DAMAGE LOCALIZATION AND QUANTIFICATION IN SEISMIC-EXCITED STRUCTURES USING OPTIMIZATION-BASED ALGORITHM

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ABSTRACT: This paper presents a new algorithm for structural damage identification and quantification in seismic-induced structures through an optimization approach. In the method, a sensitive cost function is proposed by means of statistical components of the recorded displacement time history of a structure under earthquake excitation. Earthquake duration is divided into several sub-intervals, and a statistical parameter named moment generating function is employed for extracting main information of the response time history. The optimization problem is introduced by means of fitting data strategy, and the problem is solved by a new evolutionary optimization algorithm, named the competitive optimization algorithm. This optimization algorithm is a new global search strategy, which has recently been presented for dealing with different optimization tasks. In the presented method, not only we can detect the location and severity of damages, but also the time of damage occurrence can be identified. To illustrate the efficiency and robustness of the proposed method, different numerical examples were considered. Most of the obtained results show the good performance of the algorithm for damage identification and estimation.

1. Introduction

Seismic and wind excitations may cause some structural damage in mechanical and civil engineering structures that should be identified to insure further use of structures after earthquake or storm event. Moreover, in active or semi-active control of engineering structures excited by earthquake ground motions or wind, it would be helpful for accurate and effective control of a structure if an online damage identification algorithm can identify the exact stiffness of the structure by analysing the measured structural responses during an excitation.

There are different studies in the literature which are aimed at identifying structural damage by time domain analysis. Some researchers employed signal processing techniques by means of the empirical mode decomposition (Xu and Chen, 2004; Yang, Lei, Lin and Huang, 2004), the wavelet transform (Hou, Noori and Arnand, 2000; Demetriou and Hou, 2003), and the combination of the wavelet transform and the empirical mode decomposition (Li, Deng and Dai, 2007) for estimating some information about the time of damage occurrence from structural response's time histories. Shinde and Hou (2005) presented a sifting process via the wavelet packet analysis for decomposing structural response's signals into their components with different frequency contents. They inspected the reliability of approach by comparing obtained results with those coming from the empirical mode decomposition-based approach and verified the method by numerical and experimental studies. It can be concluded that the method has a good performance when it is incorporated with the classical Hilbert transform. Also, Dong, Li and Lai (2010) proposed a damage index using the empirical mode decomposition and the vector autoregressive moving average for structural damage identification. The main idea of their method can be summarized in attempting for identifying damage by inspecting the related abrupt changes in the energy distribution of structural responses at high frequencies. It is worth saying that although in all of the mentioned signal processing-based methods the presence of some spikes in decomposed structural response's signals can detect some structural damages; these methods cannot provide any other information about the details of detected damages. To overcome this limitation, recently, Yang and Nagarajaiah (2014) developed a free-baseline and output-only damage identification method based on the wavelet transformation and the independent component analysis for estimating damage location and the instant of damage occurrence. However, it is impossible to estimate damage magnitude by using this approach. On the other hand, other researchers proposed algorithms based on the least-squares method (Yang and Lin, 2005), the time-window technique (Lin, Wang, Wu and Wang, 2005), neural networks (Park, Kim, Hong, Ho and Yi, 2009), the auxiliary particle filtering technique (Xue, Tang and Xie, 2009), for localizing and quantifying damages as well as estimating damage occurrence time.

This paper is aimed at presenting a novel method for damage localization, quantification, and for estimating the time of defect event in structures induced by earthquake accelerations. Damage detection problem is defined as an optimization problem and an objective function is introduced based on the calculated statistical Moment Generating Function (MGF) of the displacement response's time histories for a given time segment. For finding an optimal solution, a powerful evolutionary optimization algorithm, the Competitive Optimization Algorithm (COA) (Atashpaz-Gargari and Lucas, 2010), is employed. For investigating the capability and reliability of the presented algorithm, two numerical examples included a one-bay one-story steel frame and a two-span concrete beam with different damage scenarios were studied.

2. Competitive Optimization Algorithm

The COA is a global search optimization method that is inspired from a socio-political competitive event (Atashpaz-Gargari and Lucas, 2010) and was used in a number of studies for optimization tasks (Bagheri, Ghodrati Amiri and Haghdoust, 2014; Zare Hosseinzadeh, Bagheri and Ghodrati Amiri, 2013). Some of its main advantages are listed below:

- It can simultaneously search several points of the solution domain with a considerable speed.
- It can provide a list of optimum variables instead of just a single solution.
- It is well organized for parallel computers.
- It has a simple mathematical approach for searching solution domain and is a derivative-free algorithm.

The aim of this algorithm is to find a global minimum or maximum of the argument $\mathbf{y}$ of a given function $f(\mathbf{y})$. Similar to other evolutionary optimization algorithms, this algorithm is started with an initial population that called '*country*'. Some of the best countries are selected as the main countries, which are the initial candidates for the optimal solution. The rest of the initial population is considered as '*colony*' and are divided among the main countries. After the initialization process, the main countries begin to improve their colonies and attempt to absorb new colonies. This is called the assimilation process, which is modeled by moving all of the colonies toward the main country along different optimization axis. To ensure that many positions are explored in search of the minimal cost, the assimilation of the colonies by the main countries does not occur through the direct movement of the colonies toward them. A random path is induced by a random amount of deviation which is added to the direction of the movement (Atashpaz-Gargari and Lucas, 2010). If during the assimilation process, a colony reaches a position with lower cost than the main country, they will switch their positions and the algorithm will continue with the main country in the new position. The presenting approach and the competition among the main countries for absorbing different colonies continue in an iterative scheme. Finally, the optimal solution will be reached when only one point is available on the searching domain and almost all countries are added on that point. More details of this optimization approach can be found at (Atashpaz-Gargari and Lucas, 2010; Bagheri, Razeghi and Ghodrati Amiri, 2012).

3. Proposed Method

3.1. Extracting Main Information of Structural Responses

This section is aimed at introducing an effective parameter for extracting the main information of the structural displacement time history by dividing it to several segments. Assume the structural displacement time history **U**(*t*) is divided into N_s segments. For the *i*-th segment, the displacement response is presented as:

$$\mathbf{U}_i(t) = \mathbf{U}(t_i \le t \le t_{i+1}) \tag{1}$$

Eq. (1) can be rewritten by considering structural response for each DOF as below:

$$\mathbf{U}_i(t) = \left[\mathbf{u}_{i,1}(t) \quad \mathbf{u}_{i,2}(t) \quad .... \quad \mathbf{u}_{i,N_d}(t)\right]^T \tag{2}$$

where $\mathbf{u}_{i,j}$ is the structural response's time history of the *j*-th DOF for the *i*-th time segment. Based on a statistical function named the MGF, we attempt to assign a unique amount for displacement responses in a given time segment which can reflect the basic information of the structural response in a meaningful way. The most important property of the MGF is its dependence on the variance and mean values of data. The MGF is a function that can produce each order of statistical moment. This function is defined based on the Riemann–Stieltjes integral as below (Shilov and Gurevich, 2012):

$$MGF = \int_{-\infty}^{\infty} e^{\alpha x} p(x) dx \tag{3}$$

where x is the considered signal, $p(x)$ denotes the Probability Density Function (PDF) of x, and α is a positive integer number. By knowing the MGF, one will be able to define the probability distribution, completely. For a random variable with normal distribution, the PDF can be expressed as:

$$p(x) = \frac{1}{\sqrt{2\pi}\sigma} e^{-\frac{(x-\mu)^2}{2\sigma^2}} \tag{4}$$

where μ and σ^2 are the mean and variance values of x. For this distribution, the MGF can be presented as below:

$$MGF = e^{\mu\alpha + \frac{\sigma^2\alpha^2}{2}} \tag{5}$$

Therefore, by having a displacement time history, we can divide it to several segments and calculate MGF for each segment by sampling some points of the response via basic statistical relations for estimating mean and variance values. The MGF is calculated as:

$$MGF_{i,j} = e^{\mu_{i,j}\alpha + \frac{\sigma_{i,j}^2\alpha^2}{2}} \tag{6}$$

where $\mu_{i,j}$ and $\sigma^2_{i,j}$ are the mean and variance values of the structural displacement response of the *j*-th DOF for the *i*-th segment, respectively.

3.2. Damage Detection Approach

In the previous section, we introduce the MGF as a unique variable, which can represent the main information of the time history response in the different time segments. This section explains the basic premise of the proposed method for damage identification. The main contribution of our method is defining damage detection problem as an optimization problem by introducing a cost function, which is based on minimizing the error function between calculated MGF from measured and numerical structural displacement time histories. In the following, at first, the mentioned two set of MGF are introduced, then the cost function is presented.

By means of Eq. (6), we can express the MGF vector for the measured structural responses in the *i*-th time segment as below:

$$\mathbf{MGF}_i^m = \left\{ MGF_{i,1}^m \quad MGF_{i,2}^m \quad \dots \quad MGF_{i,N_d}^m \right\}^T \tag{7}$$

In the numerical model of the structure, it is assumed that damage appears by some reduction in the stiffness matrix of damaged element. Thus, in the analytical model of damaged structure, the stiffness matrix of elements can be defined as:

$$\mathbf{K}_e^d = (1-d_e)\mathbf{K}_e^u \tag{8}$$

whereas $\mathbf{K}_e^d$ and $\mathbf{K}_e^u$ are the damaged and undamaged stiffness matrices of the *e*-th element, respectively; d_e is the damage severity of the *e*-th element which is a number between 0 and 1 for healthy and fully damaged elements, respectively. By assembling the stiffness matrix of all elements and calculating the global structural stiffness matrix for a known set of damage severities, we can find structural time history responses under a known external excitation. Then, we can calculate the numerical MGF vector for a given set of damage severities as:

$$\mathbf{MGF}_i^d = \left\{ MGF_{i,1}\Big|_{(d_1,d_2,\dots,d_N)} \quad MGF_{i,2}\Big|_{(d_1,d_2,\dots,d_N)} \quad \dots \quad MGF_{i,N_d}\Big|_{(d_1,d_2,\dots,d_N)} \right\}^T \tag{9}$$

where $\mathbf{MGF}_i^d$ is the MGF vector for the analytical structural responses of the *i*-th time segment, and $MGF_{i,j}$ is the MGF for the analytical structural displacement of the *j*-th DOF for the *i*-th time segment. Finally, the error function **e** for the *i*-th time segment is defined using the MGFs computed from the numerical model and the monitored structure as follows:

$$\mathbf{e}_i = \mathbf{MGF}_i^m - \mathbf{MGF}_i^d\left(d_1,d_2,\dots,d_N\right) \tag{10}$$

and the objective function for the *i*-th time segment is formulated as:

$$Cost_i\left(d_1,d_2,\dots,d_N\right) = norm\left\{\mathbf{e}_i\right\} \tag{11}$$

We solve this objective function by the COA to find the global minimum point, which is the result of the damage localization and quantification for the *i*-th time segment. After that, we can obtain the result of damage magnitude in all structural elements for earthquake duration by repeating the presented procedure for all time segments.

4. Numerical Studies

4.1. One-Story Steel Frame

As the first example, consider a one-span one-story steel frame as shown in Fig. 1. Finite element model of this structure consists of three elements (two columns and one beam) with two free nodes and six DOFs. For all elements, modulus of elasticity and mass density of structural material are equal to E=200 *GPa* and ρ=7850 kg/m^3, respectively. For columns, mass per length, the moment of inertia, and cross section area are considered as m=117.75 kg/m, I=3.3×10^{-4} m^4, and A=1.5×10^{-2} m^2, respectively; for horizontal beam, those are considered as m=119.32 kg/m, I=3.69×10^{-4} m^4, and A=1.52×10^{-2} m^2, respectively. Two damage scenarios under the 1940 El-Centro earthquake and the 1994 Northridge earthquake were simulated in this example which are presented in Table 1. The length of time interval is 0.5 sec for the 1994 Northridge earthquake excitation while it is 1 sec for the 1940 El-Centro earthquake excitation.

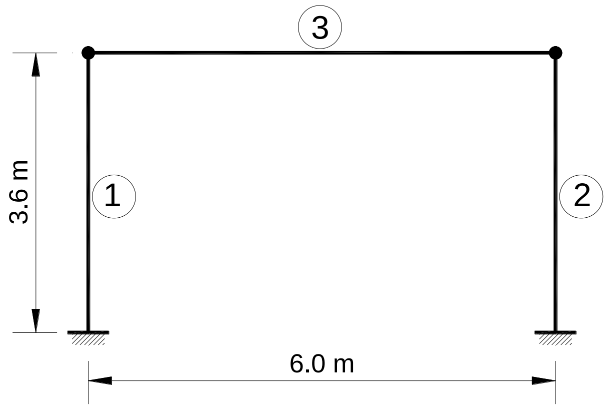

Fig. 1 – Finite element model of the one-story steel frame

Table 1 – Damage scenarios for the one-story steel frame.

Damage scenario	Earthquake	Scenario explanation
1	1994 Northridge	10% damage at t=8.5 sec in element 1
2	1940 El-Centro	10% damage at t=26.0 sec in element 1, and 20% damage at t=15.0 sec at element 3, +10% damage at t=26.0 sec at element 3

Obtained results for two damage patterns in an ideal case (free noise state) are shown in Figs. 2 and 3. It is obvious that not only can the presented method identify the time of damage occurrence, but also it can quantify damages with a high level of accuracy. In real SHM programs, structural responses are contaminated by random noises. Therefore, inspecting the applicability of the proposed method in the presence of noises is desirable. For this purpose, we repeated above-mentioned scenarios when the input time histories are polluted with 2% of normal distributed random noises. It worth noting that we de-noise input signals using wavelet-based de-noising approach for decreasing the effects of noises on the applicability of the method. More information about de-noising approach can be found in (Rizzo and Lanza di Scalea, 2006). Figures 2 and 3 show the obtained results in this case. It can be concluded that although the input data were polluted with random noises, the proposed method is able to identify damage properties with high level of accuracy. It is worth noting that although there are few time instants which report some damages in the healthy elements, the values of damage severity are so little and they are not more appreciable.

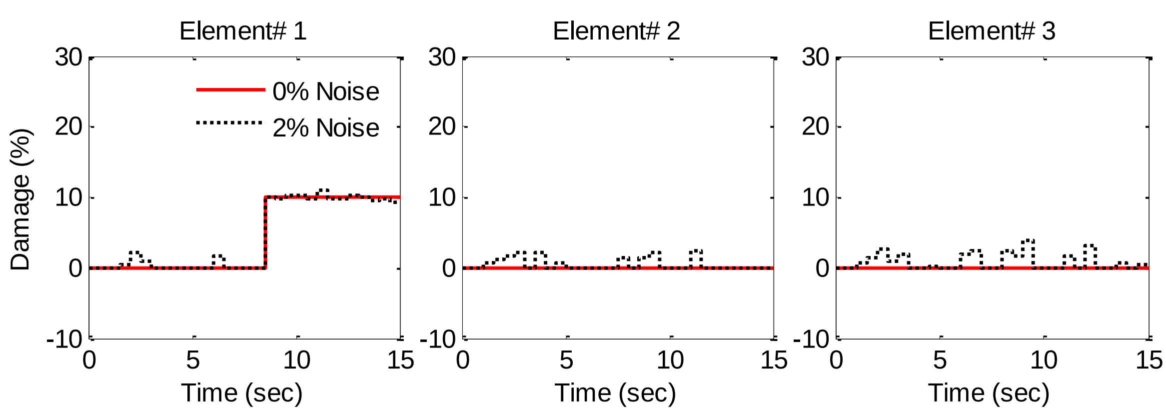

Fig. 2 – Damage detection results for the one-story steel frame for the first scenario

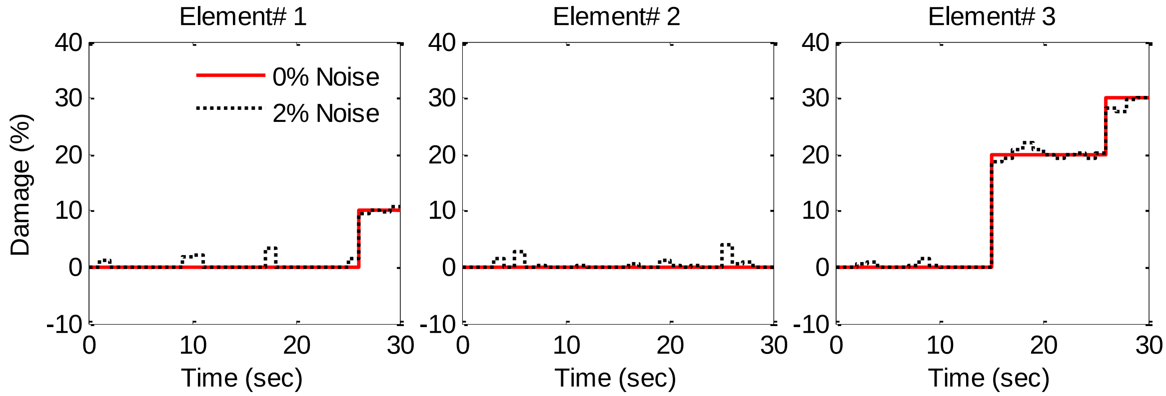

Fig. 3 – Damage detection results for the one-story steel frame for the second scenario

4.2. Two-Span Concrete Beam

The second example is devoted for damage identification in a two-span concrete beam. Figure 4 provides the finite element model of this structure. As can be seen, the finite element model consists of ten beam elements and eight free nodes that each of them has two DOFs, and this structure has 19 DOFs. Material properties for all elements of this beam were considered as following: Young's modulus E=25 GPa, and mass density ρ=2500 kg/m^3. In addition, cross sectional area and the moment of inertia of elements are A=0.35 m^2, I=0.01429 m^4, respectively. For this beam, two damage scenarios were considered that are listed in Table 2. The length of time segment are same as the previous numerical example.

Obtained results for two damage scenarios with an ideal condition are shown in Figs. 5 and 6. These identified defects are exactly same as the simulated defects in the beam. Same as the results shown for the previous example, the results of this part are emphasized the robustness and efficiency of the proposed approach for structural damage identification in time domain.

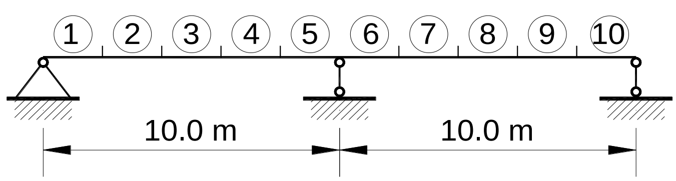

Fig. 4 – Finite element model of the two-span concrete beam

Table 2 – Damage scenarios for the two-span concrete beam.

Damage scenario	Earthquake	Scenario explanation
1	1994 Northridge	10% damage at t=5.5 sec in element 2
2	1940 El-Centro	20% damage at t=14.0 sec at element 5, and 10% damage at t=19.0 sec in element 7, and 15% damage at t=25.0 sec in element 1.

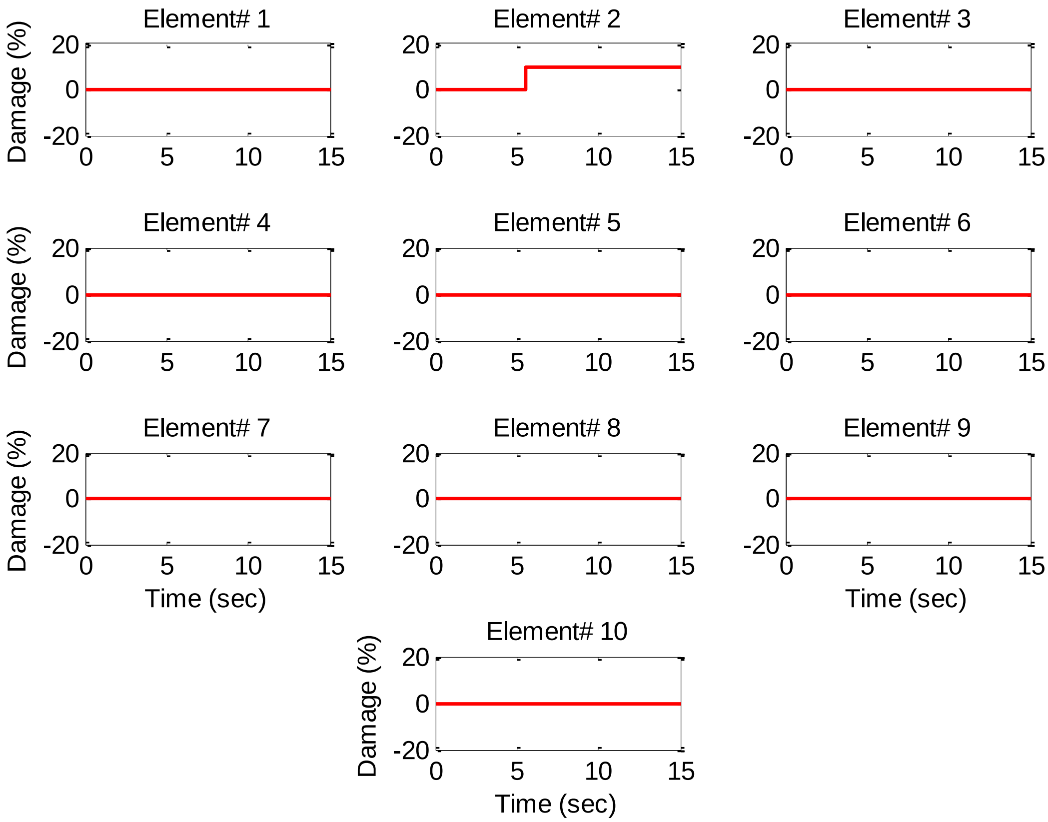

Fig. 5 – Damage detection results for the two-span concrete beam for the first scenario

5. Conclusions

In this paper, an effective and new damage prognosis algorithm in time domain was presented which is applicable to engineering structures induced by earthquake accelerations. The identification method was based on an optimization way that the objective was to minimize the error between the MGFs computed from the numerical model and the monitored structure. The presented method is able to localize and quantify damage and also estimate the time of damage occurrence during seismic excitations.

The capability and efficiency of the presented algorithm were numerically validated by means of two numerical examples, namely a one-bay one-story steel frame and a two-span concrete beam. These structures were studied with several damage scenarios which were induced with the 1940 El – Centro earthquake and the1994 Northridge earthquake. The obtained results introduced the proposed algorithm as a robustness and viable method for damage identification in time domain for seismic-excited structures.

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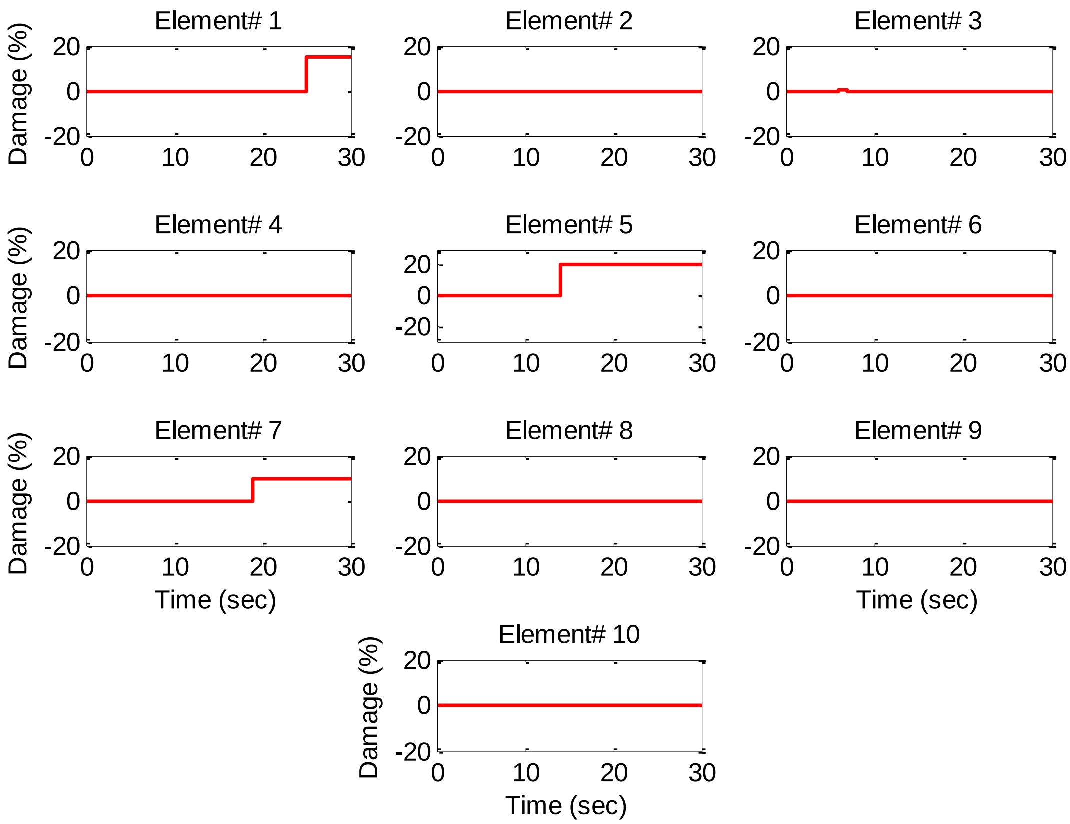

Fig. 6 – Damage detection results for the two-span concrete beam for the second scenario

7. References

ATASHPAZ-GARGARI, Esmaeil, LUCAS, Caro, "Imperialist Competitive Algorithm: An Algorithm for Optimization Inspired by Imperialistic Competition", *IEEE Congress on Evolutionary Computation,* Singapore 2007, pp. 4661-4667.

BAGHERI, Abdollah, GHODRATI AMIRI, Gholamreza, HAGHDOUST, Jamshid, "New Method for the Estimation of Strong Ground Motions based on the Colonial Competitive Algorithm", *KSCE Journal of Civil Engineering*, Vol. 8, No. 5, 2014, pp. 1403-1410.

BAGHERI, Abdollah, RAZEGHI, Hamid Reza, GHODRATI AMIRI, Gholamreza, "Detection and Estimation of Damage in Structures using Imperialist Competitive Algorithm", *Shock and Vibration,* Vol. 19, No. 3, 2012, pp. 405-419.

DEMETRIOU, Michael A., HOU, Zhikun, "On-line Fault/Damage Detection Schemes for Mechanical and Structural Systems", *Journal of Structural Control*, Vol. 10, No. 1, January/March 2003, pp. 1-23.

DONG, Yinfeng, LI, Yingmin, LAI, Ming, "Structural Damage Detection using Empirical-Mode Decomposition and Vector Autoregressive Moving Average Model", *Soil Dynamics and Earthquake Engineering*, Vol. 30, No. 3, March 2010, pp. 133-145.

HOU, Zhikun, NOORI, Mohammad, ARNAND, R.S., "Wavelet based-Approach for Structural Damage Detection", *Journal of Engineering Mechanics (ASCE)*, Vol. 126, No. 7, July 2000, pp. 677–683.

LI, Helong, DENG, Xiaoyan, DAI, Hongliang, "Structural Damage Detection using the Combination Method of EMD and Wavelet Analysis", *Mechanical Systems and Signal Processing*, Vol. 21, No. 1, January 2007, pp. 298-306.

LIN, Chi-Chang, WANG, Chieh-Erh, WU, Huang-Wen, WANG, Jer-Fu, “On-Line Building Damage Assessment based on Earthquake Records”, *Smart Materials and Structures*, Vol. 14, No. 3, 2005, pp. 137-153.

PARK, Jae-Hyung, KIM, Jeong-Tae, HONG, Dong-Soo, HO, Duc-Duy, YI, Jin-Hak, “Sequential Damage Detection Approaches for Beams using Time-Modal Features and Artificial Neural Networks”, *Journal of Sound and Vibration*, Vol. 232, No. 1-2, June 2009, pp. 451-474.

RIZZO, Piervincenzo, LANZA DI SCALEA, Francesco, “Wavelet-based Feature Extraction for Automatic Defect Classification in Strands by Ultrasonic Structural Monitoring”, *International Journal of Smart Structures and Systems*, Vol. 2, No. 3, 2006, pp. 253-274.

SHILOV, G.E., GUREVICH, B.L., “Integral, Measure, and Derivative: A Unified Approach”, *Dover Publications*, 2012.

SHINDE, Abhijeet, HOU, Zhikun, “A Wavelet Packet based Sifting Process and its Application for Structural Health Monitoring”, *Structural Health Monitoring*, Vol. 4, No. 2, June 2005, pp. 153-170.

XU, You-Lin, CHEN, J., “Structural Damage Detection using Empirical Mode Decomposition: Experimental Investigation”, *Journal of Engineering Mechanics (ASCE)*, Vol. 130, No. 11, November 2004, pp. 1279-1288.

XUE, Songtao, TANG, Hesheng, XIE, Qiang, “Structural Damage Detection using Auxiliary Particle Filtering Method”, *Structural Health Monitoring*, Vol. 8, No. 2, March 2009, pp. 101-112.

YANG, J.N., LEI, Y., LIN, S., HUANG, N., “Hilbert-Huang based Approach for Structural Damage Detection”, *Journal of Engineering Mechanics (ASCE)*, Vol. 130, No. 1, January 2004, pp. 85-95.

YANG, Jann N., LIN, Silian, “Identification of Parametric Variations of Structures based on Least Squares Estimation and Adaptive Tracking Technique”, *Journal of Engineering Mechanics (ASCE)*, Vol. 131, No. 3, March 2005, pp. 290-298.

YANG, Yongchao, NAGARAJAIAH, Satish, “Blind Identification of Damage in Time-Varying System using Independent Component Analysis with Wavelet Transform”, *Mechanical Systems and Signal Processing*, Vol. 47, No. 1-2, August 2014, pp. 3-20.

ZARE HOSSEINZADEH, Ali, BAGHERI, Abdollah, GHODRATI AMIRI, Gholamreza, “Two-Stage Method for Damage Localization and Quantification in High-Rise Shear Frames based on the First Mode Shape Slope”, *International Journal of Optimization in Civil Engineering,* Vol. 3, No. 4, November 2013, pp. 653-672.