In-plane retrofitting of masonry structures by using GFRP strips in the bedjoints

Elshan Ahani1 · Fathollah Osmanzadeh1 · Mir Naghi Mousavi1 · Behzad Rafezy1

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Abstract
Many unreinforced masonry structures were vulnerable in the past earthquakes and required retrofitting. The vulnerability of masonry structures could solve by providing numerous retrofitting approaches. However, the lack of appropriate methods, which could provide a solution for historical masonry structures with lesser effects on their façade, is demanding. In this study, two one-third scale masonry wall specimens made by clay bricks were tested under constant vertical and cyclic lateral loading. The specimens consist of an unreinforced wall and a wall retrofitted by GFRP strips. This study investigates the seismic behavior of unreinforced masonry walls before and after using GFRP strips on their bedjoints. To this purpose, various patterns of using GFRP strips have been studied by simplified micro-modeling. The results indicate that the proposed retrofitting technique could improve the lateral strength and stiffness of the unreinforced masonry wall with a considerable increase in the absorbed and dissipated energy and ductility content that changes the wall’s brittle failure to the ductile one. The proposed method could apply to the modern historical structures in which cement mortar has been used as an adhesive between the masonry layers.

Keywords Masonry walls · Retrofitting · Seismic behavior · GFRP strips · Simplified micro modeling

1 Introduction
A large percentage of the existing buildings have made of unreinforced masonry (URM) walls, which have not designed by seismic codes. Manifested reports from the earthquakes in the past 50 years demonstrated that the vulnerability of URM structures to seismic excitations is much more than other types of structural systems (McGuire and Leyendecker 1974; NAHB Research Center 1994; D’Ayala 2013; NZSEE Part-C8 2017; Fabbrocino et al. 2019). Due to devastating earthquakes, many URM structures were thoroughly demolished (NAHB Research Center 1994; Anagnostopoulos et al. 2004; D’Ayala and Paganoni 2011; Dizhur et al. 2011; NZSEE Part-C8 2017). While the survived URMs, are
severely damaged or exposed to failure (D’Ayala and Paganoni 2011; Dizhur et al. 2011; Bostenaru-Dan et al. 2013; Papayianni 2015; Cannizzaro et al. 2017). Many of these structures are monumental buildings that of high value and could be an attraction point for centuries. The ingredients used as an adhesive between the layers of the masonry units in many historical buildings were lime mortar. From the initial decades of the 20th Century, lime mortars used in masonry structures were gradually replaced by cement mortars (Gerns and Wegener 2003), which have congruity more than formerly used mortar layers. This issue leads to constructing monolithic taller buildings that are more susceptible to lateral loads. The brittle behavior of masonry units and discontinuity of load-bearing elements (Calvi et al. 2004; Lin et al. 2016) make these structures more vulnerable against lateral loads and oblige the URM walls to react as the only load-bearing system against seismic loads that barely survive. Thus, almost every URM wall needs to be considered for probable retrofitting methods with lesser expenses and more efficiency. In the current study, a different method of retrofitting for the modern historical masonry structures introduced.

Numerous experimental (Konthesingha et al. 2013; Wang et al. 2016; Knox et al. 2018; Arslan and Celebi 2019) and analytical (Chaimoon and Attard 2007; Dhanasekar and Haider 2008; Senthivel and Lourenço, 2009; Dolatshahi and Aref 2011; Abdulla et al. 2017) researches have been performed to evaluate the seismic behavior of masonry walls. Masonry walls are the only resisting elements in unreinforced masonry buildings exposed to earthquakes and unpredicted lateral loads. Several handy techniques in past decades were proposed to improve the seismic performance of existing URM walls. The prevailing retrofitting techniques could be categorized as but not limited to surface treatment like Ferro cement, shotcrete, etc. (Tasnimi and Rezazadeh 2012; Lin et al. 2016; Shabdin et al. 2018), grout injection (Dadras et al. 2019; Doran et al. 2019; Ferreira et al. 2019), external reinforcement (FEMA 172; Borri et al. 2009; Corradi et al. 2019) and center coring (FEMA 172; Dadras et al. 2019; Paret et al. 2019). Various researchers discussed the disadvantages of these methods, like decrease in necessary space (ElGawady et al. 2004; Cannizzaro et al. 2017), increase in structural mass (ElGawady et al. 2004; Cannizzaro et al. 2017; Ferreira et al. 2019) and the loss of architectural facades (ElGawady et al. 2004; Papayianni 2015; Dadras et al. 2019). Hence some researchers are concentrated on retrofitting structures by using Fiber Reinforced Polymers (FRP) in recent decades. Also employing stainless steel rebars inside the bedjoints (Binda et al. 2001; Valluzzi et al. 2005; Corradi et al. 2018), most probably in the interior part of the masonry, as a contemporary approach in constructing and most recently retrofitting of masonry wall due to its flexibility and ease in use are becoming popular. The advantages of using FRP polymers are their high mechanical strength, low weight, and ease of application (Wang et al. 2017). Externally bonded FRP strips used on the masonry walls increase the tensile and shear capacity, ductility, and lateral strength during the earthquake. Many researchers have studied retrofitting of the masonry walls by using different FRP patterns (Capozucca 2010; Luccioni and Rougier 2011; Santa-Maria and Alcaino 2011; Kalali and Kabir 2012a, 2012b). In a comprehensive study performed by Mazzolani (2015), the research obtainments related to employing the FRP fibers the essence of the research studies in 10 countries to assess the various aspects, including their advantages and disadvantages, have developed. The study proposed the modern strengthening techniques obtained from recent advancements in retrofitting the Mustafa Pasha Mosque, Fossanova Gothic Cathedral, Byzantine St. Nicholas Church, and a Greek Temple. Base isolation, applying the vertical and horizontal carbon fiber ties, and Carbon Fiber Reinforced Polymer (CFRP) belts and carbon rods along the bedjoints are the strengthening approaches that were employed. The latter technique was used in Mustafa Pasha Mosque and explained in detail in the studies performed by Krestevka et al. (2008)
and Protioli et al. (2011), according to which retrofitting was considered by defining three testing phases. The first testing is related to the original model, and the second and third phases are related to the experimentally retrofitted samples of the minaret and the mosque, respectively. Concerning the concurrently performed numerical analysis, the plastic strain content of the minaret in the connection area with the mosque identified. The same failure of developed crack patterns with wide depth in the first phase detected. As a solution, CFRP belts in the latter prototypes of both experimental and numerical studies for the mosque and minaret utilized. Rods of carbon fiber in the bedjoints of specific layers of the masonry shear walls to increase the integrity also employed. On the other hand, the high cost of FRP materials, which is about ten times that of structural steel, attenuates the use of FRP in the building industry (Burgoyne 2009; Sarker et al. 2011), unless it is used in historical monuments that have priceless value and the utilizing amount is managed. Using FRP conveys executive difficulties and an increase in expenses. Furthermore, in extreme conditions that require challenging solutions, they could confront difficulties. State of the art related to the fire resistance of masonry structures retrofitted by FRP materials is considerably narrow. Yet some studies related to the behavior of stone masonry samples confined by GFPR and CFRP after exposure to high-temperature fires (Estevan et al. 2020), the behavior of masonry walls due to high temperatures (Nadjai et al. 2006; Bošnjak et al. 2020), and also the resistance of FRP strengthened masonry structures subjected to the blast loads (Buchan and Chen 2007). Due to the limited studies, the picture related to the behavioral aspects of fire effects is still incomplete, and scattering information concerning the behavioral content of retrofitted masonries subjected to exposure to high temperatures and actual fires yields various faces of reality. Regarding the (Chowdhury et al. 2007; Bisby and Stratford 2013; Dong et al. 2016) studies, the low performance of FRP material subjected to high temperatures is a matter of concern. Moreover, because of the expansion, thermal bowing, and considerable reduction in mechanical properties of masonry walls subjected to fire (Nadjai et al. 2006; Bošnjak et al. 2020; Oliveira et al. 2021), their low performance in high temperatures is observable. It is indirectly predictable that the masonry-FRP combination could not have acceptable performance subjected to fire. Therefore, utilization of the covering and supplemental fire insulation in many studies (Buchan and Chen 2007; Chowdhury et al. 2007; Bisby and Stratford 2013; Frigione and Lettieri 2018) to retain the mechanical performance of FRP elements was proposed. Unlike fire resistance, the studies related to the durability of masonry retrofitted by FRP are widely traceable in literature (Modena et al. 2016; Kubica et al. 2020). The durability of masonry-FRP combination subjected to moisture and submersion (Maljaee et al. 2014; Vaculik et al. 2020), thermal and hygrothermal exposure (Maljaee et al. 2016; Frigione and Lettieri 2018; Vaculik et al. 2020), saline solution (Gharachorlou and Ramezanianpour 2010; Frigione and Lettieri 2018; Hu et al. 2020), corrosive conditions (Frigione and Lettieri 2018; Hu et al. 2020), freeze and thaw (Frigione and Lettieri 2018) were discussed profoundly in the reports. According to Hu et al. (2020) durability of Basalt Fiber Reinforced Polymers (BFRP) subjected to various conditions and even thermal exposure is appropriate. In the same study, the high creep of Glass Fiber Reinforced Polymer (GFRP) and its sensitivity to the alkaline environments, and suitable performance of CFRP due to creep and its inability to use in insulating conditions due to electrical conductivity, was reported. Almost the same observations for GFRP were declared by Frigione and Lettieri (2018). To debrief, the durability of the masonry-FRP set as an integrated system subjected to different extreme conditions is better than bare masonry. Concerning the literature, the FRP debonding in exceptional conditions was observed, which were related to thermal exposures in most of the cases.
In the current study, the FRP strips are used inside the bedjoints of the wall, which to some extent decline the complicated instruction of using FRP in structural systems. FRP materials are liable to shear loads (Burgoyne 2009) that make these substances sensitive to the sharp and pointy beds. Hence in the prevailing study, the bed for these materials was prepared with more accuracy, and using them in tension was controlled by numerical simulations before experimental studies. The utilization of FRP as a structural instrument facilitates many difficulties in the building industry; however, instruction deficiencies for using these materials in design codes and standards are comprehensively tangible. Many premier standards consist of but are not limited to FIB 24 (2003), ACI 369R (2011), and ACI 562M (2013) have provided instruction for rehabilitation and/or retrofitting of RC structures. The rehabilitation procedures of using FRP materials proffered by ACI 562M (2013) mostly referred to ACI 440.2R (2008), which do not cover the retrofitting principles of masonry structures. FIB 24 (2003) confines the considerable extent of the retrofitting process with FRP to RC columns. En-1998-3 (2005) may consider FRP as valid retrofitting material for existing structures, including masonry if it is used as a ring that wrapping the damaged or vulnerable to damaged elements. FEMA 172 (1992) proposed numerous retrofitting techniques for existing URM walls, including increasing the shear capacity by adding elements on the sides of the walls, filling the openings with appropriate elements, and using center coring. These methods do not satisfy the proposed requirements of Standards for the Treatment of Historic Properties with Guidelines for Preserving, Rehabilitating, Reconstructing Historic Buildings (1995) and ignore the 36 CFR 68 (2012) law for historic masonry structures. Utilization of FRP as a retrofitting element for existing steel, reinforced masonry, and reinforced concrete structures are provided and consented by FIB 14 (2001), CAN/CSA-S806 (2012), CAN/CSA-S6 (2006), and FEMA 308 (1998). However, many strictly related standards to unreinforced masonry structures such as FEMA P774 (2009) and ACI 530 (2011) have not considered FRP an acceptable material for retrofitting masonry structures. Although detailed information about the retrofitting procedure of historical buildings is provided by ASCE 41 (2006), only from the indirect statement sum-up of ASCE 31 (2003) and ASCE 41 (2006) can be deduced use of FRP for retrofitting of existing URM walls could be eligible. Yet no principle for the using procedure was presented. Hence, the requirement for studying the effects of FRP for both URM and RM walls may be tangible.

Due to the different properties of mortar joints, masonry walls are considered non-homogeneous parts. Hence, modeling of masonry structures due to anisotropy of the units, the dimension of the masonry units, joint width, material properties of the masonry units and mortar, the bed joints and head joints arrangement, and the working quality is considerably formidable (Lourenço 1994). The failure mode of the masonry walls could be diagonal sliding along the mortar joints, toe crushing, or rocking. The test outcomes of numerous retrofitted walls with FRP polymers demonstrate that a wide variety of failure modes consist of masonry cracking in tension, crushing of masonry in compression, masonry shear-sliding, the fiber-reinforced composite failure, FRP delamination, or their combination would take place. In many tests, the FRP strips were glued to historic clay bricks with a laminable surface, which leads to the delamination of FRP as a dangerous mechanism of brittle failure (Capozucca 2010). Sometimes delamination of FRP strips from masonry walls before the formation of failure makes the strips inert in the retrofitting process. In this paper, the FRP strips placed inside the grooves didn’t separate from the masonry wall before the failure.

Since the use of mortar in the building industry and masonry structures initiated from the early 1920s (Gerns and Wegener 2003), some of the modern historical buildings not
older than 100 years and constructed by cement mortar as a replacement for lime mortar could be retrofitted by this method. The mechanism of resistance in shear depends on the geometry of masonry panels, boundary conditions, the magnitude of the vertical loads, and characteristics of the interface bond between masonry units and mortar (Capozucca 2011). Many of the historical monuments are consist of masonry walls that play a crucial role in cultural beliefs and tourism. Seismic vulnerability evaluation of these buildings is always one of the most significant issues in civil engineering. Hence they should retrofit by methods that won’t affect the appearance and architecture, besides increasing the strength and ductility. Due to the complexity of the geometry, these buildings could not be retrofitted by ordinary methods. In this study, the retrofitting of masonry walls that consist of the vertical and lateral load-bearing elements has been taken into account by using a particular layout of FRP polymers. This method may use in retrofitting the walls of historical masonry structures, which shall have diminutive effects on the façade of their well-formed edifices. The mansion of Urmia municipality, the former palace of Tabriz municipality (Sa’at Tower), Stanley Dock Tobacco Warehouse of Liverpool, Monadnock Building of Chicago, and Sheikh Safi al-Din Khānegāh and Shrine Ensemble of Ardabil are the samples with historical values that could retrofit with the proposed method. This research is an endeavor to improve the integrity of modern historical masonry structures by retrofitting them with the aid of GFRP strips with lesser interventions in their façade and residual weight of the modified structure.

2 Research significance

Retrofitting brick walls with traditional methods has many executing problems, including considering architectural changes that are not feasible in many cases. The suitable physical and mechanical characteristics of FRP, which consist of lightweight, high strength to weight ratio, high strength in tension, moisture resistance, flexibility, durability, high stability, and corrosion resistance, encourage researchers to apply it. Most recent studies have been dedicated to fixing FRP strips on masonry walls either vertically or horizontally (Marcari et al. 2007; Capozucca 2010;Gattulli et al. 2017; Reboul et al. 2018). However, the applied methods that have an essential value in historical structures somehow inflict the façade. The use of FRP inside the bedjoints, which is the concentration point of the current study, may somehow resolve the problem. Since the historical structures are old and the bricks may not be in a row on out-of-plane direction, or the first layer of bricks may be weathered, they do not stick well and could be the cause of detachment of the FRP strips (Prakash and Alagusundaramoorthy 2008; Grande et al. 2013; Marcari et al. 2013). The problem referred to the inappropriate placement of the FRP strips in many cases. The modified placing of the FRP strips considerably diminishes the separation and is considered a solution in this research. Retrofitting historical masonries, the legacies of which should transmit to future generations can impose substantial expenses on the nation. The high outlay of the initial ingredients (Borri et al. 2009; Burgoyne 2009; Sarker et al. 2011) and the preparation requirements (Bostenaru-Dan et al. 2013; Dadras et al. 2019; Doran et al. 2019) may be the reason and proof for this allegation. Hence, an alternative method that requires lesser preparation and a more straightforward application procedure is desirable. However, due to the priceless value of the historical buildings, satisfying the condition is hard to achieve. Therefore, using FRP inside the bedjoints may decrease the complicated blueprints of retrofitting, the time, the cost, and could be applied by the workforce with
ease. According to numerous standards (Issue No. 376 2007; NSET 2009), the main types of problems and basic damage patterns observed during earthquakes in masonry buildings encompass non-integrity of wall and collapse of the building due to the rapid cracking and disintegrating of the various parts and their brittle nature. In this regard, the studies could lead to proffering better outcomes. According to many studies, low ductility (NAHB Research Center 1994; Tasnimi and Rezazadeh 2012; NZSEE Part-C8 2017; Dadras et al. 2019) and high weight (NZSEE Part-C8 2017; Shabdin et al. 2018; Aghabeigi et al. 2020) were accused as the main reasons for failure in masonry. Hence, most retrofitting procedures may solve the ductility but remain silent about the weight. Regarding the proffered facts, offering a retrofitting method that increases the integrity with lesser intervention in the façade and the final weight could be of high value.

3 Experimental program

3.1 Test setup and instrumentation

The specimens were constructed on a concrete foundation with 80 mm thickness, and the foundation was connected to the rigid floor with 12 high-strength bolts. A reinforced concrete beam was mounted on the walls to apply vertical and lateral loads. Many studies employ the vertical load in a way that the ratio of imposed stress to the compressive strength has minimal effects on the lateral behavior of masonry and failure type of the specimens. According to Celano et al. (2021), the amount could be less than 20%. In that study, the essence of mechanical characteristics of many other pieces of research has also been reflected, most of which resided in the determined range. Concerning other performed studies, this ratio could be in the range of 4 to 20% (Haach et al. 2010; Garofano et al. 2016; Guerreiro et al. 2018; Rezapour et al. 2021). In the current study, the vertical load has been adjusted in a manner that the stress to the compressive strength ratio of the masonry prism is about 5%. The 20 kN vertical load was applied by the 50 kN actuator and placed on steel rollers upon the I section with direct contact to the RC beam. To barricade the application of unwanted loads to the specimens the plumbness of the vertical loading system was checked by appropriate instruments. Yet there is a chance that due to the limited out of plumbness of the load cell, the actuator, or the supporting superstructure, an out-of-plane loading, unwantedly has been being imposed, and led to its contribution in the load-bearing process of the specimens. It could affect the stiffness and other aspects of the samples. The rollers expand the vertical load on the loading surface of the wall’s upper edge and hinder the creation of false stiffness. The lateral cyclic load was applied by a 1000 kN hydraulic actuator to the concrete beam. The reaction of the walls was undertaken by the rigid floor, which has no significant deformation under the lateral cyclic load. A 200 kN load cell took place between the hydraulic actuator and the lateral load system. The loading procedure was initiated by applying vertical load with linear increasing amplitude. Then the lateral cyclic load was applied. The lateral displacements of the walls under cyclic loading were measured by Linear Variable Differential Transformers (LVDT) located on the upper sides of the masonry wall. Two horizontal bars on each side of the test specimen were fixed to the loading plates on the rear and front faces of the wall. The sidebars hung from the superstructure let the vertical loading apply to the specimens through the related actuator without any interference. The
loading commenced by direct loading inducement of the actuator to the former side, and the reverse loading was performed by engaging the horizontal loading bars to the rear loading plate. A detailed schematic of the test setup and the utilized instruments for the loading procedure has provided in Fig. 1. The lateral loading procedure with increasing cyclic steps continued until the failure of the specimens. The applied force-controlled loading pattern of the present study with 3kN steps has depicted in Fig. 2.
3.2 Test specimens

Because of the high expenses of experimental samples, two specimens have been mounted in the structural laboratory of the Sahand University of Technology, and the rest of the studies to increase the research depth were performed numerically. Experimental studies for assuring the prosperity of the proposed method, and numerical studies to assess their efficiency, were carried out. However, providing more accurate outcomes for complicated structures requires further apprising, and this study is performed to evaluate the viability of the introduced method. Concerning the principles of BHRC (2005), the width of loadbearing masonries shall not be less than 0.2 m, and the length of them without tie columns shall not exceed 6 m. The maximum height of a masonry wall without tie beams shall be less than 4 m, accordingly. Since most of the modern historical masonries, to some extent, resided inside these limitations, a masonry wall with a respective height, width, and thickness of 3 m, 5.25 m, and 0.3 m were assumed as the prototype for this study. The selection criteria for evaluating the unreinforced masonry and the dimension also reside within the constraining principles of En-1998-1 (2004). Concerning the instrumental limitations and further execution costs, the one-third scale models were constructed in the laboratory. Hence, the height, width, and thickness of the selected wall specimens with the aspect ratio of 1.74 were 1 m, 1.74 m, and 0.1 m, respectively. Because of the executive restrictions, a minute difference between the aspect ratios of the prototype and experimental models took place. Despite scaling the experimental specimens, the size of masonry units was retained. Since the provided sizes for the masonry units are various, it could be hypothetically authentic to use smaller units to acquire the required scale for the research specimens.

The equilibrium equation of the URM was allocated to determine the equivalent required GFRP strips. The hypothesis of prepared equilibrium equations of the URM was made the design rationale and needed GFRP strips area for retrofitting. The required amount in each layer according to the simplifying assumptions of the equilibrium equation has been obtained. The equation was provided according to Fig. 3 and presented in respectively explained Eqs. (1)–(3) equations.

\[
V = \begin{cases} 
\text{nb}h\gamma G_m E_b & \geq E_m \\
\text{nb}h [\epsilon (E_b - E_m) + \gamma G_m] E_b & < E_m 
\end{cases}
\]

(1)

where \( E_b \) and \( E_m \) are the elasticity modulus of masonry unit and mortar joint, \( b \) and \( h \) are the width and thickness of the masonry units, \( n \) is the numbers of existing layers in the equilibrium element, \( \gamma \) is the shear strain of the selected element, and \( G_m \) is the shear modulus of mortar joint, respectively. The maximum amount of shear considering the \( nF_s \geq V \) inequality, where \( F_s \) is the imposed tensile force of the GFRP strips located inside the bed-joints, according to Eq. (1), always took place when the elasticity modulus of the masonry unit is more than its peer mortar joint. Correspondingly, the minimum design force for the assigned strips may acquire according to Eq. (2).

\[
F_s = bh\gamma G_m
\]

(2)

The minimum required area of GFRP strips by expanding Eq. (2) would lead to more operational relations as presented in Eq. (3).

\[
A_s = bh \frac{\gamma}{2(1 + \nu_m)} \frac{E_m}{\varepsilon_s E_s}
\]

(3)
Masonry units made of clay are prevalent construction units in the Middle East. The average measured dimension for the selected units was 195.4×98.5×56.3 mm. The average thickness of the mortar joints is 10 mm with a water:cement:sand ratio of 1:2:6. The studied wall specimens are consist of a bare wall and a wall retrofitted by 10 mm width GFRP strips, which placed along the bed joints on one side of the wall. In Table 1, the evaluated experimental models have been demonstrated. Mechanical properties of

**Table 1** Characteristics of experimental specimens

| Description                        | Specimens 1 Unretrofitted masonry wall | Specimens 2 Retrofitted masonry wall |
|------------------------------------|--------------------------------------|-------------------------------------|
| Height, width and thickness (mm)   | 1000 × 1740 × 100                     | 1000 × 1740 × 100                    |
| Aspect ratio (width/height)        | 1.74                                 | 1.74                                |
| Thickness of mortar joints (mm)    | 10                                   | 10                                  |
| Cement/sand ratio of mortar        | 1/3                                  | 1/3                                 |
| Masonry unit size (mm)             | 195.4×98.5×56.3                       | 195.4×98.5×56.3                      |
| Number of GFRP rows                | None                                 | 14                                  |
| Thickness of GFRP strips (mm)      | None                                 | 10                                  |

![Figure 3](image) Preparing equilibrium equation of masonry partial element for determining the retrofitting requirements of unreinforced masonry wall
masonry walls are affected by the characteristics of the constituent elements (bricks and mortar), crafting skills, and the interface interaction within the assemblage. The mechanical properties of bricks were determined by unidirectional compressive and tensile tests. The mean compressive strength of ten specimens of masonry units was 9.20 MPa, and the average flexural strength of the bricks was 0.31 MPa according to ASTM C67-11 (2011) test principles. The compressive and tensile test results of the studied specimens are presented in Table 2. The standard test results of three (40 × 40 × 160mm³) flexural and six (50×50×50mm³) compressive mortar samples, for 28 days curing, had an average strength of 4.10 MPa and 33.21 MPa, respectively. The modulus of elasticity and the Poisson’s ratio of concrete beams on the wall and the rigid floor were obtained according to ASTM C469/C469M-14 (2014). To determine the characteristics of masonry panels according to ASTM C1314-11 (2018), compressive and flexural strength tests were performed. The GFRP laminates were used for retrofitting the wall with the ultimate strength of 2300 MPa and the elasticity modulus of 90GPa. The thickness of GFRP strips used for retrofitting was 0.24 mm. The results of material tests performed to determine the properties of wall elements have been shown in Table 3.

This study concentrated on comparing the in-plane failure modes of the retrofitted and URM wall specimens and examining the ability of the existing verified analytical models to predict the lateral strength of URM and retrofitted walls with different numbers of GFRP layers.

To this purpose, the GFRP fabric was cut into 20 mm width and 1940 mm length strips (200 mm of the strips were bent and fixed on the unretrofitted side of the wall). The GFRP strips longitudinally bent over and glued two times to make four-layer strips. Horizontal bedjoints of the retrofitted layers were carved 20 mm to provide an embedding place for the GFRP strips, as presented in Fig. 4b. To prepare a flatbed for FRP strips and better integrity between the wall and the strips, the created cavities filled with putty, the details of which have been demonstrated in Fig. 4c. Finally, GFRP strips were glued into the grooves, as depicted in Fig. 4d. The GFRP strips were used on one side of the masonry, and also the grooving took place on one side of the bedjoints. Since the GFRP strips are used as longitudinal reinforcing elements, it is a tangible requirement to restrain these reinforcements, but lapping for FRP polymers in the provided condition is practically impossible. Hence, they turned to the wall other side to retain the reinforcement integrity and barricade the possible detachments. For the longsome walls, the procedure could perform by bending the strips into the penetrated hole on the wall. According to initial numerical results placing GFRP strips in one-third of the masonry joints could satisfy the retrofitting requirements. However, GFRP strips were poised in all the joints to increase reliability.

### 3.3 Experimental results

The experimental results of the test specimens are discussed in terms of lateral strength and drift. Figure 5 illustrates the cracking patterns of the specimens at the final stage of testing. In general, a mixed shear-flexural failure mode was observed in the test specimens. The failure of the first wall was the combination of diagonal crack and bed joint sliding, which is a brittle failure and takes place in a short fraction with sudden collapse. Specimen 1 approximately failed in 56kN with 0.73% drift. In the first three steps of cyclic loading for Specimen 2 (10kN), the wall was in the elastic state, and no cracking was detected. The initial flexural cracks started in the 12th step (37kN loading force and 0.38% drift). The flexural cracks expanded on both sides of the wall, and then the bricks were crushed in the
| Sample | Sample dimension (mm) | Rupture force (kN) | Compressive strength (MPa) | Sample | Sample dimension (mm) | Rupture Force (kN) | Flexural Strength (MPa) |
|--------|----------------------|--------------------|-----------------------------|--------|----------------------|--------------------|------------------------|
| 1      | 198.5×97.3           | 185                | 9.58                        | 1      | 191.5×94.8×55.9      | 3.12               | 0.405                  |
| 2      | 186.3×97.1           | 177                | 9.78                        | 2      | 194.0×93.0×55.9      | 1.66               | 0.253                  |
| 3      | 192.3×99.0           | 180                | 9.45                        | 3      | 185.0×102.0×55.0     | 2.21               | 0.271                  |
| 4      | 195.7×97.5           | 167                | 8.75                        | 4      | 194.2×103.2×54.0     | 2.80               | 0.354                  |
| 5      | 203.5×100.0          | 250                | 12.29                       | 5      | 195.7×101.2×58.7     | 2.56               | 0.276                  |
| 6      | 202.0×96.0           | 215                | 11.09                       |        |                      |                    |                        |
| 7      | 196.0×96.0           | 121                | 6.30                        |        |                      |                    |                        |
| 8      | 204.2×103.2          | 151                | 7.17                        |        |                      |                    |                        |
| 9      | 198.9×99.0           | 179                | 7.90                        |        |                      |                    |                        |
| 10     | 199.0×93.0           | 156                | 9.67                        |        |                      |                    |                        |
| Average| 197.64×97.81         | 164.6              | 9.20                        | Average| 192.08×98.84×55.9   | 2.47               | 0.31                   |
| Deviation| 5.42×2.73           | 35.26              | 1.77                        | Deviation| 4.24×4.61×1.75     | 0.56               | 0.07                   |
### Table 3 Material Properties

| Material          | Properties                                | Specimen no | Deviation | Test standard                  |
|-------------------|-------------------------------------------|-------------|-----------|--------------------------------|
| Masonry unit      | Compressive strength = 9.20 MPa           | 10          | 1.774     | ASTM C67-11                    |
|                   | Flexural strength = 0.31 MPa             | 5           | 0.065     | ASTM C67-11                    |
| Concrete          | Elasticity modulus = 24,100 MPa          | 2           | 832.4     | ASTM C469/C469M-14             |
|                   | Poisson’s Ratio = 0.22                    | 2           | 0.005     | ASTM C469/C469M-14             |
| Mortar            | Compressive strength (28 days) = 33.21 MPa| 6           | 0.986     | ASTM C109/C109M-11             |
|                   | Flexural strength (28 days) = 4.10 MPa    | 3           | 0.470     | ASTM C348-08                   |
| Masonry prism     | Compressive strength = 2.18 MPa          | 5           | 0.799     | ASTM C1314-11                  |
|                   | Tensile strength = 0.06 MPa              | 5           | 0.004     | ASTM E518/E518M-15             |
|                   | Cohesive strength = 0.14 MPa             | 5           | 0.014     | ASCE/SEI 31-03 and ASTM G115-04|
|                   | Coefficient of internal friction = 0.53  | 5           | 0.040     | ASTM G115-04                   |
| GFRP strip        | Ultimate strength = 2300 MPa             | Wrap 600G   |            | According to QUANTOM®          |
|                   | Elasticity modulus = 9 × 10^4 MPa        | Wrap 600G   |            | According to QUANTOM®          |
lower corners due to toe-crushing failure mode. Then bed joint sliding failure occurred in the lower line of the first row. In the last stage, the wall lost its strength by FRP strips separation from the wall. The second specimen failed in the 24th step (70kN loading force and

Fig. 4 Retrofitting process of Specimen 2; a Schematic view; b Carving of masonry layer; c Putty pasting inside the carved layers; d Poising of FRP strips in mortar joints

Fig. 5 The finalized crack pattern of the scrutinized specimens; a Specimen 1; b Schematic view of the Specimen 1; c Specimen 2; d Schematic view of the Specimen 2
1.7% drift). The wall acts as an integrated object, and soft failure mode occurred according to Fig. 5c. Detailed crack pattern and final failure of the studied specimens have provided in Fig. 5b, d. Since the loading protocol is force-controlled, the load amount at every loading cycle reached the determined limits.

At the early stage of loading, the specimens are in the linear elastic range, and the loading and reverse loading is symmetric. Due to the low tensile strength of the masonry, a loss of stiffness and strength occurred in the first half of each loading cycle, which caused an increase in displacement in the reverse loading. This unsymmetrical behaviour between the loading and reverse loading increases at the final stages of the tests. The use of GFRP strips in the second specimen improved the consistency of the masonry and consequently decreased the deviations between the displacements of the loading and reverse loading in the first cycles. The GFRP strips lost their connection with the masonry in the last cycles, although they had increased the ductility of the specimen. Hence it is assumable that the hysteresis curves don’t have symmetry in the displacement axis. The hysteresis curve of experimental specimens and the extracted superposition of the hysteresis loops have presented in Fig. 6.

The failure mode in Specimen 1 was diagonal cracking. The minimal period between the initiation of the failure and its finalization led the structure to undertake considerable shear forces at the loading early stages. It is inferable that the unretrofitted specimen has a brittle failure since the failure of the specimen took place in shear. However, the failure pattern in Specimen 2 was initiated with flexural rocking failure and eventuated with shear toe crushing failure. According to the outcomes, the retrofitting may change the failure mode and provided more deformation capability to the structure. The displacement capacity of Specimen 2 is 100% more than Specimen 1. The absorbed energy of the proposed retrofitting method is about 6950 kN.mm, approximately 2.5 times more than the unretrofitted specimen, and the energy dissipated by the retrofitted and unretrofitted specimens is about 2670 kN.mm and 260 kN.mm, respectively. Concerning the extracted results, the ability of the retrofitted sample to dissipate energy due to its superior plastic content and higher load-bearing capacity is 10 times that of the unretrofitted specimen. The broader width of the load–displacement cycles displays that the energy absorption ability of the retrofitted specimen is considerably improved. The failure of the unretrofitted specimen in lower drifts was amended by utilizing GFRP, which also improves the tensile strength and prevents the sudden fracture of the masonry. Considerable increases in the flexibility of the structure due to retrofitting procedure authenticate the preliminary assumptions about the prosperity of using GFRP at the bedjoints as a retrofitting method.

Stiffness degradation
Initial stiffness of masonry is calculated by measuring the slope of the line tangent on the load–deflection curve at the origin. The linear elastic stiffness was

![Fig. 6 Superposition of the hysteresis loops for the reference and retrofitted specimens](image-url)
calculated according to Tomaževič (1999). This method combines the principles of ATC-40 (1996) and FEMA440 (2005) standards, which have been established based on equal energy definition. According to this definition, the enclosed areas under the load–displacement and the ideal bilinear curve should be identical. To simplify the behavior of historical URM walls, several researchers considered idealized bilinear envelopes for horizontal force–displacement cyclic in-plane experimental behavior (Magenes and Calvi 1997; Haach et al. 2010). Therefore, to obtain the stiffness, the secant stiffness of each cycle and peak lateral load of the $i^{th}$ cycle together with its corresponding displacement should be acquired. Since the stiffness degradation rate depends on the damage of the wall, the secant stiffness of each cycle was calculated to evaluate the evolution of damage during the loading process. The secant stiffness at each loading cycle was calculated according to Eq. (4).

$$k_{si} = \frac{H_{max,i}}{d_{Hmax,i}} \tag{4}$$

where $k_{si}$ is the secant stiffness of each cycle, $H_{max,i}$ is the peak lateral load of the $i^{th}$ cycle, and $d_{Hmax,i}$ is the displacement corresponding to the peak lateral load of the $i^{th}$ cycle. The final secant stiffness of the structure is calculated according to Eq. (5).

$$k_s = \frac{H_{max}}{d_{Hmax}} \tag{5}$$

where $H_{max}$ is the maximum resistance of the wall during the shear test and $d_{Hmax}$ is the corresponding displacement. The stiffness staged degradation for both specimens was proposed according to Tomaževič (1999) and is demonstrated in Fig. 7. In the figure, the load–displacement and bilinear curves for the studied specimens are also presented. The unreinforced masonry and retrofitted wall present the lowest and highest stiffness degradation, which are approximately 68% and 64% of $0.6d_{Hmax}$, respectively.

![Figure 7](image-url)

**Fig. 7** a Bilinear idealized diagram for masonry walls proposed by Tomaževič (1999); b Experimental and idealized horizontal load–displacement diagrams of the studied specimens
In Fig. 7, $d_{cr}$ is the displacement corresponds to the initial wall cracking, $d_e$ is the elastic displacement, $d_{H_{max}}$ is the displacement correlated to $H_{max}$, $d_u$ is the idealized displacement point, $d_{max}$ is the ultimate horizontal displacement, $H_{d_{max}}$ is the ultimate horizontal force during the test, $H_{cr}$ is the horizontal force at the formation of the first significant cracks in the wall, $H_u$ is the wall ultimate force, and $H_{max}$ is the maximum horizontal force.

**Ductility factor** The primary weakness of clay brick masonry structures during earthquakes is the lack of ductility, which is mistakenly considered as the lack of resistance in most cases. Ductility may determine by the bilinear load–displacement curve, in which the maximum displacement is divided by the first yield displacement. The corresponding equation for this definition has provided in Eq. (6).

$$\mu_u = \frac{d_u}{d_e}$$  \hspace{1cm} (6)

The elastic displacement can be explained by Eq. (7).

$$d_e = \frac{H_u}{K_e}$$ \hspace{1cm} (7)

The effective stiffness of the wall could be extracted from Eq. (8), which is proposed by Tomaževič (1999).

$$K_e = \frac{H_{cr}}{d_{cr}}$$ \hspace{1cm} (8)

The referring value for $H_u$ proposed by Tomaževič (1999) is considered to be 90% of $H_{max}$. According to the experimental outcomes and corresponding bilinear curves, in the current study, $H_u$ was considered 84% of $H_{max}$, close to the extracted results of Tomaževič (1999). Table 4 shows the results of bilinear idealization for the walls. According to the table, ductility and elastic stiffness have increased in the retrofitted wall by 4.3 and 1.86, respectively. Comparing the results with the experimental outcomes illustrated a proper convergence between experimental and numerical stiffness. The comparison between the characteristics of the retrofitted and the unretrofitted specimens has been presented in Fig. 8.

According to the outcomes of the experimental, numerical, and theoretical studies, the summary of the compared results between the unretrofitted and the retrofitted specimens is presented in Fig. 8. By the results, the load-bearing capacity and initial stiffness of the retrofitted specimen increased by 22% and 86%, respectively. The ductility content ($\mu$) of the retrofitted specimen is 2.29 times more than the unretrofitted specimen. In this regard, the increase in the ductility content of the retrofitted specimen is considerably more than the corresponding increase in the stiffness, which leads the behavior of the structure to a ductile failure.

**Table 4** Summary of the results for bilinear idealized curves of the studied specimens

| Specimen | $H_{cr}$ (kN) | $d_{cr}$ (mm) | $K_e$ (kN/mm) | $H_{max}$ (kN) | $H_u$ (kN) | $d_e$ (mm) | $d_u$ (mm) | $\mu$ |
|----------|---------------|---------------|---------------|----------------|-------------|------------|------------|------|
| Specimen 1 | 29.12 | 0.56 | 52.39 | 56.38 | 50.74 | 0.97 | 5.99 | 6.2 |
| Specimen 2 | 37.1 | 0.38 | 97.63 | 68.57 | 61.71 | 0.63 | 16.81 | 26.6 |

† Springer
4 Numerical studies

There are numerous valuable historical buildings with complex architecture that impose researchers to replace the costly experimental investigations for assessing their behavior during the earthquakes by numerical modelings (Milani et al. 2017; Valente et al. 2017). According to the outcomes of many studies (Sandoval and Roca 2012; Ahani et al. 2019), the finite element method (FEM) is the most accurate and widely used tool for analyzing masonry structures that have also used in numerical simulations of this research. In the current study, The Concrete Damaged Plasticity (CDP) criterion, a modified form of the Drucker–Prager criterion, is used for numerical simulation (Kmiecik and Kaminski 2011).

4.1 Simulation

The simplified micro-modeling approach for numerical simulation of the masonry specimens was appropriated. According to this approach, mortar thickness and brick–mortar interfaces are squeezed to a zero-thickness surface. Afterward, the dimensions of the brick units expand to retain the geometry of a masonry wall (Senthivel and Lourenço, 2009). For numerical simulations and assessments, ABAQUS CAE commercial software was employed. Joints were modeled using interface elements and softening on the cohesion and friction angle. Masonry units were modeled by linear 3D, eight-node cubic C3D8R elements. The CDP criterion was employed to model the post-failure behavior of masonry units. The COH3D8 element was used in modeling the adhesives. Young’s modulus of the wall was considered the sum of the stiffness of the bed joints and the elasticity modulus of the units, which is presented in Eq. (9) (Sandoval and Roca 2012).

\[
k_n = 0.583E_b
\]

where \( k_n \) is the stiffness of the bed joints and \( E_b \) is the experimentally measured Young’s modulus of the masonry units. Interface shear stiffness as the other elastic property of the
joints by assuming the applicability of the theory of elasticity has been calculated directly from the normal stiffness, which presented in Eq. (10) (Senthivel and Lourenço 2009).

\[ k_s = k_n / (1 + \mu) \]  

where \( \mu \) is Poisson’s ratio (assumed to be 0.15), and \( k_s \) is the interface shear stiffness. The inelastic properties of the unit-mortar interface consist of tensile strength, first fracture energy, and second fracture energy, the details of which explained in Table 5 and considered in numerical modeling (Lourenco 1996). This adjustment has been described in more detail by Lourenco (1996), Senthivel and Lourenço (2009), and Sandoval and Roca (2012). The GFRP composite was modeled by S4R 3D shell elements that have three rotational and three translational degrees of freedom at each node. By coupling the degrees of freedom of the solid elements to the shell elements, the mismatch between them reconciled.

Since no separation between the GFRP strips and masonry wall during and after the test procedure was observed, it is considered that the separation between GFRP strips and the masonry wall won’t take place. Therefore, the tie element was used for numerical modeling of the GFRP-wall connection. The analyses were carried out by the direct displacement control method and considering the geometric nonlinearity. The concrete beam on top of the walls was assumed rigid to reduce the computational operations.

In performing the nonlinear analysis, essential factors are the loading procedure and the analysis approach (Prakash 2008). The incremental displacement was applied to the wall-tied upper beam and constrained in all directions except the loading axis. The load increments were determined at every loading step by arc-length control scheme. The wall was tied to the foundation at the seat, and the foundation supports were fixed in all directions. The loading was assigned in 2 steps to simulate the experimental condition. The vertical load was applied first and took a quarter of the overall loading time. The lateral load was appropriated at the rest of the loading period afterward. The load application procedure is linear. The strips in the masonry layers were considered to remain elastic. The thickness of the composite reinforcements was 0.24 mm as in the experimental study. The numerical model of the masonry panel by coupling masonry elements nodes with composite strips obtained, which corresponds to a perfect bonding between the masonry units and the composite strips.

### 4.2 Verification

In Fig. 9, the load–displacement curves obtained for the studied specimens and comparison with experimental results have been presented. The difference between the final strength of experimental and numerical models for specimens 1 and 2 are 18.2% and 13.1%, respectively. The difference between the initial stiffness of the studied specimens is 7.5% for specimen 1 and 7.7% for specimen 2. The results of experimental and numerical specimens were in a proper convergence. At the final stage, the cracks of specimen 1 (unretrofitted masonry) were formed with a smaller width, while due to the use of GFRP strips in Specimen 2 (retrofitted masonry), a wide diagonal crack was observed. The GFRP strips reclaimed the retrofitted wall as an integrated homogeneous object like a shear wall. This proposed method of retrofitting changed the shear-sliding failure mode to the sliding failure mode (Fig. 10). According to these outcomes, in the retrofitted specimen, the wall is entirely involved in load-bearing due to the increase in the integrity of the wall. In this regard, the amount of plastic strain as an indicator of partial or thorough damage
Table 5  Characteristics of bricks and mortar joints used in numerical modeling

| Masonry unit | Young’s modulus | Poisson’s ratio | Density | Dilation angle | Elastic properties | Plastic properties | Concrete damage plasticity |
|--------------|-----------------|----------------|---------|----------------|--------------------|--------------------|---------------------------|
| Concrete     | 4900 MPa        | 0.15           | 1.38 × 10⁻⁶ kg/mm³ | 27.5°          | Kn, Ks, Kt         | Density            | Tension-cut off           |
| Mortar       | 2622 MPa        | 1141 MPa       | 2.12 × 10⁻⁶ kg/mm³ |                | fn, fs, ft         | Normal behavior     | Contact                   |
|              |                 |                |         |                | 0.157 MPa          | 0.216 MPa          | 0.53                      | 1.96 MPa                  |
Fig. 9  Comparison of load- displacement curve of the experimentally studied specimens with numerical outcomes

Fig. 10  Approximate crack formation places in analytical model for Specimen 1 and Specimen 2
considerably decreased. The stress status of numerically evaluated specimens after applying the vertical loads and at the final stage has shown in Fig. 11.

According to the provided counters in Fig. 10, the strain value declined from 0.054% in specimen 1 to 0.028% in specimen 2. In this regard, the strain value in the retrofitted specimen halved and expanded to the entire wall in exchange. This homogenous expansion of the strain in specimen 2 leads to ductile rocking failure presented in the experimental study. The outcomes of the numerical analysis for specimen 1 also cover the extracted results from the empirical research study, the observed failure of which in the unretrofitted wall occurred in brittle diagonal cracking failure.

According to numerical outcomes, using GFRP strips in one-third of masonry layers satisfies the expected behavior of the structure. Further numerical analyzes by reducing the number of layers are to evaluate whether lesser GFRP strips may cover the anticipated behavior of the augmented masonry or the retrofitting procedure becomes inert. Since both the numerical and experimental outcomes in both have an appropriate convergence, it is expectable that the retrofitting results of the masonries with lesser GFRP strips with the number of retrofitting layers between the initial specimens could have acceptable accuracy.

### 4.3 The effects of number of layers

To find the optimum reinforcement for the unretrofitted specimen by using the proposed retrofitting technique, the effect of 5 retrofitting arrangements for 3, 4, 5, 7, and 14 GFRP layers with 0.24 mm thickness on the initially evaluated wall in addition to the bare wall examined and results have compared. The increase in stiffness and strength was assessed by comparing the results with the unretrofitted specimen. The arrangement of GFRP rows is presented in Fig. 12. The ratio of the used GFRP strips to the wall area according to the width and the thickness of the used GFRP strip, the ultimate strength, and the strength increase ratio by comparing them to the unretrofitted specimen have presented in Table 6. By inserting 3 layers of GFRP strips in horizontal bed joints due to the small $A_{\text{GFRP}} / \text{wall}$ ratio, the space between the rows of the GFRP layers does not affect the strength.

![Fig. 11 Numerically evaluated specimens before and after application of cyclic horizontal loads](image-url)
considerably, while the further increase of GFRP strips leads to a change in the strength increase rate from 4.5% to 12.1% (Table 6). By increasing the number of GFRP layers up to 14 layers, the wall strength was increased by 45.9%. Placing GFRP strips in all the horizontal bed joints concerning the executive and economic viewpoints is problematic. Hence, considering the financial issues, the optimum layers to insert GFRP strips are ranged from 4 to 7 rows in horizontal bed joints. The stress status of further evaluated numerical specimens after applying the vertical loads and at the final stage is shown in Fig. 13. The load–displacement curve of numerically evaluated samples has presented in Fig. 14.

### 5 Conclusion

This study aimed to provide a method of retrofitting masonry walls, which barely affect the façade of the building in a manner that increases the ductility content of the complex. While one of the main problems of masonries is their brittle behavior, an increase in ductility could considerably improve their lateral behavior. According to the experimental and numerical experiences, the following results were obtained.

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![Image](image_url)

**Fig. 12** The schematic view of the bare model and application procedure of GFRP strips in expanding numerical studies. GFRP strips considered to apply in 3–14 layers of initially evaluated models to consider the lateral behavior of each model and introduction of the most proper retrofitting model.

| Model number | Number of GFRP row | GFRP thickness (mm) | $(A_{FRP}/A_{wall})$ % | Ultimate force (kN) | Increase rate (%) |
|--------------|--------------------|---------------------|-----------------------|---------------------|------------------|
| 1            | 0                  | 0                   | 0                     | 56.11               | 0                |
| 2            | 3                  | 0.96                | 0.03                  | 58.66               | 4.5              |
| 3            | 4                  | 0.96                | 0.04                  | 62.85               | 12.1             |
| 4            | 5                  | 0.96                | 0.05                  | 67.33               | 19.9             |
| 5            | 7                  | 0.96                | 0.07                  | 70.62               | 25.8             |
| 6            | 14                 | 0.96                | 0.14                  | 81.89               | 45.9             |

---

**Table 6** Used FRP percentages for retrofitting of Specimen 1 (Unretrofitted Specimen)
Fig. 13  The stress status of the numerically excavated specimens with different GFRP layers after application of gravity loads and at the final stage of loading.
The proposed retrofitting method leads to a gradual failure of the wall. Hence, the integrity of the wall, even after failure retained. The first crack in the wall took place in the high lateral load values. Due to their high elasticity modulus and correspondingly stiffness content, FRP strips commence absorbing a considerable amount of the lateral load when the cracks form.

Reinforcing the wall utilizing the proposed method increases the shear capacity, load-bearing capacity, initial stiffness, and in-plane strength. Although the intended retrofitting method increased both stiffness and ductility parameters, ductility augmentation was more than stiffness. Since this method of retrofitting does not increase the structure weight, which directly leads to the absorbing of further seismic loads, the load-bearing capacity by the increase in the ductility content shall enhance.

GFRP strips change the brittle failure to a soft failure mode. In the first experimentally studied specimen, the lateral load underwent with the shear strength of the mortar. Since the mortar had low shear strength, the wall collapsed in mixed diagonal crack and bed joint sliding failure mode. The second specimen has the horizontal GFRP strips, which bear the lateral forces in companion with the mortar shear strength. The wall strength loses from weaker row with bed joint sliding mode, failure of which was more ductile than the first studied experimental specimen.

Ultimate strength, maximum displacement, and absorbed energy of the retrofitted walls with 14 rows of GFRP layers are 23.1, 2.08, and 2.5 times larger than the first experimentally studied wall, respectively. While, the dissipated energy of the employed retrofitting method is about 9 times greater than the unretrofitted specimen, which is due to the better flexible content of the retrofitted sample. The high maximum displacement of the retrofitted wall indicates that the wall has a higher plastic content and tensile strength, which could be a notification for collapse. The outcomes for samples with 3, 4, 5, and 7 GFRP layers are almost close, while the difference with the 14 GFRP layers is tangible.

According to the results, a retrofitted wall converts cracks with a greater depth to small cracks with less thickness. GFRP strip makes all parts of the wall act as an integrated object and resists the lateral load. The experimental observations, which were also proved by numerical studies, demonstrate that the strain distribution in retrofitted specimens is more homogenous, leads to the formation of lesser plastic...
strain points in the specimens, and increases the load-bearing capacity and prosperity of the retrofitted specimens.

**Author contributions** All authors contributed to the study’s conception and design. The logic related to assuming the required amount of the materials presented in the initial equations of the manuscript was performed by [E.A.]. The leading portion of the experimental studies, numerical modeling, and the initial draft of the empirical and numerical studies are carried out by [F.O.] and the cooperation of [M.N.M.] and [E.A.]. The modification and preparing the final tables, figures, and modified numerical studies have been done by [M.N.M.]. The paper preparation and adjustment were carried out by [B.R.] and [E.A.]. The early draft of the manuscript is prepared by [E.A.], and all authors commented on previous versions of the manuscript. All authors read and approved the final manuscript.

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**Data Availability** Since some outcomes of this study didn’t publish in any other Journal, all data, models, and the results that support the findings of this study are available from the corresponding author upon reasonable request.

**Declaration**

**Competing interests** All authors certify that they have no affiliations with or involvement in any organization or entity with any financial interest or non-financial interest in the subject matter or materials discussed in this manuscript.

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