a

Feasibility of retrofitting solutions for an old RC wall-frame building in Lisbon

Viabilidade de soluções de reforço para um edifício antigo de BA em Lisboa

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Abstract

This work addresses the problem of strengthening one of the most vulnerable class of existing reinforced concrete buildings in Lisbon, namely Reinforced Concrete (RC) wall-frame buildings, designed and built under old codes and engineering practices.

The seismic performance assessment of an old RC dual wall-frame structure is performed aiming to identify the deficiencies and failure modes of this building typology and to propose efficient retrofitting solutions. Shear failure in members can occur in old RC buildings designed without considering adequately the effect of horizontal actions or in buildings with low concrete strength or without sufficient transverse reinforcement. This failure mode impairs the deformation capacity of the structure and, hence, has an important influence on the seismic performance of the buildings. The effectiveness of different strengthening strategies to improve the seismic response of the structure is investigated, at the same time seeking for a solution with a low economic and structural impact.

Keywords: RC building / RC wall / Seismic retrofit / Steel bracing / FRP

Resumo

Este trabalho aborda o problema do reforço de uma das classes mais vulneráveis de edifícios existentes em betão armado em Lisboa, a saber, os edifícios em pórtico de betão armado (BA), dimensionados e construídos sob códigos e práticas antigas.

A avaliação do desempenho sísmico de uma estrutura antiga mista pórtico-parede de BA é realizada com o objetivo de identificar as deficiências e os modos de colapso desta tipologia de edifício e propor soluções eficientes de reforço. O colapso por corte dos elementos estruturais é esperado que ocorra em edifícios antigos de BA uma vez que: foram dimensionados sem considerar adequadamente o efeito de ações horizontais, foram contruídos com betão de baixa resistência e com insuficiente armadura transversal. Este modo de colapso, por corte, reduz a capacidade de deformação e a ductilidade da estrutura e, portanto, tem uma influência importante no desempenho sísmico dos edifícios. A eficácia de diferentes estratégias de reforço para melhorar a resposta sísmica da estrutura é investigada, procurando, simultaneamente, uma solução com baixo impacto económico e estrutural.

Palavras-chave: Edifício em BA / Parede em BA / Reforço sísmico / / Contraventamentos metálicos / FRP

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1 Introduction

In many cities, reinforced concrete (RC) frame and dual wall-frame building structures represent an important portion of the building stock. This is the case in Lisbon, Portugal, where many of these buildings were not seismically designed or were designed based on early seismic codes (pre-1980). Therefore, they have structural deficiencies that are reflected in high seismic vulnerability. Furthermore, many cities, such as Lisbon, are characterised by medium to high seismic hazard, which, combined with a large exposure of vulnerable buildings, results in high seismic risk. The vulnerability of this type of buildings was evident during the latest earthquake in Mexico, where most of the collapsed RC buildings were old (pre-1985) non-ductile structures [1]. In particular, RC walls designed to withstand low seismic forces (if any) are more vulnerable to brittle shear failure and tend to cause earlier collapse of the building.

Assessment of existing buildings is required to identify structural deficiencies at local and global levels. Then, the most adequate retrofitting strategy, or a combination of them, should be selected to improve the performance of the building [2,3]. Guidelines such as the fib Bulletin on Seismic Assessment and Retrofit of Reinforced Concrete Buildings [2] and the ASCE 41-17 Seismic Evaluation and Retrofit of Existing Buildings [4] provide guidance on the cases where each measure is most effective. Each technique has its own advantages and drawbacks and is selected based, primarily, on technical criteria. The choice depends on: (i) the locally available materials and technologies, (ii) cost considerations, (iii) the disruption of use it entails and the duration of the works, (iv) architectural, functional and aesthetic considerations or restrictions, etc. There are two main objectives in seismic retrofitting, mainly to reduce demand or to increase capacity, and three main properties to examine: strength, stiffness and deformation capacity.

In Di Ludovico *et al*. (2017) [5], a careful analysis and data collection of the reconstruction process after the L'Aquila earthquake was performed, showing that for RC buildings, the application of FRP composite systems was the most commonly used technique for local strengthening interventions, followed by RC jacketing of members and the application of steel bracings. Furthermore, in most buildings more than one strengthening technique was used to improve the seismic capacity of the structure; for instance the use of steel bracings to increase the lateral stiffness and strength of an existing building may imply the need for beam-column joint strengthening in order to ensure adequate resistance to the localized actions induced by the bracing system.

In the present study, the effectiveness of two strengthening methods involving diagonal X steel braces and the application of Fibre Reinforced Polymers (FRPs) are evaluated in a case study building, representative of a RC building typology in Lisbon. The evaluation of the seismic vulnerability of the structure is based on the performance-based assessment procedures and on the structural safety conditions prescribed by Part 3 of Eurocode 8 (EC8-3) [6] for the case of reinforced concrete buildings.

2 Case study building

For this work, a building belonging to the typology of RC wall-frame buildings built within 1960 and 1980 in Lisbon, is selected (Figure 1). The building is an eight-storey (ground floor plus seven storeys above ground) structure, characterized by an open ground storey and infills in the upper storeys. The structure is symmetric with respect to the Y direction and moderately asymmetric along the X direction. It features three main RC frames in the longitudinal direction (X) and two stiff RC cores that provide an acceptable lateral stiffness in the transverse (Y) direction. The building, which was designed according to the pre-70s building codes has non-ductile behaviour and insufficient detailing, e.g. (i) smooth longitudinal reinforcing bars; (ii) columns and RC walls with low confinement and tie reinforcement (lower than 1%); (iii) beams framing eccentrically to the columns.

The building was designed according to the old Portuguese codes for reinforced concrete and for earthquake resistant design. The first national standard to explicitly specify seismic resistance was the Construction Safety Standard against Earthquakes (RSCCS), introduced in 1958 [7]. Seismic loads were defined as horizontal static forces equivalent to the inertia forces induced by the earthquake and were obtained by multiplying the mass of the structure by a seismic coefficient. The base shear coefficient for the case study building is equal to 0.1, corresponding to seismic Zone A of the old 1958 RSCCS code. Thus, the design base shear can be estimated as 3552.8 kN, which corresponds to 10% of the total weight of the building, which amounts to 35527 kN.

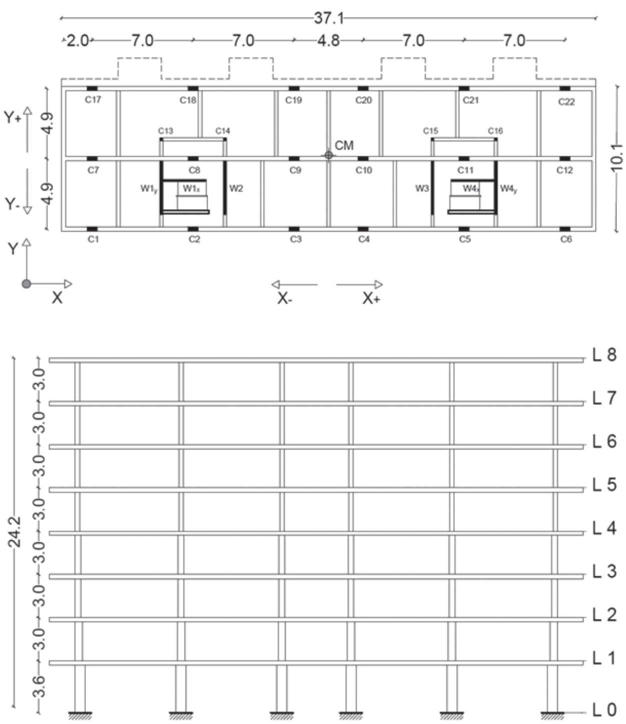


Figure 1 Case study building, dimensions in [m]: structural plan layout and cross-section variation along the height

The safety checks of the structural members were purely force-based, consisting in the comparison of internal force demands with the members' resistance. In the design of the RC frames, the structural actions (dead load, imposed loads, shrinkage, temperature variations and seismic load) were combined according to the code, in order to obtain the most unfavourable loading scenario. Thus, for the case study building (Figure 1) the horizontal forces are resisted in the longitudinal X-direction by the RC frames and in the transverse Y-direction by the RC walls of the staircase and lift core.

The dimensions and reinforcement properties of the two T-shaped RC walls (Wall 1 and Wall 4), in the X and Y direction, are shown in Table 1, where ρ_{long} is the longitudinal reinforcement ratio and ρ_h is the horizontal reinforcement ratio. The longitudinal steel reinforcement ratio for columns ranged between 0.2% and 4.3% of the gross section area, while the transverse reinforcement consisted of smooth steel stirrups, 6 mm in diameter, equally spaced at 20 cm along the entire member length (non-seismically detailed transverse reinforcement).

2.1 Structural modelling

The first step of the seismic performance assessment procedure consists of the development of a three-dimensional structural model. The RC wall-frame building is modelled in OpenSees [8].

Table 1 Summary of T-shaped wall's properties

Wall		Dimension (m)	ρ_{long} (%)	ρ_h (%)
W1y - W4y	L 0 - 1	0.25 x 4.00	0.65	0.16
	L 1 - 2	0.25 x 4.00	0.35	0.16
	L 2 - 8	0.25 x 4.00	0.27	0.16
W1x - W4x	L 0 - 1	0.15 x 3.00	0.18	0.17
	L 1 - 8	0.15 x 3.00	0.18	0.17

Force-based beam–column elements and a fibre modelling approach are employed for beams, columns and RC walls. In the model, only flexural nonlinear behaviour is considered. he main features of the building are replicated in the model, such as the infills and the smooth reinforcing bars, typical of RC structures built in the 60s. As for the smooth rebars, a simple approach is considered, involving the reduction of the Young modulus and the maximum strength of the reinforcing steel to simulate the increase of the member flexibility due to strain penetration effects. Based on the results of a previous work [9], it is decided to reduce the Young modulus of the rebars by 40% and the maximum strength of the rebars of the RC walls at the ground storey level by 30%.

2.2 Seismic safety assessment

The assessment is based on nonlinear static analysis. The target displacements of the structure are determined with the N2 method, the nonlinear static procedure prescribed in Part 3 of Eurocode 8 (EC8-3) [6]. To define the seismic action, the elastic response spectrum for soil type B defined in Part 1 of Eurocode 8 (EC8-1) [10] is adopted. The Significant Damage (SD) Limit State (LS) is considered for the seismic assessment. Thus, according to the Portuguese National Annex of EC8-1, the seismic hazard is represented by a ground motion with a return period of 475 years and a PGA of 0.153 g for seismic action Type 1, which corresponds to the most severe seismic action for the case study building.

The seismic performance evaluation is conducted based on the assessment procedures prescribed in EC8-3 which, in simple terms, consist of comparing chord rotations and shear demands with the values of ultimate chord rotation and shear strength defined in the European code. The results of a previous study [11] indicated that the most severe failure mode of the building corresponds to the shear failure (brittle failure mechanism) of the RC walls in both X and Y directions, while the columns exhibit reasonable flexural behaviour, developing a stable flexural response. The shear resistance of RC walls and columns is calculated by means of shear resistance to web crushing, $V_{Rd,max}$, and shear resistance as controlled by the stirrups, $V_{Rd,s}$. The minimum of the two values are adopted in the seismic performance assessment process. In Figure 2, the shear strengths obtained with the expressions provided by EC8-3 and EC2-1 and ATC-40 [12] are compared, showing very similar results. The shear failure of the wall in the X direction (Figure 2, left) is reached for a very low value of PGA. As stated before, this result is due to the very low amount of horizontal reinforcement (see Table 1).

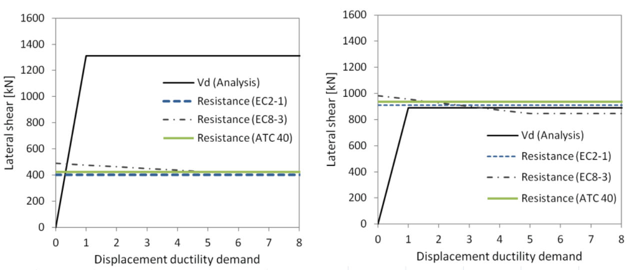


Figure 2 Comparison of shear demand and shear strength (obtained by different approaches) for the RC walls in the X direction (left) and Y direction (right)

3 Definition of retrofitting strategies

The effectiveness of two retrofitting strategies are evaluated with the aim of improving the performance of the RC walls in the X direction, which, as shown in Section 2.2, are more vulnerable to brittle shear failure and tend to cause earlier collapse of the building. This improvement of behaviour may be achieved by adopting one of the following approaches or strategies, or even combining them: (i) by reducing the seismic demands on members, (ii) by increasing the member capacities.

The deformation capacity and shear strength of individual members may be significantly upgraded through FRP-wrapping, without introducing a relevant change to their stiffness. This solution is investigated in Section 3.1. Reduction of seismic demand on the walls through retrofitting may be achieved by increasing the lateral stiffness. The lateral stiffness can be increased by adding a new lateral load resisting system to take almost all the full seismic demand, *e.g.*, steel bracing or new concrete walls. In this work, the effectiveness of applying steel braces at the ground storey level to reduce the shear demand on the RC walls is investigated (Section 3.2). This partial strengthening, as opposed to a global one, has the double aim to reduce the cost of intervention and allow the continued usage of the building during the retrofitting work.

3.1 Retrofitting using FRP

Externally bonded Fibre Reinforced Polymers (FRPs) are used in seismic retrofitting in order to enhance or improve: (i) the deformation capacity of flexural plastic hinges, (ii) deficient lap splices, (iii) shear resistance. To improve the shear capacity of brittle components, the FRP overlay should be applied with the fibres mainly in the direction in which enhancement of shear strength is pursued. Unlike beams, columns and walls are subjected to a constant shear force within each storey. Hence, if shear strengthening is needed, it should be uniform throughout the height of the vertical element in a storey. Moreover, as the shear demand alternates between opposite values, the main direction of the FRP should be horizontal [3].

According to Annex A of EC8-3, the total capacity, as controlled by the stirrups and the FRP, is evaluated as the sum of the contribution from the existing concrete member and the contribution from the FRP. The contribution of FRP to the shear capacity for full wrapping with FRP or side bonded FRP strips may be calculated, respectively, with Equations (1) and (2), which correspond to Equations A.22 and A.23 of EC8-3, respectively:

$$V_{Rd,f} = 0.9 \cdot d \cdot f_{fdd,e} \cdot 2 \cdot t_f \cdot (\cot\theta + \cot\beta) \cdot \frac{W_f}{S_f} \tag{1}$$

$$V_{Rd,f} = 0.9 \cdot d \cdot f_{fdd,e} \cdot 2 \cdot t_f \cdot \frac{\sin\beta}{\sin\theta} \cdot \frac{W_f}{S_f} \tag{2}$$

where d is the effective cross sectional depth, θ is the strut inclination angle and β is the angle between the strong fibre direction in the FRP strip (or sheet) and the axis of the member, t_f is the thickness, w_f is the width and s_f is the spacing of the strip (or sheet), $f_{fdd,e}$ is the design FRP effective debonding strength, which is different for fully wrapped or side bonded FRP (Equations A.24 and A.30 of EC8-3, respectively).

Table 2 Shear contribution of FRPs

	n_f	$V_{Rd,f}$ (kN)
Fully wrapped	1	986
	2	1697
Side bonded	1	746
	2	994
	5	1496

In this work, carbon FRP sheets are chosen. They are characterised by an elastic modulus of 240 GPa, thickness, $\bar{t}_f$, of 0.167 mm and ultimate strength of 3800 MPa. In Table 2 the values of the FRP contribution to the shear resistance, $\bar{V}_{Rd,f}$, are reported for different number of layers, n_f. Considering the shear demand in the X direction, as shown in Figure 2, to enhance the shear resistance of the wall it would be necessary to fully wrap the wall with two layers of FRPs. Alternatively, where required by architectural constraints, the U-shaped or side-bonded striped FRPs could be applied in more than two layers, for a maximum number of five layers [3].

3.2 Retrofitting with steel braces

The application of steel braces in selected bays of an existing RC building is effective for global strengthening, provided that a reliable, well detailed and technically sound connection between the steel elements and the existing concrete members is ensured [13,14]. Architectural constraints related to strengthening schemes can be addressed through alternative choices of bays to be braced. A strengthening solution could be achieved by using X, V or inverted V bracing. Alternative retrofitting methods of non-ductile RC frames include the use of eccentric steel braces with vertical shear links as energy dissipation elements. Among these alternatives, diagonal X bracing is the most common technique, providing a considerable increase in terms of lateral strength and stiffness of the building [15]. Nevertheless, the application of X bracing in an existing building can lead to possible side effects, particularly on columns attached to the bracing system [16].

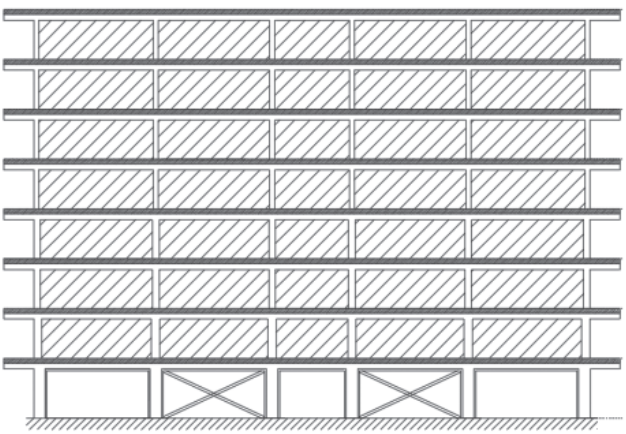


Figure 3 Retrofitting scheme

A strengthening intervention using concentric X-diagonal steel braces is proposed herein in order to make the analysed RC frame compliant with the performance requirements of the LS of SD, under the corresponding seismic action defined in the National Annex of EC8-3. Figure 3 shows a possible layout for the bracing system. The diagonals are composed by hot-rolled, circular hollow section (CHS) steel profiles, directly connected to the beam-column nodes of the bay of the RC frame. This connection is considered to behave as a "nominally pinned joint" as defined in Part 1-8 of Eurocode 3 [17], *i.e.* it can transmit the internal axial forces without developing significant moments. As no specific rules for the design of hybrid RC-steel systems exist in EC8-1, the provisions of the latter concerning steel frames with concentric braces were taken as a reference for the design of the new steel braces. Regarding the lower and upper limits for the non-dimensional slenderness $\bar{\lambda}$ of the braces, clause 6.7.3 (1) of EC8-1 states that $1.3 \leq \bar{\lambda} \leq 2.0$, for buildings with more than two storeys. The lower limit is defined to avoid overloading the frame's columns in the pre-buckling stage (*i.e.*, when compressed and tensioned diagonals are active).

The braces' sections (listed in Table 3) were selected after a preliminary analysis with the objective to reduce the shear demand on the RC walls in the X direction, keeping these RC elements in the elastic region, as it will be shown in Section 3.3. For architectural reasons, no braces were applied in the Y direction. Moreover, a previous study [11] indicated that the building has an acceptable seismic performance in that direction.

Table 3 Section profile of steel braces

Bay's dimension [m]	Brace type	$\bar{\lambda}$	N_{cr} [kN]	$N_{pl,Rd}$ [kN]	$N_{b,Rd}$ [kN]
7x3.6	CHS 219.1x5.9	1.30	716.4	983.7	681.3

N_{cr} – Euler's critical load; $N_{pl,Rd}$ – yield resistance of the gross section; $N_{b,Rd}$ – buckling load

3.2.1 Modelling options related to the steel braces

The modelling of the steel braces was based on the conclusions of the studies conducted by [18]. The inelastic force-based beam-column element available in OpenSees was used for simulating the hysteretic behaviour of the steel braces. This element accounts for large displacements by embedding the basic system in a corotational framework. A force-based finite element formulation with five integration sections was used to implement this model.

The connections between the steel braces and the RC frame are modelled as pinned regarding the out-of-plane rotation. In this approach, a zero-length element is defined in OpenSees. This element is defined by two coincident nodes that are connected by a linear elastic spring. The rigidity of the connection (*i.e.* the gusset plate) is modelled using rigid elastic elements as proposed by Hsiao *et al*. (2012) [19]. Concerning the constitutive law defining the cyclic behaviour of the steel material, the Menegotto and Pinto (1973) [20] model was employed, combined with the isotropic hardening rules proposed by Filippou *et al*. (1983) [21], with the following mechanical properties (mean values): (i) Modulus of elasticity (initial elastic stiffness): E_s = 210 GPa; (ii) Yield stress (mean value): f_{ym} = 343.75 MPa; (iii) Strain hardening parameter: μ = 0.005; (iv) Steel specific weight: Y_s = 7850 kg/m^3.

While the above-referred modelling aspects are consensual among authors, others like the number of finite elements per individual brace and the brace's initial camber, Δ0, are not. It is however widely accepted that braces should be divided at least in two FE's, that are offset (initial camber) at mid-length of the brace, as to trigger flexural buckling. Based on the results of a parametric study [18], in this work, the braces are modelled with four force-based elements, allowing the consideration of an initial geometrical imperfection with values within the range of 0.1% of the brace length, and 5 integration points per element.

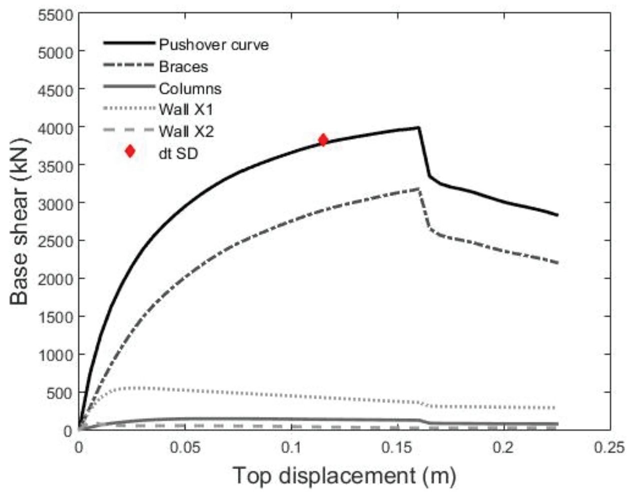
3.3 Results

Figure 4 shows the pushover curves, *i.e.* base shear *versus* top displacement at the centre of mass, for the X direction (the results in the positive and negative direction are identical, as the structure is symmetric with respect to the Y axis). A modal load pattern is used in both directions because it is more consistent when compared to the results of non-linear time history analysis, as shown in a previous study [1]. The strengthening solution which involves the application of the steel braces is depicted in Figure 4a, while the strengthening solutions which involves the application of FRP in the RC walls is shown in Figure 4b. The target displacements, obtained by applying the N2 method for the PGA prescribed in Lisbon for SD limit state, are depicted as a red dots in Figure 4. They amount to 0.11 m for the structure retrofitted with steel braces (Figure 4a) and to 0.118 m for the structure retrofitted with FRP. The latter is the target displacement also of the original structure.

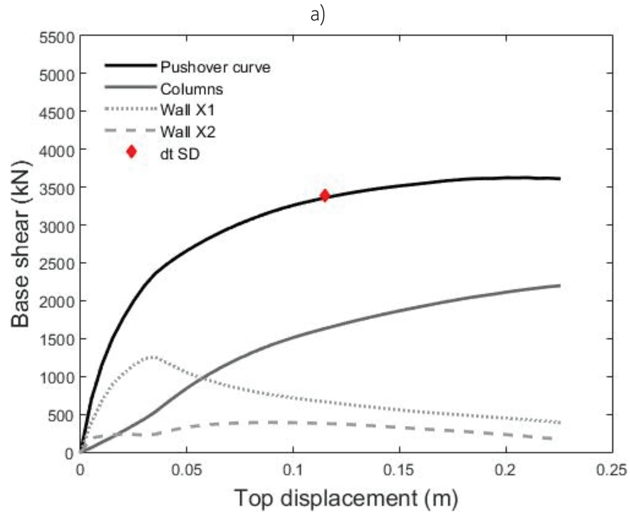
In Figure 4, the total base shear carried by the columns is labelled as "Columns", while the base shear carried by the first RC walls in the X direction is labelled as "Wall X1" and the second as "Wall X2" (assuming a positive direction of the load – Figure 1) and correspond to W1x and W4x in Figure 1. It is evident that the use of steel bracing significantly reduces the potential for shear failures in the walls at the ground storey level. The shear demand is reduced as to keep the RC walls in the elastic region (see Figure 2). The strengthening solution which involves the application of the FRP is represented in Figure 4b. As stated before, the application of the FRP does not modify the stiffness of the structural elements but increases the shear strength of the walls, allowing them to reach their flexural capacity without developing a brittle mechanism.

By comparing the two pushover curves (black solid lines) it is evident that the application of the steel braces at the ground storey did not result in a significant increase of the lateral strength (Figure 4a). It is also worth noting that the fundamental periods *f* the structure, which are 0.90 and 0.87 seconds in the longitudinal (X) and transverse (Y) directions, respectively, do not show any substantial difference after retrofitting. On the other hand, the absolute displacements at the lower stories are reduced. The distribution of the inter-storey drifts as well as the lateral displacements along the height of the structure are shown in Figure 5a and Figure 5b, respectively, where the results for building repaired with FRP are compared with the building retrofitted with braces.

The shear D/C ratio in the RC walls of the two retrofitted schemes at the SD limit state are plotted in Figure 6. All shear D/C ratios fall below unity, indicating the positive effects achieved with brace retrofitting in reducing the lateral demand imposed to the shear walls. A similar result can be observed for the FRP retrofitting, which increases the shear capacity of the walls leading to D/C ratios below one. It is noted that the partial retrofitting at the ground storey results in a decrease in wall shear at the lower storey, but it does not have a negative influence on the upper storeys of the building, where the D/C ratio is unchanged (although shear walls may experience higher demand from the effects of higher modes of vibrations).

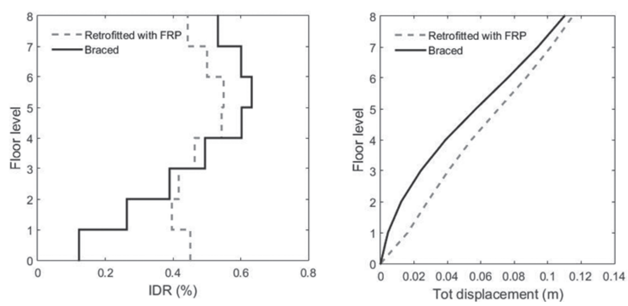


a)



b)

Figure 4 Pushover curves in the X direction for the retrofitted building with (a) steel braces and (b) FRP



a) b)

Figure 5 (a) Inter-storey drifts (IDR) and (b) lateral displacements profile in the X direction

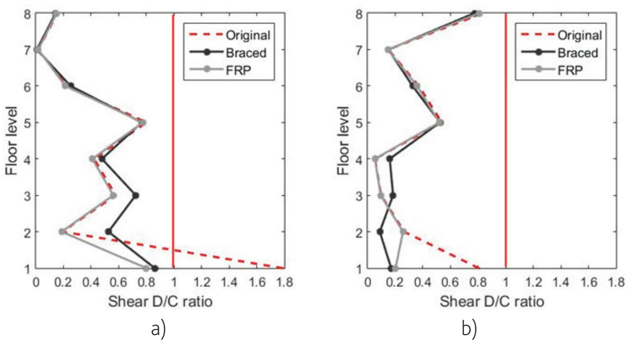


Figure 6 Shear D/C ratio in RC Wall1x (a) and Wall4x (b)

3.4 Costs of retrofitting

In the following paragraphs a description of the construction manufacturing costs, required to carry out the retrofitting operation using the two techniques examined, is reported. The unit price of each process includes the cost of materials, labour, transportation and rental, which must be added to the safety costs and profit of the construction companies. In order to consider reliable and realistic repair costs, the CYPE database [22], which contains detailed and up-to-date construction costs for the Portuguese construction practice, has been utilized. The prices not included in this database were deducted from the prices charged by construction companies operating with these techniques.

Table 4 shows a summary of the cost of retrofitting with C-FRP (carbon FRP) strips and sheets, including scaffolding, partial demolition and reconstruction of partition walls and finishing works. The estimated total cost of this intervention is 15829 Euro, considering full wrapping of the T-shaped RC walls at the ground storey with C-FRP sheet and strips.

Table 4 Cost of retrofitting with FRP

Description	Unit	Unit cost [Euro/unit]
Scaffolding	m^2	17.62
Demolition of interior partition wall	m^2	4.51
C-FRP laminates	m^2	97.88
C-FRP sheets	m^2	106.53
Reconstruction of interior partition wall	m^2	26.82
Plastering	m^2	4.29

Based on the indications provided by the manufacturers, the steel braces installation has an average unit price of 5.00 Euro / kg (varying from 3 to 9 Euro / kg), which includes material costs, production costs and on-site assembly costs (Table 5). Considering that four X - braces must be installed, two on each side of the structure (Figure 3), and that the weight of the CHS 219.1x5.9 is 31 kg/m, the total weight of the steel for these devices is 1984 Kg. The total cost for structural reinforcement through the installation of BRBs is estimated as 15277 Euro, thus a slightly more economic solution.

Table 5 Cost of retrofitting with steel braces

Description	Unit	Unit cost [Euro/unit]
Scaffolding	m^2	17.62
Steel Bracings	kg	4.30

4 Conclusions

The study reported in this paper addressed the issue of strengthening one of the most vulnerable class of existing reinforced concrete buildings in Lisbon, namely buildings with an open ground storey (pilotis), designed and built under old codes and engineering practices. The feasibility of performing partial strengthening of such buildings was examined, with the ultimate aim to develop an efficient retrofitting plan for this typology.

Two local methods of retrofitting were used, the first involving the partial strengthening at the open ground storey with steel braces, the second involving the FRP-wrapping of single elements (individual walls). FRP composite materials have received increasing attention in the past few decades as a potential solution for retrofitting of existing RC structures.

The purpose of this the study was to design two seismic local interventions, applied to a group of members that suffer from structural deficiencies in order to obtain the desired seismic performance. By means of nonlinear static analyses the seismic performance of the existing and retrofitted buildings was evaluated. By comparing the pushover curves, it was evident that the two solutions led to comparable results in terms of maximum strength and stiffness and allow mitigating the main vulnerabilities of the original building.

From the cost analysis, it turned out that the use of the technique steel bracing is the most economical solution and it also requires lower times of intervention. As a final consideration, local methods of intervention, as opposed to a complete strengthening to comply with current standards for new buildings, are perhaps the only retrofitting possibility that might be acceptable by the owners of such buildings, for two important reasons: (i) low cost of intervention and (ii) continued usage of the building during the retrofitting work.

Future developments of this work will include the implementation of a vulnerability loss assessment methodology, ideally in combination with a cost-benefit analysis framework to guide the decision regarding the retrofitting intervention.

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