Seismic Rehabilitation of a Full-Scale RC Structure using GFRP Laminates

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Synopsis: The SPEAR (Seismic PErformance Assessment and Rehabilitation) research Project is specifically targeted at existing under-designed structures and, in its framework, the core of the experimental activity is the series of full-scale pseudo-dynamic tests on a torsionally unbalanced three-storey RC structure, carried out at the ELSA Laboratory of the Joint Research Centre. As one of the main goals of the project is to pursue a better understanding of the potential of seismic rehabilitation methods, the experimental activity of the SPEAR project has foreseen pseudo-dynamic tests both on the 'as-built' and the FRP-retrofitted full-scale structure. In the paper, the strategy of the retrofitting intervention, consisting in the application of glass fiber wrapping, is described and the performance of the specimen in the two different configurations during the PsD tests is described. Through the experimental data, the effectiveness of the retrofitting strategy is thus assessed.

Keywords: frame; full-scale; GFRP; irregular; pseudo-dynamic; RC; seismic repair

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INTRODUCTION

One of the major sources of hazard in southern European Countries is represented by a number of existing under-designed RC structures that are non-compliant with current codified requirements for earthquake resistance. They have been designed following older codes and construction practice and not ensuring adequate provisions for earthquake-induced lateral loads. Among them, plan-wise asymmetric structures are quite common. Given the economic costs of demolishing and re-building under-designed structures, it is necessary to enforce a more rational approach for the seismic assessment and rehabilitation of existing structures in order to reliably identify hazardous buildings and conceive rehabilitation interventions aimed at the most critical deficiencies only. The SPEAR (Seismic PErformance Assessment and Rehabilitation) research Project, currently being carried out by a consortium of European Partners, is specifically targeted at existing under-designed-structures: evaluation of current assessment and rehabilitation methods, development of new assessment and retrofitting techniques, contribution to the improvement of current codes are some of its main goals.

In the framework of SPEAR, a series of tests on small members and subassemblies has been carried out; however, the core of the experimental activity is the series of full-scale pseudo-dynamic tests of a torsionally unbalanced three-storey RC frame structure, recently carried out at the ELSA Laboratory of the Joint Research Centre. In the SPEAR structure, the issues brought about by plan-irregularity in older structures are further enhanced by generally poor local detailing, low amount of steel longitudinal bars, insufficient confinement, weak joints and older construction practice. As above mentioned, one of the main goals of the project is to pursue a better understanding of the potential of seismic rehabilitation methods among which FRP materials certainly represent one of the most promising techniques. For this reason, the experimental activity of the SPEAR project has foreseen pseudo-dynamic tests both on the as-built and the FRP-retrofitted full-scale structure.

REHABILITATION OF RC UNDERDESIGNED STRUCTURES WITH FRP

The seismic rehabilitation of existing underdesigned structures (i.e., structures not designed to withstand lateral loads) can be achieved by means of different available techniques whose selection is generally made based on the deficiencies that the theoretical analysis or the observed post-earthquake damage point out. A classification of seismic rehabilitation methods is included in FEMA 356 guidelines (2000) where the following strategies are identified: local modification of components, removal or lessening of existing irregularities and discontinuities, global structural stiffening, global structural strengthening, mass reduction, seismic isolation, supplemental energy dissipation. The strategy adopted in the full-scale tests described in following sections belongs to the category of local modification of components pursued by installing FRP laminates. The FRP technique provides several advantages over traditional methods such as RC or steel profile jacketing and steel encasement that have been widely used in the past. It can allow overcoming disadvantages like difficulty of ensuring perfect bond, collaboration between old and new parts, loss of space, construction time, high impact on building functions, durability issues and mass increase. Laboratory outcomes confirmed the potential of FRP techniques for the upgrading of RC columns (Bousias et al., 2004) and joints (Antonopoulos and Triantafillou, 2002; Prota et al., 2004); the results obtained at element or subassemblage level were validated by tests on full-scale structures (Pantelides et al., 2004; Balsamo et al., 2005).

When FRP materials are used for seismic strengthening or rehabilitation of an existing RC structure, its global deformation capacity can be improved either by increasing the ductility of plastic hinges without their relocalization or establishing a correct hierarchy of strength by relocalizing the plastic hinges. Since the SPEAR project was focused on exploring the potential of a "light" rehabilitation intervention, the strategy followed in the rehabilitation of the full-scale structure presented in this paper was driven by the first of the two above mentioned options. Recalling that for a given

RC structure its global deformation capacity is governed by the plastic deformation capacity of its columns and beams, and that underdesigned structures generally lack of plastic deformation capacity of the columns, a “light” rehabilitation should aim at increasing their confinement, thus boosting the ductility of the compressive concrete and the rotation capacity of the plastic hinges. For typical axial load levels, the confinement of column ends has a strong influence on the cross-sectional ductility, but does not affect significantly its strength; this means that column strengthening should not modify the hierarchy of strength of the structure. However, it could be appropriate to increase the shear capacity of exterior beam-column joints by installing FRP laminates; this could allow preventing the shear failure of exterior joints that is brittle and could be detrimental to the global performance (Calvi et al., 2002).

DESCRIPTION OF THE STRUCTURE

The SPEAR structure represents a three-storey RC structure typical of old constructions realized in southern European Countries without specific provisions for earthquake resistance. Its design aimed at obtaining a gravity load designed (GLD) frame and was performed using the concrete design code enforced in Greece between 1954 and 1995 as well as both construction practice and materials typical of the early 70s. The structure is regular in elevation: it is a three-storey building with a storey height of 3 meters. Its plan configuration is depicted in Figure 1: it is non symmetric in two directions, with 2-bay frames spanning from 3 to 6 meters. The plan layout is characterized by the presence of a balcony on one side. One of the two bays is longer than the other by 1 m in the weak direction and 0.5 m in the strong directions; this increases the plan irregularity, shifting the centre of stiffness (CS) away from the centre of mass (CM) (Mola, et. al. 2004).

The concrete floor slabs are 150 mm thick, with bi-directional 8 mm smooth steel rebars, at 100, 200 or 400 mm spacing. The structure has the same reinforcement in the beams and columns of each storey. Beam cross-sections are 250 mm wide and 500 mm deep. They are reinforced by means of 12 and 20 mm smooth steel bars, both straight and bent at 45 degrees angles, as typical in older practice; 8 mm smooth steel stirrups have 200 mm spacing. The confinement provided by this arrangement is thus very low. Eight out of the nine columns have a square 250 by 250 mm cross-section; the ninth (column C6 in Figure 1) has a cross-section of 250 by 750 mm, which makes it much stiffer and stronger than the others along the Y direction (as defined in Figure 1) which is the strong direction for the whole structure. All columns have longitudinal reinforcement provided by 12 mm bars (4 in the corners of the square columns, 10 along the perimeter of the rectangular one). Their longitudinal bars are lap-spliced over 400 mm at floor level. Column stirrups are 8 mm spaced at 250 mm, which is equal to the column width, meaning that the confinement effect is very low.

The joints of the structure are one of its weakest points: neither beam nor column stirrups continue into them, so that no confinement at all is provided. Moreover, some of the beams directly intersect other beams (see joint close to columns C3 and C4 in Figure

1) resulting in beam-to-beam joints without the support of the column. The materials used for the structure are also those typical of older practice. A concrete nominal strength of f_c= 25 MPa was assumed in design; smooth steel bars were used having a design strength of f_y= 300 MPa (Table 1). Then, concrete cubes were tested during each construction phase; mean values of concrete strength for each slab and for each floor column were obtained and are summarized in The joints of the structure are one of its weakest points: neither beam nor column stirrups continue into them, so that no confinement at all is provided. Moreover, some of the beams directly intersect other beams (see joint close to columns C3 and C4 in Figure 1) resulting in beam-to-beam joints without the support of the column. The materials used for the structure are also those typical of older practice. A concrete nominal strength of fc= 25 MPa was assumed in design; smooth steel bars were used having a design strength of *fy*= 300 MPa (Table 2). Specimens of each diameter of steel bars were also taken during construction and tested; mean values of yield strength are reported in Table 1.

BI-DIRECTIONAL PSEUDODYNAMIC TEST: RATIONALE AND SETUP

A short description of the pseudodynamic (PsD) technique used in the tests is given in this section. A more detailed description of the method and of the mathematical approach can be found in Molina et al. (1999) and Molina et al. (2004). The bi-directionality of the PsD test, consisting in the simultaneous application of the longitudinal and the transverse components of the earthquake to the structure, introduces a higher degree of complexity, from both the analytical and technical standpoint compared to unidirectional PsD tests. In the case of bi-directional tests, three degrees of freedom (DoFs) per storey need to be taken into account: two translations and one rotation along the vertical axis, as opposed to the single degree of freedom per storey that is considered in unidirectional PsD tests. Four actuators per storey have to be connected to the structure, three of which are strictly necessary; the control of a redundant actuator requires a complex control strategy. The PsD integration of the horizontal response of the structure is performed in terms of three generalized DoFs at each floor, consisting of the in-plane displacements, d_X and d_Y, and of the rotation along the vertical axis, d_θ, at the CM of the structure. They are collected in the vector of generalized floor displacements. The in-plan restoring forces, R_X and R_Y, and the torque, R_θ, are collected in the vector of conjugated generalized restoring forces. Assuming for each floor the hypothesis of rigid-body behaviour, its horizontal motion is completely described by the generalized displacements and its equations of motion are derived from the application of D'Alembert's Principle, when the whole structural mass is assumed to be concentrated at the floor level. Thus, a 3N system of equations of motion governs the structural response, where N is the number of storeys and the variables are the generalized displacements of the CM.

However, the control system used for the test is based on a set of linear actuators and displacement transducers attached at prescribed locations at each floor. For this reason, the necessary transformations between the two systems of co-ordinates are developed. The measurement of floor displacements for control purposes is achieved using high-resolution linear displacement transducers attached to each floor. During the test, the computed generalized displacement of the floor is imposed by means of the actuators with feedback from these displacement transducers; thus, in order to determine the target displacement at the transducer level, a geometric transformation is first performed. At each step, each displacement transducer is associated to an actuator acting along the same direction; once the prescribed displacements of each transducer at each step are reached, the acting axial force in each actuator is measured by its load cell. It is then necessary to express such forces as resultant generalized forces at the CM of each floor by means of a static transformation. When more than three actuators act on a rigid floor, as in this case, the use of individual displacement transducers on the structure as feedback signals for the actuators can lead to control instability. For this reason, only a number of feedback displacements equal to that of the DoFs has been used, whereas the redundant actuators have been controlled by other means with the aim of guaranteeing an acceptable distribution of loads among all the actuators themselves. A dedicated algorithm is used to compute the optimal distribution of piston loads compatible with the known set of generalized floor forces. Two different approaches are usually employed to step-by-step solve the system of equations of motion: the explicit Newmark method or the α operator splitting method, which are both particular cases of the α-generalized method, an extension of the Newmark scheme. In this case, the explicit Newmark method was used because the time step was small enough in comparison to the natural frequencies of the specimen.

The servo-control units used for the tests under study are MOOG actuators with ±0.5m stroke and a load capacity of 0.5 MN. The control displacement transducers are Heidenhain sensors with a stroke of ±0.5m and a resolution of 2μm. Each actuator is equipped with a strain-gauge load cell and a Temposonics internal displacement transducer. When modelling the structure and implementing the time integration algorithm, the structural mass considered is the one that takes into account the presence of the finishing and of the quota of the live loads which is assumed to act at the time of the earthquake. Therefore, the mass properties are those resulting from the preliminary numerical simulations. The coordinates of the CM of each floor can be calculated with reference to the system of coordinates originating in C3 and shown in Figure 2; the mass values, the coordinates of the CM, x_{CM} and y_{CM}, and the moment of inertia with respect to the CM, I, are given in Table 2. It is underlined that the laboratory full-scale structure here discussed did not have finishing and live loads on it. For this reason, in order to reproduce the corresponding stress on the structural elements, a distribution of water tanks on each floor was studied, to simulate the presence of the finishing and of 30% of the live loads at each floor; the tanks were distributed so that the gravity loads on columns would be the closest to the values used in design. A view of the loaded frame prior to testing is shown in Figure 3.

INSTRUMENTATION

The layout of the instrumentation on the structure responded to different needs and considerations, both numerical and experimental. The bi-directionality of the test made it difficult and too demanding to conceive an instrumentation layout to trace the local bi-directional behaviour of all the elements at all the storeys. Moreover, the significance of such a choice would have been debatable. Based on the extensive preliminary numerical simulations (Jeong and Elnashai, 2004), the expected damage pattern had been defined, and the elements likely to exhibit the most significant behaviour had been identified. The structure was expected to fail due to column failure, rather than developing significant damage in beams or joints; moreover, a soft-storey mechanism at the first storey was expected in the weak direction and most of the damage was then expected on top and bottom of first storey columns, with the possibility of further damage taking place at the second floor. For this reason, the local instrumentation was mainly focused on the columns at the first and second floor, with inclinometers mounted at the member ends. To capture the effects of the hooks of the bars, inclinometers were also placed above the splice level. Moreover, on the two large faces of column C6, displacement transducers were located to measure the shear deformation of the column, without including the effects of bar slippage at the bottom. Finally, the beam-on-beam intersections (close to columns C3 and C4) on the soffit of the first and second floor were chosen to be more carefully investigated because they could have experienced local torsional effects. They were both instrumented with two inclinometers (one in each direction) and two crossed displacement transducers.

TESTING OF THE AS-BUILT STRUCTURE

Artificial accelerograms obtained from the Montenegro 1979 Herceg Novi ground motion records were used as the input signal for the PsD tests. Due to the plan-irregular configuration of the structure and the possibility of interchanging the longitudinal and transverse component, a number of preliminary analyses were run, in order to define the most appropriate direction of application. The aim was to maximize the effects of torsion on the response when determining the final combination for the test. To quantify the effects of torsion on the response, the standard deviation of the displacement demand imposed on the columns was examined: the larger this parameter, the larger the influence of torsional effects. Based on this criterion, it was decided to adopt the pair of signals that consisted in the application of the X signal component in the –X direction of the reference system of Figure 2, and of the Y signal component in the –Y direction of the same reference system.

Finally, the levels of peak ground acceleration (PGA) had to be defined. This was not an easy task, considering that such level was the critical parameter in determining the damage pattern and intensity of the specimen. The aim of the tests in the unretrofitted configuration, in view of the subsequent phases of the project, was to investigate the behaviour of the structure with a significant damage, but not so severe as to be beyond repair. In fact, the following repair and retrofitting phase was intended to consist into a light intervention, meaning that the level of damage inflicted in the first round of test should have been carefully and cautiously limited. To choose the acceleration level for scaling, damage levels of the structure under the Herceg Novi record scaled to different PGA values were investigated. The degree of damage was represented by the interstorey drift demand-to-capacity ratio of the columns. Due to the torsional irregularity, some of the columns were the critical ones: C3 because it had the highest axial load, C1 and C2 because they were the edge columns farthest from the CR. Based on the preliminary analyses, it was finally decided to run the first test in the'as-built' configuration with a PGA level of 0.15g, then to run one more at the intensity of 0.2g PGA.

In Table 3, the values of the maximum absolute interstorey drifts (rotations) reached during the four PsD tests are given, for each floor and for each DOF. In the first two rows the data relative to the as-built configuration can be observed. In the same way, the maximum absolute interstorey shears (torques) in the as-built configuration are reported in the first two rows of Table4. The maximum interstorey drifts in Table 3 are compatible with the damage pattern that was observed after the two rounds of tests. The major damage concerned the ends of the square columns with crushing of concrete at all storeys. The level of damage was more significant at the 2nd storey. For each floor the most damaged member was column C3 depicted in Figure 4, where it is also possible to observe the effects of the torsion reflected by inclined cracks on the compressive sides. During tests, significant cracks opened on the tensile side of the columns at the beam-column interface. The damage on the rectangular column C6 was less significant even though crushing of concrete and cracks at the interface with beams were observed (Figure 5). Details about the experimental performance of the as-built structure can be found in Negro et al. (2004).

DESIGN OF THE REHABILITATION WITH COMPOSITES

The structure was rehabilitated using GFRP laminates with uniaxial and quadriaxial (0°-90°-±45°) fiber texture. Prior to laminates installation, unsound concrete was removed in all zones of the elements where crushing was observed (Figure 6); then, the original cross sections were restored using a non-shrinking mortar (Figure 7). In addition, all cracks were epoxy-injected. The laminates were installed by manual lay-up and impregnated in-situ. The amount of GFRP to be wrapped around the columns was designed in order to have a significant increase of the rotational capacity of the columns. This has been achieved by means of two criteria: increasing the ultimate curvature of the cross sections of the columns by confining them with FRP and avoiding a reduction of the fixed-rotation in order to obtain a plastic hinge length of rehabilitated columns comparable to that of those as-built. In order to achieve this last goal, the external reinforcement on the joints was not connected to the columns.

The eight square columns were all confined at the top and bottom using 2 plies of GFRP uniaxial laminates having each a density of 900 gr/m^2 each (Figure 8-a). At each storey, the GFRP confinement was extended for 800 mm from the beam-column interface; in some cases, such length was increased up to 1000 mm in order to account for the more extended concrete damage. Then, the beam-column joints corresponding to the corner square columns (C2, C5, C7 and C8) were strengthened using 2 plies of quadriaxial GFRP laminates having each a balanced density of 1140 gr/m^2. This joint reinforcement was extended on the beams by 200 mm on each side (Figure 8-b) in order to U-wrap it and ensure a proper bond (Figure 8-c). Since column C6 Since column C6 has a sectional sides ratio equal to 3, shear could have controlled its behavior rather than flexure. For this reason, column C6 was wrapped for its entire height with two plies of the same quadriaxial GFRP laminates used for the above mentioned joints. Once the wrapping of the column was completed at each storey, the joint was strengthened. In this case the quadriaxial GFRP reinforcement was installed on both outer and inner parts of the joint. For the outer part, the joint reinforcement had the height of the beam; it was extended for 200 mm on the adjacent members (Figure 9-a) and then U-wrapped (Figure 9-b). The same philosophy was followed for the inner part, even though the presence of the slab determined an height of the external reinforcement equal to 350 mm (Figure 10-a); the extension of adjacent beams and the U-wrap were equal to those of the outer part (Figure 10-b).

TESTING OF THE REHABILITATED FRAME

Once FRP-retrofitted, the structure was then tested under the same input ground motion, at first with a PGA level of 0.20 g PGA, to have a direct comparison with the previously executed experiment, then with a PGA level of 0.30g. In the last two rows of Table 3), the maximum interstorey drifts reached during the tests in the FRP retrofitted configuration can be observed. In the same way, the maximum absolute interstorey shears (torques) in the FRP retrofitted configuration are reported in the last two rows of Table 4). In general, the experimental behaviour of the rehabilitated structure has been very close to that expected according to the rehabilitation design; no brittle mechanisms have occurred (i.e., shear failure of beams or significant damage of joints). It has been observed a very ductile behaviour of the columns (Figure 11); the damage of the unstrengthened joints have highlighted an incoming failure of beams due to crushing of concrete and the initiation of a shear crack pattern of the joints themselves (Figure 12-a), whereas no visible damage has been detected on the strengthened joints (Figure 12-b).

ORIGINAL VS REHABILITATED FRAME PERFORMANCES

In Figure 13, a comparison is made between the as-built and the FRP structure's responses, in terms of displacements: the displacement time histories in the X direction at the second storey (the largest ones) for the two structural configurations and three PGA levls are reported. In the same way, in Figure 14, the time histories of the interstorey drifts at the CM in the X direction (the weak one) are compared. In Figure 15, the time histories of the flexible edge columns drifts in the X direction for the two structural configuration and three levels of PGA are also compared. This allows a better understanding of the relative importance of the rotational and the translational DOFs in the different structural configurations. The trends shown by the X direction displacements and drifts apply to the Y direction, too. Finally, Table 3 and Table 4) allow a quick comparison between the maximum values of drifts and shears to be drawn.

It can be seen that the retrofitting intervention provided the structure with a very significant supply of extra ductility, with respect to the original configuration, which was almost totally lacking the appropriate capacity to withstand even the 0.20g PGA level of excitation. On the contrary, after the vertical elements and the joints were wrapped with glass fibers, the structure could withstand the higher (0.30g PGA) level of excitation without exhibiting relevant damage. The maximum displacements reached during the latter test were around 160mm at the second storey, with a roughly 50% increase with respect to the maximum values reached during the 0.20g PGA test in the unreftrofitted configuration. Drifts also increased by around 50%, for example at the second storey in the weak direction (106mm vs. 57.1mm). From Figure 14 and Figure 15, it can be observed that the FRP intervention did not change the torsional behaviour of the structure, which is consistent with the local modification of components approach. In fact, the rotational DOF has the same strong relevance in both configurations; the strongest torsional effects are just shifted in time, in the two cases: for the as-built structure the rotational DOF gets the most active around 12s in the time history, whereas in the FRP retrofitted one at the high level of excitation, the strongest rotational effects are around 10s.

CONCLUSIONS

The paper presented the experimental performance of a full-scale underdesigned RC frame subjected first to bi-directional PsD tests in the as-built configuration and then retested after have been rehabilitated using GFRP laminates to confine the ends of the columns and to strengthen the corner beam-column joints. The preliminary analysis of test results herein performed highlights that the FRP rehabilitation enabled the structure to exhibit a larger displacement capacity compared with the as-built, thus withstanding a level of excitation in two directions higher than that applied to the as-built without attaining brittle mechanisms.

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Table 1 – Design and actual material properties

Concrete				Steel bars		
Story	Member	Actual f_c (MPa)	Nominal f_c (MPa)	Bar	Actual f_y (MPa)	Nominal f_y (MPa)
1st	column	24.73	25	8mm	358	300
	slab	26.70	25			
2nd	column	26.70	25	12mm	327	300
	slab	27.53	25			
3rd	column	25.32	25	20mm	321	300
	slab	27.39	25			

Table 2 – Coordinates of the CM and mass moment of inertia I at each floor

Storey	Mass (kN)	x_{CM} (m)	y_{CM} (m)	I (kN m^2)
1	672.64	-1.58	-0.85	15007
2	672.64	-1.58	-0.85	15007
3	628.04	-1.65	-0.94	13634
total	1973.32			

Table 3 – Maximum absolute interstorey drifts reached during the four PsD tests at each floor and for each global DOF

	TEST/FLOOR	FLOOR 1	FLOOR 2	FLOOR 3
MAXIMUM I-S DISPLACEMENTS	0.15g PGA AS BUILT	X DIR: 15.0mm Y DIR: 11.6mm θ ROT: 3.3mrad	X DIR: 36.2mm Y DIR: 19.7mm θ ROT: 5.8mrad	X DIR: 24.2mm Y DIR: 18.2 mm θ ROT: 4.05 mrad
	0.20g PGA AS BUILT	X DIR: 24.5mm Y DIR: 30.06mm θ ROT: 4.2mrad	X DIR: 57.1mm Y DIR: 47.0mm θ ROT: 9.9mrad	X DIR: 35.8mm Y DIR: 32.1mm θ ROT: 7.27mrad
	0.20g PGA FRP	X DIR: 32.1mm Y DIR: 39.7mm θ ROT: 7.04mrad	X DIR: 55.3mm Y DIR: 47.6mm θ ROT: 11.9mrad	X DIR: 34.1mm Y DIR: 47.6mm θ ROT: 7.75mrad
	0.30g PGA FRP	X DIR: 59.3mm Y DIR: 42.3mm θ ROT: 7.7mrad	X DIR: 106.0mm Y DIR: 55.9mm θ ROT: 13.3mrad	X DIR: 63.5mm Y DIR: 50.7mm θ ROT: 7.38mrad

Table 4 – Maximum absolute interstorey shears reached during the four PsD tests at each floor and for each global DOF

	TEST/FLOOR	FLOOR 1	FLOOR 2	FLOOR 3
MAXIMUM I-S SHEARS AND TORQUES	0.15g PGA AS BUILT	X DIR: 176 kN Y DIR:259 kN θ ROT: 875 kNm	X DIR: 161 kN Y DIR: 253 kN θ ROT: 730 kNm	X DIR: 126 kN Y DIR: 146 kN θ ROT: 608 kNm
	0.20g PGA AS BUILT	X DIR: 193 kN Y DIR: 274 kN θ ROT: 955 kNm	X DIR: 165 kN Y DIR: 213 kN θ ROT: 728 kNm	X DIR: 110 kN Y DIR: 166 kN θ ROT: 709 kNm
	0.20g PGA FRP	X DIR: 208 kN Y DIR: 286 kN θ ROT: 1070 kNm	X DIR: 163 kN Y DIR: 258 kN θ ROT: 817 kNm	X DIR: 112 kN Y DIR: 171 kN θ ROT: 716 kNm
	0.30g PGA FRP	X DIR: 190.2kN Y DIR: 277 kN θ ROT: 1000 kNm	X DIR: 168.3 kN Y DIR: 276.0 kN θ ROT: 801 kNm	X DIR: 122.8 kN Y DIR: 101.9 kN θ ROT: 631.4 kNm

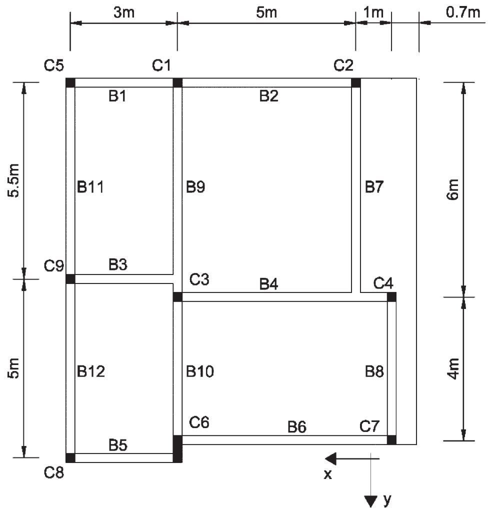


Figure 1 — Plan view of the SPEAR structure

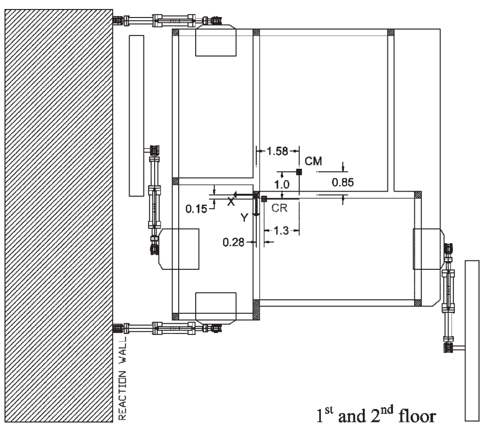


Figure 2 – Load arrangement and location of the CM of the structure

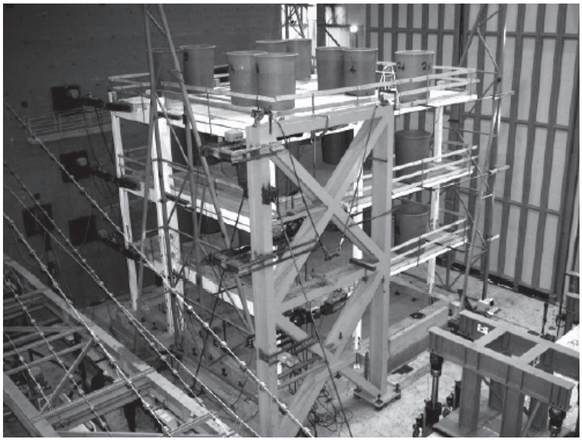
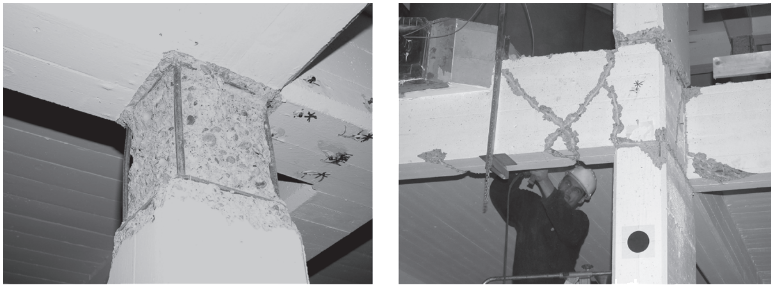


Figure 3 – View of the SPEAR structure before a PsD test

Figure 4 – Damage on column C3 at 1st storey

(a) (b)

Figure 5 – Damage on column C6 at 1st (a) and 3rd (b) storeys

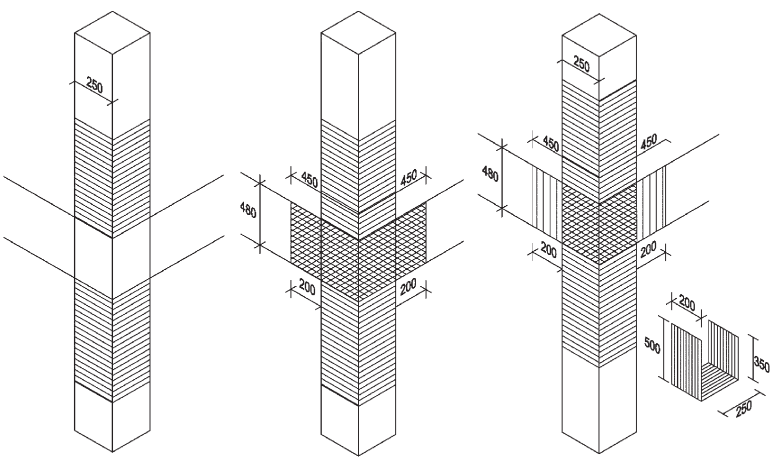


(a) (b)

Figure 6 – Columns C3 (a) and C6 (b) at 1st story after removal of unsound concrete

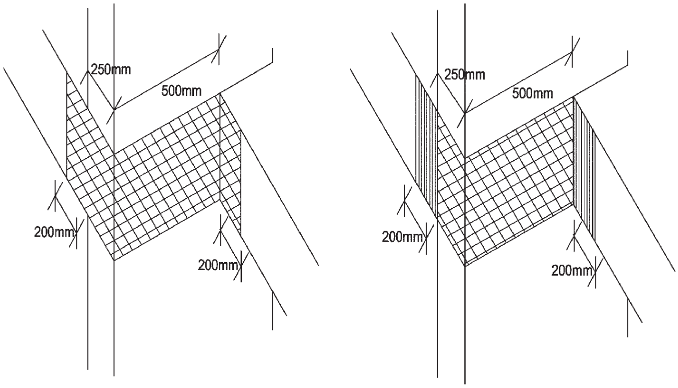
(a) (b)

Figure 7 – Columns C3 (a) and C6 (b) at 1st story after restoration of original cross-sections



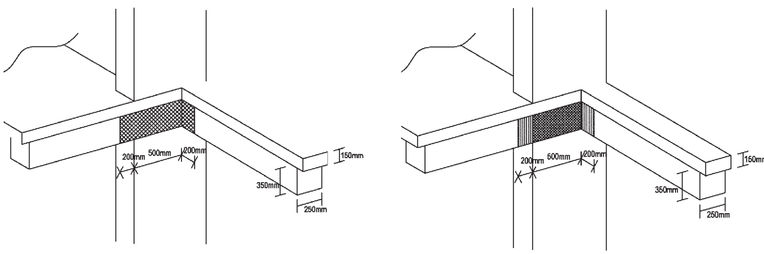
(a) (b) (c)

Figure 8 – Rehabilitation of exterior columns and joints: column wrapping (a), joint strengthening (b), U-wrapping of joint strengthening on the beams (c)



(a) (b)

Figure 9 – Outer portion of joint of column C6



(a) (b)

Figure 10 – Inner portion of joint of column C6

(a) (b)

Figure 11 – View during test in progress from column C2 (a) and C6 (b)

(a) (b)

Figure 12 – Damage at unstrengthened joint of column C9 (a) and at corner joint of column C2 (b)

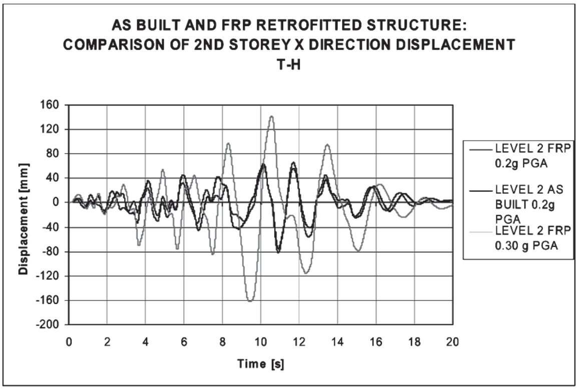


Figure 13 – Comparison between the response in the 'as-built' and the unretrofitted configuration in terms of displacements

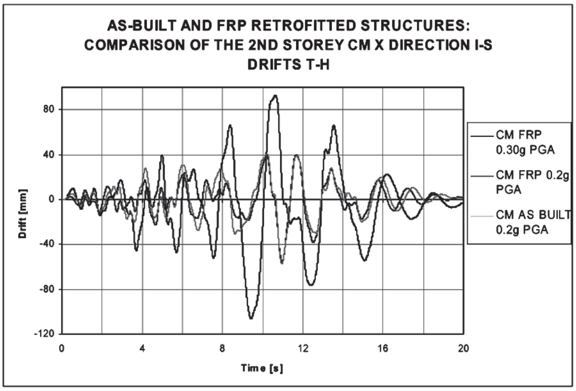


Figure 14 – Comparison between the response in the 'as-built' and the unretrofitted configuration in terms of interstory drifts at the CM

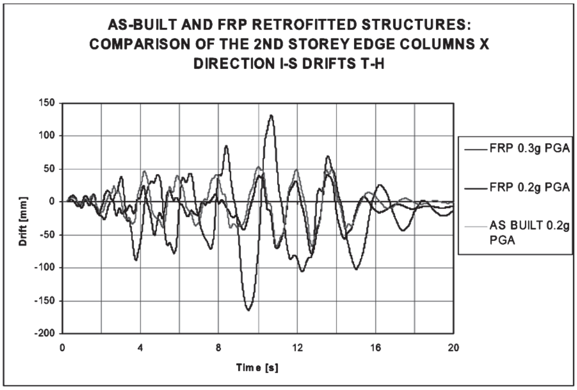


Figure 15 – Comparison between the response in the 'as-built' and the unretrofitted configuration in terms of interstory drifts at the flexible edge