FULL-SCALE SHAKING TABLE TESTS ON DEFICIENT RC BUILDINGS STRENGTHENED WITH FRP COMPOSITES

M.A. Ciupala[(1)], K. Pilakoutas[(2)], P. Papastergiou[(3)], R. Garcia[(4)] , I. Hajirasouliha[(5)], M. Guadagnini[(6)]

[(1)] *Senior Lecturer, University of East London, School of Architecture, Computing and Engineering, UK,* m.a.ciupala@uel.ac.uk
[(2)] *Professor, University of Sheffield, Department of Civil and Structural Engineering, UK,* k.pilakoutas@sheffield.ac.uk
[(3)] *Research Associate, University of Sheffield, Department of Civil and Structural Engineering, UK, p.papastergiou@sheffield.ac.uk*
[(4)] *Assistant Professor, University of Warwick, School of Engineering, Reyes.Garcia@warwick.ac.uk,*
[(5)] *Senior Lecturer, University of Sheffield, Department of Civil and Structural Engineering, UK, i.hajirasouliha@sheffield.ac.uk*
[(6)] *Senior Lecturer, University of Sheffield, Department of Civil and Structural Engineering, UK, m.guadagnini@sheffield.ac.uk*

Abstract

The paper discusses the results from experimental programmes including full-scale shaking table tests on two deficient one-span two-storey RC frame buildings strengthened with Carbon Fibre Reinforced Polymer (CFRP) composites. The tests were performed at one of the world class shaking table testing facility, the AZALEE shake table at the Commissariat à l'Énergie Atomique (CEA), Laboratory in Saclay, France, as part of two European (EU) funded research projects (EU ECOLEADER project and EU FP7 SERIES/ BANDIT project). The aim of these experimental programmes was to investigate the effectiveness of externally bonded CFRP reinforcement in improving the seismic behaviour of deficiently designed RC buildings. To simulate typical substandard design, the reinforcement of beam-column joints and columns of the two full-scale frames had inadequate detailing. In both projects, initial shaking table tests were carried out to assess the seismic behaviour of the bare frame buildings. Columns and joints were subsequently repaired and strengthened using CFRP composite material for the ECOLEADER frame, and a strengthening solution consisting of CFRP composite material and Post-Tensioned Metal Straps for the BANDIT frame. The frames were then subjected to incremental seismic excitations to assess the effectiveness of the strengthening solutions in improving the global and local frame performance. Whilst the original bare frames were significantly damaged at a peak ground acceleration (PGA) of 0.15-0.20g, the strengthened frames resisted severe shaking table tests up to PGA=0.50-0.60g without failure. Moreover, the strengthening interventions enhanced the interstorey drift ratio capacity and proved to be very effective in addressing the seismic deficiencies of substandard buildings.

Keywords: seismic strengthening; shaking table tests; FRP composites; deficient RC buildings

1. Introduction

In many European countries as well as in developing countries, much of the existing Reinforced Concrete (RC) building stock has been designed according to old design codes, with no or little seismic provisions, in conjunction with poor material or construction practices. Consequently, many existing buildings suffer from deficient lateral load resistance or insufficient energy dissipation capacity, resulting in loss of strength during earthquakes and subsequent collapse. Major earthquakes (Turkey, 1999; China, 2008; Italy, 2009; Chile and New Zealand, 2010; Nepal, 2015) highlighted the seismic vulnerability of the deficiently designed and detailed RC buildings, resulting in significant human and financial losses.

In order to reduce the vulnerability of these buildings to earthquakes and increase their load carrying capacity, externally bonded Fibre Reinforced Polymer (FRP) composite materials have been considered as a feasible and cost-effective solution for the seismic strengthening of deficient buildings [1]. This is due to their well-known high strength to weight ratio, high resistance to corrosion, excellent durability, ease and speed of in-situ application, and flexibility to strengthen selectively the members that are seismically deficient [2].

The seismic behaviour of deficient full-scale RC buildings strengthened with FRP composite materials has been assessed through several experimental tests. Most of them were pseudo-dynamic tests [3-6] or quasi-static lateral load tests [7,8]. The results of these studies confirmed the effectiveness of the FRP materials in improving the seismic behaviour of the strengthened frame buildings. However, none of the above mentioned studies investigated the seismic behaviour of full-scale, FRP strengthened RC frames using shaking table tests.

This paper discusses results from experimental programmes including shaking table tests on deficient one-span two-storey RC frame buildings strengthened with Carbon Fibre Reinforced Polymer (CFRP) composites. All tests were performed on the AZALEE shake table at the Commissariat à l'Énergie Atomique (CEA), Laboratory in Saclay, France, as part of two European (EU) funded research projects (EU ECOLEADER project and EU FP7 SERIES/ BANDIT project). The aim of these experimental programmes was to study the effectiveness of externally bonded CFRP reinforcement in improving the seismic behaviour of deficient RC frames.

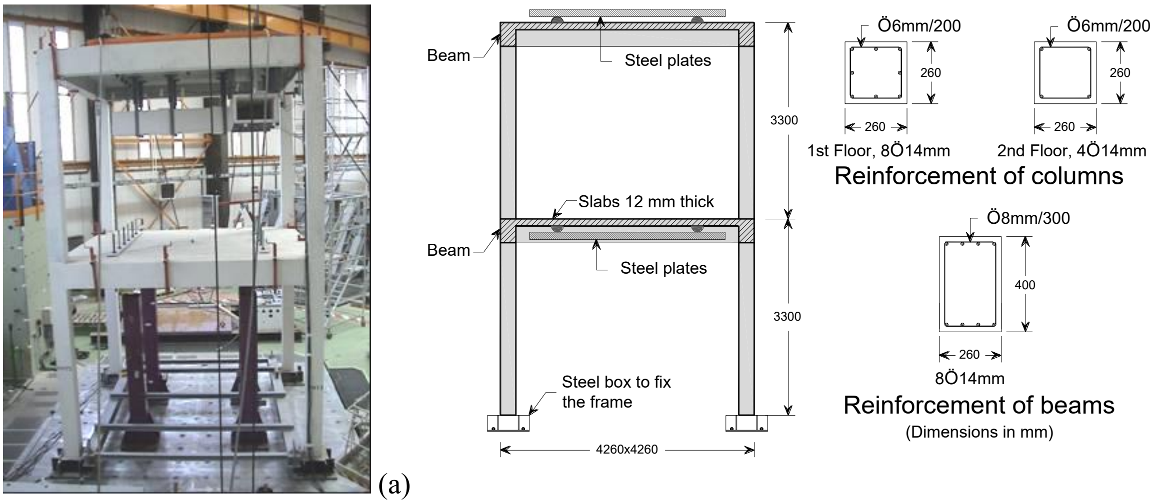
2. ECOLEADER project

2.1 Project overview

The main aim of this project was to investigate experimentally the performance of existing deficient RC frame buildings using CFRP composite materials. A RC frame was designed and built according to typical old pre-seismic construction practice of southern Europe. This is representative of substandard buildings typically found in countries with old design codes, with no or little seismic provisions, including in developing countries. A series of initial shaking table tests were performed until significant damage was achieved. Subsequently, the damaged frame was repaired, and columns and beam-column joints were strengthened using externally bonded CFRPs, followed by additional shaking table tests.

2.2 Geometry of the RC frame, material properties and set-up of tests

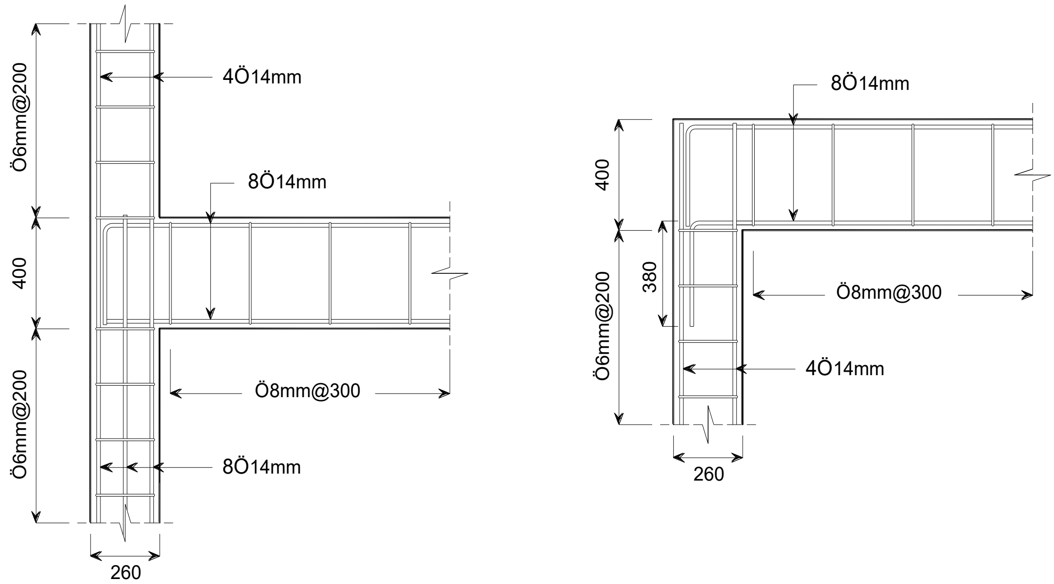
The tested frame building was a RC one-span two-storey frame, regular in plan and elevation, designed using old European earthquake-resistant provisions from the 60's [9, 10]. As a result, columns and beam-column joints were expected to experience significant damage during the initial shaking tests. The frame had the dimensions in plan of 4.26×4.26 m and had a constant storey height of 3.30 m. The general view of the frame, along with details of the general geometry, element sections and corresponding reinforcement are shown in Fig.1.



(a) (b)

Fig. 1 - ECOLEADER frame building: (a) general view, (b) geometry of the frame and structural elements

The detailing of the reinforcing steel in beam-column joints is shown in Fig.2. The concrete compressive strength and modulus of elasticity were fc=20 MPa and Ec=25545 MPa, and the yield and ultimate strength of steel reinforcement were fy=551 MPa and fu=656 MPa, respectively. The manufacturer's mechanical and physical properties of the utilised unidirectional CFRP composite material were: the tensile strength f_{FRP}=1350 MPa, modulus of elasticity E_{FRP}=105 GPa, and layer thickness of 0.48 mm. An additional mass of 9.0 ton was attached to each slab to simulate real loading conditions (Fig.1).



(a) (b)

Fig. 2 - Detailing of reinforcing steel in beam-column joints at: (a) 1st storey and (b) 2nd storey

Linear variable displacement transducers (LVDTs) and acceleration transducers were installed at each storey to monitor the response history during the shaking tests. The LVDTs were attached to an external rigid frame to facilitate the measurements and quantify the residual displacements after each test.

Unidirectional horizontal input shakings were applied to the frame, using increasing peak ground accelerations (PGA) levels ranging from 0.05g to 0.4g. A single ground motion record was used based on the Eurocode 8 (EC8), soil profile type C spectrum [11]. The natural frequencies of the frame for the first two modes of vibration were obtained using white noise as input signal before the start and after each test.

2.3 Tests results and discussion

The initial series of tests with PGAs varying between 0.05g and 0.40g produced significant damage on the frame, especially at the ends of columns (weak column-strong beam damage mechanism). After these tests, the damaged frame was strengthened locally with externally bonded CFRP composites using a wet lay-up technique. The main scope of the strengthening was to produce a strong column-weak beam mechanism, which is in line with current seismic design provisions. Before strengthening, the damaged concrete was repaired using repair mortar and the main cracks were injected with epoxy resin. One vertical CFRP sheet (parallel to the columns' axes) was attached at the interior and exterior faces of columns ends to enhance their flexural strength (Fig.3a). Beam-column joints at both storeys were also strengthened using one orthogonal sheet to avoid a premature shear failure, as shown in Fig.3b. In order to prevent premature debonding of the sheets applied to the strengthened joints, two thin strips of CFRPs were wrapped around the beams ends (Fig.3b). Additionally, CFRP confinement was used to further increase the column capacity and to avoid possible buckling and premature debonding of the longitudinal sheets along the columns' axes (Fig.4). The existing transverse reinforcement was sufficient to prevent shear failure in beams and columns, therefore no additional CFRP was required in this case.

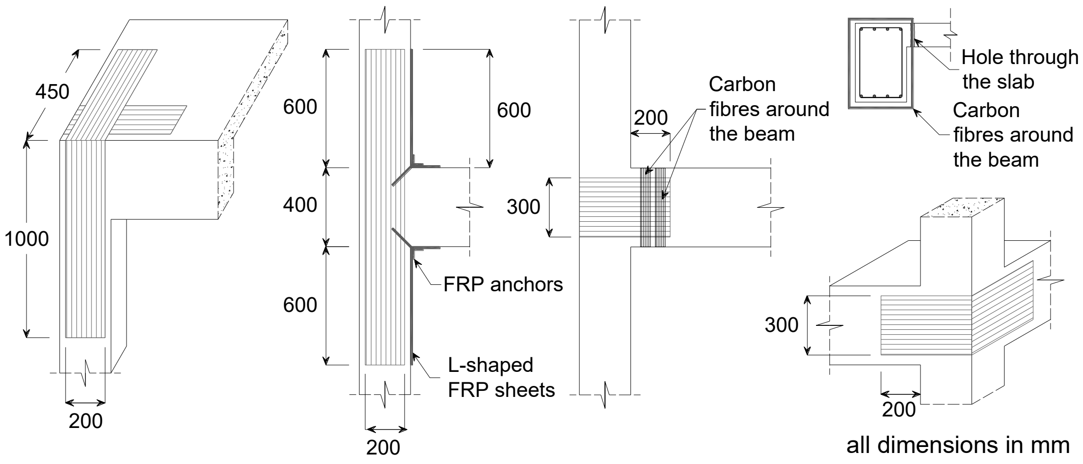
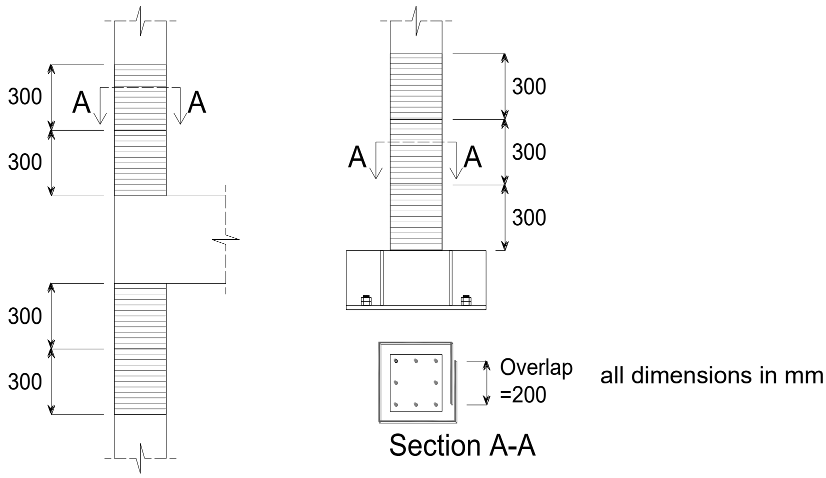


Fig. 3 - Strengthening of the frame using CFRP material at (a) exterior faces of columns, and (b) exterior zone of beam-column joints

Following the CFRP strengthening of the frame, a second series of shaking table tests were conducted with PGA levels ranging from 0.05g to 0.50g. No evident damage appeared during the test at PGA= 0.05g. The first damage to the CFRP sheets at the 2nd storey columns was observed after the test at PGA level of 0.20g. No further damage was noted at columns or CFRP sheets until after the final tests, although some fracture of the sheets occurred at beam-column joints. The adopted strengthening strategy was effective in preventing debonding of the CFRP sheets, as this type of failure was not observed during the tests. However, after the test at a PGA of 0.50g, significant concrete cracking occurred on the beams (Fig.5a), concrete cover spoiling under the CFRP confinement at the end of columns (Fig.5b) and at joints (Fig.5c). Fig.6 shows the deformed shape at maximum deformation for the bare and CFRP strengthened frame. It was noted that the use of CFRP strengthening resulted in a significant reduction in the inter-storey drift of 67% at the second floor (from 3.9% to 1.3%) for the PGA of 0.4g and delayed the deterioration of the frame due to bond-slip of the reinforcing steel at beams and columns.



(a) (b)

Fig. 4 - Confinement of the columns using CFRP composite material at (a) columns ends, and (b) bottom of 1st storey columns

(a) (b) (c)

Fig. 5 - Damage after the test at PGA=0.5g on the CFRP strengthened frame:
(a) beam end at 1st storey, (b) bottom of column 1st storey, (c) 2nd storey joint

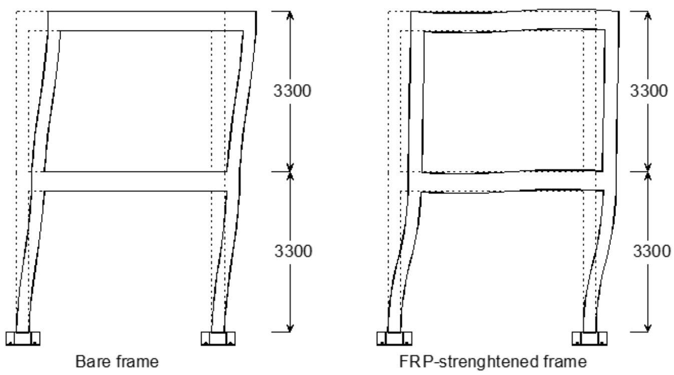


Fig. 6 - Deformed shape of the bare and CFRP strengthened frame at PGA=0.40g

Table 1 shows the tests performed for the bare and the CFRP strengthened frame, as well as the natural periods for the first two modes of vibration. The aplication of the CFRP confinemnent reduced the fundamental period of vibration T_1 by 50%, from 1.47s after the test at PGA=0.40g for the bare frame to 0.73s under the white noise for the CFRP shrengthenered frame. After the test at PGA=0.50g, the 1st and 2nd natural periods of the CFRP strengthened frame increased by 56% and 43%, respectively, compared to those prior to the application of the CFRP stregthening. These results showed the effectiveness of the CFRP confinement in increasing the stiffness of the stengthened framed compared to the bare one and the increase in its ability to experience a higher level of seismic excitation.

Table 1- Test sequence of ECOLEADER frame and dynamic characteristic after each test [10]

Bare frame			CFRP Strengthened frame		
Test	**T1 (s)**	**T2 (s)**	**Test**	**T1 (s)**	**T2 (s)**
White noise	0.53	0.18	White noise	0.73	0.23
0.10g	0.74	0.23	-	-	-
0.20g	0.94	0.28	0.20g	0.94	0.28
0.30g	1.14	0.38	-	-	-
0.40g	1.47	0.39	0.40g	1.02	0.30
			0.50g	1.14	0.33

3. BANDIT project

3.1 Project overview

This project studied the effectiveness of a ‘dual’ innovative strengthening solution using CFRP composite material and Post Tensioned Metal Straps (PTMS) in improving the performance of a full-scale RC frame building with deficient beam-column joints which was subject to previous damage. The frame was tested as part of an intensive experimental programme, including five phases of shaking table tests. The proposed dual strengthening solution permitted a direct comparison of the effectiveness of the two techniques using full-scale shaking table tests at high PGA levels. In this paper, only test phase 4 of the project is described, focusing on assessing the performance of the dual strengthening solution combining CFRP and PTMS, whilst the other test phases are reported in [12, 13].

3.2 Geometry of the RC frame, material properties and set-up of tests

The geometry of the full-scale two-storey one-span frame building was similar to that of the frame tested as part of the ECOLEADER project, as discussed in Section 2 and [14]. The regular dimensions in plan of the frame building were 4.26×4.26 m and its constant floor height was 3.30 m (Fig.7a-b). The cross section of columns was 260×260 mm and longitudinal column bars were of diameter Ø14 mm (Fig.7c). The column bars were lapped over a length l_b=25Ø=350 mm just above the 1st floor joints (Fig.7d) to simulate old construction practices.

The beams had a cross section of 260×400 mm in the X direction and 260×300 mm in the Y direction (Fig.7c). The smaller beam depth was selected to increase the flexibility of the building in the Y direction, thus undergoing large interstorey drifts, therefore damage, in this direction during the tests at higher PGA levels. The longitudinal beam reinforcement had a diameter of Ø14 mm. The bottom beam reinforcement was anchored into the joints with no hooks/bends for a length of 220 mm (approximately 16Ø) to study the effect of deficient bar anchorage on the structural response (Fig.7d). This short anchorage length would be considered insufficient to develop the full capacity of the Ø14 mm bars according to Eurocode 2 [15]. The top beam reinforcement of the 1st floor joints was anchored using 90° bends and U-shaped hooks, in order to provide adequate anchorage according to Eurocode 2 requirements, whilst the top and bottom beam reinforcement was anchored into the 2nd floor joints for 220 mm in both X and Y directions (Fig.7e). The beam reinforcement at the 2nd floor joints were welded to the longitudinal column reinforcement to repair the building after the initial damage. The column-to-beam relative flexural strength ratio ($\Sigma MR_{col}/\Sigma MR_{beam}$) in the bare frame did not satisfy the strong column-weak beam strength condition recommended in current seismic design codes. To prevent shear failures outside the joints, the columns were reinforced with Ø6 mm stirrups spaced at 200 mm centres, and the beams were reinforced with Ø8 mm stirrups spaced at 250 mm centres. The stirrups were closed with 90° hooks instead of 135° hooks typically required by current seismic codes. As no confining stirrups were provided in the beam-column joint cores, significant damage was expected to occur at these locations. The top and bottom of the 120 mm thick slabs were reinforced with 10 mm bars, spaced at 100 mm centres in both directions.

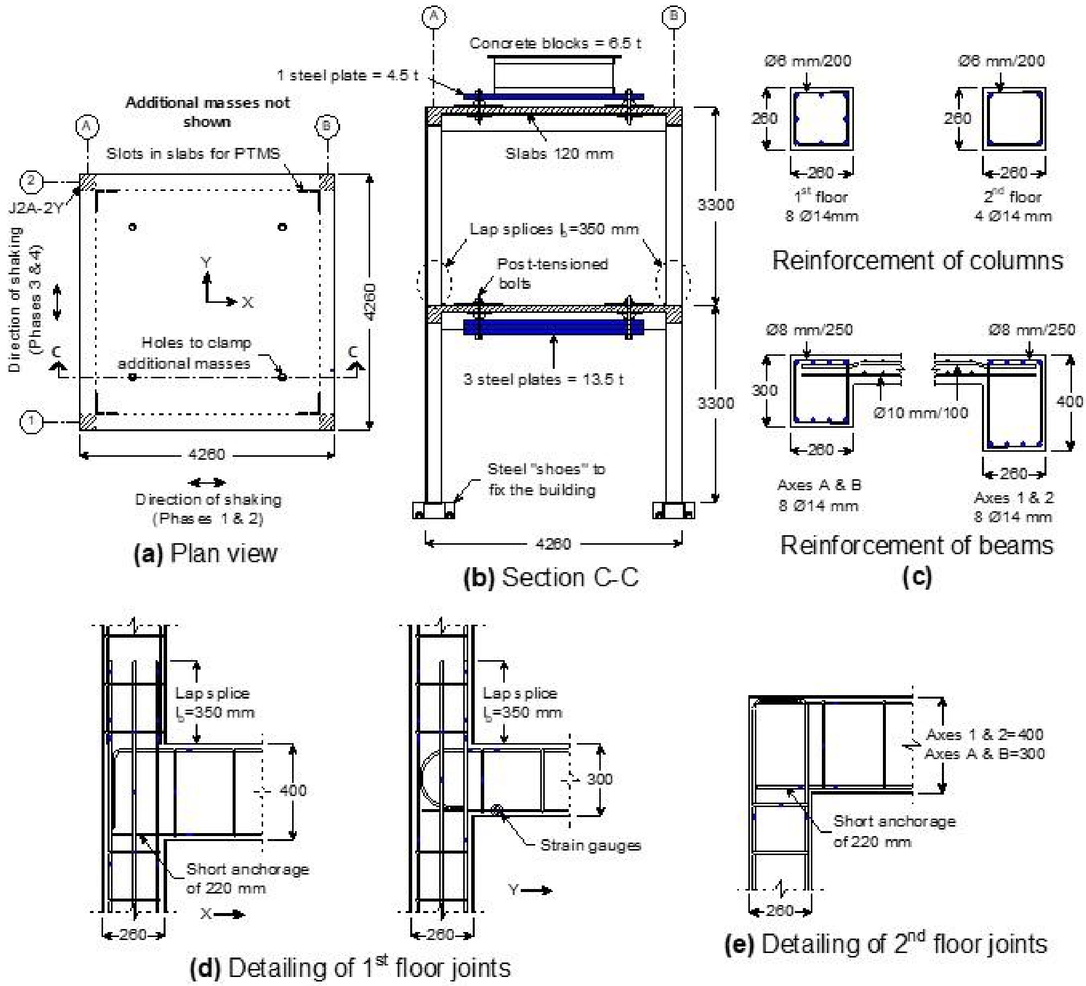


Fig. 7 - General geometry and reinforcement detailing of BANDIT frame building (units: mm), partially adapted from [12]

The building was cast using two batches of ready-mixed normal-strength concrete, one for each floor. Each floor was cured for seven days in the formwork and then kept under standard laboratory conditions.

The mean properties of the concrete compressive strength (f_{cm}) and elastic modulus of each batch were 30.7 MPa and 24.300 GPa, respectively for the 1st floor, and 24.6 MPa and 21.2 GPa, respectively for the 2nd floor. Although the resulting concrete strengths are higher than often encountered in existing substandard RC buildings (f_{cm}<20 MPa), the strength of the concrete used in this study is representative of typical constructions built during the 1970-80s in the Mediterranean, Middle-East and Latin America regions. Due to difficulties in finding low-strength plain bars, the building was reinforced using ribbed bars grade S500 complying with NF A 35-016-1 [16]. Bidirectional (0°-90°) CFRP sheets were used for strengthening, with 70% of the fibres oriented in the main longitudinal direction of the sheets. The manufacturer's mechanical and physical properties of the utilised CFRP sheets were: the tensile strength f_{FRP}=1350 MPa, modulus of elasticity E_{FRP}=105 GPa, and layer thickness of 0.48 mm. Commercially available high-strength (f_u=1100 MPa) metal straps with nominal cross section 0.8×25 mm and zinc corrosion-resistant surface coating were used for the PTMS strengthening.

The dual strengthening intervention aimed at providing a strong column-weak beam damage mechanism by preventing the premature failure of the joints and by enhancing the flexural capacity of the columns, thus complying with current seismic design provisions. Fig.8 shows a general view of the strengthened building, where Fig.8a shows the PTMS installed so that it creates a confining 'mesh' around the beam-column joints, whilst Fig.8c shows the CFRP strengthened joint. The layout of CFRP sheets was similar to that used to retrofit the ECOLEADER frame building presented in section 2.3 (see also [12, 14]).

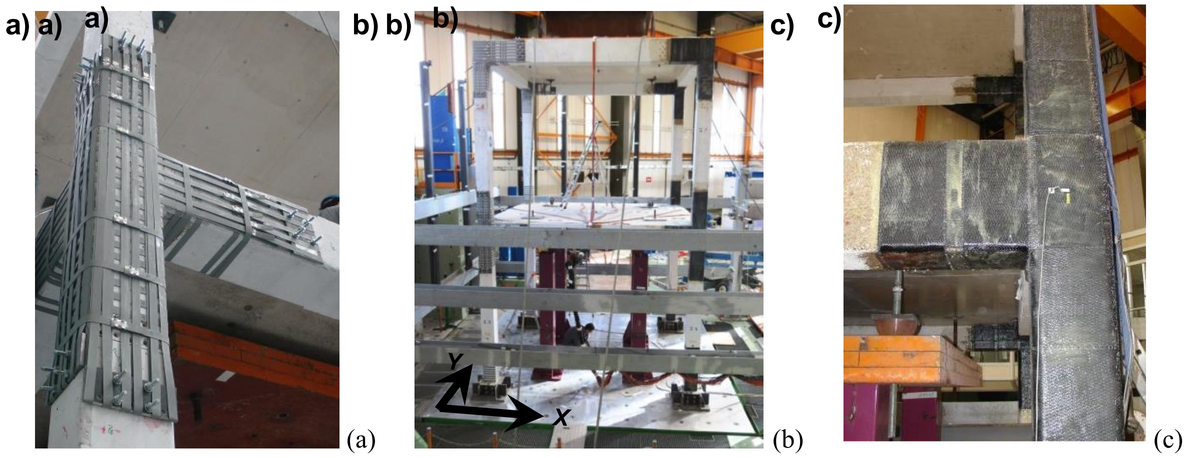


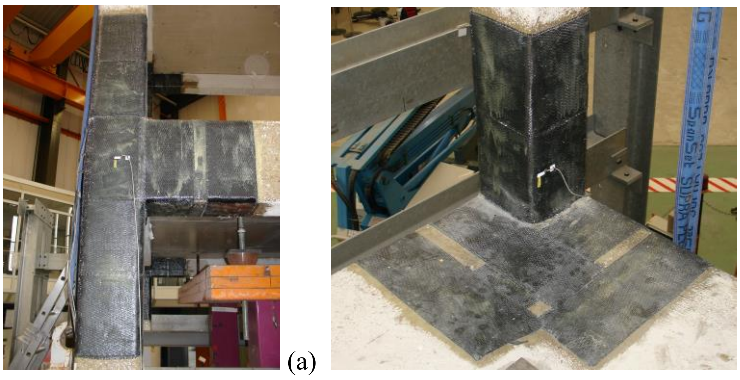
Fig. 8 - View of the frame building after the CFRP/PTMS strengthening: (a) PTMS strengthened joint, (b) general view of the strengthened frame on shake table, (c) CFRP strengthened joint

A comprehensive presentation of the tests carried out as part of the experimental programme is reported in [13, 17], which consisted in 30 shaking table tests conducted in five phases. The building was tested up to a PGA of 0.35g during the first three phases of testing, in order to study the seismic performance of the deficient RC frame building and assess the effectiveness of the CFRP and PTMS strengthening techniques. The performance of the two strengthening solutions using CFRP and PTMS was compared in testing phase 4, during which the frame was subjected to input records in the direction parallel to frames A and B in Fig.7a. A subsequent testing phase (phase 5) was carried out to study the behaviour of the CFRP and PTMS strengthened frame under severe three-dimensional shaking.

Only the results from phase 4 are presented and discussed in section 3.3 to highlight the potential of the studied strengthening solutions.

3.3 Tests results and discussion

The seismic excitation applied to the building after the repairs and dual CFRP/PTMS strengthening (phase 4) ranged from PGA 0.05g to 0.35g. In general, as the CFRP sheets and metal straps covered the concrete surfaces, damage at the joints could not be observed during the tests. However, popping and metallic sounds during tests (especially after the test at PGA=0.20g) indicated that the CFRP and metal straps were effectively carrying considerable tension forces. After the tests were halted, a visual inspection indicated that no apparent damage was caused on the CFRP sheets or on the straps. Overall, the CFRP sheets maintained their physical integrity (Fig.9) and the metal straps maintained most of the post-tensioning force, with the exception of some longitudinal straps placed along the columns of the 1st floor joints. Some cracking was also evident at the beam-column joints, but only on frame B (Fig.7a) where the PTMS strengthening was applied. The damage observed in the frame in phase 4 indicated that the repairs and CFRP/PTMS strengthening intervention prevented the premature shear failure of the deficient joints.



(a) (b)

Fig. 9 - View of the CFRP strengthening after the testing phase 4 at PGA=0.35g
(a) external view of 1st storey joint, (b) internal view of 1st storey joint

Table 2 shows the tests performed for the bare and the CFRP strengthened frame, as well as the first two natural periods of vibration of the frames.

Table 2 - Test sequence of BANDID frame and natural periods after each test [13]

Phase	**Test direction and condition**	**PGA (g)**	**X direction**		**Y direction**	
			T1 (s)	T2 (s)	T1 (s)	T2 (s)
1	X – Bare frame	undamaged	0.48(b)	0.18	0.52(b)	na
		0.10g	0.68	0.25	-	-
		0.15g(a)	0.88	0.29	-	-
4	Y Repaired; CFRP/PTMS strengthened frame	Initial	0.75	0.23	0.84	0.25
		0.10g	-	-	0.93	0.27
		0.20g	-	-	0.99	0.28
		0.30g	-	-	1.03	0.29
		0.35g	-	-	1.09	0.30

(a) After the test, cracks were resin-injected and damaged concrete replaced with high-strength epoxy mortar;
(b) T_{iX} = 0.48 (s), T_{iY} = 0.52 (s)

The analysis of the global lateral stiffness of the structure (K) during testing phase 4 was performed, based on the implementation of an equivalent SDOF model. This showed that the repairs and strengthening restored the stiffness of the building from K=1430 kN/m (end of phase 3) to 2510 kN/m (beginning of phase 4), resulting in an enhancement of 75% (Fig.10). At the end of phase 4 (PGA=0.35g), damage accumulation degraded the stiffness of the original bare frame building by 77% (from 6600 kN/m at the beginning of phase 1 to 1500 kN/m). Despite this severe stiffness reduction, the limited structural damage observed after phase 4 indicated that the building was capable of sustaining seismic shaking at higher seismic intensities. Further tests confirmed that the stability of the building was not compromised even during the shake table tests at PGA of 0.60g. In general, the adopted CFRP strengthening solution was very effective in preventing debonding of the CFRP sheets under sever shaking. As regards the PTMS technique, some push type seals started to shear off at such high level of PGA shaking. However, the metal straps maintained about 70% of their initial post-tensioning force, thus confirming the reliability of the PTMS technique at high seismic excitation levels.

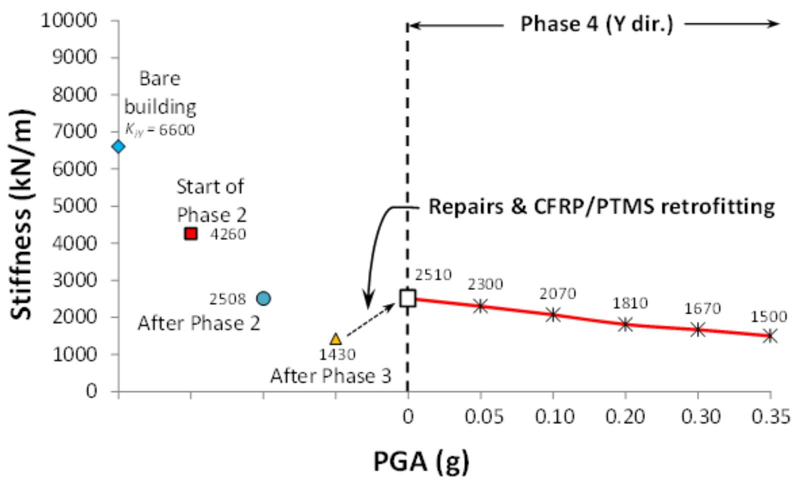


Fig. 10 - Lateral stiffness during testing phase 4 (Y direction)

Existing guidelines for seismic assessment and retrofit of existing buildings such as ASCE/SEI 41-06 [18] establish limits on acceptable values of maximum interstorey drift ratios, implying that exceeding these limits is a violation of a performance objective. According to ASCE/SEI 41-06, maximum transient drift ratios of δ=1.0%, 2.0% and 4.0% correspond to Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) performance levels, respectively.

The maximum interstorey drift ratios of the 1st and 2nd floor of the CFRP and PTMS-strengthened frames are shown in Fig.11. This figure presents the results from two equidistant displacement transducers fixed on each exterior face of the slabs, labelled as “CFRP” and “PTMS” according to the strengthening technique used on the frame. It is shown that the 1st and 2nd floors of the strengthened frame had similar δ_s at all PGA levels. This implies that both CFRP/PTMS strengthening solutions led to a more uniform damage distribution over the building height, which allowed a better exploitation of the available capacity of members. The building remained safe within the CP performance level during the last test at PGA of 0.35g (δ=2.92%). Overall, maximum drift values of the “PTMS” frame were only 2-6% larger than those of the “CFRP” frame, thus indicating that some negligible in-plane torsion occurred during the tests.

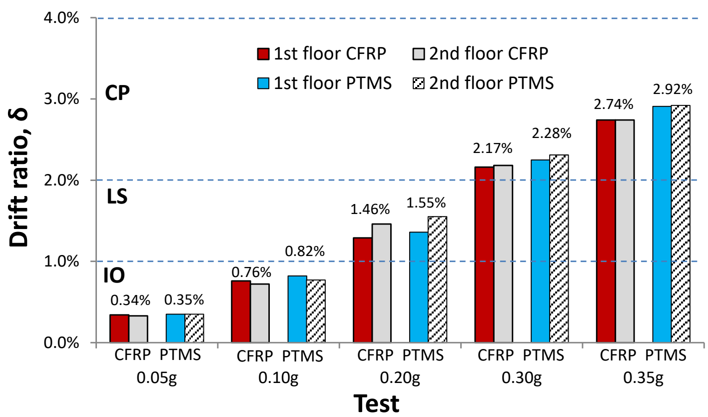


Fig. 11 - Interstorey drift ratio of 1st and 2nd floors during phase 4 (Y direction)

4. Conclusion

The two EU-funded research projects discussed in this paper investigated experimentally the effectiveness of externally bonded CFRPs at improving the behaviour of seismically deficient full-scale two-storey RC frames.

In both projects, the shaking table tests demonstrated that the adopted local strengthening strategy using CFRP composite material was effective in achieving a strong column-weak beam mechanism, which is in line with current seismic design provisions. The repair of cracks and the adopted strengthening strategy were also effective in restoring the lateral stiffness and the initial dynamic characteristics of the RC frames.

The proposed CFRP and dual CFRP/PTMS strengthening intervention also led to a more uniform damage distribution and, therefore, to a better exploitation of the available capacity in the structure.

In comparison to the original bare building, the strengthening intervention enhanced the interstorey drift ratio capacity of the 1st and 2nd floors. Both of the examined buildings remained safe within the Collapse Prevention performance level and without evidence of potential collapse.

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