Seismic Strengthening of Deficient RC Buildings Using Post-Tensioned Metal Straps: An experimental Investigation

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LISBOA 2012

SUMMARY:
This paper presents some of the preliminary results of the European research project BANDIT (SERIES Programme). The project investigates experimentally the efficiency of Post-Tensioned Metal Straps (PTMS) strengthening at improving the seismic performance of a seismically deficient full-scale two-storey RC building tested on a shaking table. To simulate typical deficient buildings, the reinforcement of columns and beam-column joints were poorly detailed. The first two test stages performed on the BANDIT building are summarised here. In Stage 1, the bare building was tested to produce damage and evaluate its basic performance. After the initial tests, the damaged building was repaired and subsequently strengthened using PTMS. In Stage 2 the building was re-tested to examine the efficiency of the PTMS technique. The results show that the PTMS strengthening improved substantially the seismic performance of the deficient building.

Keywords: Post-Tensioned Metal Straps, seismic strengthening, shake table tests, RC deficient buildings

1. INTRODUCTION

Much of the existing building stock in Europe and many developing countries has been designed using old standards with little seismic provision and often suffers from poor material and construction practices. As a consequence, these buildings have deficient lateral load resistance, insufficient energy dissipation and can rapidly lose their strength during earthquakes, leading to collapse. Extensive human and economic losses during past strong earthquakes (Turkey 2003; Pakistan 2005; China, 2008; Indonesia and Italy, 2009; Haiti, 2010) have highlighted the seismic vulnerability of substandard reinforced concrete (RC) buildings. The strengthening of seismically deficient structures provides a feasible and cost-effective approach to improving their load carrying capacity and reducing their vulnerability.

Over the last years, the use of external Post-Tensioned Metal Straps (PTMS) has offered engineers an alternative solution for strengthening deficient RC members (Frangou et al, 1995; Moghaddam et al, 2010a and b). Comparatively to other strengthening techniques, PTMS possess advantages such as ease and speed of in-situ application, ease of removing/replacing the strengthening, relatively low cost of materials, flexibility to strengthen selectively only those deficient members, and the opportunity of applying active confinement before the strengthened element is subjected to load. The PTMS technique involves the post-tensioning of high-strength steel straps (bands) around concrete members using hydraulically-powered standard strapping tensioning tools as those used in the packing industry. To maintain the post-tensioning force, the straps are mechanically-fastened using metal clips clamped with a pneumatic sealer (jaws). Therefore, the effective lateral stress (active confinement) applied on the concrete enhances the load carrying capacity and ductility of the strengthened member. Whilst the PTMS technique has proven extremely effective in the strengthening of medium-scale beams and columns (see Figure 1), its effectiveness has never been tested on deficient full-scale RC structures subjected to simulated earthquake excitations.

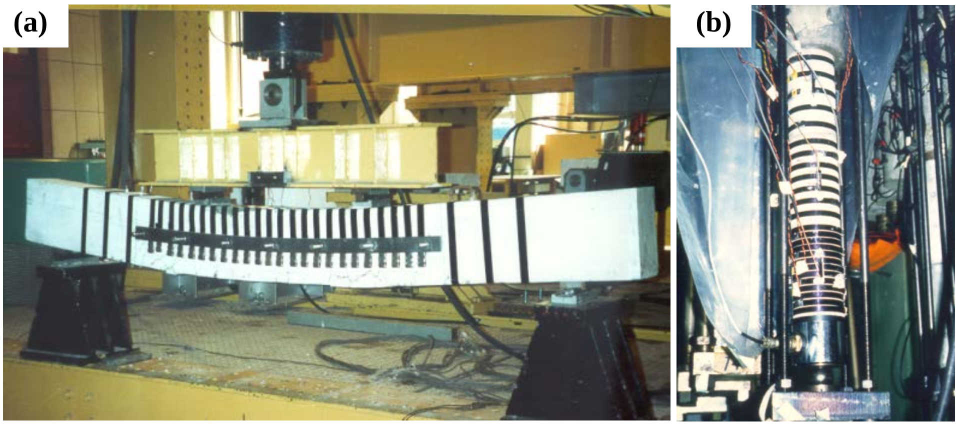

Figure 1. PTMS strengthening on (a) beams, and (b) columns

The EU-funded BANDIT project (part of the SERIES Programme) investigates experimentally the efficiency of PTMS strengthening at improving the seismic performance of a seismically deficient full-scale two-storey RC building. The building was tested on the AZALEE shaking table of the EMSI laboratory (Laboratoire d'Etudes de Mécanique SIsmique), at CEA Saclay (France). To simulate typical deficient buildings of Mediterranean and developing countries, the BANDIT building had inadequate detailing of reinforcement in beam-column joints and columns. This paper summarises the results from the first two test stages performed on the BANDIT building. In Stage 1, the bare building was tested using uniaxial shaking to produce significant damage and evaluate its basic performance. After the initial tests, the damaged building was repaired and subsequently strengthened using PTMS. In Stage 2 the building was re-tested to examine the efficiency of the PTMS as a post-earthquake strengthening alternative. The results are discussed in terms of natural frequencies, maximum displacements and damage observed after each test stage.

2. EXPERIMENTAL PROGRAMME

2.1. Geometry of the building

The building was a one-bay two-storey moment-resisting frame regular in plan (4.26×4.26 m) and with a constant storey height of 3.30 m (see Figure 2a and b).The columns had a cross section of 260×260 mm as shown in Figure 2d. The longitudinal reinforcement of the first floor columns consisted of eight 14 mm bars placed along the column perimeter, whilst only four 14 mm bars located at the column corners were used for the second floor columns. The columns were reinforced in shear using transverse stirrups of 6 mm diameter spaced at 200 mm centres. The cross section of the beams in the X direction was 260×400 mm, whilst the beams in the Y direction had a cross section of 260×300 mm. The main flexural reinforcement consisted of four 14 mm bars at the bottom and four 14 mm bars at the top. The design flexural capacities of the 1st storey columns and beams were 39 and 92 kNm ($\Sigma M_{Rcol}/\Sigma M_{Rbeam}$=0.85). Therefore, the behaviour of the BANDIT building was expected to be dominated by a strong beam-weak column mechanism. The beam had transversal reinforcement consisting of 8 mm stirrups spaced at 300 mm centres. The geometry and reinforcement of beams and columns in the test direction (X direction, see Figure 2c) were identical to those used in ECOLEADER Project (Garcia et al, 2010). To simulate dead and live loads, additional masses were fixed to the 120 mm thick concrete slabs. Three steel plates with a total mass of 13.5 tonnes were fixed beneath the 1st floor slab, whilst one steel plate and twelve concrete blocks were clamped to the top of the 2nd floor slab adding a total mass of 11.0 tonnes. The estimated self-weight of the building was 20.4 tonnes.

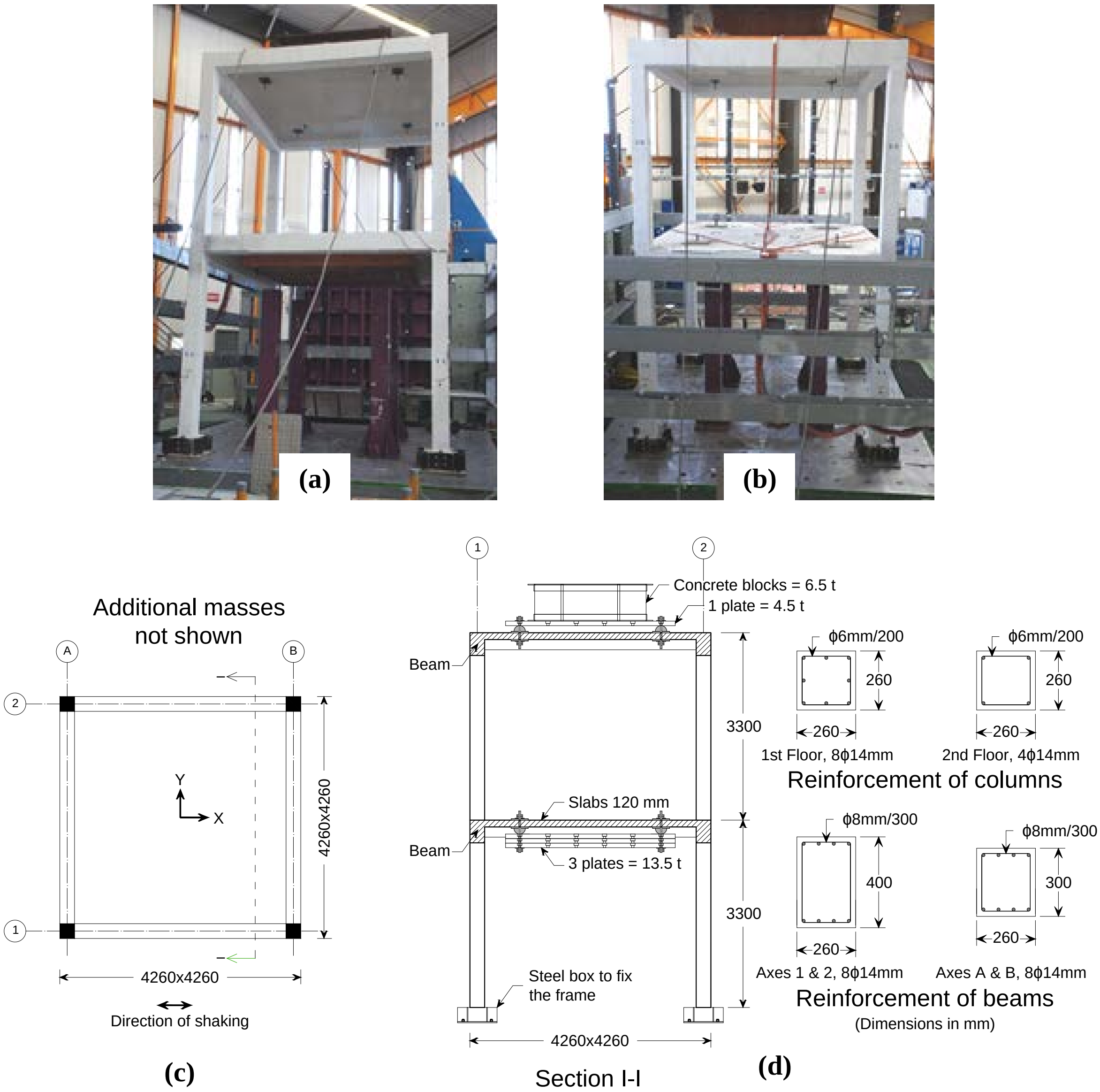

Figure 2. BANDIT building (a) general view axis A-A; (b) view axis 2-2; (c) plan view, and (d) geometry of building and members

To replicate old construction practices, no transverse reinforcement was positioned in the beam-column joints (see Figure 3). In addition, the beam bottom reinforcement was only anchored into the joint for a length of 230 mm (approximately $17d_b$) and no hooks or bends were provided. This short anchorage length was estimated to be insufficient to develop the full capacity of the 14 mm bars. The longitudinal bars of the columns were lapped (lap length l_b=25d_b=350 mm) just above the joint core. As a consequence, columns and beam-column joints at the 2nd floor were expected to experience significant damage during the initial shaking tests.

Columns and beam-column joints were identified using an alphanumeric ID code that contains the location of the structural member in plan and elevation (see Figure 2c and d). The first number and letter of the ID stand for the axes' intersection at which the structural member is located, whilst the number after the hyphen denotes the floor number. For instance, joint 1A-1 corresponds to a joint located at the intersection of axes 1 and A on the 1st floor.

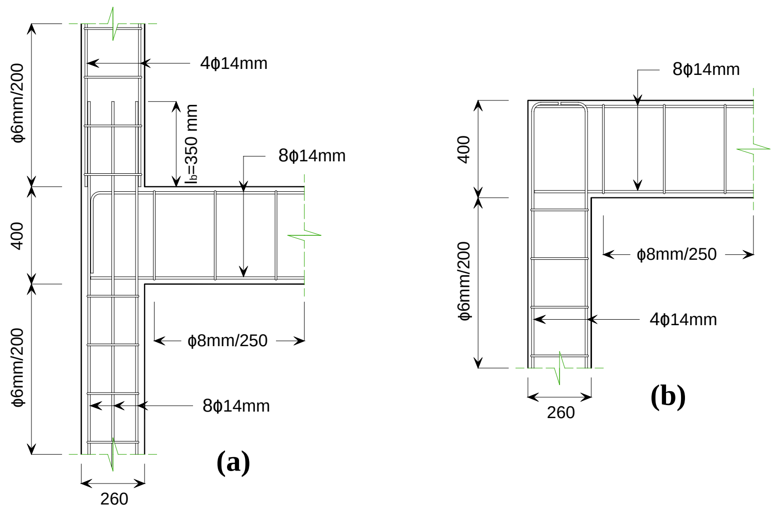

Figure 3. Detailing of columns and beam-column joints at (a) 1st floor, and (b) 2nd floor, X direction

2.3. Material properties

The mechanical properties of the Φ14 mm longitudinal reinforcement were obtained from tensile tests on three sample bars. The average yield and ultimate strength were f_y=526 MPa and f_u=616 MPa, respectively. Two batches of concrete were used to cast the building, one batch for each floor. The average compressive strength and secant elastic modulus at 0.4f_c for each batch were obtained from compressive tests on nine standard cylinders (150×300 mm) and were: f_c=30.8 MPa and E_c=23.9 GPa for the 1st floor; and f_c=25.5 MPa and E_c=21.7 GPa for the 2nd floor. High-tensile, high-ductility steel straps with a cross section of 0.8×25 mm were utilised for the strengthening (PTMS). The average mechanical properties of the straps were obtained from coupons tested at The University of Sheffield, which gave a yield and ultimate strength of f_y=1000 MPa and f_u =1100 MPa, respectively, and an elastic modulus E_s=230 GPa. The ultimate strength compares reasonably well with the minimum tensile strength f_u=950 MPa provided by the manufacturer (Megadyn® France).

2.3. Test set-up and ground motion record

The structure was instrumented with displacement and acceleration transducers at each floor to monitor the displacement and acceleration histories during the experiments. Unidirectional horizontal input shaking table tests were carried out on the building using increasing levels of Peak Ground Accelerations (PGAs). A single artificial ground motion record was used based on the Eurocode 8 (EC8) soil profile type C spectrum (CEN, 2004) as shown in Figure 4. The total duration of the record was 30.0 s. The natural frequencies of the structure were obtained using white noise as input signal before and after each shaking test. The accelerations recorded at each floor were subsequently post-processed to identify the natural frequencies of the first two modes of vibration.

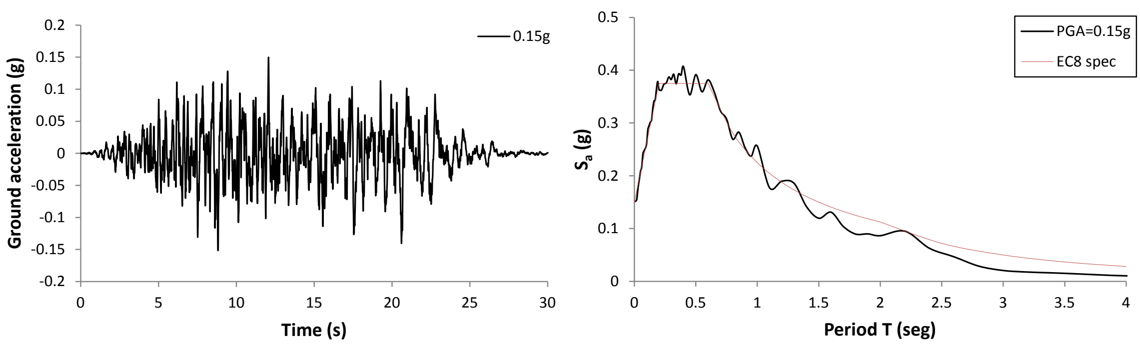

Figure 4. (a) Artificial input record for Stages 1 and 2, and (b) match to the EC8 spectrum for damping=5%.

3. TEST RESULTS STAGE 1: BARE BUILDING

In the first stage of the BANDIT experiments, the bare building was subjected to unidirectional shake table excitations. Increasing levels of PGAs were applied to the building ranging from 0.025 to 0.15g. Table 3.1 presents the 1st and 2nd mode natural frequencies (f_1 and f_2, respectively) along with their accumulative reduction (Δf) after each test. The table also presents the equivalent 1st and 2nd mode natural periods (T_1 and T_2, respectively). As shown in Table 3.1, two tests were performed at a PGA level of 0.15g. During the first of these tests, some issues were encountered with the test equipment and therefore the test was halted after 20.0 s. Consequently, some of the higher amplitude cycles of the ground motion were not applied. The test was subsequently repeated to apply the full duration of the record (0.15g bis in Table 3.1).

Table 3.1. BANDIT results from tests in Stage 1: bare building

Condition	1st mode f_1 (Hz)	Δf_1	2nd mode f_2 (Hz)	Δf_2	1st mode T_1 (s)	2nd mode T_2 (s)
Undamaged	2.09	NA	5.57	NA	0.48	0.18
After PGA=0.025g	1.88	-10%	4.98	-11%	0.53	0.20
After PGA=0.05g	1.68	-20%	4.64	-17%	0.60	0.22
After PGA=0.10g	1.46	-30%	4.04	-27%	0.68	0.25
After PGA=0.15g	1.29	-38%	3.69	-34%	0.78	0.27
After PGA=0.15g bis	1.14	-45%	3.40	-39%	0.89	0.29

Table 3.1 clearly shows the reduction in 1st and 2nd mode frequencies of the structure as a consequence of damage accumulation. After the test PGA=0.15g bis, the 1st natural frequency of the building dropped by 45%. The residual stiffness after this test was approximately only 30% of the original undamaged stiffness of the building, which indicates severe damage in the structure.

As expected, most of the damage occurred at the beam-column joints of the 2nd floor, see Figure 5. Also, splitting cracks were clearly visible at the bottom of the 2nd floor columns where the longitudinal reinforcement was lapped (refer to Figure 3a). A detailed visual inspection also revealed limited diagonal cracking at the 1st floor joints and horizontal cracking at the 2nd floor columns. No significant damage occurred at the 1st floor columns. The tests were halted after the test PGA=0.15g bis as the desired target level of damage was achieved.

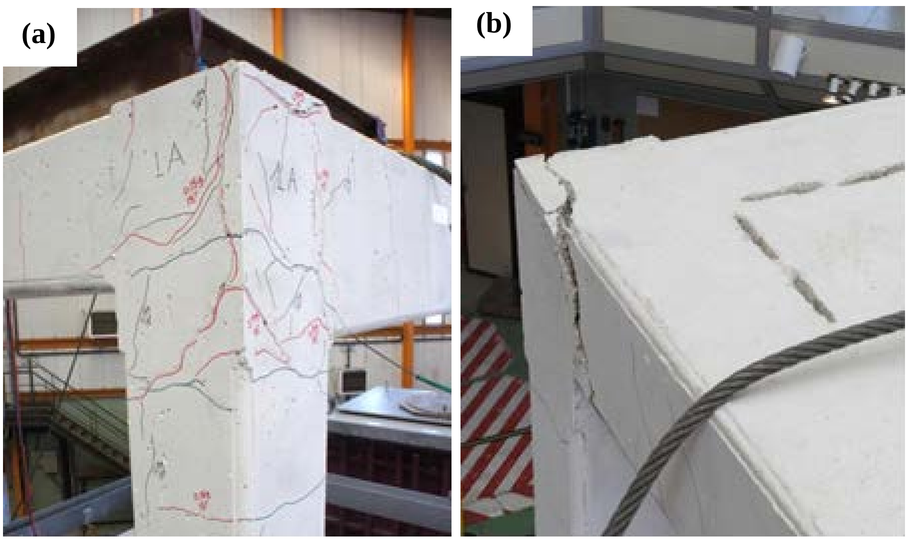

Figure 5. Damage at the 2nd floor joints (a) joint 1A-2 and (b) joint 2A-2.

4. PTMS STRENGTHENING

After the first series of tests, the damaged concrete was repaired using high-strength repair mortar and the cracks were injected with epoxy resin. Subsequently, beam-column joints and columns were strengthened locally using PTMS. The main objective of the local intervention with PTMS aimed at increasing the strength capacity of columns and beam-column joints, but without modifying significantly the original stiffness characteristics of the building.

The PTMS strengthening was installed using the following procedure. First, four 10 mm thick steel plates were fixed to the columns and two plates to the beams at the locations shown in Figure 6. The plates were held in place using high-strength bolts HIT-V M12×120 (steel 8.8) inserted in 14 mm holes prefilled with HIT-RE 500-SD resin. To start the strapping, 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 the 2 layer metal straps. The columns ends of both floors were initially confined using nine horizontal straps (with 1 layer) placed at 50 mm centres. To prevent shear failure of the beams, six straps with 1 layer were also wrapped around the beams ends. Subsequently, eight straps with 2 layers were provided parallel to the longitudinal beams axes (i.e. horizontally) to provide confinement to the beam-column joint. These straps were also required to compensate the possible premature pullout failure of the beam bottom reinforcement. Next, six straps with 2 layers were provided along each outer face of the columns (parallel to the columns axes) to increase their flexural capacity. After the 2-layer straps were installed on beams and columns, the nuts of the bolts securing the steel plates were fully tightened. Additional confining straps (1 layer only) were placed horizontally around the beams and columns to prevent excessive buckling of the 2-layer horizontal and longitudinal straps during the tests. This process was repeated until all the joints were strengthened. For the case of the 2nd floor joints, the 2-layer longitudinal straps parallel to the column axes were bent at 90 degrees at the slab edges and secured to two steel plates located on the top of the slab.

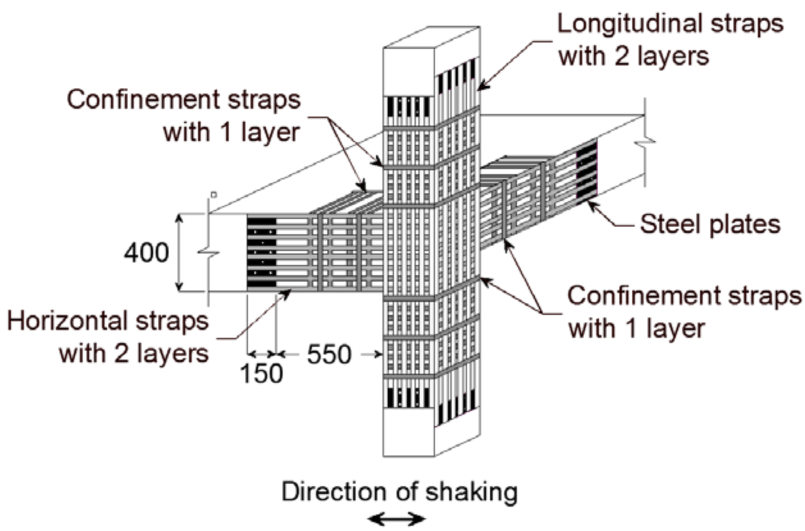

Figure 6. PTMS strengthening intervention at 1st floor beam-column joints

Figure 7 shows close-up views of the 1st and 2nd floor joints after the PTMS strengthening. As can be seen, the steel straps provided a regular orthogonal confinement grid around the beam-column joints and columns ends. The total strapping time for each joint varied from 2 to 3 hrs, which proves the ease and speed of application of the proposed method. In addition, the added weight of the straps and steel plates did not increase significantly the total weight of the structure.

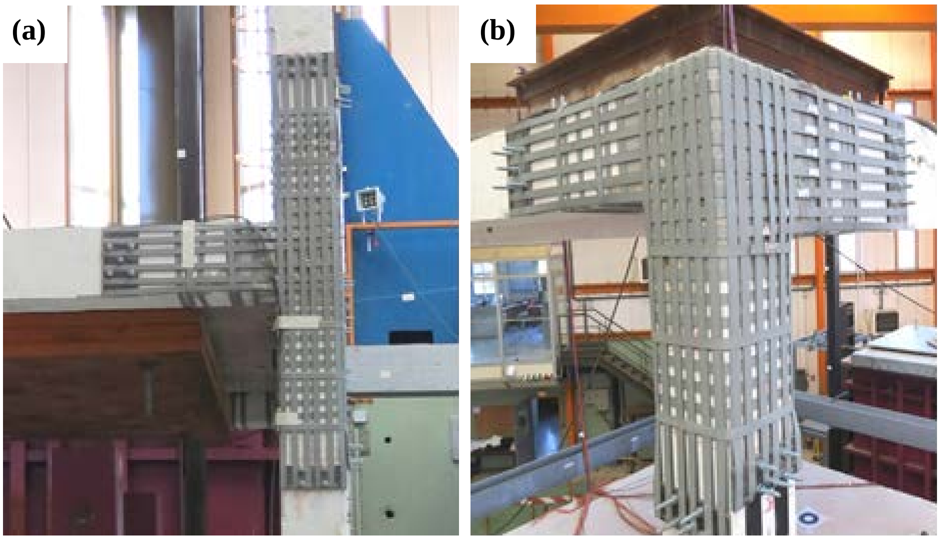

Figure 7. PTMS strengthening of (a) joint 1A-1, and (b) joint 1A-2

5. TEST RESULTS STAGE 2: PTMS-STRENGTHENED BUILDING

In Stage 2, the tests were repeated using similar levels of PGA to those imposed in Stage 1. Seismic shaking started at PGA=0.05g and was increased progressively in multiples of 0.05g up to 0.35g as listed in Table 5.1. The results show that the initial frequency at the beginning of Stage 2 (1.65 Hz) was 45% higher than the natural frequency after test PGA=0.15g bis in Stage 1 (f_1=1.14 Hz, see Table 3.1). This implies that the structural stiffness of the building was substantially recovered as a result of the crack resin injection, concrete repair and subsequent PTMS strengthening. Moreover, a frequency of 1.65 Hz is equivalent to the frequency that characterised the post-cracked behaviour of the specimen after the test at PGA=0.05g in Stage 1.

Table 5.1. BANDIT results from tests in Stage 2: PTMS-strengthened building

Condition	1st mode f_1 (Hz)	Δf_1	2nd mode f_2 (Hz)	Δf_2	1st mode T_1 (s)	2nd mode T_2 (s)
Initial	1.65	-	5.01	-	0.61	0.20
After PGA=0.05g	1.56	-5%	4.66	-7%	0.64	0.21
After PGA=0.10g	1.50	-9%	4.59	-8%	0.67	0.22
After PGA=0.15g	1.47	-11%	4.33	-14%	0.68	0.23
After PGA=0.25g	1.28	-22%	3.90	-22%	0.78	0.26
After PGA=0.35g	0.99	-40%	3.70	-26%	1.01	0.27

The results in Table 5.1 show that after the test at PGA=0.35g, the 1st mode frequency of the building dropped by 40% as a consequence of damage accumulation. This frequency degradation is comparable to that obtained at the much lower PGA level of 0.15g in Stage 1. Hence, the local strengthening of joints and columns effectively controlled the stiffness degradation of the structure, which became critical only at higher PGA levels.

Although only limited local damage in the columns was visible during the tests (see Figure 8a), the drop in frequency suggested that more significant damage had taken place. After the tests were halted, a thorough inspection of the building was carried out and no apparent damage of the PTMS strengthening or steel plates was visible, with only some of the longitudinal straps (parallel to the columns axes) showing evidence of minor buckling at the 1st floor beam-column joints. The removal of the straps revealed significant damage in the beam-column joints, particularly at joints 1A-1, 1A-2, 2A-1 and 2A-2 (see Figure 8b). The test results show that whilst the level of damage produced in Stage 1 was critical for the building, damage was effectively controlled during Stage 2 and the global stability of the building was never compromised.

Figure 8. (a) View of joint 1A-2 during tests in Stage 2; and (b) joint 1A-1 after removing the PTMS

The experimental results also show that during Stage 1, the inadequate reinforcement detailing led to premature failure of the 2nd floor before the capacity of the 1st floor joints was fully mobilised. This was confirmed by the limited diagonal cracking observed at the 1st floor joints after the removal of the PTMS. Conversely, in Stage 2 the PTMS intervention increased significantly the shear strength of the 2nd floor joints and controlled better their structural deterioration. Accordingly, seismic forces redistributed among the building's members and the 1st floor joints were subjected to higher seismic force demands, which in turn produced severe damage (see Figure 8b). This suggests that the PTMS strengthening allowed a better exploitation of the available member strengths over the building height.

6. COMPARISON OF MAXIMUM FLOOR DISPLACEMENT RESULTS

Maximum displacements provide an insight into the global behaviour of the test structure. Table 6.1 reports the maximum absolute floor displacements from the tests performed in Stages 1 and 2. As a consequence of damage accumulation, floor displacements of the PTMS-strengthened building are consistently larger than those obtained for the bare building. At PGA levels of 0.05g and 0.10g, 1st and 2nd floor displacements of the PTMS-strengthened building are approximately 26 and 17% larger than those obtained during the first series of tests.

Table 6.1. Maximum floor displacements from BANDIT tests

	Floor no.	Bare building (Stage 1), mm	PTMS-strengthened (Stage 2), mm
PGA=0.05g	2	17.5	20.5
	1	10.5	13.3
PGA=0.10g	2	44.7	52.0
	1	24.8	29.4
PGA=0.15g	2	81.8[a]	78.9
	1	31.3[a]	41.6
PGA=0.25g	2	-	125.9
	1	-	60.7
PGA=0.35g	2	-	162.3
	1	-	75.3

[a] Results from test 0.15g bis

Table 6.1 also shows that at a PGA level of 0.15g, the maximum 2nd floor displacements in both test Stages were very similar. However, the 1st floor displacement in Stage 2 increased by 33%. Based on these results and on the observed damage, it is clear that the applied strengthening system was very successful at controlling the development of damage in the 2nd floor joints and columns. Compared to the maximum floor displacements achieved in Stage 1, the maximum 1st and 2nd floor displacements at PGA=0.35g in Stage 2 increased by 100 and 140%, respectively. It can be concluded that the adopted PTMS strengthening strategy was very effective at improving the global deformation capacity of the building.

6. CONCLUSIONS

This paper presented preliminary results from the first two stages of the EU-funded BANDIT project. The efficiency of Post-Tensioned Metal Strap (PTMS) strengthening was investigated experimentally through shake table tests on a seismically deficient full-scale RC building. In Stage 1, the building was tested in bare condition. After the initial tests, the damaged building was repaired, subsequently strengthened using PTMS and re-tested in Stage 2. From the tests results, the following conclusions are drawn:

1. The initial shake table tests on the deficient bare building (Stage 1) produced significant damage at the 2nd floor joints and lapped columns. Conversely, the joints and columns at the 1st floor experienced limited damage. This confirms that the design of the bare structure was inadequate and would lead to extensive damage at the 2nd floor before the capacity of the joints at the 1st floor could be fully mobilised. Tests in Stage 1 were performed at increasing levels of PGA until the desired target level of reparable damage was reached.

2. The results of the shaking table tests demonstrated that the adopted local strengthening strategy using PTMS was effective at increasing the seismic capacity of the building (Stage 2). Whilst the bare building resisted a maximum PGA=0.15g before a critical level of damage was reached, the PTMS-strengthened building sustained a maximum PGA of 0.35g. The epoxy-injection of cracks and the adopted strengthening strategy were also effective at restoring the initial post-cracked dynamic characteristics of the RC building.

3. The PTMS strengthening increased significantly the deformation capacity of the building by reducing the expected local structural damage in the 2nd floor beam-column joints and columns. For the same PGA intensity of 0.15g, the maximum 2nd floor displacements in Stages 1 and 2 were similar, but the 1st floor displacements increased by 33% in Stage 2. The PTMS intervention also increased the shear strength of the 2nd floor joints and controlled better their structural deterioration. Accordingly, seismic forces redistributed among the building's members and the 1st floor joints were subjected to higher seismic force demands which produced severe damage. This suggests that the PTMS allowed a better exploitation of the available member strengths over the building height.

4. The preliminary results presented here show that the PTMS strengthening improved significantly the seismic behaviour of the deficient damaged building. Consequently, the proposed method is feasible and very attractive for quick post-earthquake strengthening.

AKCNOWLEDGEMENTS

The research leading to these results has received funding from the European Community's Seventh Framework Programme [FP7/2007-2013] for access to CEA (Commissariat à l'Energie Atomique) under grant agreement n° 227887. The first author wishes to acknowledge the financial support provided by CONACYT and DGRI-SEP through its Complementary Scholarship programme. The second and fifth authors acknowledge the financial support provided by Damascus University and Al-Baath University, respectively. The authors also would like to gratefully acknowledge Thierry Chaudat of EMSI laboratory for his contributions during the early stages of BANDIT project. The PTMS strengthening was performed by the Industrial Partner Nuvia Travaux Spéciaux (Agence Ile de France).

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