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Numerical investigations on the seismic performance of a scaled-down RC frame structure pre- and post-strengthening with composite membrane

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Abstract

Due to the rapid expansion of urban areas around the world and the ever-increasing number of reinforced concrete (RC) structures serving a variety of functional purposes, their behaviour in case of major seismic events has become of great interest to researchers and decision makers alike. Although initially designed to ensure certain levels of safety in case of earthquakes, the damage accumulated in a reinforced concrete structure during its lifetime due to seismic events will ultimately require its strengthening in order to comply with the new seismic design regulations.
The paper presents the results obtained by numerical analyses of the seismic performance of a scaled-down RC frame structure retrofitted with a composite membrane. The 1/3-scale symmetric structure was designed according to the specifications of the European norms and following the guidelines in the national annex for Romania. The RC frame structure was previously damaged during a series of shake table tests. The numerical model was developed based on the initial undamaged state of the frame structure and was validated by comparing the numerical results to the experimentally obtained results. 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 numerical model was then updated to include the effect of strengthening by means of composite membrane. The obtained results were used to estimate the outcomes of a new series of shake table tests on the strengthened model following the same loading set-up in terms of seismic motions.

Key words: numerical model, nonlinear THA, seismic performance, composite membrane

1 Introduction

Urban areas continue to attract more and more inhabitants due to better life conditions in terms of comfort and convenience. This prompted an increase in the housing and office buildings demand. Reinforced concrete (RC) structures represent a large percentage of the built stock in all densely populated areas or areas where human activity is met. A significant number of cities around the world were built in highly active seismic areas and are constantly subjected to severe earthquakes but mostly to frequent seismic motions of medium and small magnitudes [1-3]. Every structure is designed to fulfil its intended purpose and to ensure the safety of its occupants. However, the accumulation of damage due to severe or moderate loading scenarios, e.g. earthquakes, during its lifetime may require that the structure be repaired or strengthened to either continue to serve its purpose or to comply with the new safety demands stipulated in the design codes [4].

Initially developed for and used in aerospace engineering, fibre reinforced polymeric (FRP) composite materials quickly became the material of choice for designers when it came to strengthening the 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 [5]. FRP composite materials possess excellent properties, such as high tensile strength, high stiffness, corrosion resistance and, perhaps the most important one, light weightiness.

Columns are considered to play a significant role in the overall stability of structures during seismic events because they are expected to resist both the lateral cyclic loads induced by the earthquake and the gravitational loads. FRP composite materials proved their effectiveness [6] in strengthening the already damaged columns due to past seismic events, as demonstrated by a series of recent shake table tests [7, 8].

The advancement of knowledge regarding the seismic behaviour of RC frame structures is slowed by the lack of experimental data documenting the realistic responses of structures tested under seismic scenarios and is mainly caused by the expensive nature of the involved testing equipment (shake tables) as well as by the complexity of such experimental investigations. Numerical simulations came as an alternative to experimental investigation in many research areas of civil engineering [9]. Perhaps the main advantage of numerical simulations is the possibility of investigating the influence of a large number of parameters varying over large intervals on the behaviour of structural systems subjected to seismic motions. However, the numerical model needs to be calibrated and validated by means of experimental results in order to obtain reliable results. Currently, there are two approaches to the numerical modelling 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 [10]. 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, yet it fails 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 the numerical analyses on the seismic behaviour of a damaged RC frame structure pre- and post-strengthening with composite membrane. The RC frame structure was previously damaged during a series of shake table tests aimed at investigating the short column behaviour during seismic excitations [11]. 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. The average compressive strength, obtained from uniaxial compression tests on 9 cylindrical specimens (100 mm x 200 mm) was f_c' = 35 MPa at 28 days. The modulus of elasticity, computed as an average of 8 measurements, was $E_{c,28}$ = 34.03 GPa [11].

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 the ultimate tensile strengths were determined by means of direct tensile tests and they were f_y = 513 MPa and f_u = 626 MPa, respectively [11]. The stirrups (shear reinforcement) were made of Grade250 steel.

2.3 Repairing mortar and composite membrane

The damaged part of the columns were repaired by means of MAPEGROUT HI-FLOW fibre reinforced mortar. 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 fibre reinforced polymer strengthening solution consisted in a unidirectional high strength basalt fibre membrane that was applied at the level of the short columns where most of the damage occurred, the ends of both longitudinal and transversal beams and the node of the RC frame structure. The mechanical properties of the basalt fibres and of the composite material, fabric and resin, 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 [g/m²]	**Tensile strength** [MPa]	**Tensile modulus of elasticity** [GPa]	**Tensile strength** [MPa]	**Tensile modulus of elasticity** [GPa]	**Elongation at failure** [%]
400	2500	70	2300	70	2.5

3 Numerical model

3.1 Geometry

The geometry of the model is 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 [11]. 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.

In the case of the numerical model, the fixed supports were considered at the level of the steel frame used to fix the model on the shake table, resulting in a total height of 1360 mm.

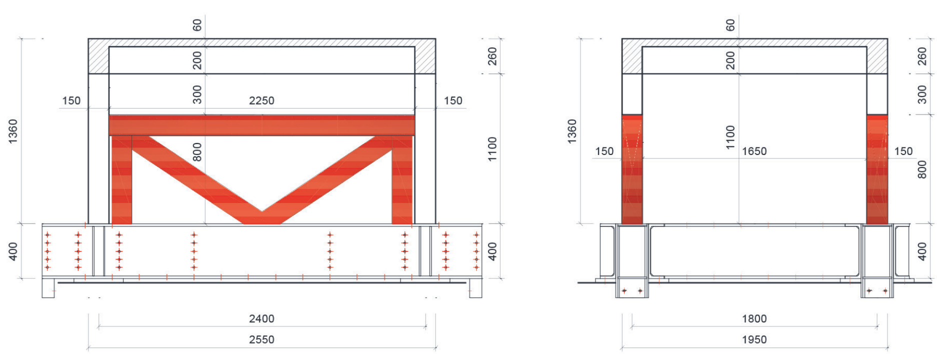

Figure 1. Geometry of the model (steel frame to simulate the partial infill wall is shown in red)

3.2 Material models

The concrete part of the model was considered with the mechanical properties previously presented. Since the short columns were rebuilt using fibre reinforced mortar and then confined with basalt fabric, different levels of restoring the initial strength were assumed: 70 %, 80 %, 90 % and full strength.
The cross-sections of both the beams and the columns were modelled by means of layered-section option. In this way, any nonlinear behaviour could be captured during the analysis. The concrete located inside the reinforcement cage was modelled as confined-concrete taking into account the Takeda model whereas the cover concrete was modelled as unconfined concrete. For the confined part of the columns, located between the steel frame and the lower face of the beams, the whole concrete part was modelled as confined concrete. A bilinear stress-strain model was adopted for both the longitudinal and the shear reinforcements. The numerical model is shown in Fig. 2.

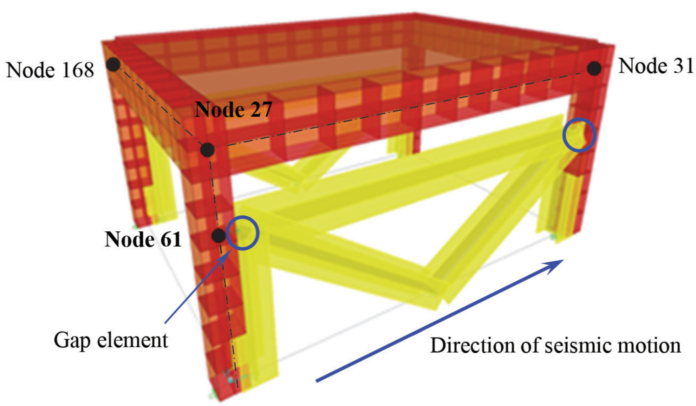

Figure 2. Numerical model (SAP2000)

The connection between the steel frame, simulating the partial infill wall, and the columns of the model were considered in the form of “gap” connections. This type of connection allowed for high stiffness in compression and zero stiffness in tension and alternatively allows for the contact between the columns and the steel frame or their complete separation during the seismic motion. 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 [12].

3.3 Loading scenarios

The self-weight of the model was automatically computed from the geometry of the structure and the material characteristics. The additional weight added on the slab for the experimental set-up [11] was considered as a uniformly distributed load over the entire 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 [13], 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 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	5	5Hz_+6dB	0.20
2	1Hz_+6dB	0.20	6	5Hz_+9dB	0.28
3	1Hz_+9dB	0.28	7	5Hz_+18dB	0.56
4	5Hz_0dB	0.10	8	EQ_0dB	0.14

4 Results and discussions

4.1 Fundamental period of vibration

Before proceeding with the numerical analysis, the fundamental period of vibration was determined and compared to that of the scaled down model to be tested on the shake table test. The results obtained are T_{exp} = 0.0411 seconds, whereas $T_{numerical}$ = 0.0434 seconds, with only 7.53 % difference between them.
The four graphs shown in Fig. 3 shown the change in the fundamental period of vibration, for each restoring degree of strength and stiffness, with each loading scenario from Table 2. As mentioned previously, the degree of restoring the strength and stiffness after strengthening with the FRP composite membrane was assumed to vary between 70 % and 100 % (fully restored) from the initial, undamaged model.

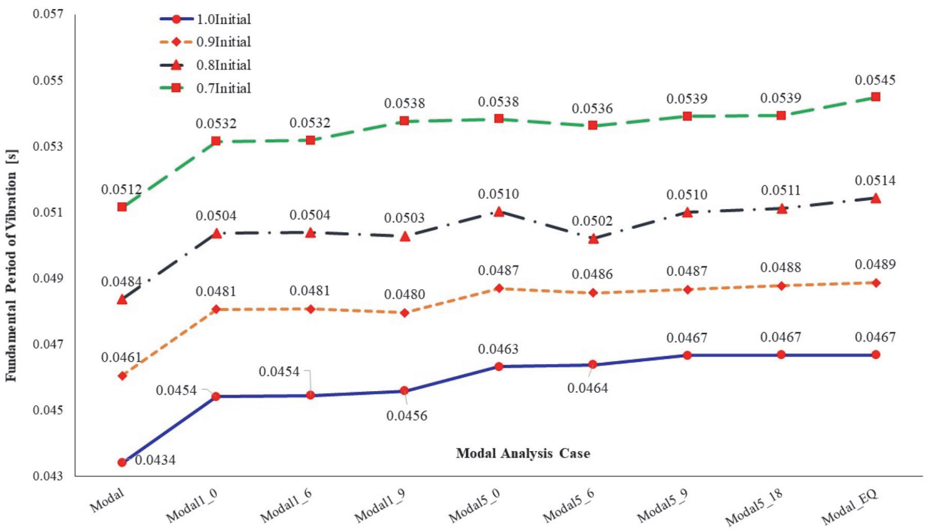

Figure 3. Change in the fundamental period of vibration considering the loading scenarios

The modal analysis cases were named in such a way so as to reflect the input frequency of the shaking motion and the amplitude of the input signal in dB. Hence, Modal5_6, for example, represents the modal analysis run after the sine-beat with an input frequency of 5 Hz and a signal amplitude of 6 dB, 0.20g, as shown in Table 2, was applied.

In 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, for restoring levels of 80 %, 90 % and 100 %. When the level of restoring was assumed to be 70 %, there was a slight increase in the fundamental period of vibration with increasing the amplitude of the input signal. 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, especially for 0.9Initial and 1.0Initial cases. Based on the results obtained, it can be concluded that there is a clear accumulation of damage which can be seen from the increase in the value of the period of vibration. Moreover, the lower the assumed restoring level strength and stiffness of the short columns, the higher the fundamental period of vibration because the model becomes more flexible.

4.2 Maximum drift in the short columns

Fig. 4 presents the change of the maximum drift ratio in the short columns, due to repeated shaking motions, for each of the assumed restoring levels. The lateral displacements at the lower and top sections of the short columns were recorded during the numerical analyses, e.g. for nodes 27 and 61 from Fig. 2. The data obtained was used to compute the drift ratio corresponding to the 300 mm height of the short columns, as seen in Fig. 1.

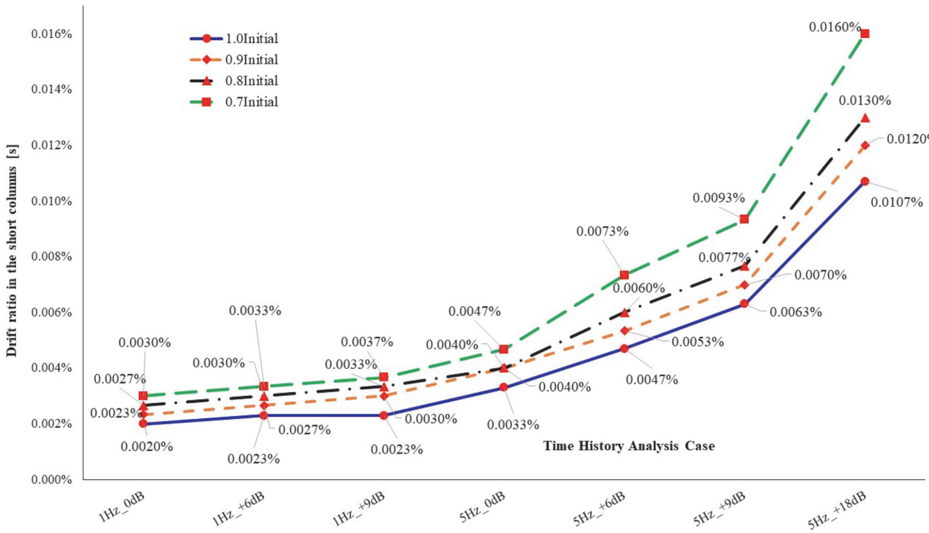

Figure 4. Maximum drift ratio in the short columns

Fig. 4 shows that all four restoring levels have a similar trend in terms of maximum drift ratio in the short columns. The increase in the values of the drift ratio from the first non-linear THA case to the last one ranges between 487 % and 535 %.

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 between 0.01 % and 0.016 %. 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.

5 Conclusions

The paper presents the results obtained by numerical analyses on the seismic behaviour pre- and post-strengthening with a FRP composite membrane. The RC frame structure was previously damaged during a series of shake table tests aimed at investigating the short column behaviour 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 numerical analyses were conducted in order to obtain a broad image of what is expected of the strengthened model during a subsequent series of shake table tests after strengthening. Since the strengthened part cannot be removed from the structure, four levels of restoring the strength and stiffness are assumed for the numerical analyses.

Based on the obtained results it can be concluded that there is a clear accumulation of damage which can be seen from the increase in the value of the period of vibration.

Moreover, the lower the assumed restoring level strength and stiffness of the short columns, the higher the fundamental period of vibration because the model becomes more flexible.
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 between 0.01 % and 0.016 %, depending on the considered restoring level. 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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