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ANALYSIS OF SUBSTANDARD RC BUILDINGS STRENGTHENED WITH AN INNOVATIVE POST-TENSIONED METAL STRAPPING TECHNIQUE

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Abstract

This paper investigates analytically the efficiency of an innovative Post-Tensioned Metal Strapping (PTMS) strengthening technique at improving the seismic behaviour of a reinforced concrete (RC) building. A full-scale two-floor building with substandard reinforcement detailing at the columns and beam-column joints was tested on a shake table as part of the EU-funded research project BANDIT. After the bare building was initially damaged significantly, it was repaired and strengthened with PTMS to perform additional seismic tests at various Peak Ground Acceleration (PGA) levels. The PTMS technique improved considerably the seismic performance of the tested building. Whilst the bare building experienced critical damage at an earthquake of PGA=0.15g, the PTMS-strengthened building sustained a PGA=0.35g earthquake without compromising stability. In this study, the experimental data from the initial shake table tests are used to calibrate analytical models through nonlinear time-history analyses. To simulate deficient beam-column joints, models of steel-concrete bond-slip and bond-strength degradation under cyclic loading are considered. The results show that the PTMS strengthening intervention significantly enhanced the seismic behaviour of the substandard beam-column joints, as well as the deformation capacity of the building. It is also shown that the analytical models of both bare and strengthened buildings provide a reasonable estimate of the maximum deformation demands for earthquake excitations with different PGA levels.

Keywords: shake table tests; RC buildings; steel strapping; nonlinear time-history analysis; inter-storey drift

1. Introduction

High human and financial losses caused by recent severe earthquakes in developing countries (e.g. China, 2008; Indonesia, 2009; Haiti, 2010; Turkey, 2011; Nepal, 2015) have highlighted the high seismic vulnerability of the existing building stock, much of which comprises non-ductile reinforced concrete (RC) frame buildings built before the introduction of modern seismic design guidelines. The collapse of many of these substandard structures has been often attributed to failure of beam-column joints due to inadequate detailing of the joint reinforcement. Different conventional retrofitting techniques have been examined in the past to enhance the performance of substandard joints, such as concrete/shotcrete jacketing [1-3] and steel jacketing/steel plates [3-5]. However, these techniques can be highly invasive, labour intensive and they usually increase the mass of the building. More recently, the use of externally bonded Fibre Reinforced Polymers (FRP) has proven very effective at enhancing the seismic performance of deficient joints [e.g. 6, 7-13]. In comparison to the above traditional retrofitting techniques, FRP materials have advantages such as high resistance to corrosion, excellent durability, high strength to weight ratio, and ease and speed of in-situ applications. However, the relatively high initial cost of the FRP materials may prevent its wide use in developing countries, where the material costs account for a large proportion of the expenses.

Pioneering research at the University of Sheffield [14, 15] has led to the development of a novel technique for retrofitting RC beams and columns using Post-Tensioned Metal Straps (PTMS). The technique uses high-strength ductile steel straps post-tensioned around RC elements using air-operated strapping tools such as those used in the packaging industry. To maintain the tensioning force at PTMS after post-tensioning, the strips are fastened mechanically using jaws and push type seals. This provides active confinement to the members, thus increasing their ductility and capacity before applying load. In comparison to traditional steel/concrete jacketing, PTMS retrofitting has advantages such as ease and speed of application, low material cost and ease of quickly removing/replacing damaged strips and seals. As no adhesives or sophisticated equipment is used, PTMS retrofitting is also expected to provide cost-effective strengthening solutions due to lower material costs. Previous tests have confirmed the effectiveness of the PTMS technique at enhancing the capacity of columns under compression [16], beams with short lap-splices [17] and substandard beam-column joints subjected to cyclic load [18]. More recently, the technique was proven extremely effective at enhancing the seismic behaviour of a full-scale PTMS-strengthened RC building subjected to shake table tests [19]. However, the behaviour of PTMS-strengthened structures has not been investigated numerically. Calibrated numerical models can be efficiently used for assessing the seismic performance of buildings using different seismic records and strengthening strategies.

This study aims to investigate analytically the efficiency of the PTMS strengthening technique at improving the seismic behaviour of the substandard reinforced concrete (RC) building tested on a shake table as part of the EU-funded research project BANDIT. The experimental data from the initial shake table tests are used to calibrate analytical models through nonlinear time-history analyses. To simulate the cyclic behaviour of deficient beam-column joints, the effects of steel-concrete bond-slip and bond-strength degradation are considered. This paper details the adopted strengthening strategy, discusses global results from the shake table tests and presents the analytical approach used to calibrate Finite Element (FE) models. This study is part of a comprehensive research effort focusing on the seismic retrofitting of substandard RC buildings using PTMS [19] and PTMS composites [20, 21].

2. Experimental programme

The seismic performance of a substandard RC building was investigated through a series of unidirectional full-scale shaking table tests. The building was initially damaged significantly, and then repaired and strengthened with PTMS before additional seismic tests were performed. A summary of the testing programme is given in the following sections and more details can be found in references [19] and [22].

2.1 Geometry, material properties and set-up of experiments

The selected substandard one-bay two-floor building was regular in plan and elevation, and was designed to have inadequate reinforcement detailing at the columns and beam-column joints. The general geometry, element sections and corresponding reinforcement are shown in Fig. 1. To simulate permanent and variable loads, additional masses of 13.5 and 11.0 tonnes were fixed to the 1st and 2nd floor slabs using steel plates and concrete blocks. The compressive strength and elastic modulus of the concrete used to cast the building were f_c=30.7 MPa and E_c=24.3 GPa for the 1st floor, and f_c=24.6 MPa and E_c=21.2 GPa for the 2nd floor. The yield and ultimate strength of the longitudinal bars were f_y=526 MPa and f_u=616 MPa.

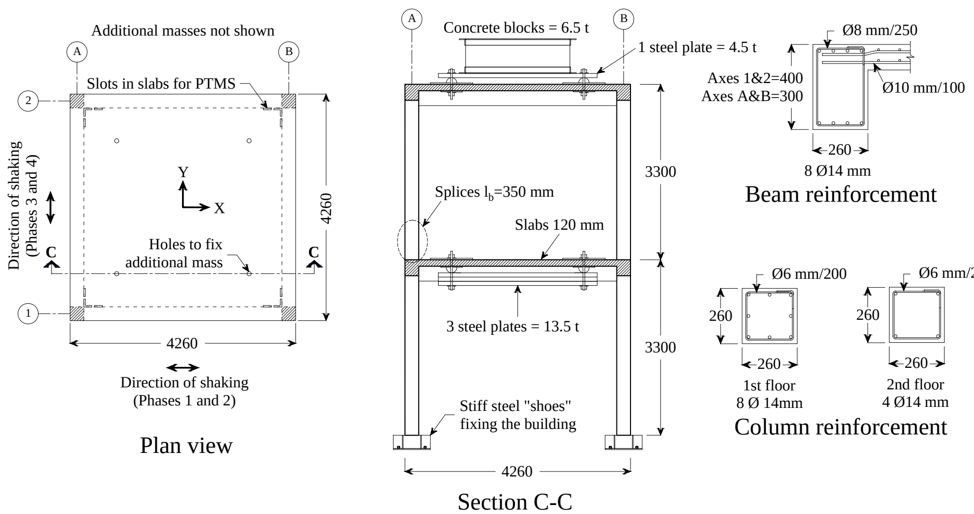

Fig. 1 – General view (left) and geometry (right) of the bare building (adapted from reference [19])

The structure was instrumented with displacement and acceleration transducers at each floor to monitor the displacement history during the experiments. In this study, maximum drift was selected as an indicator of damage to structural and non-structural elements and was used for validation of the FE models based on the results of nonlinear time-history analyses.

2.2 Tests on the bare and PTMS-strengthened structure

Unidirectional horizontal input shaking table tests were carried out on the original building using increasing PGA accelerations ranging from 0.05g to 0.15g. A single artificial record was used, based on the Eurocode 8 soil type C spectrum [23]. Natural frequencies of the structure were obtained using white noise as input signal before and after each test. For this purpose, a low intensity excitation containing a frequency range of 0.5-50 Hz was used. The accelerations recorded at the base and at each floor were then post-processed to identify the natural frequencies of the first two vibration modes. As expected, significant damage was observed at the 2nd floor beam-column joints and at the 2nd floor columns after the initial tests (see Fig. 2). Little damage was observed in the beams and in the 1st floor joints.

Fig. 2 – Damage in 2nd floor joints after the test PGA=0.15g on original building

After the initial tests, the damaged joints and columns were strengthened locally using PTMS. The main purpose of the intervention was to produce a beam mechanism, which is in line with modern seismic design philosophy. Before the intervention, the damaged concrete was repaired and the main cracks were filled by injecting epoxy resin. The PTMS was applied adopting the following procedure:

1. Six 10 mm thick anchor steel plates were fixed to the columns, beams and top of the 2nd floor slab using steel bolts inserted in holes prefilled with epoxy mortar. After the adhesive set, the plates were positioned and partially tightened with nuts and washers leaving a small gap of approximately 1 mm between the plates and the concrete faces, which was necessary to secure two layers of metal straps.

2. Nine horizontal straps at 50 mm spacing were placed at column ends to provide confinement and increase their shear strength.

3. One layer of straps was inserted in the pre-formed slots of the slabs to confine the beam ends and also increase their shear capacity.

4. Eight straps (two layers each) were installed parallel to the longitudinal beams axes (i.e. horizontally) at 50 mm centres to provide confinement to the beam-column joint. These straps were anchored around the steel plates.

5. Six straps (two layers each) were provided along the outer faces of the columns (parallel to the column axis) to enhance their flexural capacity. For the 2nd floor joints, the six straps were bent at 90° at the slab edges and secured to steel plates located on the top of the 2nd floor slab. Afterwards, the nuts of the bolts securing the steel plates were tightened by hand using a spanner to prevent the loss of prestressing in the straps.

6. Finally, one layer of confinement straps was placed around beams and columns to prevent excessive buckling of the horizontal and longitudinal straps installed at Stages 4 and 5.

All straps were fastened using “push type" seals of 25 mm length. Fig. 3 shows a typical beam-column joint after the PTMS strengthening. As can be seen, the metal strapping provided a confining grid around the beam-column joint and columns ends.

Fig. 3 – View of a typical PTMS-strengthened 2nd floor joint

After the PTMS strengthening, further shaking table tests were conducted with PGA accelerations ranging from 0.05g to 0.35g. No significant damage was observed at the PTMS strips or the concrete. Whilst the building was clearly capable of sustaining seismic excitations at higher PGA intensities, the tests were halted after a PGA level of 0.35g to assess the local damage at the joints and the global condition of the structure. The majority of the straps were intact at this stage. The structural damage was maintained within repairable limits as further shaking table tests were planned on the building, as reported in reference [22]. Overall, the results indicated that the PTMS strengthening was very effective at improving the seismic performance of the building.

Fig. 4 – Damage at 1st floor joints after tests on the PTMS-strengthened building

3. Analytical Modelling

3.1 Modelling of the bare and PTMS-strengthened buildings

The experimental results on the bare and strengthened buildings were used to calibrate numerical models developed in DRAIN-3DX [24]. Because of the symmetry in the structure, the buildings were modelled in 2D for computational efficiency (Fig. 5 left). Beams and columns were modelled using a fibre element (element 15) with distributed plasticity. To increase the accuracy of the analysis, each section comprised of discrete steel and concrete fibres (Fig. 5 right). The behaviour of the steel reinforcement and concrete was characterised by the constitutive models given in Eurocode 2 [25]. The effect of accumulated stiffness degradation was included in the analyses by considering a stiffness degradation factor in the stress-strain relationship of concrete. Concentrated loads were assigned at intermediate nodes of the beams to simulate the distributed load from slabs and beams. Additional nodes added at the top and bottom of the outermost column elements simulated the actual geometry of columns and beam-column joints. The masses at each floor were lumped at the two corresponding exterior nodes and calculated assuming a concrete density equal to 24 kN/m^3. Elastic damping was introduced through a mass damping coefficient and an element stiffness coefficient using a Rayleigh damping model [26].

Trial values of 3 to 5% were assigned to the 1st mode of vibration, and 2 to 4% for the 2nd mode. Second order (P-Δ) effects were also included in the analysis.

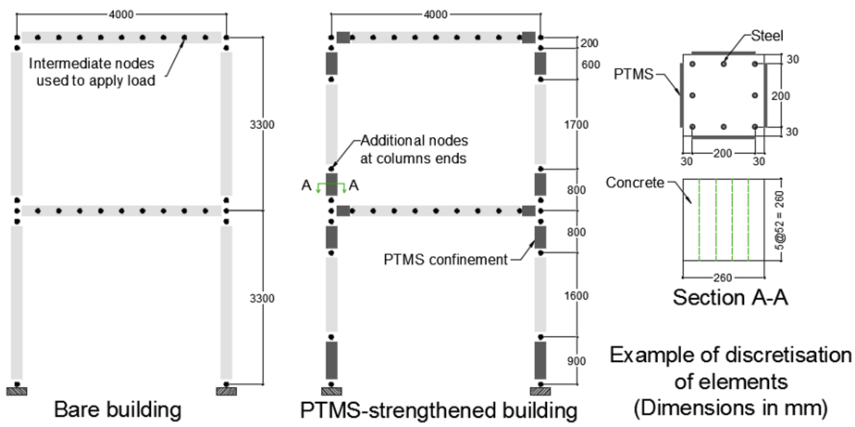

Fig. 5 – building models (left) and fibre elements (right) used in DRAIN-3DX for the building

In the analytical models, deformations occurring at the joints were taken into account by using zero-length connection hinges at column ends. Previous studies [27] have shown the need of considering the additional deformations generated by bond-slip of steel bars. The fibre properties used for the elements were chosen to model bond-slip within the beam-column joint and including degradation parameters. Partial degradation was initially assigned to both bond-stiffness and bond-strength. Gap properties were assigned at the connection face to simulate crack opening. In order to model the strengthened building, additional nodes were added to define the PTMS-confined zones of the columns. The effect of the PTMS confinement was introduced by using the constitutive model for confined concrete proposed by Moghaddam et al. [16]. Ultimate compression strength and ultimate strain used for the analysis were f_{cc}=38.4 MPa and ε_{cc}=0.115 for the 1st floor, respectively, whereas such values were f_{cc}=33.1 MPa and ε_{cc}=0.128 for the 2nd floor. The straps fixed parallel to the beams and columns axes modelled using fibres with the steel material properties obtained from coupon tests. Bond degradation parameters were reduced to reflect the effect of the additional confinement provided by the PTMS, as well as the longitudinal straps along the columns' axes.

4. Model calibration and discussion

The experimental data from the shake table tests were used to calibrate the analytical models developed in DRAIN-3DX [24]. Natural frequencies obtained from white noise tests are compared with analytical results in Table 1. As can be seen, for the first two modes of vibration, the dynamic properties of the bare and strengthened buildings are well captured (within a 5-10% accuracy) by the analytical models.

Table 1 – Structural frequency of the buildings obtained from tests and analysis (in Hz)

Mode No.	Bare building		PTMS-strengthened	
	Tests	Analysis	Tests	Analysis
1	2.09	1.90	1.65	1.75
2	5.57	5.24	5.01	4.56

The experimental and analytical displacement histories of the 1st and 2nd floors for the bare and PTMS-strengthened buildings are compared in Fig. 8 and Fig. 9. Due to space limitations, only the results for PGA of

0.05g and 0.15g are presented in this paper, which are representative of the elastic and post-cracking range of behaviour. In spite of some differences, the illustrated results indicate that the predicted and measured displacements compare reasonably well along the entire time duration of the excitation. Note that the test at a PGA level of 0.15g was halted after approximately 20.0 s due to resonance issues with the shaking table (shown as a × symbol in Fig. 7).

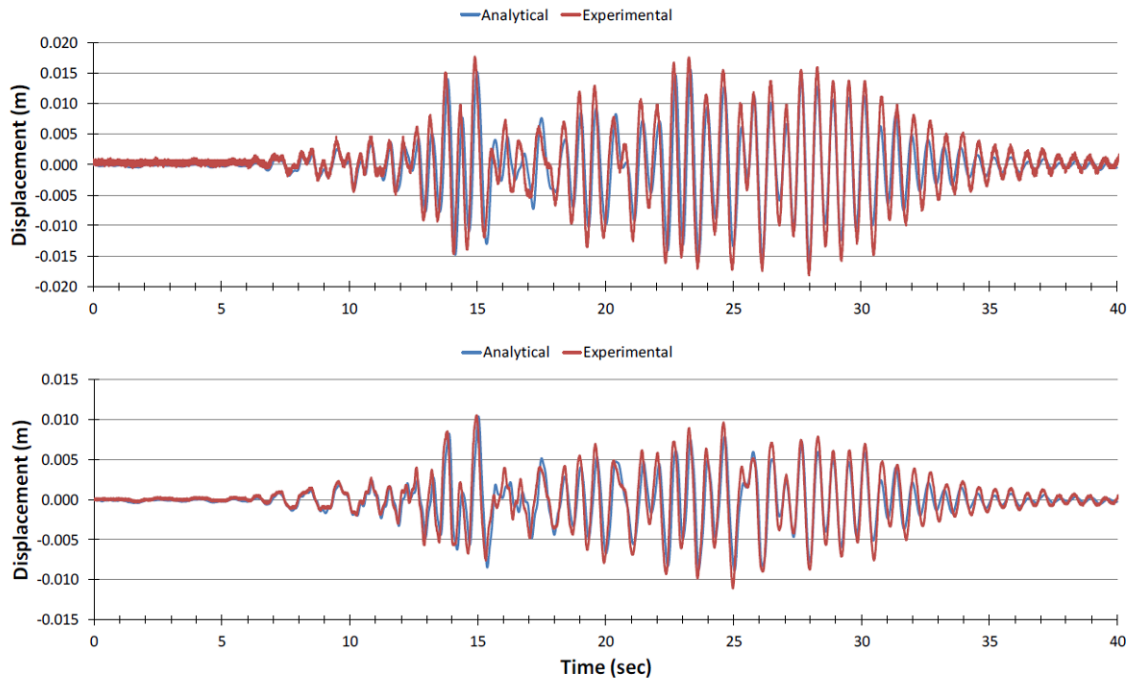

Fig. 6. Results from nonlinear time history analysis for 2nd (top) and 1st (bottom) floors at PGA=0.05g, bare building

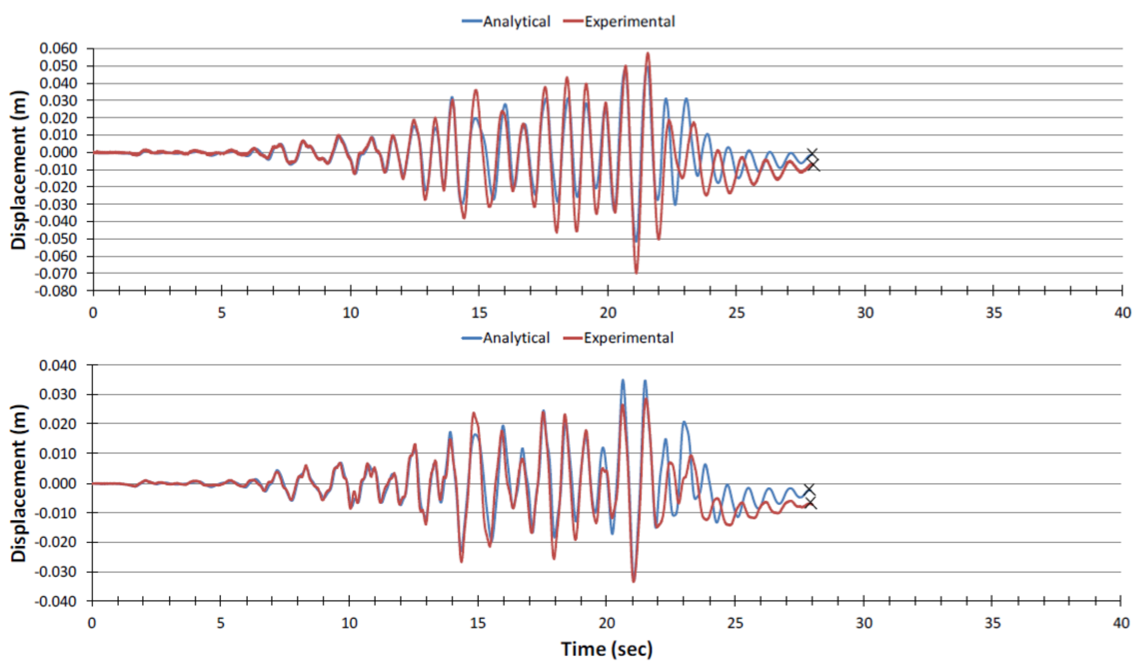

Fig. 7. Results from nonlinear time history analysis for 2nd (top) and 1st (bottom) floors at PGA=0.15g, bare building

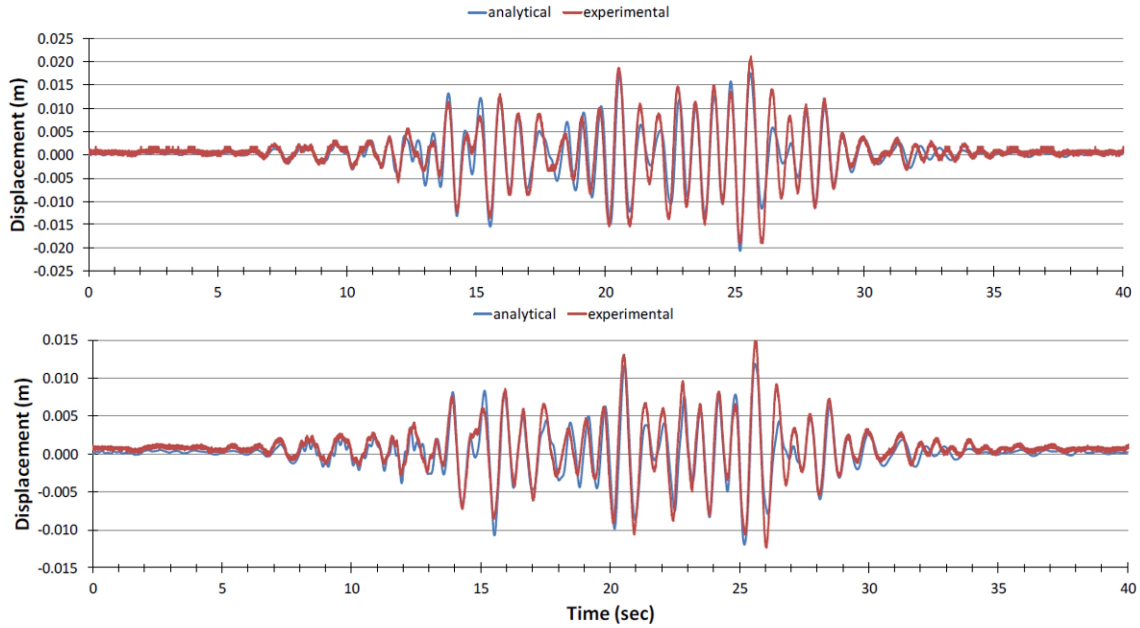

Fig. 8. Results from nonlinear time history analysis for 2nd (top) and 1st (bottom) at PGA=0.05g, PTMS-strengthened building

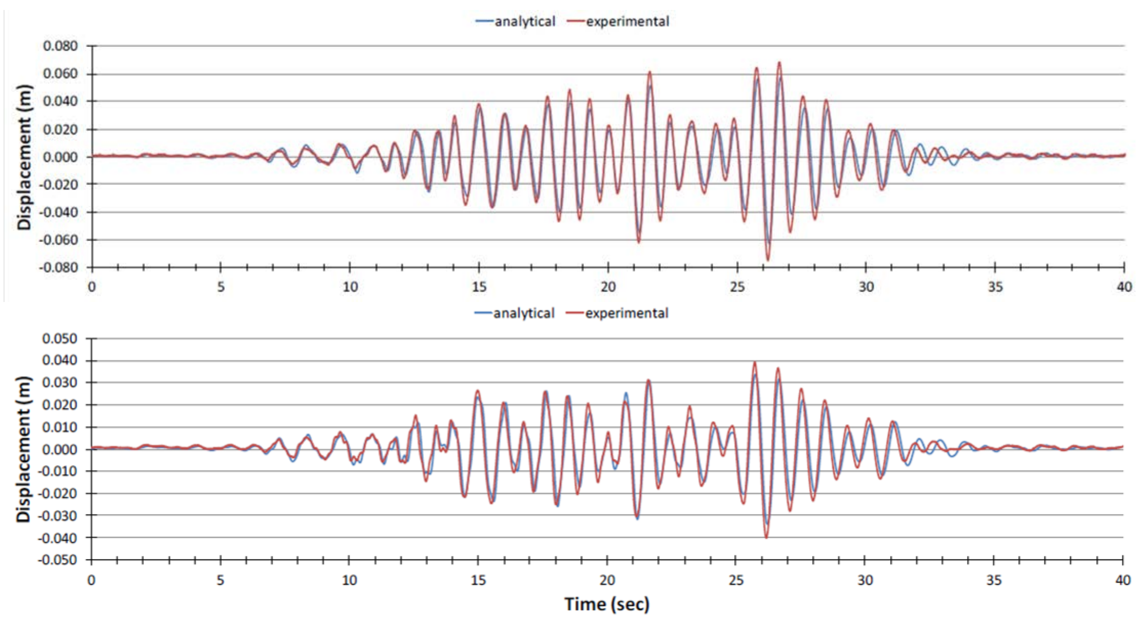

Fig. 9. Results from nonlinear time history analysis for 2nd (top) and 1st (bottom) at PGA=0.15g, PTMS-strengthened building

Table 2 compares the maximum inter-storey drift ratios obtained from experimental tests and analytical models for different PGA levels. In general terms, the analytical models for both bare and strengthened buildings tend to slightly underestimate the storey drifts after a PGA of 0.25g. However, the analytical models of both bare and strengthened building provide a reasonable estimate of the maximum drifts for earthquake excitations with different PGA intensities. This may be attributed to the fact that the PTMS strengthening changed the behaviour of the bare building by preventing the joint failure at the 2nd floor.

Table 2 – Inter-storey drift ratios from experiments and analysis (in %)

PGA	Floor	Bare building		PTMS-strengthened	
		Tests	**Analysis**	**Tests**	**Analysis**
0.05g	2nd	0.3	0.3	0.3	0.3
	1st	0.3	0.2	0.3	0.3
0.15g	2nd	1.2	0.7	1.1	0.9
	1st	0.8	0.9	1.0	0.8
0.25g	2nd	-	-	2.0	1.5
	1st	-	-	1.6	1.5
0.35g	2nd	-	-	2.6	2.2
	1st	-	-	2.0	2.3

5. Conclusions

This paper presented an experimental and analytical investigation on the efficiency of the PTMS strengthening technique at improving the seismic performance of a deficient full-scale two-floor RC building tested on a shake table. The results from the analyses show that the use of PTMS prevents failure of the 2nd floor beam-column joints and increases significantly the deformation capacity of the substandard building. The analytical models of both bare and strengthened buildings provide a reasonable estimate of the maximum inter-storey drifts for earthquake excitations with different PGAs. Such calibrated numerical models can be efficiently used for assessing the performance of PTMS-strengthened buildings using different seismic records and strengthening strategies.

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