Retrofitting the precast RC wall panels using externally bonded CFRP laminates

PhD Thesis – English Summary

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This work is relevant to the field of earthquake engineering and addresses the seismic behaviour of the reinforced concrete walls. The objectives of the thesis are to investigate the seismic performance of the precast reinforced concrete walls and reveal the effects of the seismic retrofit by externally bonded carbon fibre reinforced polymers laminates. The experimental program consisted of ten quasi-static cyclic tests on near-full scale precast reinforced concrete wall specimens. The experimental variables referred to the opening type and the strengthening condition. The influence of the cut-out opening size on the shear strength, stiffness and was considerable. The retrofitting technique by means of CFRP-EBR laminates improved the behaviour characteristics, primarily in terms of maximum load bearing capcity and maximum displacement; however, certain limitations were identified on the use of this strengthening system in reversed cyclic applications.

1. INTRODUCTION

1. Framework

This paper addresses the seismic response of precast reinforced concrete walls and the possibility to retrofit them. In the communist era of Romania there was a high demand for new housings for the citizen in the urban area, thus the need for buildings with good seismic response and fast development was needed. The answer came in form of the typical Eastern Europe apartment buildings made with wall structural systems made either of precast reinforced concrete large panels (PRCLP) or monolithic. In terms of functionality this apartment buildings are not up to today's living standards and the owners of such apartments are in search for solution to improve the comfort of their homes by modifying the existing floor plan. However, this proves to be quite difficult as the reinforced concrete wall structural system is not modular, forcing the occupants to create new openings in the existing walls or to increase the existing openings by cut-outs. In the past few decades, there have been considerable advancements in the design procedures of reinforced concrete (RC) shear walls for new construction, such as the new performance-based seismic design and capacity design principles. Alongside these advances for new buildings research has begun in the field of retrofitting of these buildings with reinforced concrete walls as structural system, nonetheless the fact that there are so many opening possibilities, different design of the panels during the years and distinct retrofitting strategies, the search continues for the most effective retrofitting procedure and the understanding on of the cut-outs on the seismic behaviour.

2. Motivation and objectives

The research theme is focused on experimental studies on the behaviour of precast reinforced concrete large panels subjected to reversed cyclic loading, simulating the seismic action. The objectives of this thesis are to provide further understanding and improve the knowledge on several key discussions.

- How does the precast reinforced concrete wall panels perform and fail when subjected to in-plane reversed cyclic loading?
- Can the ERB-CFRP strips retrofitting technique restore the walls initial load bearing capacity?
- How does the retrofitting procedure influence the failure modes of the elements?
- Does the size and type of opening or weakening influence the structural behaviour and capacity at limit states?
- What is the energy dissipation capacity of the shear walls with different openings and how does it compare to the retrofitted counterparts?
- Are design code provisions for estimating the shear strength accurate?
- How does the results obtained compare to others found in literature?

3. Overview of the thesis

The thesis is composed of six chapters and two appendices totalising more than 180 pages. The thesis focuses on ten quasi-static cyclic tests on near-full scale precast wall panels.

In the first chapter, the framework of the thesis is introduced, the motivation and objectives that lead to the research are given and an overview of the thesis is presented.

Chapter 2 offers a general presentation of the seismic zones in Romania and the ways scientist try to predict the earthquakes. Additionally, the distribution of the most popular and widely used construction system for apartment buildings in Romania is given, with the presentation of the prototype used for the experimental campaign. Afterwards, a review of the literature on RC wall tests is presented, followed by a description of retrofitting strategies and materials and an extensive review on retrofitted RC wall tests.

In Chapter 3 a detailed description of the experimental program is given. The chapter commences with the timeline of the tests alongside the description of the experimental specimens in terms of concrete outlines, steel reinforcing details. Subsequently, the properties of the three structural material types are presented. Special attention was paid to the description of the retrofitting strategies employed in the research and the technique used to apply them. In the followings the test set-up, the loading protocol and boundary conditions are detailed. The experimental program is concluded by the instrumentation of the specimens.

In the 4th Chapter the results obtained by the ten cyclic wall tests are rendered in two ways, namely primary results and detailed test logs. The primary results consist in the load-displacement response, the loading and displacement histories, the final cracking pattern, a brief description of the observed behaviour and failure mode, and a limited number of photographs on failure details. Another extensive part of the chapter is the analysis of the results. In accordance to the general seismic performance characteristics of the lateral load resisting members the following analysis types were undertaken: strength and ductility analysis, displacement analysis, stiffness analysis, theoretical study and energy dissipation analysis. In addition to the measured response analyses, the observed behaviour aspects peculiar to concrete members were also addressed through cracking analysis.

In Chapter 5 the conclusions are drawn with respect to the retrofitting effect of the FRP-EBR system on the cyclic response of the precast reinforced concrete wall panels. The chapter is concluded by an account of the author’s publications and his personal contribution to this work.

In the last chapter, an outlook is provided for future research directions.

The Appendices contain supplementary descriptive information consisting in charts which would have been disruptive if presented in the main body of the thesis. In Appendix A a comparison between similar specimens in regards to their force-drift ratio analysis is made and in Appendix B the detailed test logs for all specimens are given

2. LITERATURE REVIEW

1. Seismic zones in Romania

Romania is situated in the southeaster part of Central Europe and is the twelfth country of Europe, considering its area of 238391 km2 [1]. In the southern part, Romania borders Serbia and Bulgaria, part of this border being the Danube (1075 km). Other borders are with Hungary, Ukraine, Moldova and the Black Sea. The country is crossed by the Carpathian Mountains which are a subdivision of the larger Alps-Himalayan System part of the Alpide belt. According to USGS [2] the Alpide belt is the second most seismically active region worldwide accounting for 17% of the largest earthquakes. Located in the subcrustal lithosphere in the beginning of the Eastern Carpathians Vrancea region, the most dangerous seismic zone in Romania, is characterized by earthquakes of intermediate-depth.

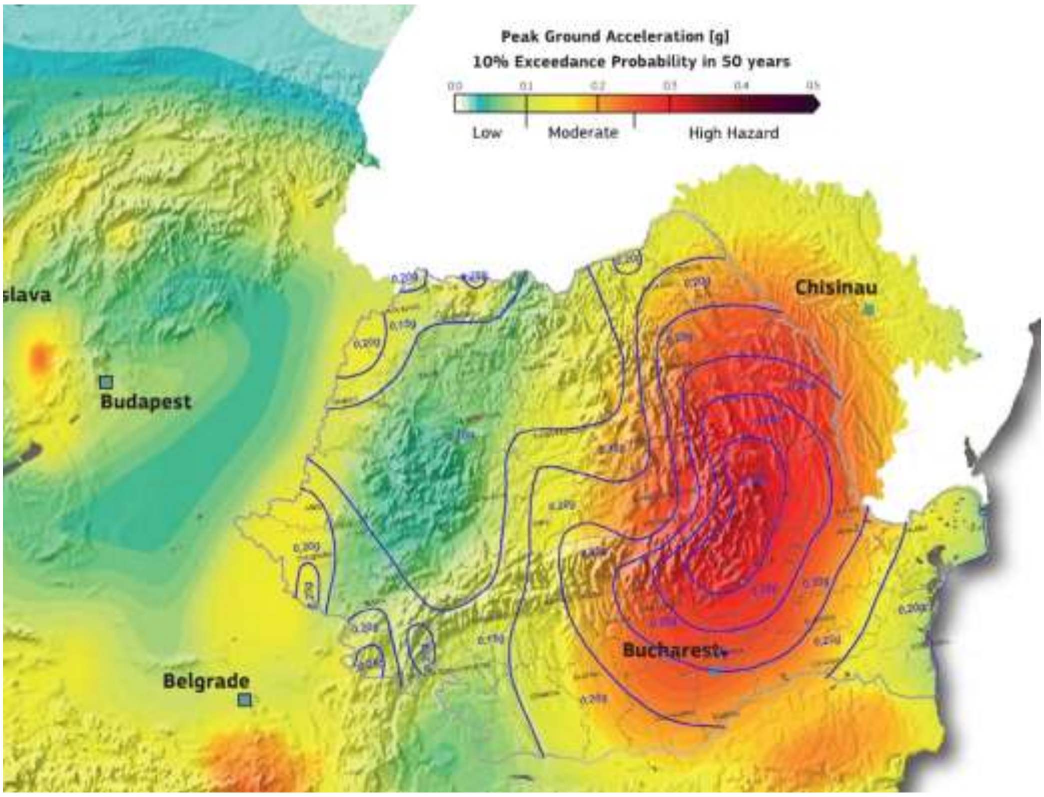


Figure 2-1 SHARE map and P100-1 2013, peak ground accelerations for Romania overlay.

“Seismic hazard describes a natural phenomenon associated with an earthquake and can be quantified by a level of severity (e.g., peak ground acceleration, macro-seismic intensity), its occurrence frequency and location” [2]. Seismic hazard prediction is very intricate and arduous because of the specificity of each regions characteristics and classic seismic hazard analysis is not relevant ever to strong and deep earthquakes in Vrancea. [3]. A comprehensive state of the art in regards to geodynamics and intermediate-depth seismicity in the area was done by Zadeh et al. [2], proposing further perspective studies to improve several key aspects like which kind of studies should be conducted in the region to explain the unresolved problems, the better understanding of the lithosphere and the mantle beneath Vrancea and how to better the earthquake forecast and develop reliable seismic models.

2. Precast reinforced concrete large panel buildings in Romania

In Romania one of the most widely spread construction type of apartment buildings are the ones with precast reinforced concrete walls. The popularity of these structural system gained momentum in the post second world war era, when a large number of apartments were needed in the urban areas. According to [4] the urban population living in apartment blocks in 1966 was 17.4% and it has increased to over 42% by 1977 reaching a staggering 71.4% by the early nineties. In Figure 2.2 the distribution of dwellings based on their construction material [5] is presented and analysing the chart one can see that most of them are precast dwellings and reinforce concrete ones with a combined share close to 50%. A short examination of the large-panel structures was published by Demeter et al. [6].

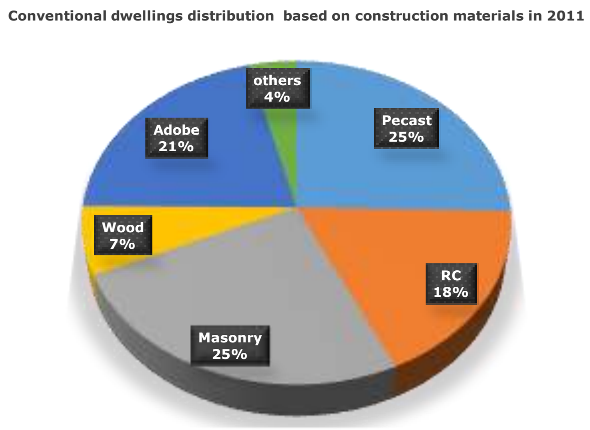


Figure 2-2 Dwellings distribution based on construction materials

Romania has a population of approximately 20 million people [7], the highest number for population was measured in 1992 with over 22 million people living in the country. Although the trend of stable population is descending, Romania is the 7th largest country in European Union by population [8] and the 58th worldwide [9]. When it comes to where Romanians live, 35%, of the population lives in apartment buildings, meaning close to 7 0000 000 people [5]. Out of these we can say that 2.75 million people live in precast apartment buildings and over 2.50 million live in reinforced concrete apartment buildings. As the peak construction period for these buildings was over 30 years ago, most of the people living in apartment building have an old dwelling constructed in the socialist era. For the current thesis, a typical large panel building was chosen, the Romanian Project Type 770-81 built after the 1977 earthquake in 1982, the same one used in previous work related to seismically performance of sheer precast walls, done by D. Istvan [10] and C. Todut [11] within this experimental program. Besides the existing two shear walls from previous works, 4 new shear walls were cast, the specimens tested are build according to the P100 regulations from 1981, the revised regulation after the 1977 earthquake.

3. RC walls seismic laboratory test

Precast reinforced concrete shear walls have been in use in buildings for many years and had positive behaviours during major earthquake occurrence throughout the history. Several major studies were developed in order to investigate the seismic response of these panels when subjected to in plane loading.

4. Retrofitting procedures using FRP for RC

Several different retrofitting procedures for reinforced concrete structures exist in the literature. Based on the elements characteristics and purpose the appropriate retrofitting procedure can be chosen. An introduction into FRP, properties, process and concepts are presented by M. A. Masuelli [12] who outlines the large functionality that these products (Aramids, Composites, Glass-FRP, and Carbon FRP) have. Several different recent retrofitting procedures using FRP are presented in this chapter.

Fibres of aromatic polyamide in which 85% of the amide linkages are attached directly to two aromatic rings are called "aramid". These are one of the first polymers used in retrofitting existing structures. Aramid Fibre Reinforced Polymer (AFRP) can replace the classic reinforcement since they are not prone to corrosion, accelerated aging tests on aramid fibre reinforcement was done by Soroushian et al. [13].

Another material that is not very commonly used in retrofitting procedure is the basalt fibre reinforced polymers (BFRP). These fibres were first developed by the Moscow Research Institute of Glass and Plastic in the 1950s by melting basalt rocks. A review of these basalt fibres and their composites was made by V. Fiore et al. [14]. The first concern with this material was the safety, since it presents similar composition to asbestos. However, studied showed that it poses no threat to humans and can be safely used. [15, 16]. The mechanical properties of these fibres are similar to those of glass fibre when it comes to tensile strength, but the Elastic modulus of these fibres is higher [17].

Glass Fibre Reinforced Polymers (GFRP) as the name implies is a composite made out of glass fibres contained with a polymer matrix. The GFRP is more widely spread in retrofitting of old, existing constructions and new ones than the aramid ones. One of the main advantages is that the GFRP is easier to produced and it is being used in different domains successfully for a long period of time. As in the case of AFRP the GFRP comes in several forms.

The most popular way of retrofitting existing RC structures is by using Carbon Fibre Reinforced Polymers (CFRP). Compared to the other presented FRP systems the carbon has the highest elasticity modulus and the largest tensile strength. The only downside of CFRP is cost, since it is significantly more expensive than other FRP systems. Many studies have been made in regards to the use of CFRP for structural rehabilitation of reinforced concrete structures, in terms of shear strengthening, flexural strengthening, confinement of concrete etc

5. Retrofitted RC walls seismic laboratory test

In the literature, there are several researchers whose experimental test regarding the retrofitting procedure for shear walls have improved the knowledge on the behaviour and capacities of these elements.

3. EXPERIMENTAL PROGRAM

1. Introduction

The research objective is to simulate the behaviour of RC structural walls subjected to seismic actions. The experimental program involved testing scaled models of PRCLP designed and constructed according to the code provisions that were in effect in the 1980's. the experimental test was conducted in the Reinforced Concrete Structures Laboratory of Civil Engineering Department, Construction Faculty, Politehnica University Timisoara, Romania. The presented research is a continuation of a larger investigation regarding PRCLP started by I. Demeter [10] and continued by C. Todut [11].

Ten experimental tests were carried out on six different specimens known as PRCWP [13-18], two existing ones from the previous stage tests and four new build one, starting April 2013 until July 2015, as presented in Figure 3.1. The tested specimens had different opening types and cut-outs. Several specimens were retrofitted prior-to-damage or post-damage using EBR-CFRP strips or EBR-CFRP combined with NSM.

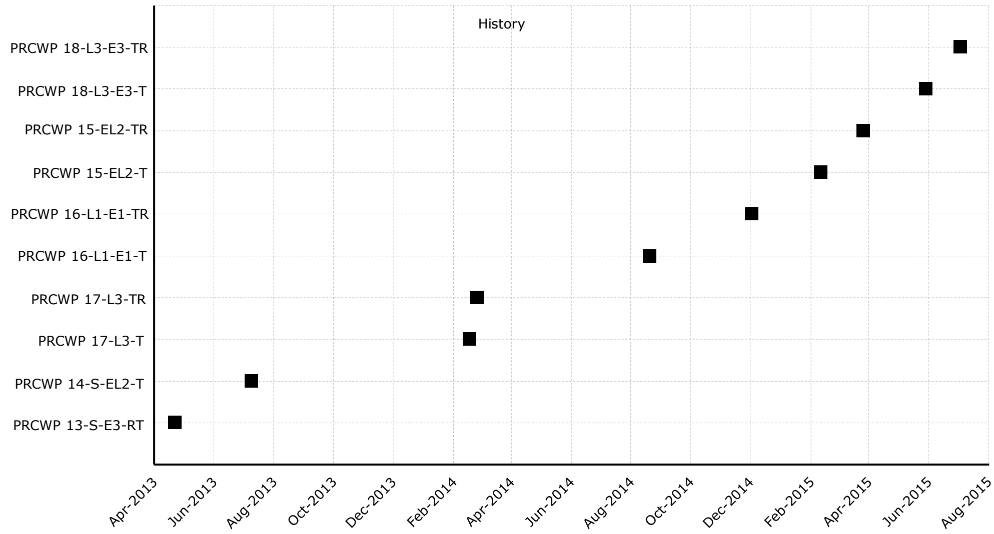


Figure 3-1 Testing timeline

2. Test Specimens characteristics

The experimental wall specimens were constructed and designed according to the 1980 Romanian large panel manufacturing practice. In this thesis, the prototype wall specimens were chosen to be the interior wall denoted I54-2a and the exterior longitudinal panel denoted E 36-7. This wall types can be considered representative for the post 1977 large panel building and can be found in several different plans of the 770-series. Considering the limitations imposed by the testing facility and the lifting capabilities of the bridge crane available in the laboratory, all the tested elements are scaled down with a reduction factor of 0,83, corresponding to 1:1.2 model. Each experimental test specimen can be referred to as a wall because it represents one storey, one bay and one plane member. In order to simulate the connection joints towards the adjacent panels of the as-build conditions, all tested units have T-shaped boundary elements which also prevent the out-of-plane displacement during the in-plane lateral loading. These elements are referred to as wings.

The specimens tested in this thesis were fabricated on two different sites. Two of the six specimens, namely 13 and 14, were manufactured in Timisoara in 2007 with the reinforcement and concrete provided by the construction firm. The other four specimens were manufactured in Stei, county Bihor, at a concrete station within a construction firm. All were cast in horizontal position and vibrated in the formwork.

Figure 3-2 Construction of specimens

As the two specimens manufactured in Timisoara were solid elements and in this thesis the author wanted two specimens with openings cut-outs, two opening were designed and cut-out from the specimens in the laboratory, by removing a door-shaped portion from a solid wall the rectangular outline of the web-panel changes and it becomes a frame-like element composed of two piers and a spandrel beam at the sides and above the opening, respectively. For the experimental program the author had to test the specimens as unstrengthen first and afterwards, repair and strengthen them, in order to be able to retest them. Thus, the specimens were named using distinguishing notations by adding at the end of the tested specimens name the suffix T, TR or RT. T stands for element unstrengthen tested prior to damage, TR for element tested, retrofitted post damage then rested and RT stand for elements retrofitted prior to damage and tested. All specimens had some kind of initial opening and the reinforcement is in accordance with the opening type. In the coupling beam above each opening there is a spatial steel reinforced cage comprised of 4 Φ10 mm diameter bars with Φ8 mm stirrups. The spatial cage length of the narrow door or window opening differs from the one of the large door or window opening. The PRCWP 15-EL2 specimen had a vertical spatial steel reinforcement cage on the entire height of the left pier, the cage comprised of 6 Φ6 mm diameter bars and Φ6 mm stirrups. In all specimen piers that were not reinforced with spatial cages vertical and horizontal steel bars and welded mesh was provided. In the parapet of all specimens welded mesh of Φ8 mm diameters cold drawn wires and 150 mm centres was present. It can be seen for the specimens having opening cut-outs how the reinforcement was placed in order to simulate the opening cutting through the bars and the welded mesh.

3. Material properties

Materials used in the wall test consisted of concrete, steel reinforcement, CFRP reinforcement and repair mortar. Tests, in accordance to [18], were carried out on the concrete from all walls, while for the walls manufactured in Timisoara, material test were also carried out, by Demeter [10], on the steel in compliance with [19, 20].

The concrete was provided by the concrete station were the walls were constructed, and was prepared in accordance to the concrete recipe for C16/20 concrete clas. From each specimen six 150 mm edge cube samples were obtained, after the concrete had settled and hardened for one or two days the samples were removed from the moulds and were placed into water basin and maintained there during strength development.

The tests on the compressive strength of the cubes were carried out at the Laboratory of Civil Engineering Department, Construction Faculty, Politehnica University Timisoara, Romania, using the Universal testing machine of 2000 kN, the samples were tested after a hardening time of minimum 39 days and maximum 66 days, so the coefficient βcc(t), given in Eurocode 2 [21], for the concrete strength variation in time was not considered necessary. The results of the compression test obtained are similar for all samples, indicating a good and constant concrete quality, however, it can be seen that the concrete compressive strength indicated, for PRCWP 17-L3, a class that is superior to the desired C16/20. This can be attributed to the recipe used that was made in order to ensure each time a minimum quality of C16/20. It can be seen from Table 1 that the compressive strength of samples from PRCWP 18-L3-E3 is extremely low for the reason that the samples had frozen in the moulds before placing them in water. Fortunately, during the transportation and the test it was observed that the specimen did not have the same low compressive strength so a similar strength to the other ones was assumed.

Table 1 Properties of the concrete

Element	$f_{cm,cube}$ **(N/mm²)**	f_{cm} **(N/mm²)**	f_{ck} **(N/mm²)**	**Class**
PRCWP 15-EL2	27,35	22,43	16,91	C16/20
PRCWP 16-L1-E1	29,03	23,80	17,95	C16/20
PRCWP 17-L3	34,1	27,96	21,08	C20/25
PRCWP 18-L3-E3	10,25	8,41	6,34	C6/7,5

For the specimens manufactured in Timisoara, tensile tests were performed using the Universal Testing Machine of the Steel Structures Laboratory from Politehnica University Timisoara, for each type of reinforcement used, namely: smooth (OB37), ribbed (PC52) hot-rolled bars and cold-drawn ribbed welded mesh (STNB). These tests were done by Demeter [10]. For the specimens manufactured in Stei the properties of the reinforcement are considered to be the typical Romanian reinforcements characteristics.

For the repair of the concrete that was very damaged, high strength mortar was used form Mapei. Two types of mortar were used, one was the Mapegrout Rapido which has the property of fast setting time between 50-60 minutes [22], the other mortar used was the Mapegrout Easy Flow GF, which is a pre-blended, one-component thixotropic cementitious mortar, made from sulphate-resistant hydraulic binders, polyacrylonitrile synthetic fibres, inorganic, fibres, organic corrosion inhibitors, special admixtures and selected aggregates [22].

Three types of FRP composites were used in this thesis, carbon fibre laminates from Mapei for the main strengthening system, unidirectional carbon fibre fabric used for the anchors, carbon fibre grid used as confinement. The properties of the experimental CF-products and resin matrix are taken from data sheets provided by the producer.

4. Repair and Strengthening

The author of this thesis adopted two strengthening strategies for the experimental specimens, one using Near Surface Mounted (NSM) Carbon Fibre Reinforced Polymers (CFRP) combined with Externally Bonded Reinforcement (EBR) Carbon Fibre Reinforced Polymers (CFRP) and the other using Externally Bonded Reinforcement (EBR) Carbon Fibre Reinforced Polymers (CFRP) laminates. The strengthening strategies are adopted according to the observed behaviour and failure mode of the unstrengthen (reference) specimens, with focus on the critical zones. Each specimen has its own unique strengthening strategy based on what was observed during the unstrengthen tests. The general characteristics of the behaviour of the reference specimens were as follows: shear cracking of the piers was the main reason for failure, concrete crushing of the spandrel-pier connection joint, concrete crushing of the bottom extremities of the piers.

The first retrofitting strategy was adopted for the prior to damage strengthened specimens 13 S-E3-RT, the only specimen that was not tested initially as unstrengthen. Considering the reinforcement of the specimen, which is specific to a solid element, and the opening type of a large door (E3), the specimen had two symmetrically reinforced piers connected by the spandrel. The author of this paper presents the retrofitting strategy only for one side of the specimen, however one should keep in mind that both sides of the specimen were symmetrically retrofitted.

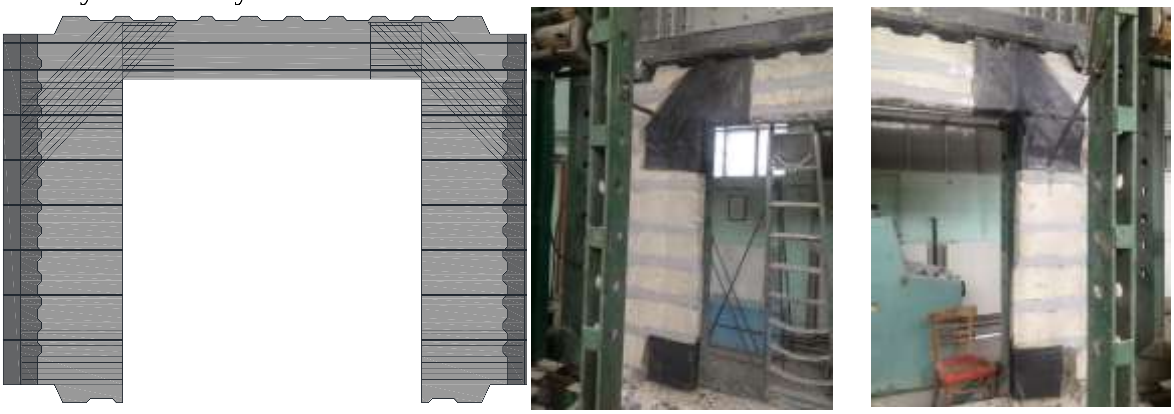


Figure 3-3 Strengthening strategy using NSM-CFRP combined with ERB CFRP

All specimens manufactured in Stei were retrofitted using externally bonded (EBR) Carbon Fibre Reinforced Polymers (CFRP) laminates, and the retrofitting strategy was applied post-damage. Each of the tested specimens had slightly different distribution of the strengthening materials on their surface depending on the behaviour of the unstrengthen specimen and on the cracks pattern distribution. Some of the details regarding the retrofitting strategy for each specimen are presented in Figure. 3.4. Similarly, to the previous retrofitting strategy, in the thesis only one side of the element is presented, however both sides were symmetrically reinforced. The dimensions of the CFRP laminates were the same for all the specimens, they were 3,2 cm wide and 1.4 mm thick and anchored using either CFRP mesh or CFRP grid. Given the fact that all walls had some type of opening, the spandrel had one CFRP plate running alongside the opening for all the specimens, however, the position of the plate differs.

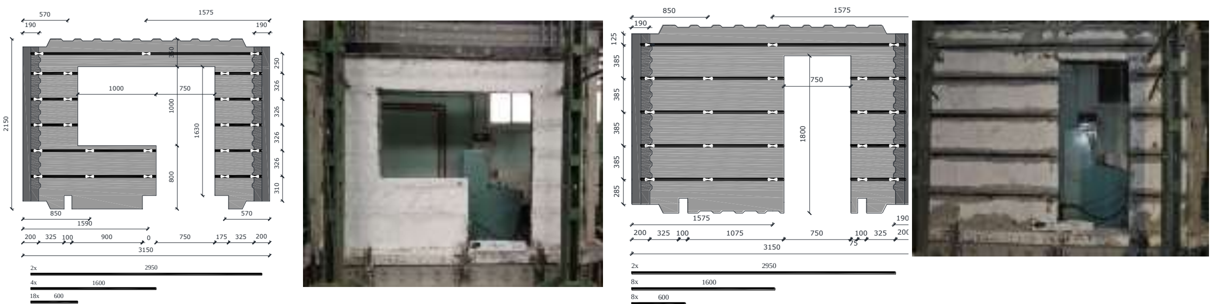


Figure 3-4 The position of the EBR CFRP laminates

5. Experimental test set-up

The test set-up configuration comprises of four vertically steel reaction frames of 1000 kN capacity, anchored on the existing steel anchorage point in the Reinforced Concrete Structures Laboratory, two truss type steel lateral reaction frames of 1000 kN capacity, the hydraulic loading device, electric pump, hand pumps, a series of hydraulic jacks, cylinders and hoses of 375 bar working pressure. For transmitting the forces to the specimen two loading beams were used, one placed at the top of the element named cap beam and one at the bottom of the element named base beam. These beams were designed by Demeter [10] to be used in all the experimental test that followed, hence the high steel percentage in the beam and the high concrete class of the beams. The loading beams consist of two U300 steel channels and a reinforced concrete T beam, these were connected using Φ20 mm threaded rods. For the top panel, horizontal edges shear keys were formed and at the beam end shear steps were provided. In order to connect the beams to the reaction frames, special connection details were provided. For the base beam Φ70 mm steel bolt hinges and for the cap beam ball bearings at the end. For the experimental wall anchorage to the base beam Demeter designed a lap-welding of 4-5 vertical continuity steel rebar, depending on the opening type of the specimen, however Todut in her experimental program improved the system by also lap-welding them steel rebar from the specimen to the steel L channels which were fixed to the base beam. In all the test presented in this thesis the improved version of the anchorage was used. The space between the two beams and the wall specimen was filled with high strength repair mortar, the same one used to repair the crushed concrete in the strengthened tests.

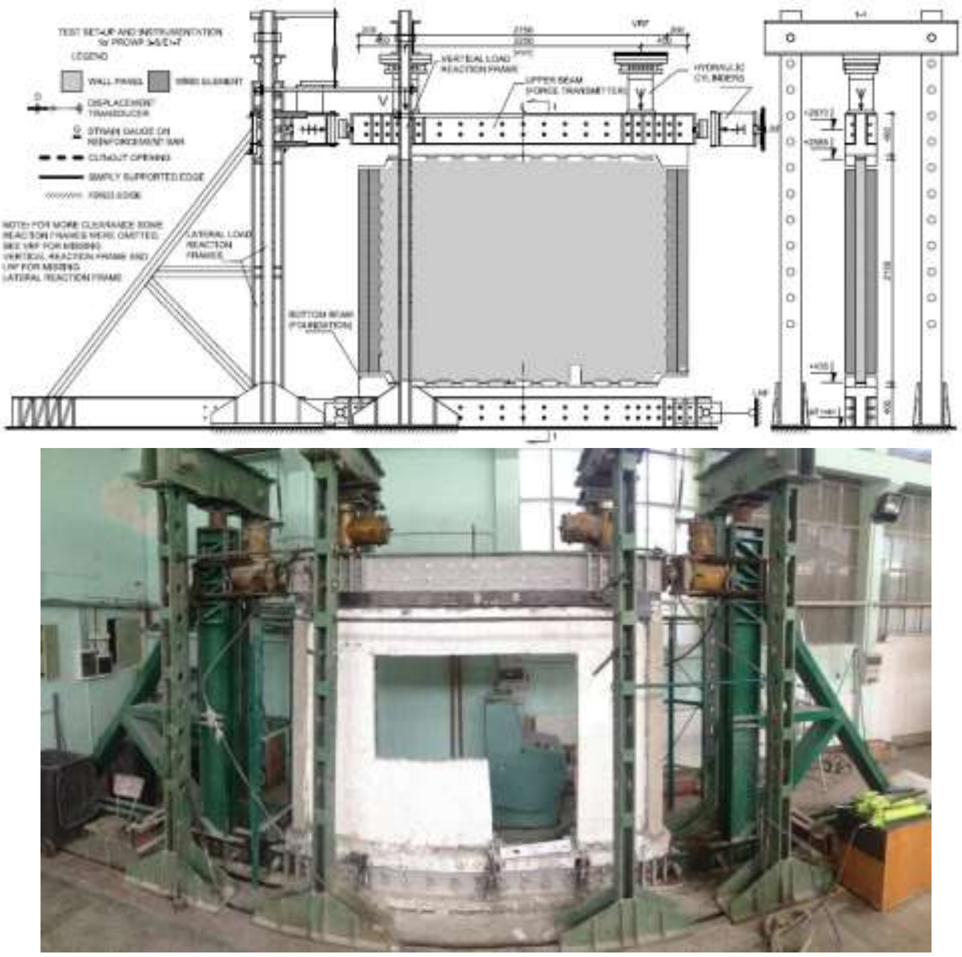


Figure 3-5 Experimental test set-up

6. Loading protocol

The test specimens were subjected to quasi-static in-plane reversed cyclic seismic loads and alternating gravitational loads. For the seismic (lateral) loads a reversed cyclic, displacement controlled increasing displacement amplitude was adopted. The displacement control was taken as the horizontal drift computed as the difference between the horizontal displacement measured at the top of the specimen and the horizontal displacement measured at the bottom of the specimen. This distance is approximately equal to 2150 mm so the 1% drift ratio corresponds to 21,5 mm, computed with the formula:

$$R\% = \frac{drift}{height} * 100$$

The displacement amplitudes were multiplies of the base value computed above, thus the increment (ΔR%) of 0.1% was applied. To replicate the real behaviour of the specimens, two hydraulic jacks were used at the top of the loading beam, in order to simulate the gravity loading conditions at the base of a five-storey building. These axial loads were comprised of a constant part and an alternating one. This was necessary to restrain the rocking rotation of the loaded specimens, this was required due to the fact that the base beam was not fixed to the strong floor, thus having no vertical tension reaction forces.

7. Boundary conditions

The experimental program exhibits a restrained rotation type of boundary condition for all the wall specimens. By adopting this boundary condition, we were able to stimulate the shear behaviour combating the flexural one, by reducing the shear spam. The test set-up is featuring an almost zero overall base moment through the hinged end connections. However, in case of specimens with openings, like the ones presented in this thesis, it is possible for some interior moments to develop. These moments are limited by the increasing axial loads

8. Instrumentation

The performance of the tested specimens was assessed by measuring two quantities, namely the displacements and the forces. For each specimen, a total of 10 displacements and three pressures were measured. The displacements were measured using linear potentiometers, they were fixed either to an independent steel frame or directly on the specimen. For the pressure measurement, piezo-resistive transducers were mounted on the hydraulic hoses. A total of three measurements were needed, one for the lateral loads and two for the axial loads.

4. RESULTS

In this chapter the results obtained by the nine cyclic wall tests are presented. All the recorded data was subjected to an intense screening and removing of any data acquisition "bugs", by eliminating all unsuccessful cycle attempts, removing all duplicated values, and by reassigning the correct sign to the force when changing of the loading direction. Basically, this smoothing operation was carried out by eliminating several data lines from the recorded data file, in order to have fluent and readable graphs and diagrams. After the analysis of the data we have obtained the following important aspects: the general observed behaviour and failure modes of the walls during the experimental tests, the force-drift ratio analysis, the dissipation of energy, the ductility of the elements, the stiffness degradation, weakening assessment, and computation of the final cracking pattern.

1. Failure details and behaviour of the reference specimens

The general behaviour aspects that were observed during the test of the reference, unstrengthen specimens, consists of a significant number of cracks, appearing in all regions of the element, concrete crushing at the base and corners of the openings and reinforcement yielding. For each specimen a detailed overview of the behaviour was created.

2. Failure details and behaviour of the FRP strengthened specimens

In general, the behaviour aspects, of the wall specimens that were tested, were as expected and similar to the unstrengthen reference specimens. The characteristics observed during the tests, consists of reopening of the existing cracks, from previous tests, FRP laminates debonding, anchorages failure, new cracks appearing, and concrete crushing. However, during these tests no FRP laminates has failed.

3. Force-drift ratio data analysis

In order to observe the performance of the tested specimens, several types of analysis on the force and drift ratio responses were necessary. The general seismic performance of the lateral load bearing members is best observed by the following types of analysis: hysteresis loops, envelope curves (cyclic envelope M2 and monotonic envelope M1), backbone envelopes T1 and T2

The data processing lead off with the hysteresis loops specific constituents being specified.

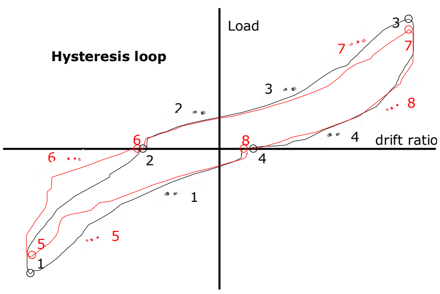


Figure 4-1 Cycles' loading points and subsections

In Figure 4.1 the hysteresis loop loading points and its subsections are presented, at a general x displacement level of a general two reversed cycles load-displacement response. Each cycle is made of four subsections: two opposite loading sections and two opposite unloading sections. Each of these sections are bounded by two points: peak loading point (corresponding to the target displacement) and the reloading point (corresponding to the curve-to-axis intersection). Note that point 8 coincide with point 0 for the next loop.

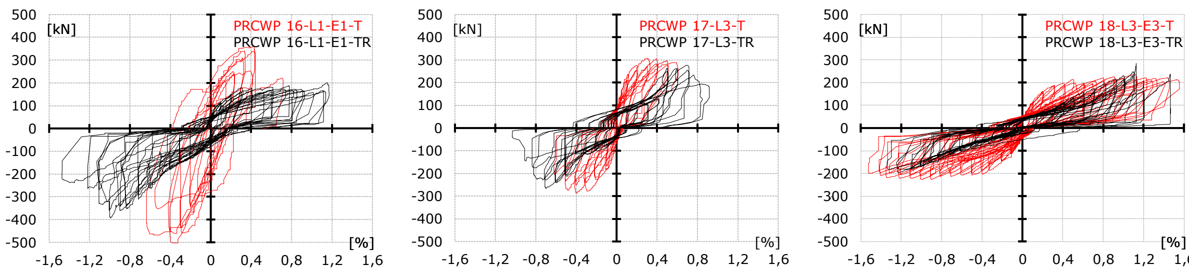


Figure 4-2 Comparison between hysteresis loops

When comparing the hysteresis loops of the reference specimens and the retrofitted ones presented in Figure 4.2, one can see that the drift ratio was increased for the retrofitted elements and the lateral load resistance was completely restored in case of PRCWP 18-L3-E3, and in case of 17-L3 it was restored to 93% compared to the reference one. In case of 16-L1-E1-TR, the lateral load of the reference specimen was not reached in case of the retrofitted one, this is due to the difference in behaviour caused by the malfunction of the pressure gauge.

The cyclic envelopes can be constructed for any type of cyclic response diagrams. Given the fact that the experimental test presented in this thesis are displacement (drift) controlled, the cyclic envelopes were obtained for the load drift response. For the construction of the cyclic envelope the peak loading points for each cycle, namely 1 and 3 for the first cycle and 5 and 7 for the second one, were interconnected through the increasing displacement points, thus obtaining one envelope for each cycle, Envelope C1 for the first cycle and envelope C2 for the second one. In order to obtain the average cyclic envelope referred to as Envelope M2, the arrhythmic mean between the peak loading points from C1 and C2 was computed, then by connecting these average loading points the Envelope M2 was constructed. For the construction of the monotonic envelope the arrhythmic mean of the absolute values for the peak loading points at each cycle, namely 1, 3, 5 and 7 was computed, then by connecting these average loading points the Envelope M1 was constructed.

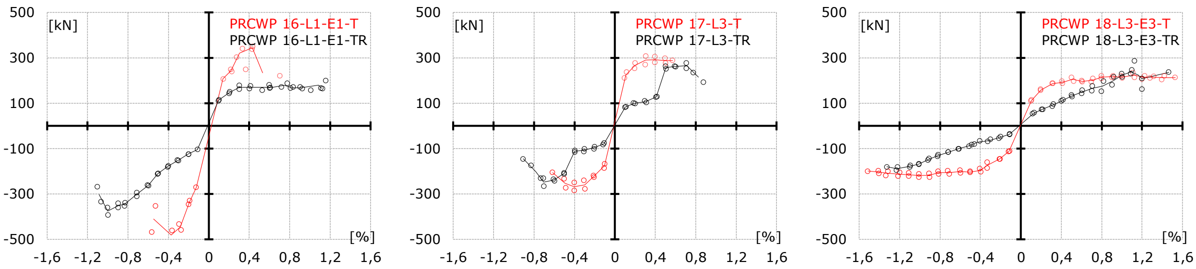


Figure 4-3 Cyclic envelopes M2 comparison

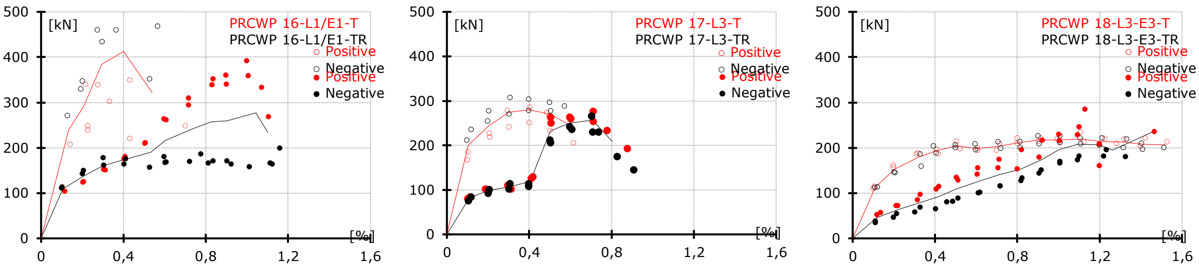


Figure 4-4 Cyclic envelopes M1 comparison

The author of this thesis adopted the tri-linear backbone envelopes model, due to the fact that it is the closest one to the real response, for each specimen two types of backbones envelopes are presented. Several other different backbone envelopes exist: bi-linear [23, 24] or tetra-linear [25]. The construction of the tri-linear envelope involves the definition of three displacement point.

For the first type of backbone envelope presented (Type 1) the three defining points are as follows:

(1) the cracking point - which is the point when the diagonal crack appeared,
(2) the peak loading point – is the point where the lateral force was highest
(3) the failure point – where the specimen lost at least 20% of its load bearing capacity

For the second type of backbone envelope presented (Type 2) the three defining points are as follows:

(1) the yield point - which is the point when later force is at 0.85 of the peak loading point,
(2) the peak loading point – is the point where the lateral force was highest
(3) the failure point – where the specimen lost at least 20% of its load bearing capacity

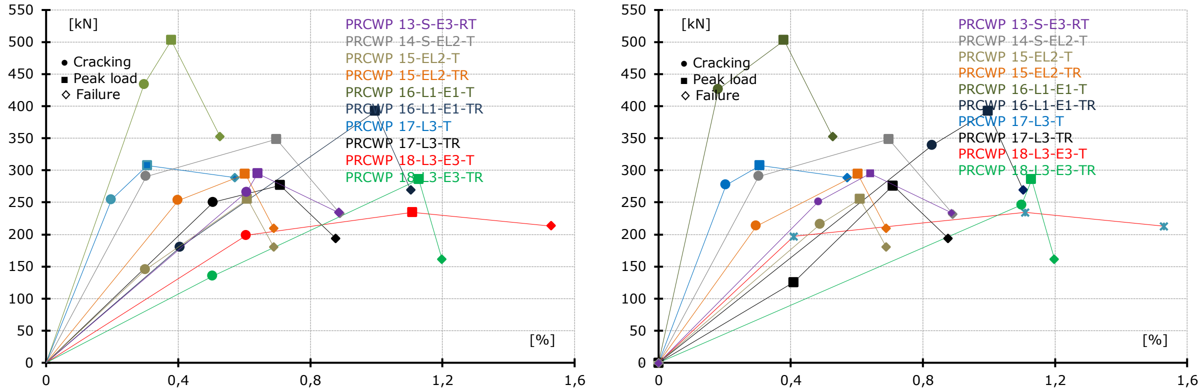
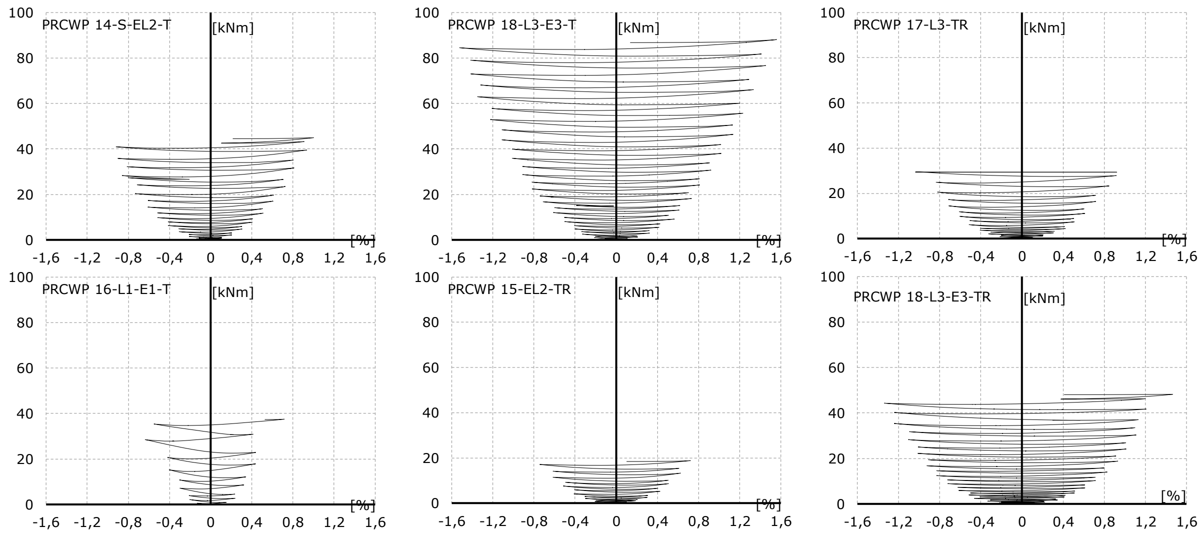


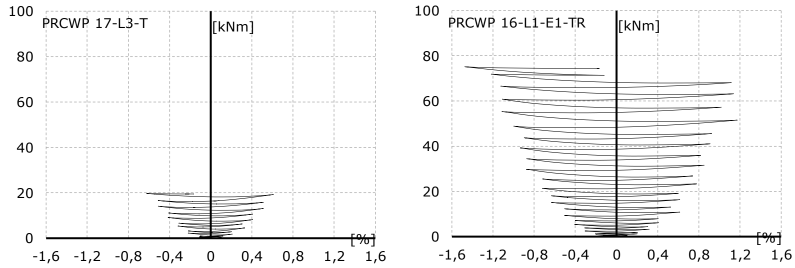
Figure 4-5 Backbone Envelope comparison: (left) Type 1; (right) Type 2

4. Energy dissipation analysis

Of particular significance to the behaviour of precast reinforced large panel buildings is the energy dissipation capacity. In the case of PRCWP, the energy dissipated is calculated by using the area under the force-displacement hysteresis loops as presented [171, 172, 59]. In this thesis, the amount of Energy Dissipated is noted with ED and the Cumulative Energy Dissipated is noted with CED. At each cycle, the ED and CED in both positive and negative direction were computed given us the results per half cycle. . In order to calculate the Cumulative Energy Dissipated (CED) the following incremental equation for the integration of the load displacement hysteresis loop was used:

$$CED_j = CED_{j-1} + (\delta_j - \delta_{j-1}) \times \left(\frac{V_j}{2} + \frac{V_{j-1}}{2}\right)$$





A definitive conclusion and observation cannot be drawn from the energy dissipated since some dissipated more energy than others without an obvious reason

Figure 4-6 Cumulative energy dissipated vs drift ratio

5. Strength and ductility analysis

The shear strength ranges between 234,5 kN and 502,5 kN. For the unstrengthen specimens the highest recorded value was obtained by 16-L1-E1, which had the smallest opening. Following this observation, it can be seen that the strength of the specimens decreases as the opening size increases. Given the fact that the specimens had slightly different classes of concrete, it seems that the presence of the parapet is not so influent in the maximum shear resistance of the walls. For the strengthen specimens the strength ranged from 276,5 kN to 392 kN with the maximum shear being obtained by the 16-L1-E1 TR the same specimen as in the unstrengthen tests. These results show that all specimens with similar openings had similar strength. In Figure 4.7 all the strength for all specimens is presented both as absolute value and as normalized one compared to the maximum strength obtained

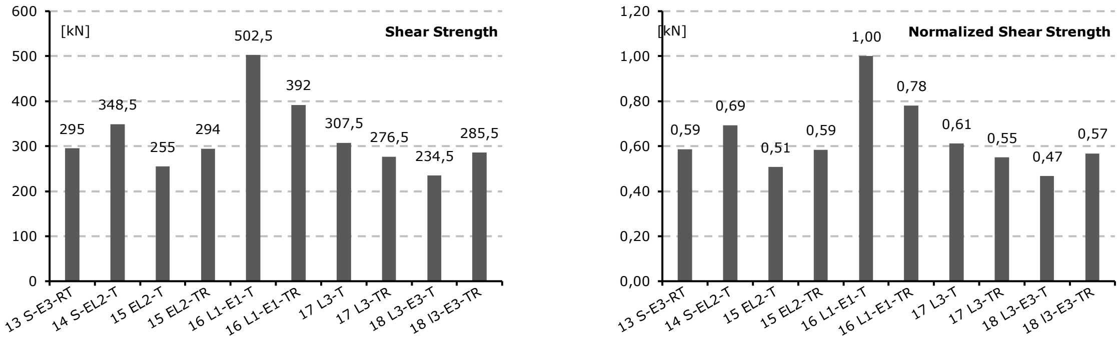


Figure 4-7 The shear strength for the tested specimens

When calculating ductility ratios, the definition of the yield deformation (displacement, rotation and curvature) often causes difficulty since the strength-deformation relation may not have a well-defined yield point. Different investigators used different definitions, according to Priestley [26] it is defined as the intersection of the initial tangent stiffness with the nominal strength, the intersection of the secant stiffness through first yield with nominal strength, and the displacement at first yield, while Park [27] said it is the yield displacement of the equivalent elastic-perfectly plastic system with reduced stiffness found as secant stiffness at 75% of the peak lateral load of the actual system. In this thesis, the μ0.85 method was used as in [11] which states that the ductility ($\mu=\Delta u/\Delta y$) is the ratio between the ultimate displacement (Δu), corresponding to the loss of 20% of the lateral force resistance of the specimen, and the displacement corresponding to 85% of the maxim lateral load on the ascending curve

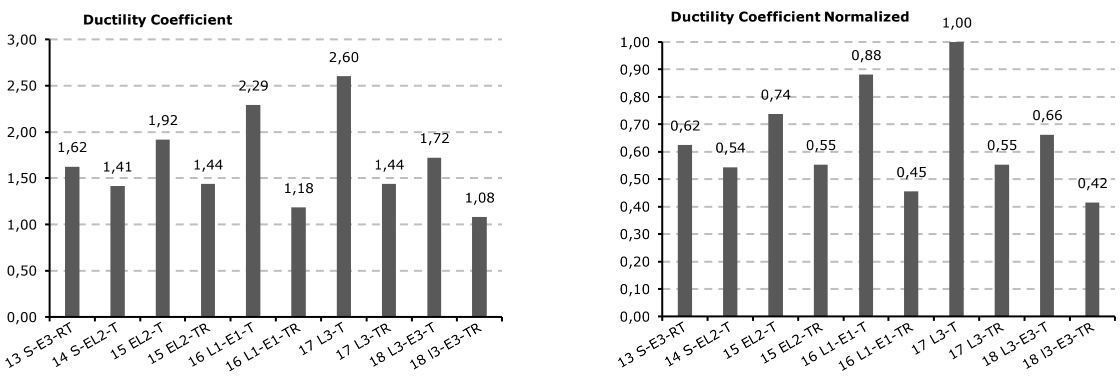


Figure 4-8 Ductility Coefficient

6. Displacement analysis

The main characteristics of the seismic response are defined on the backbone curves as the cracking point, the yield point, the peak loading point and the failure point. As the cracking point is assigned by the author own judgement it seemed unprofessional to use it in the displacement analysis.

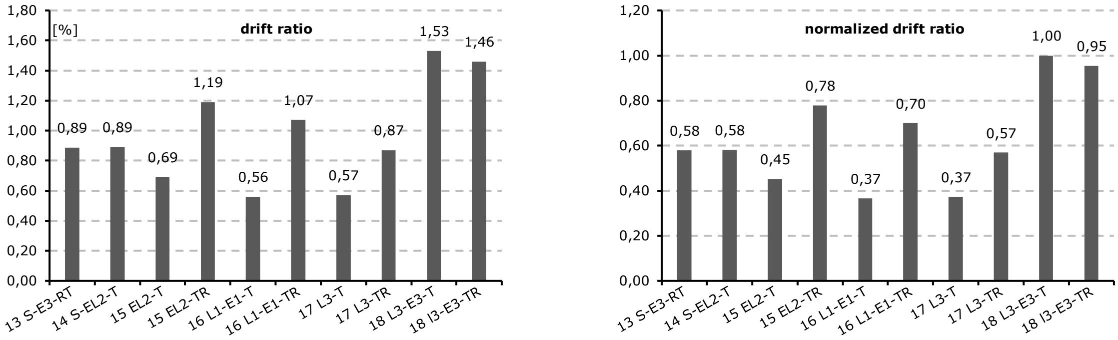


Figure 4-9 Drift ratio for all tested specimens

7. Stiffness analysis

The stiffness of a structural member is defined as its rigidity and is the ratio between the applied load and the resulting deflection. In order to plot the stiffness degradation, the monotonic envelope M1 was used, on the envelope the secant stiffness is the slope of a line connecting the origin point to a point on the curve as defined in [10]. The first point on the graph corresponds to 2,15 mm (0,1%) drift ratio and is called the initial stiffness. In Figure 4.10 the stiffness degradation is plotted for all elements.

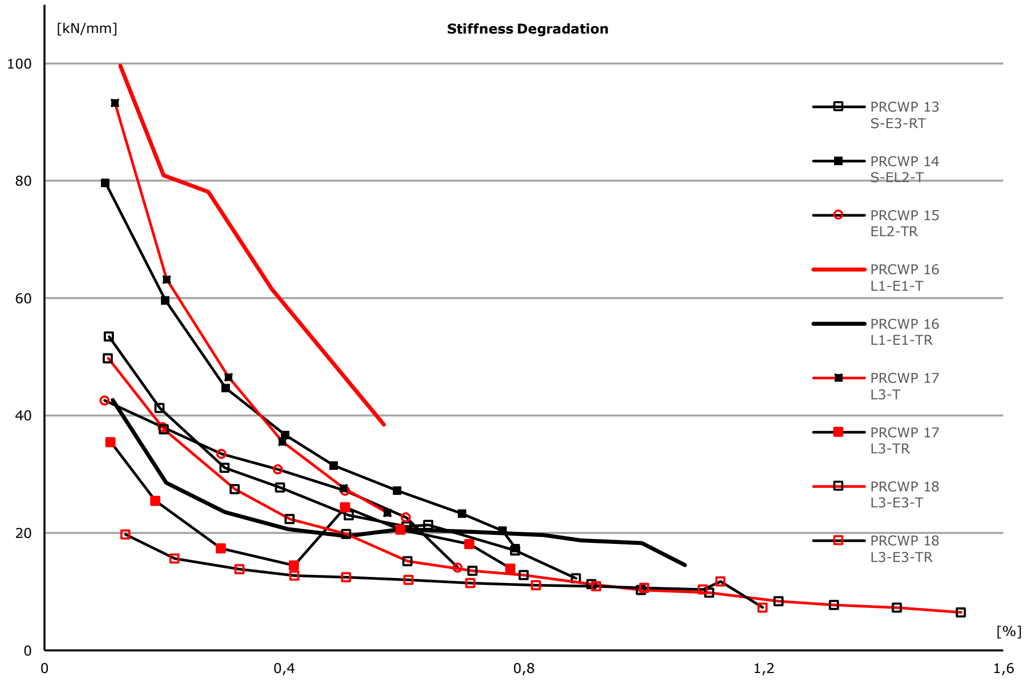


Figure 4-10 Stiffness degradation comparison

8. Theoretical study using Eurocode and CNR provisions

For the reference specimens the provisions in Eurocode 2 [21] section 6 were used to determine the shear resistance. The shear resistance is defined in terms of $V_{Rd,c}$, $V_{Rd,s}$, $V_{Rd,\ max}$
For the retrofitted specimens the provisions in CNR-DT 200 R1/2013 [28] were used to determine the shear resistance taking into account the retrofitting system.

In Eurocode 8 it is assumed that $V_{Rd,max} > V_{Rd,s}$ so the maximum shear resistance is limited by the yielding of the shear reinforcement. It can be observed that the predictions based on the shear resistance of the yielding of the shear reinforcement are highly conservative for the tested elements. For the case were the shear resistance is limited by the crushing of the compression strut it is observed that the results using the Eurocode provisions are overestimated. Although for walls with aspect ratio (α_s≤1,5), referred to as large lightly reinforced walls, designed according to DCM (medium ductility), no reduction factor is specified. However, using the reduction factor for high ductility class, 0.4 of the of the value determined in other regions than the critical base one, it can be seen from Figure 4.11 that this reduction factor seems to be very close to the reality as reported also by Postelnicu et al. [29] and Todut [11].

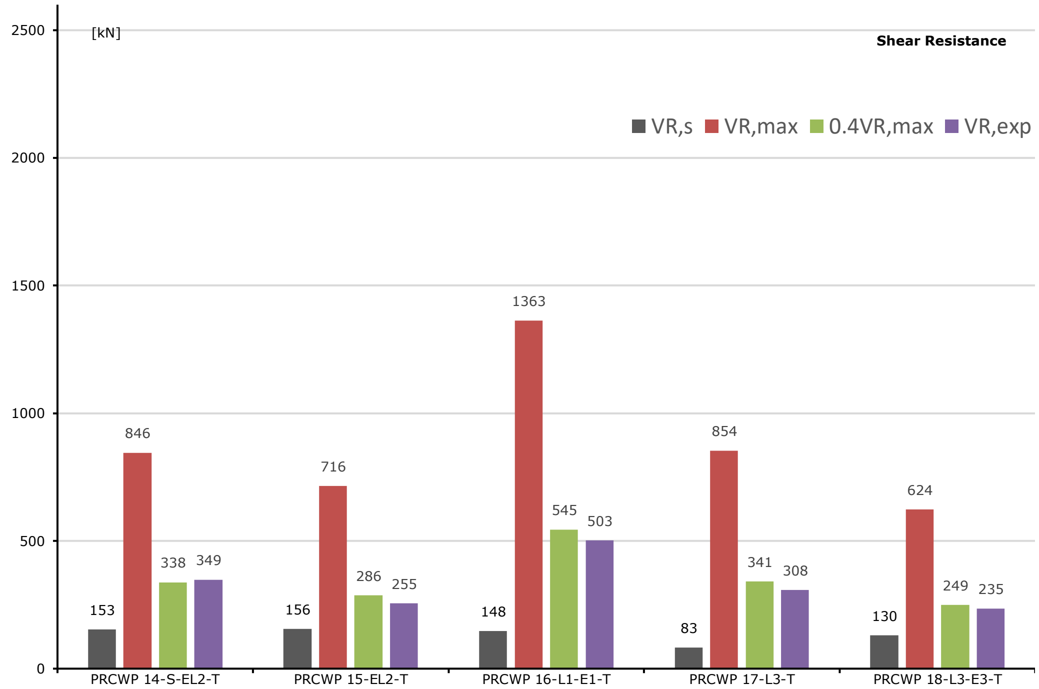


Figure 4-11 Shear resistance of reference specimens comparison

In Figure 4.12 the shear capacity of the element is computed based on CNR provisions. It can be seen that the shear capacity of the specimens with FRP is larger than that of the specimens resistance limited by the crushing of the compressive struts. However, the experimental test show that the element does not reach the computed shear force, the failure force being similar to initial test. By computing the shear capacity of the specimens with the angle of the compressed struts with respect to the member longitudinal axis of 45° Figure 4.13, as assumed in the codes, we can see that the overestimation of the shear capacity of the element limited by the crushing of the compressive struts is higher. Applying the factor of 0.4 we are close to the experimental results.

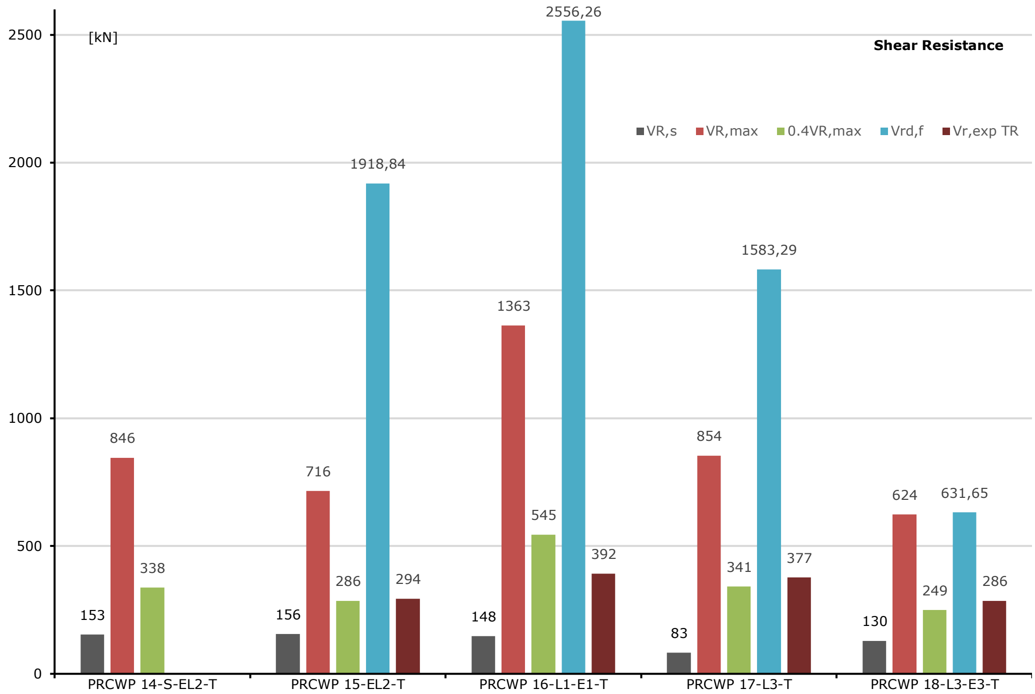


Figure 4-12 Shear resistance retrofitted specimens comparison

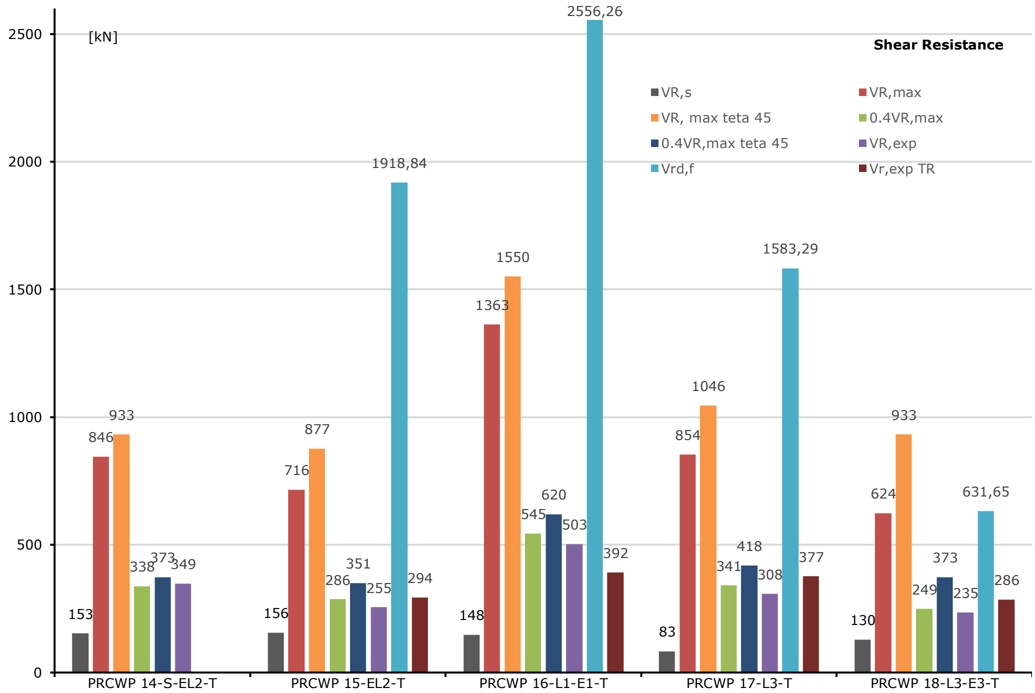


Figure 4-13 Shear resistance comparison between computed elements

9. Cracking Pattern

A very important behavioural aspect of the reinforced concrete elements, besides the measured response is the cracking patter. In concrete elements cracks appear very early if the element is subjected to increasing load/deflection. In this thesis, the author monitored only one face of the specimen, namely the front face, because the back face was occupied with monitoring equipment so it was not possible to have access to that face. It should be mentioned

that the cracking pattern was inspected also on the back face, after the testing was completed and the equipment was removed, and it was observed that the cracking pattern was very similar on both faces, most of the cracks from the front face being visible on the back face as well.
The expected behaviour of reinforced concrete walls subjected to in-plane lateral loading is either flexural or shear. Each behaviour is characterized by different cracking patterns, for the flexural behaviour one can expect horizontal cracks to appear, while the shear behaviour is characterized by inclined cracks. Still in some cases it is not unusual to observe cracks that start horizontally, typical for flexural behaviour, then changing their direction becoming inclined, typical for shear. This case is known as flexural-shear behaviour

5. CONCLUSIONS AND PERSONAL CONTRIBUTIONS

1. Conclusions

This thesis addresses the subject of retrofitting PRCWP subjected to seismic loads using CFRP materials. Within this chapter are reported the main conclusions that derived from this research, that was performed on six close to full scale precast reinforced concrete wall panels. The results obtained are equivalent to the real behaviour since the specimens are 1:1.2 real scaled elements.

Regarding the load bearing capacity of the specimens, it can be said that the retrofitting solutions adopted in this thesis were successful managing to restore the load bearing capacity of most of the specimens and in several cases to increase it.

Comparing the maximum displacements of the refence specimens and the retrofitted ones, one can conclude that the retrofitting procedure increases the displacement of the specimens, the displacement was increased by the retrofitting method in most cases.

The failure of the reference specimens was characterized by extensive cracking of all the areas, most common places for the cracks to develop were at the left and right pier and in the spandrel at the opening corners.

In case of the retrofitted specimens the main failure criteria was the debonding of the FRP system. In case of the EBR-CFRP combined with NSM-CFRP system the CFRP used for confinement debonded, whilst the NSM-CFRP did not show signs of weakness. For the EBR-CFRP laminates system debonding of the laminates and failure of the anchorage was observed, however, in most cases the CFRP debonded with the crushed concrete, so the adhesion between the retrofitting system and the concrete surface was good.

In regards to the energy dissipation capacity of the specimen, a clear and definitive conclusion cannot be drawn from these tests only, more tests are required for a clearer observation of the results.

Concerning the stiffness reduction, by comparing specimens with similar concrete quality, it was observed that the stiffness is influenced by the opening, namely, the bigger the opening the smaller the stiffness.

Using EC2 expressions for the shear strength evaluation denoted that the shear bearing capacity that can be sustained by the yielding reinforcement is conservative compared to the experimental test, whilst the maximum shear resistance of the member limited by the crushing of the compression strut is highly overestimated.

2. Personal contributions

Improving the existing experimental program by:

- organizing the cut-outs for the solid elements and casting of 4 new test specimens with specific reinforcement placements and concrete quality;
- testing six near-full scale specimens all with different openings, conducting 10 tests, adding to more than 38 hours of testing time;
- testing and designing strategies for the used retrofitting systems of EBR-CFRP combined with NSM-CFRP and EBR-CFRP laminates;
- different materials used for the retrofitting of the specimens;
- adding four stabilisers to the test set-up in order to avoid the out of plane movement of the specimens.

Processing and analysing the recorded data from the instrumentation:

- more than 100 000 raw data rows and over 39 000 “clean” data rows processed;
- more than 250 diagrams generated;
- production of load vs displacement hysteresis loops;
- production of load vs displacement envelops (cyclic envelops, monotonic envelops, two types of backbone envelops);
- energy dissipation analysis (cumulative energy dissipation, cumulative energy dissipation per half cycle, cumulative energy dissipation per cycle, energy dissipation per cycle);
- strength, ductility and displacement analysis;
- stiffness analysis (initial stiffness and stiffness degradation);
- failure details observations and behaviour mode examination;
- shear strength evaluation using design code provisions;
- computation of cracking pattern.

Investigation of current similar research such as: precast shear elements, reinforced concrete structures, reinforced concrete walls subjected to seismic loads, retrofitting of concrete elements using FRPs, deign code analysis. The database contains 177 refences.

Synthesis of the results and further directions of research.

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6. FURTHER RESEARCH RECOMMENDATIONS

The current research is based on an ongoing investigation of the behaviour of the PRCWPs and the strengthening and retrofitting procedures for these elements. Given the fact that in this research a limited number of experimental specimens were tested it is obvious the need for further investigation on experimental specimens. This experimental investigation was conducted on nearly full-scale precast reinforced concrete walls subjected to shear, the walls had initial and/or cut-out openings. A large number of experimental test have to be conducted in order to obtain all the answerers in the current research and to solidify the concluded observations.

Bibliography

[1] "Romanian national Statistics Institutee," Governmental, [Online]. Available: http://www.insse.ro/cms/. [Accessed 01 February 2015].

[2] A. Ismail-Zadeh, L. Matenco, M. Radulian, S. Cloetingh and G. Panza, "Geodynamics and intermediate-depth seismicity in Vrancea (the south-eastern Carpathians): Current state-of-the art," *Tectonophysics,* Vols. 530-531, pp. 50-79, 2012.

[3] G. Mărmureanu, A. Mărmureanu, E. F. Manea, D. Toma-Danila and V. Mihnea, "Can we still use classic seismic hazard analysis for strong and deep Vrancea earthquakes?," *ROMANIAN JOURNAL OF PHYSICS,* vol. 61, no. 3-4, pp. 728-738, 2016.

[4] G. Pásztor and P. László, "Romanian Housing Problems: Past and Present," *Studia Universitatis Babes-Bolyai-Sociologia,* vol. 1, pp. 79-100, 2009.

[5] National Institute of Statistics, "Census of Population and Dwellings 2011," vol IV, Buildings, Dwellings, Households, 2011.

[6] I. Demeter, "Short history of large panel structures in Romania," *Scientific Bulletin of Politehnica University of Timisoara,* vol. 51, no. 65, pp. 87-94, 2005.

[7] National Institute of Statistics, "Census of Population and Dwellings 2011," Vol. I: stable population (rezidents) - demographic structure, 2011.

[8] "eurostat.eu," European Uninion, [Online]. Available: https://europa.eu/european-union/about-eu/figures/living_en#tab-1-3. [Accessed 03 04 2016].

[9] "cia.gov," Central Inteligence Agency, [Online]. Available: https://www.cia.gov/library/publications/the-world-factbook/rankorder/2119rank.html. [Accessed 02 03 2016].

[10] I. Demeter, Seismic Retrofit of precast RC walls by externally bonded CFRP composites, Phd. Thesis: Politehnica University of Timisoara, 2011.

[11] C. Todut, Seismic strenghtening of precast RC wall panels using FRP composites, Phd. Thesis: Politehnica University of Timisoara, 2015.

[12] M. A. Masuelli, Introduction of Fibre-Reinforced Polymers – Polymers and Composites: Concepts, Properties and Processes, INTECH Open Access Publisher, 2013.

[13] P. Soroushian, S. Ravanbakhsh and M. A. Nagi, "Laboratory Evaluation and Field Application of Concrete Reinforcement with Aramid Fiber-Reinforced Polymer Bars," *Materials Journal ,* vol. 99, no. 6, pp. 584-590, 2002.

[14] V. Fiore, T. Scalici, G. Di Bella and A. Valenza, "A review on basalt fibre and its composites," *Composites Part B: Engineering,* vol. 74, pp. 74-94, 2015.

[15] F. M. Kogan and O. V. Nikitina, "Kogan, F. M., & Nikitina, O. V. (1994). Solubility of chrysotile asbestos and basalt fibers in relation to their fibrogenic and carcinogenic action.," *Environmental health perspectives,* vol. 102, no. 5, p. 205, 1994.

[16] E. E. McConnell, O. Kamstrup, R. Musselman, T. W. Hesterberg, J. Chevalier, W. C. Miiller and P. Thevenaz, "Chronic inhalation study of size-separated rock and slag wool insulation fibers in Fischer 344/N rats," *Inhalation toxicology,* vol. 6, no. 6, pp. 571-614, 1994.

[17] V. Fiore, G. Di Bella and A. Valenza, "Glass–basalt/epoxy hybrid composites for marine applications," *Materials & Design,* vol. 32, no. 4, pp. 2091-2099, 2011.

[18] CEN, "Part 3: Compressive strength of tested specimens," in *EN 12390-3:2001 Testing of hardened concrete*, Brusseles, Belgium, European Comittee for Standardization (CEN), 2001.

[19] CEN, "EN ISO 15630-1:2002 Steel for the reinforcement and prestressing of concrete," in *Test methods - Part 1: Reinforcing bars, wire rod and wire*, Brusseles, Belgium, European Comittee for Standardization (CEN), 2002.

[20] CEN, "EN 10002-1:2001 Tensile Testing Metallic Materials," in *Tensile Testing Metallic Materials*, Brusseles, Belgium, European Comittee for Standardization (CEN), 2001.

[21] CEN/TC250, "EN 1992-1-1:2004:E Eurocode 2: Design of concrete structures," in *Part 1-1: General rules and rules for buildings*, Brusseles, Belgium, European Committee for Standardization (CEN), 2004.

[22] "www.mapei.com," MAPEI, [Online]. Available: http://www.mapei.com/public/COM/products/1066_mapegrout%20easy%20flow%20gf_gb.pdf. [Accessed 28 10 2016].

[23] M. Mansour, J. Y. Lee and T. T. Hsu, "Cyclic stress-strain curves of concrete and steel bars in membrane elements," *Journal of Structural Engineering,* vol. 127, no. 12, pp. 1402-1411, 2001.

[24] A. S. Elnashai and R. Pinho, "Repair and retrofitting of RC walls using selective techniques," *Journal of Earthquake Engineering,* vol. 2, no. 04, pp. 525-568, 1998.

[25] M. Warashina, S. Kono, M. Sakashita and H. Tanaka, "Shear behavior of multi-story RC structural walls with eccentric openings," in *The 14th World Conference on Earthquake Engineering*, Beijing, China, 2008.

[26] M. J. N. Priestley, "Performance based seismic design," *Bulletin of the New Zealand society for earthquake engineering,* vol. 33, no. 3, pp. 325-346, 2000.

[27] R. Park, "Ductility evaluation from laboratory and analytical testing," in *Proceedings of the 9th world conference on earthquake engineering*, Tokyo-Kyoto, Japan, 1988.

[28] CNR, DT 200 R1/2013 Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures, Rome: Italian National Research Council, 2014.

[29] I. Damian, E. Morariu, B. Buzaianu, T. Postelnicu and D. Zamfirescu, in Postelnicu T, editor.; Proiectarea structurilor de beton armat in zone seismice (Design of reinforced concrete structures placed in semismic areas - in Romanian), Bucharest: MarLink, 2012.