels of damage (Doebling et al. 1998). However, they have less statistical variation from random error sources than the damage features related to mode shapes (Doebling et al. 1997).

In case of buildings, an important engineering response parameter and indicator of structural performance is the interstorey drift. In fact, interstorey drift is significantly related to the structural performance levels (ATC-40 1996; FEMA-273 1996), as stated in seismic engineering. In this context, several researchers have been focused on this parameter for structural assessment. For instance, (Furtado et al. 2018) presented the effect of the mainshock and the aftershock on the structural response on RC infilled structures. The maximum interstorey drift was chosen as the structural response parameter to compare the behaviour of three types of numerical models of the same structure: the first without infill walls, the second considering only the in-plane behaviour of the infill walls and the third introducing also their out-of-plane behaviour. The same approach was proposed by (Pepe et al. 2019). The results confirm that the infill walls can significantly modify the structural response of a building, as well-known in the literature (De Angelis and Pecce 2018a and 2018b).

3 THE CASE STUDY

The sample building consists of three multi-storey RC framed systems. The building is residential and has a rectangular plant with three floors (Figure 1). The main in-plan dimensions are $L_x = L_y = 15$m and the interstorey heights are 3m. The frames have three 5m bays in x-direction and in y-direction. The floors are made of RC joists and hollow clay bricks for lightening, with a total height of 0.20m and an RC slab of 0.04m. The materials used are concrete of class C25/30 and steel of class B450C.

The structural elements (beams and columns) of the building were designed according to two different procedures. In one case the design was carried out according to the current provisions of constructions in seismic area, in the other case only the vertical loads were considered in the design to simulate the conditions of many existing buildings in Italy.

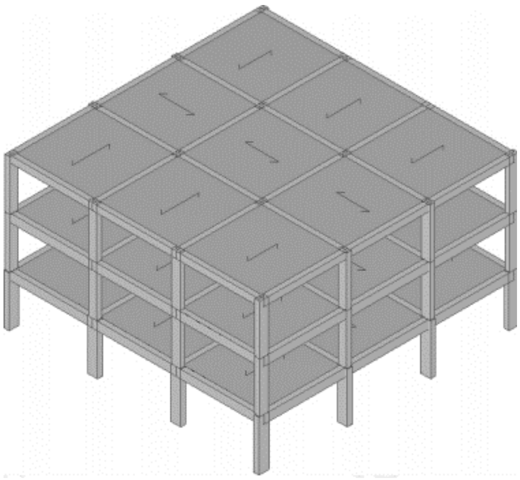


Figure 1. 3D view of the sample framed building.

The design of the frame in seismic area was carried out according to NTC2018 and EC8 provisions.

The building is on a soil of category C, with a topography coefficient T1. The seismic hazard parameters to define the elastic spectrum of acceleration are a_g=0.2583g, F_0=2.304, T^*_c=0.370s. A behaviour factor q=3.9 and a ductility class B were assumed for the design.

The self-weight load for the intermediate floor and the roof is 5.38 kN/m^2 and 4.47 kN/m^2 respectively. Live loads of 2.00 kN/m^2 were assumed for all floors. The infill walls are made of masonry with a single lining with a thickness of 0.25m (γ=8.8 kN/m^3).

The beams have a rectangular section of (30x50)cm at the first and second floors, and (30x40)cm at the third floor. The columns have a rectangular section of (30x60)cm at the first floor and (30x50)cm at the second and third floors.

The steel reinforcement was designed according to capacity design and detailing provided by EC8 (the same of NTC2018).

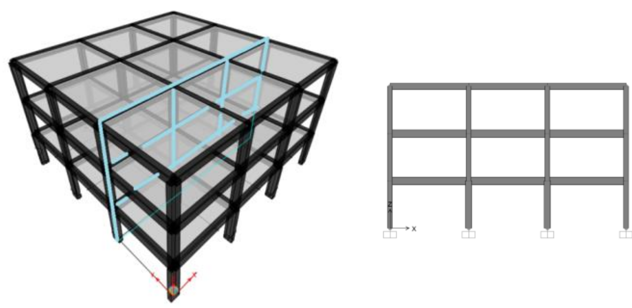


Figure 2. Two-dimensional frame considered for the analysis.

With respect to the frame designed only for vertical loads, the beams have rectangular sections of (30x50)cm at the first and second floors and of (30x40)cm at the third floors. The columns have a rectangular section of (30x40)cm at the first floor and of (30x30)cm at the second and third floors.

The steel reinforcement of the building designed only for vertical loads was defined according to the provisions of EC2 (EN 1992-1-1, 2004), the same of NTC2018, for RC buildings. Therefore, the details of this building do not fulfil the provisions of NTC2018 and EC8 for buildings in seismic area. In particular, the percentage of reinforcement in columns and beams is much smaller and the spacing of the stirrups is greater.

Numerical models of both frames were implemented using the software SAP2000 (Computer & Structure, Inc. 2016).

The structure of the building is symmetric and can be defined as regular in plane, therefore the study for the damage detection was developed only for the central frame (Figure 2).

4 THE NUMERICAL SIMULATION OF DAMAGE

First of all, the progressive damage produced by an earthquake was numerically simulated by performing an adaptive pushover analysis on the models of both frames, according to the procedure already assessed by the authors in (Pepe et al. 2019). A shape distribution of the horizontal loads proportional to the first mode of vibration was assumed. A lumped plasticity modelling approach to introduce plastic hinges at the ends of the elastic beams and columns was used. The rotations corresponding to the yielding moment and the ultimate moment were evaluated according to EC8 provisions.

Then, the variations of the modal parameters, namely frequencies and mode shapes, were monitored through modal analyses performed after each step of the adaptive pushover analysis, which is associated to a damage scenario, i.e. the formation of a new plastic hinge. In addition, the modal interstorey drift was chosen as further global damage feature.

5 THE DYNAMIC RESPONSE OF THE DAMAGED FRAMES

The first damage indices analysed are the fundamental period of vibration and the first natural frequency. In order to obtain a generalized trend of both indices, their values at each level of damage were normalised using the values computed for the undamaged structure and plotted as functions of the number of damage scenarios (Figure 3).

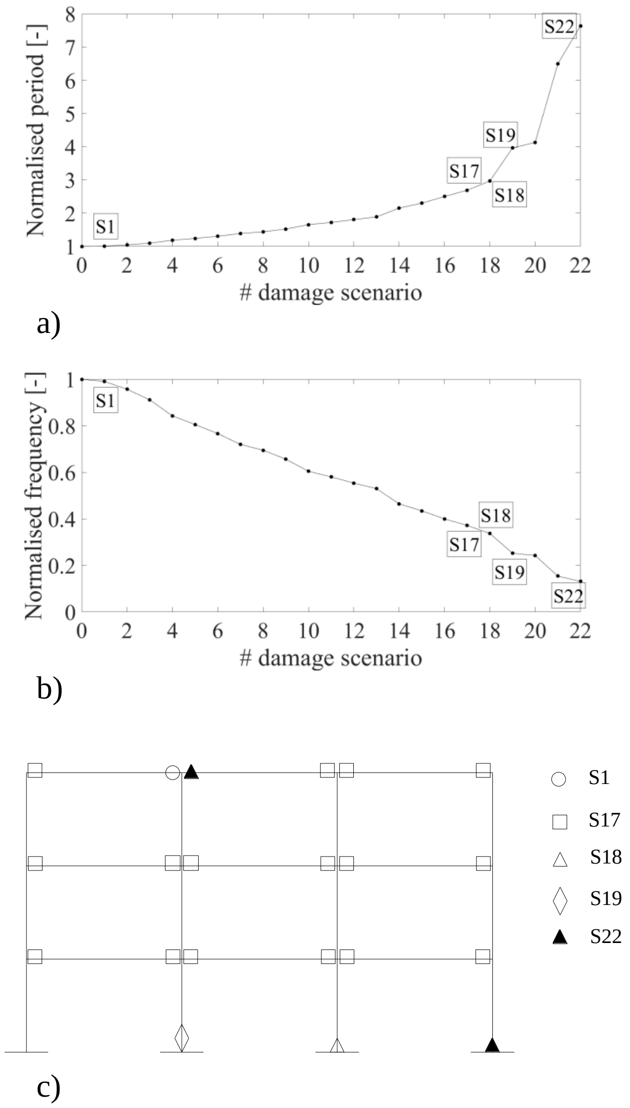


Figure 3. Frame designed for seismic loads. a) Periods; b) Frequencies; c) Plastic hinges.

The period of the undamaged frame designed to withstand seismic loads is 0.26s, but increases of 7.64 times (1.96s) with the amplitude of the seismic actions until the global mechanism of collapse occurs.

The comparison between the trend of periods (Figure 3a) and frequencies (Figure 3b) points out that the period is better than the frequency as damage index for this type of structure. In fact, the frequencies have a nearly linear trend when damage occurs in beams, as in damage scenarios S1 and S17 in Figure 3c, namely until the damage of columns, which occurs in damage scenarios S18, S19 and S22 in Figure 3c, so that it is impossible to detect the type of damage. On the contrary, the variation of the period is linear until the damage occurs in columns (damage scenarios S18, S19 and S22), then a sudden increment is a clear warning about an important damage happened in the building.

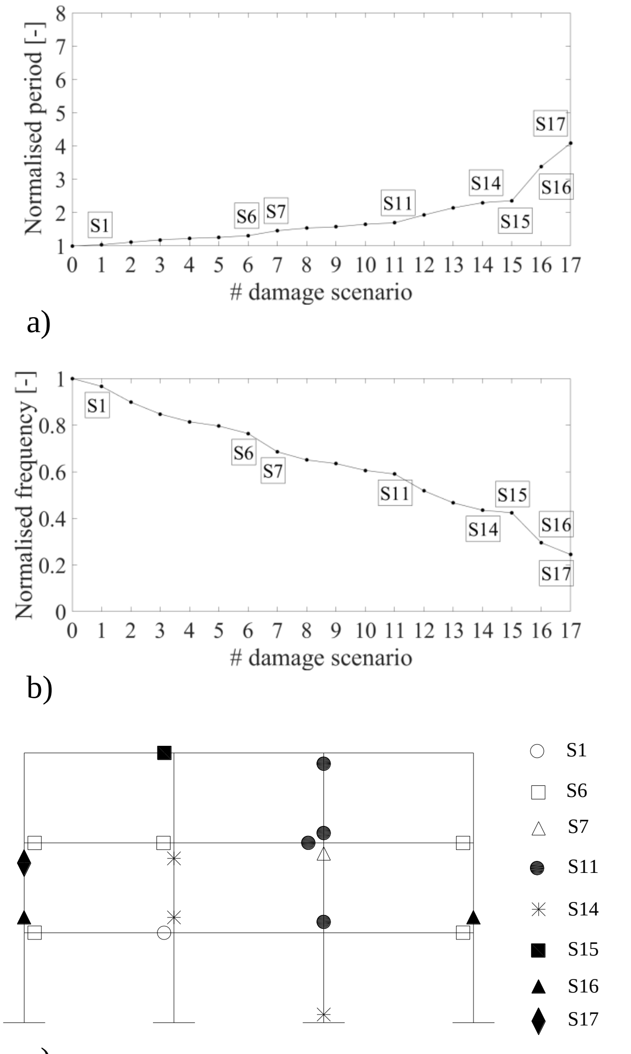


Figure 4. Frame designed for vertical loads a) Periods; b) Frequencies; c) Plastic hinges.

Focusing on the frame designed for vertical loads, it experiences a progressive plasticisation in beams and columns alternatively until a local collapse mechanism occurred at the second floor (Figure 4c).

The period of the undamaged structure is 0.35s and increases of 4.08 times (1.42s) due to damage (Figure 4a). Also in this case the period is a more significant parameter.

As in the previous case, the greatest change in the period trend slope is associated to a damage occurred in columns, as it can be seen from the values of the period at the scenarios 6-7, 11-14. Furthermore, a sharper change of the period trend slope points out that the local collapse mechanism has arisen.

To gather more information about the state of the structure, changes in mode shapes can be examined.

In Figure 5 and in Figure 6 the three mode shapes of the frame designed for seismic and vertical loads respectively, normalised using the absolute value of the maximum component, are depicted for the damage scenarios plotted in Figure 3c and Figure 4c respectively.

The form of the mode shapes of both frames is very regular at the beginning. As the damage advances, the distribution of the plasticisation in the beams of the frame designed for seismic loads does not produce a significant and systematic variation of the mode shapes which permits to localise the damage. On the contrary, the variation of the mode shapes for vertical loads shows a regular change in the first mode shape, revealing the local collapse mechanism at the second floor.

The differences between the first mode shape of both frames when the collapse mechanism occurs are clearly graphed in Figure 7.

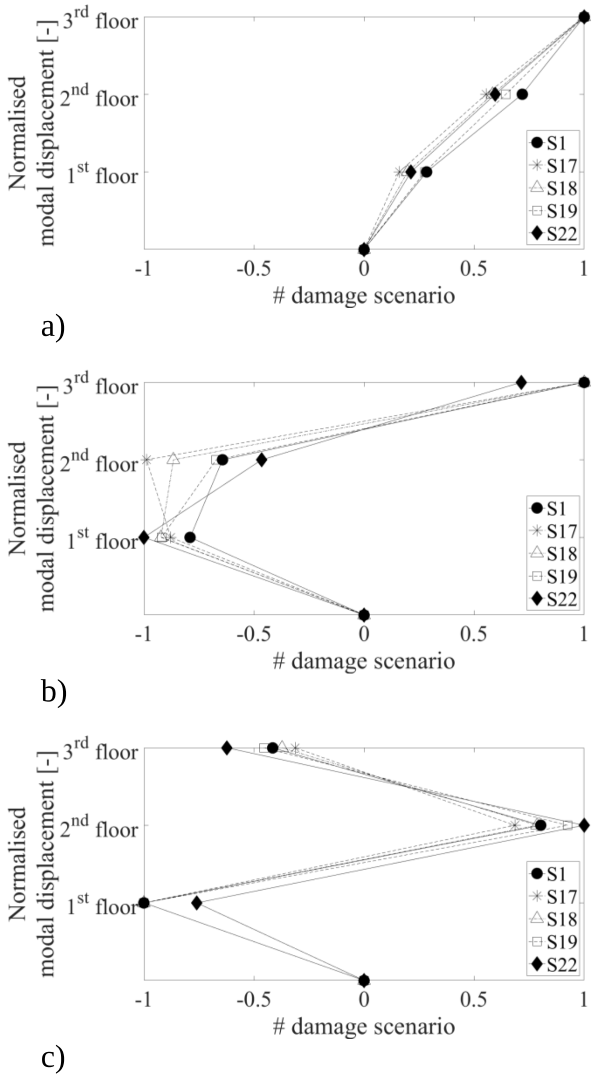
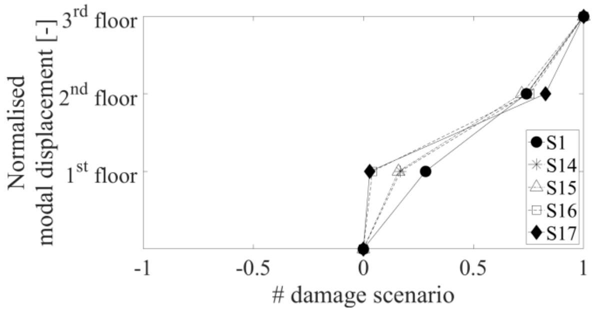
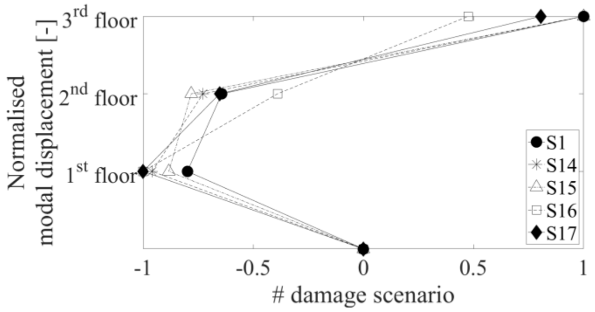


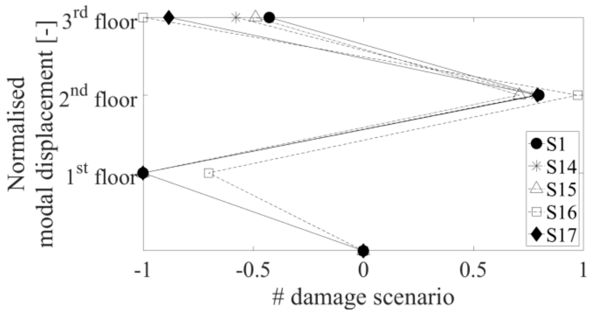
Figure 5. Mode shapes of the frame designed for seismic loads. a) First mode; b) Second mode; c) Third mode.



a)



b)



c)

Figure 6. Mode shapes of the frame designed for vertical loads. a) First mode; b) Second mode; c) Third mode.

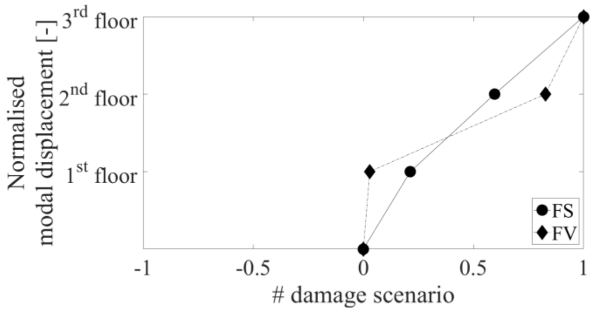


Figure 7. Mode shapes at the last damage scenario of the frame designed for seismic loads (FS) and for vertical loads (FV).

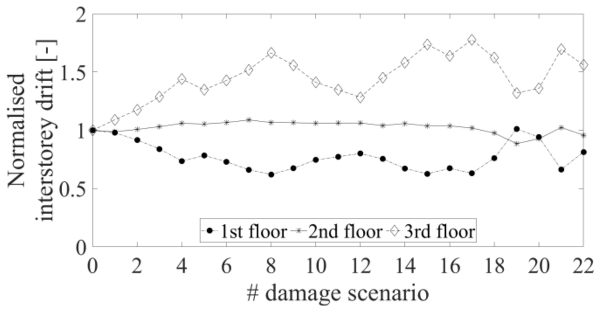


Figure 8. Modal interstorey drift evaluated from the first mode shape for the frame designed for seismic loads.

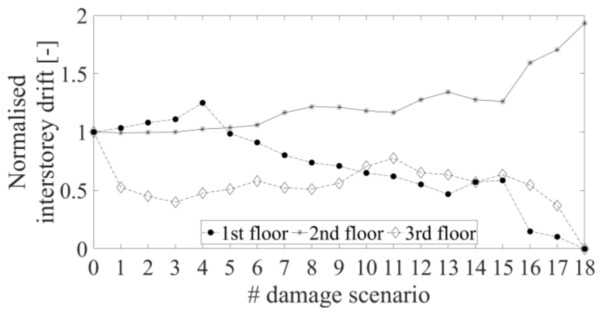


Figure 9. Modal interstorey drift evaluated from the first mode shape for the frame designed for vertical loads.

Finally, the structural response in terms of modal interstorey drift has been analysed.

It has been computed from the first mode shape as follows:

$$d_j=\frac{\phi_{i+1,j}-\phi_{i,j}}{h_{interstorey}} \quad \text{for } i=1, \ldots, n_{storeys} \qquad (1)$$

where $\phi_{i,j}$ is the component of the first mode shape at the i-th floor for the j-th damage state, $\phi_{i+1,j}$ is the component of the first mode shape at the (i+1)-th floor at the j-th damage state and $h_{interstorey}$ is the interstorey height.

The modal interstorey drift has been normalized using the value calculated for the undamaged frame. In Figure 8 its trend is depicted for each floor at each damage scenario for the frame designed for seismic loads. The first and the third floors exhibit a symmetrical trend. In particular, a clear variation of the slope of the trend is associated to a damage in a column, a tiny variation is associated to a damage in a beam, as for the periods of vibration.

The same observation about the interstorey drift trend can be made for the frame designed for vertical loads (Figure 9), even if these variations cannot be clearly detected. Unlike the variation of the modal interstorey drift evaluated for a frame designed for seismic loads, the interstorey drift of the first and the third floors of the frame for vertical loads drop to zero, whereas the drift of the second floor doubles. This means that a local collapse mechanism developed and it can be clearly identified. Furthermore, the onset of the damage in the column of the second floor is clearly pointed out at the scenario 4-7, when the drift of the first floor starts to reduce and the drift of the second floor increases.

In this respect, the interstorey drift evaluated for a frame designed for vertical loads permits only to identify the collapse mechanism and the storey where it is localized, but it does not allow to detect the progressive damage occurred before. On the contrary, for the frame designed for seismic load, it enables to establish when the damage occurs in columns, but it is difficult to establish where the damaged structural element is located in the frame.

6 CONCLUSIONS

Monitoring systems based on dynamic identification can be useful instruments to quick assess the state of a structure after seismic event. The design of an efficient sensors network and the selection of meaningful parameters are necessary.

In this paper the ability of three common damage features, namely fundamental period of vibration, mode shapes and interstorey drift, to detect and localise damage due to earthquakes has been analysed.

Two RC frames extracted from two buildings designed to withstand seismic loads and vertical loads respectively were studied by a nonlinear analysis and a modal analysis.

The modal dynamic analysis at the various steps of damage pointed out the following main results:

- the period of vibration is a damage feature better than the frequency of vibration for both framed buildings, because it magnifies the effect of damage in the columns;
- the mode shapes of the frame designed for seismic loads does not experience systematic variations while the damage is advancing;
- the variation of the first mode shape of the frame designed for vertical loads distinctly reveals the local collapse mechanism at the second floor;
- the variation of the interstorey drift evaluated for the frame designed for seismic loads clearly warns that damage is distributed on the entire structure;
- the variation of the interstorey drift evaluated for the frame designed for vertical loads clearly indicates a concentration of damage with a local mechanism at the second floor.

The results presented in this paper represent a first encouraging step of a wider study that will be developed in the future to assess the procedure of damage detection trough dynamic analysis, considering further types of structures and modelling approach.

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