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NUMERICAL SIMULATIONS OF THE SEISMIC BEHAVIOR OF A DAMAGED RC FRAME RETROFITTED WITH COMPOSITE FABRIC

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Abstract

Reinforced concrete (RC) structures are frequently met in urban areas all around the world, including areas prone to severe seismic events. Although initially designed to withstand large earthquakes the damage accumulated in a RC structure during seismic events will ultimately require the strengthening of the structure or retrofitting it to comply with the new seismic design regulations. The paper presents the results obtained by numerical analyses on the seismic behavior of a damaged RC frame structure retrofitted with a composite fabric. The RC frame structure was previously damaged during a series of shake table tests aimed at investigating the short column behavior during seismic excitations. The numerical model was developed based on the initial, undamaged, state of the RC frame structure and was validated by comparing the numerical results to the experimentally obtained results.

The RC frame structure was retrofitted by means of an epoxy resin injected in the cracked concrete located in the short column part of the model. Additionally, unidirectional basalt fiber fabric was used to confine the beam-to-column joint and the short column part. The numerical model was changed to include the new "composite" material made of cracked concrete and epoxy resin from the short column part of the RC structure. The complete stress-strain curve of the C25 concrete class used for making the columns, beams and the slab of the RC structure was experimentally determined and used in the numerical model.

A non-linear time history analysis (THA) was used in order to account for the damage accumulation in the model from one seismic record to the next. Modal analysis was run after each non-linear THA to determine any change in the fundamental period of vibration. The seismic record consisted in an artificial earthquake generated according to Eurocode 8 using soil type C spectrum, the amplitude of which was gradually increased from one loading scenario to the next.

The obtained results were used to estimate the outcomes of a shake table test on a scaled down damaged RC frame structure following the same loading set-up in terms of seismic motions. The numerical model will then be calibrated based on the experimental results in order to create an accurate simulation of the real case. This could prove to be beneficial on the long term because it offers the possibility to investigate the seismic behavior to virtually countless real or artificial earthquake.

Keywords: damaged RC frame structure; numerical model; nonlinear THA; seismic behavior

1. Introduction

Reinforced concrete (RC) structures are frequently met in urban areas all around the world, including areas prone to severe seismic events. Even though such structures were initially designed to ensure certain levels of safety in case of earthquakes, the damage accumulated in a RC structure during its lifetime due to seismic events will ultimately require its strengthening or being retrofitted to either comply with the new seismic design regulations [1] or to prevent material losses and, possible, casualties during a future earthquake.

Fiber reinforced polymer (FRP) composites were initially used more often in aerospace engineering but nowadays are quickly becoming the first choice of designers when it comes to load-bearing structural components for infrastructure applications. These new materials are now being used worldwide for new civil engineering structures as well as for the rehabilitation of the existing ones [2]. FRP materials, including carbon, basalt, glass and aramid, possess excellent properties, such as high tensile strength, high stiffness, corrosion resistance and light weightiness [3].

During seismic events, columns are considered to be vital for the overall stability of the structures because they are expected to resist the lateral cyclic loads induced by the earthquake while also being able to resist the gravitational loads. Composite materials proved their effectiveness [4] in strengthening the already damaged columns due to either past seismic events or environmental stressors such as chemical attacks [5]. Recent studies conducted on shake tables have shown the benefit of using such composite materials in either retrofitting damaged RC structures [6] or strengthening sub-standard RC frames [7,8].

A significant obstacle to the advancement of knowledge regarding the seismic behavior of existing RC frame structures is the lack of experimental data documenting the realistic dynamic responses of structures tested under real or artificial seismic scenarios [9] and are mainly caused by the expensive nature of the involved testing equipment (shake tables) as well as by the complexity of such experimental investigations. In recent years, several shaking table tests have been performed to assess the seismic behavior of RC frame structures and some of the tests were conducted even up to the structural collapse [10].

In order to overcome the above mentioned obstacle, numerical simulations using a variety of computer programs, have quickly become an alternative in many research areas of civil engineering [11,12]. The main advantage of numerical simulations consists in the fact that a large number of varying parameters can be considered in order to get a comprehensive understanding on the behavior of a structural system subjected to seismic actions [13]. However, some of the numerical simulations may prove to be computationally intensive and require powerful computers to run the analyses. Moreover, such numerical simulations need to be validated by experimental programs in order to calibrate the numerical model, with all its simplified assumptions, with the response of a structure subjected to real life loading scenarios.

Currently, there are two approaches to the numerical modeling of RC frame structures by means of the finite element method (FEM): linear elements (beam elements) or 3D continuum elements for the concrete coupled with 1D truss elements for the reinforcement [14]. The first approach is computationally much cheaper and therefore quite often used in earthquake engineering to numerically simulate the structural response during an earthquake excitation. It does, however, fail to deliver sufficient information. The latter approach is computationally more expensive but it provides detailed information of the damage evolution in the concrete and of the stresses in the reinforcement.

The paper presents the results obtained by numerical analyses on the seismic behavior of a damaged RC frame structure retrofitted with a composite fabric. The RC frame structure was previously damaged during a series of shake table tests aimed at investigating the short column behavior during seismic excitations. The numerical model was developed based on the initial, undamaged, state of the RC frame structure and was validated by comparing the numerical results to the experimentally obtained results.

2. Materials

2.1 Concrete

A C25/30 concrete strength class was considered as it represents one of the most widely used types of concrete in the construction industry. Natural aggregates with rounded edges and a maximum aggregate size of 16 mm were used together with a CEM I – 42.5R rapid hardening cement, readily available on the market [15]. The average compressive strength, obtained from uniaxial compression tests [16] on 9 cylindrical specimens (100 mm x 200 mm), was $f_{c,14}$ = 32 MPa and $f_{c,28}$ = 35 MPa at 14 days and 28 days, respectively. The corresponding static moduli of elasticity [17], computed as an average of 8 measurements, were $E_{c,14}$ = 32.68 GPa and $E_{c,28}$ = 34.03 GPa [15].

2.2 Reinforcement

The longitudinal reinforcement was made of BST500 steel, ductility class C, commonly available and used on the construction market. The yield and ultimate tensile strengths were determined by means of direct tensile tests. Their values were f_y = 513 MPa and f_u = 626 MPa for the yield and ultimate strengths, respectively [15]. The stirrups (shear reinforcement) were made of Grade250 steel.

2.3 Composite material and repairing mortar

The damaged short columns were repaired by means of MAPEGROUT HI-FLOW fiber reinforced mortar. The retrofitting process is described in the subsequent section. According to the data sheet supplied by the producer, the flexural tensile strength at 28 days is 12 MPa and the modulus of elasticity in compression is 27 GPa.

The basalt fiber reinforced polymer (BFRP) strengthening solution consisted in a unidirectional high strength basalt fiber fabric that was applied for the short columns, the ends of both longitudinal and transversal beams and the node of the RC frame structure. The mechanical properties of the basalt fibers and of the composite material are shown in Table 1.

Table 1 – Material properties of the FRP composite (as supplied by the producer)

Basalt fabric – MAPEWRAP B-UNIX-AX-400			**FRP composite – fabric + MapeWrap 21 resin**		
Mass	Tensile strength	Tensile modulus of elasticity	Tensile strength	Tensile modulus of elasticity	Elongation at failure
[g/m^2]	[MPa]	[GPa]	[MPa]	[GPa]	[%]
400	2500	70	2300	70	2.5

3. Numerical Model

3.1 Geometry and dimensions

The geometry of the model in shown in Fig. 1. The building model was a one-bay one-storey frame, with a storey height of 1360 mm and in-plane dimensions of 2550×1950 mm. The slab was 60 mm thick and was reinforced with steel mesh with the diameter of 5 mm spaced at 100 mm in both directions.

The columns had a cross-section of 150×150 mm, reinforced with four 14 mm bars as longitudinal reinforcement, and 4 mm stirrups spaced at 100 mm as shear reinforcement [15]. The beams in the X and Y directions, in-plane directions, had a cross-section of 150×260 mm (b×h). They were reinforced with four 12 mm and four 10 mm bars, respectively, as longitudinal reinforcement and 6 mm stirrups spaced at 100 mm in the shear spans and at 200 mm in the middle for both beams.

The stiffening frame, used to ensure the short column behavior, was modeled via HEA180 profile, similar to the scaled down model to be used on the shake table. The generated numerical model by means of SAP2000 software is shown in Fig. 2. The fixed supports were considered at the level of the horizontal steel frame so the total height of the model was 1360 mm.

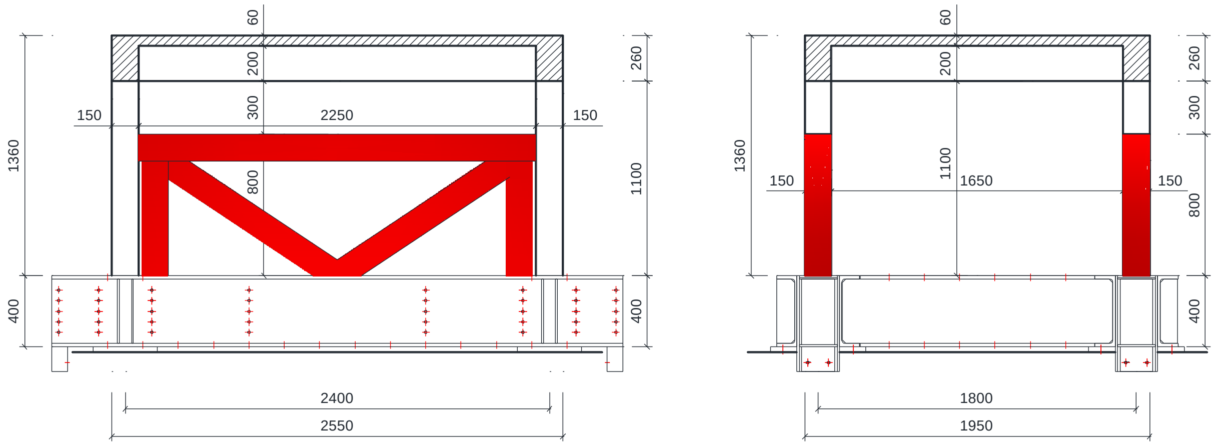


Fig. 1 – Geometry of the model (steel frame to simulate the infill wall is shown in red)

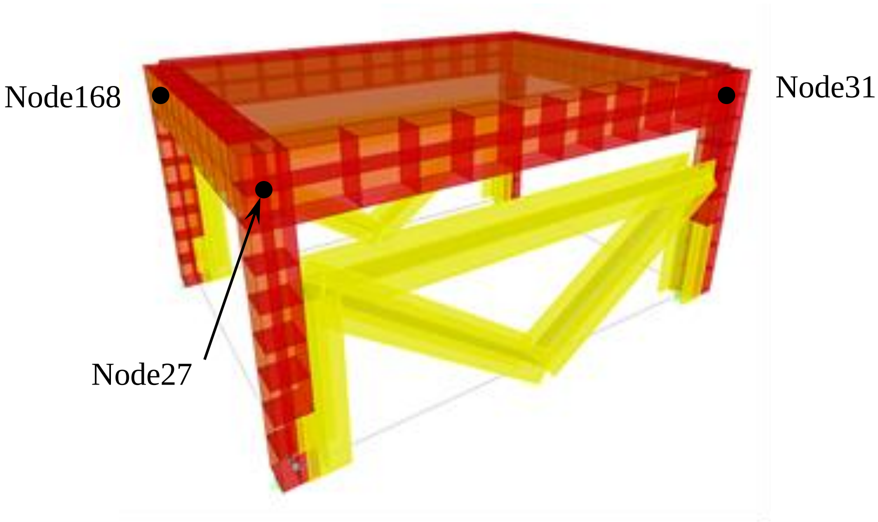


Fig. 2 – Numerical model (SAP2000)

3.2 Materials and Sections

The concrete part of the model was considered with the mechanical properties outlines in [15]. Since the short columns were rebuilt using fiber reinforced mortar and then confined with basalt fabric, it was assumed that their initial strength and stiffness was fully restored. However, in view of the future calibrations of the numerical model by means of the experimental results, the short columns might be remodeled as an equivalent concrete-like material with mechanical properties of the mortar confined by basalt fabric.

The cross-sections of both the beams and the columns were modeled by means of layered-section option available in SAP2000. In this way, any nonlinear behavior could be captured during the analysis. The concrete located inside the reinforcement cage was modeled as confined-concrete taking into account the Takeda model whereas the cover concrete was modeled as unconfined concrete.

The connection between the steel frame, simulating the infill wall, and the columns of the model were considered in the form of “gap” connections. This type of connection allows for high stiffness in the compression part of the shaking movement when the column comes into contact with the steel frame and completely disconnects the steel frame and the concrete column when the shaking movement is in the opposite direction. It has been successfully used in previous research works to simulate the pounding effect between buildings or structural components in a building in case of severe earthquakes [18,19].

3.3 Loading Scenarios

The RC frame had an estimated mass of 1.6 tones. The columns were restrained in the direction of the seismic motion so that to obtain a short column with a height of 300 mm, as seen in Fig. 1. The additional mass added on the slab, 1.2 tones, was modeled as a uniformly distributed load over the surface of the slab.

Generally, shake table experiments use one of the following wave forms: sine-beat, sine sweep, time history, continuous sinusoidal input. According to previous observations reported in the scientific literature [20,21], if there was no significant coupling between the orthogonal test axes of the specimen, single axes testing with sine beat is the preferred method of testing. Hence, the numerical model included time history analysis cases in the form of sine-beat functions. There were two cases considered in the generation of the sine function, namely two frequencies of the input motion: 1 Hz and 5 Hz. The starting amplitude of the uniaxial shaking, in the longitudinal direction of the model, was 0.1g and was gradually increased. In order to match the subsequent experimental program, the following loading scenarios were also considered in the numerical model, as shown in Table 2.

At the end of the sine-beat loading scenarios, the model was subjected to an artificial earthquake generated according to Eurocode8, soil type C spectrum, with a PGA of 0.14g.

After each sine-beat and time history analysis, a modal analysis case was run in order to determine any changes in the period of vibration of the model along the longitudinal axis. This would allow for the assessment of any potential damage that could occur in the real RC model during the shaking motions.

Table 2 – Loading scenarios (sine-beat function) and artificial earthquake

Load case	**Scenario**	**PGA [g]**	**Load case**	**Scenario**	**PGA [g]**
1	1Hz_0dB	0.10	7	5Hz_+6dB	0.20
2	1Hz_+3dB	0.14	8	5Hz_+9dB	0.28
3	1Hz_+6dB	0.20	9	5Hz_+12dB	0.39
4	1Hz_+9dB	0.28	10	5Hz_+18dB	0.56
5	5Hz_0dB	0.10	11	EQ_0dB	0.14
6	5Hz_+3dB	0.14	-	-	-

4. Results and Discussions

4.1 Period of vibration in the longitudinal direction

Before proceeding with the numerical analysis, the period of vibration in the longitudinal direction, the in-plane direction of the red frame shown in Fig. 1, was determined and compared to that of the scaled down model to be tested on the shake table test. The obtained results are shown in Table 3. It can be observed that the values obtained from the numerical analysis match very well the experimentally determined ones, with only a 7.53% difference between them.

Fig. 3 presents the change in the period of vibration of the model after each loading scenario from Table 2 was applied. The modal analysis cases were named in such a way so that to reflect the input frequency of the shaking motion and the amplitude, in dB, of the input signal. Hence, Modal1_9, for example, represents the modal analysis run after the sine-beat with an input frequency of 1 Hz and a signal amplitude of 9dB, 0.28g as shown in Table 2, was applied.

From Fig. 3 it can be observed that the shaking motions with the input frequency of 1 Hz and the PGA of up to 0.28g will create virtually no change in the period of vibration. A slight increase is observed for shaking motions with input frequency of 5 Hz. Almost constant values of the period of vibration in the longitudinal direction were obtained for all subsequent modal analyses, including the artificial earthquake.

There is a clear accumulation of damage which can be seen from the increase in the value of the period of vibration. In order to check whether this damage is significant or not, the damage index according to one of the proposed equations from the scientific literature [22] was evaluated. The obtained value of 0.06 means that the model would suffer only minor degradations according to the reference intervals reported in earlier research works [23].

Table 3 – Dynamic characteristics of the scaled down model and numerical simulation (longitudinal direction)

Case	Dynamic properties	
	f [Hz]	T [s]
Experiment	24.32	0.0411
Numerical model	23.03	0.0434

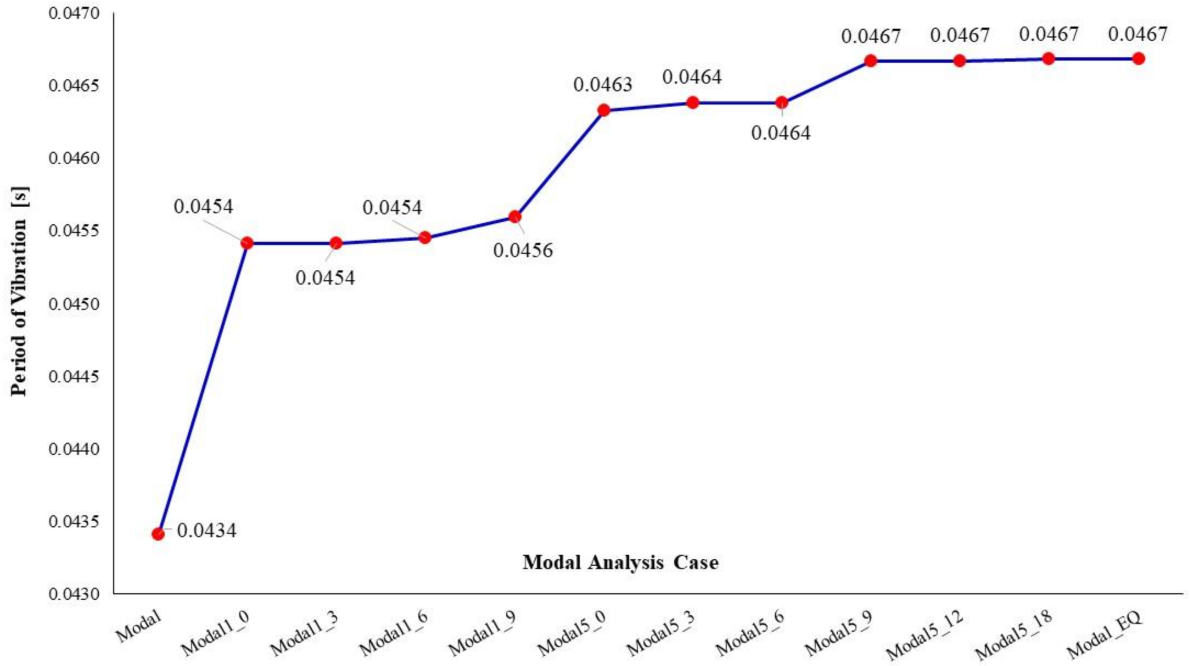


Fig. 3 – Change in the period of vibration considering the loading scenarios

4.2 Lateral displacements

The lateral displacements at the nodes of the RC frame were calculated for each loading scenario presented in Table 2. The values of lateral displacements were used as a means to determine whether degradations occurred during the non-linear analyses of the RC frame that would create a change in the stiffness in the columns. This would result in lateral displacements coupled with rotation around the vertical axis.

Fig. 4 presents the results obtained for the last nonlinear time history analysis, for the nodes 27 (shown in blue) and 31 (shown in red) located on the same side in the longitudinal direction. It can be observed that the values are matching rather well for the two selected nodes. There are differences in the positive and negative ranges of the lateral displacements due to the steel frame connected to the RC frame by means of gap elements. Therefore, for positive values of the displacement, node 31 displaces over a longer distance because the right column is not restrained in that direction. Node 27, however, displaces less because the left column comes in contact with the steel frame. Similarly in the negative direction – node 27 displaces more than node 31. These lateral displacements will be used later on to compute the maximum drift in the short columns.

Fig. 5a presents the results obtained for the last nonlinear time history analysis, for the node 27 (shown in blue) and Fig. 5b for the node 168 (shown in green) located on the same side in the transversal direction. The two graphs could not be shown in the same representation because the results are almost identical. Based on the results shown in Fig. 4 and Fig. 5 it can be concluded that no significant damages could be expected during the experimental program.

However, it should be noted that the results from the numerical analysis that were compared to the experimental ones were only in terms on initial dynamic properties in the longitudinal direction. No other calibrations were performed to the numerical model.

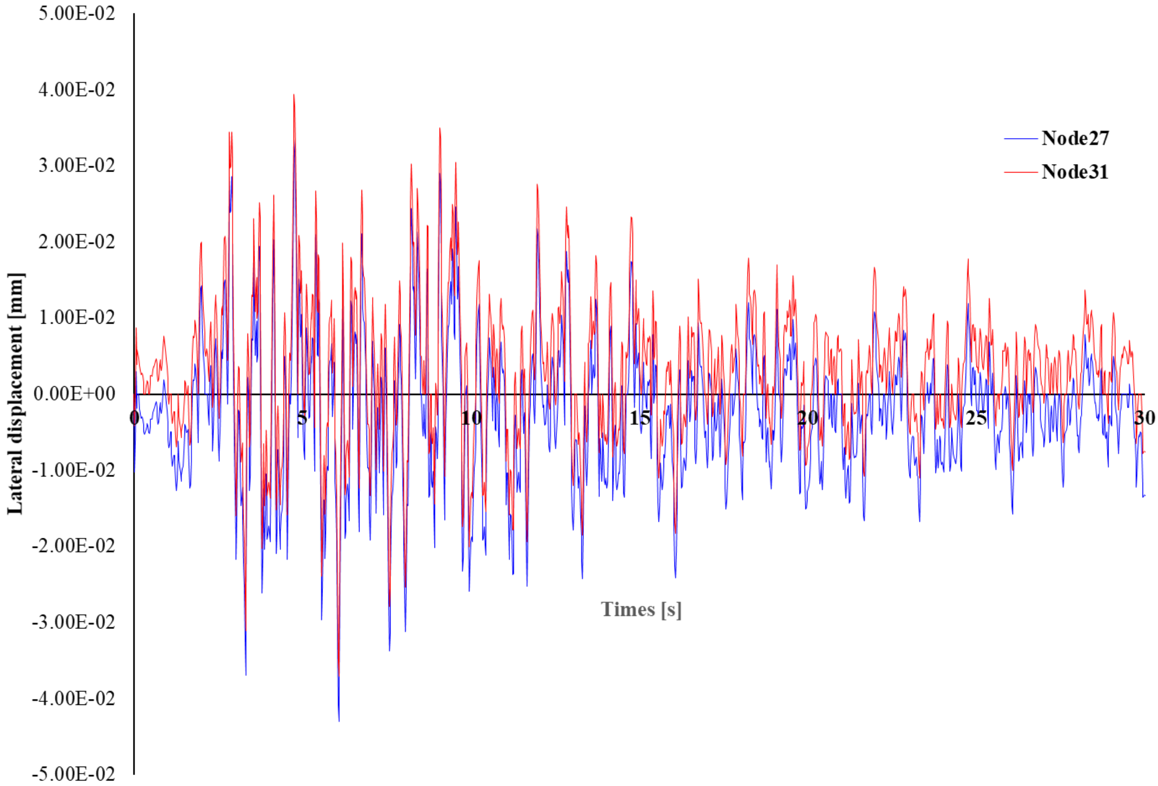
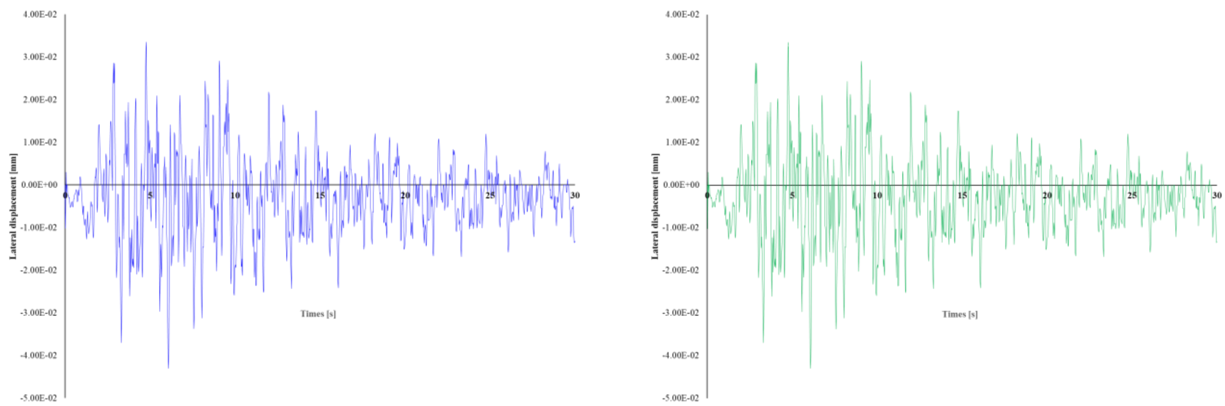


Fig. 4 – Lateral displacements for two nodes located on the same side in the longitudinal direction



a) Node 27 b) Node 168

Fig. 5 – Lateral displacements for two nodes located on the same side in the transversal direction

4.3 Maximum drift in the short columns

The lateral displacements at the lower and top sections of the short columns were recorded during the numerical analyses. The obtained data was used to compute the drift ratio corresponding to the 300 mm height of the short columns. The results are presented in Fig. 6. The short column drift ratio showed significant changes in its values by as much as 535% from the first to the last time history analysis case. Even though this change may seem significant and rather difficult to obtain in real life scenarios, the maximum drift ratio at the end of last sine-beat shaking motion was only 0.01%. This means that the model is very stiff, also proven by the values of the dynamic properties, and that there is little to no damage in the concrete columns due to repeated non-linear time history analyses.

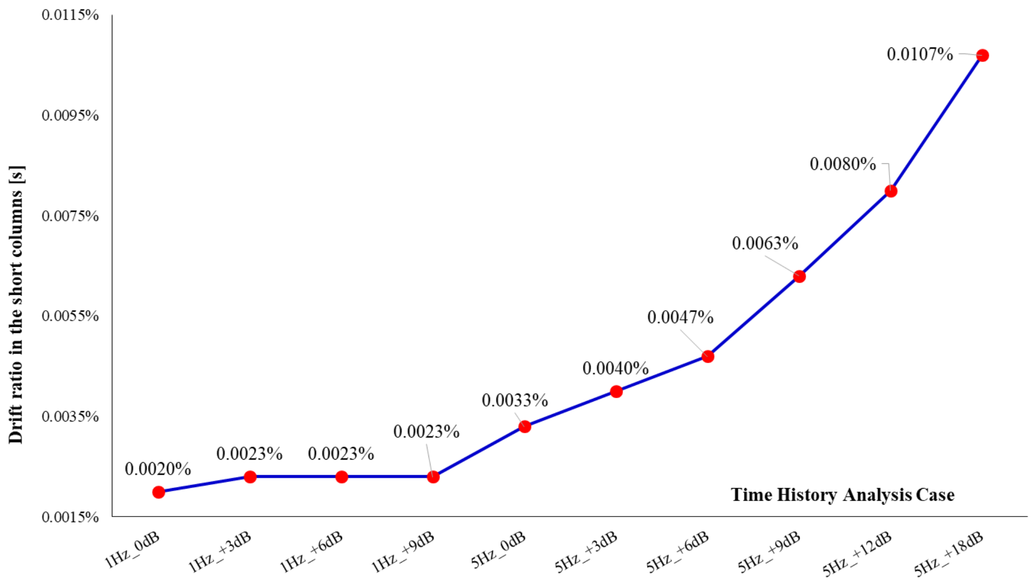


Fig. 6 – Maximum drift ratio in the short columns

5. Conclusions

The paper presents the results obtained by numerical analyses on the seismic behavior of a damaged RC frame structure retrofitted with a composite fabric. The RC frame structure was previously damaged during a series of shake table tests aimed at investigating the short column behavior during seismic excitations. The numerical model was developed based on the initial, undamaged, state of the RC frame structure and was validated by comparing the numerical results to the experimentally obtained results.

The obtained results in terms of dynamic properties of the model in the longitudinal direction show that the values obtained from the numerical analysis match very well the experimentally determined ones, with only a 7.53% difference between them.

The values of lateral displacements during each nonlinear time history analysis are used as a means to determine whether degradations occur in the RC frame that would create a change in the stiffness of the columns. This would result in lateral displacements coupled with rotation around the vertical axis. The results show that no such rotations occur, the frame only displacing along its longitudinal direction, the direction of the input motion.

The short column drift ratio shows significant changes in its values by as much as 535% from the first to the last time sine-beat history analysis case. Even though this change may seem significant and rather difficult to obtain in real life scenarios, the maximum drift ratio at the end of last sine-beat shaking motions is only 0.01%. This means that the model is very stiff, also proven by the values of the dynamic properties, and that there is little to no damage in the concrete columns due to repeated non-linear time history analyses.

The numerical model needs further calibration with the results obtained from the shake table tests on the scaled down model.

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