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Research Article

Vertical Rods as a Seismic Reinforcement Technique for Rammed Earth Walls: An Assessment

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Rammed earth (RE) is a construction material which is manufactured by compacting soil by layers within a formwork to build a monolithic wall. RE material is the subject of numerous scientific researches during the last decade because of the significant heritage of RE buildings and the sustainable properties of this material: low embodied energy, substantial thermal inertia, and natural regulator of moisture. The seismic performance of RE buildings is an interesting topic which needs to be thoroughly investigated. This paper presents a numerical study which assesses the relevancy of a seismic reinforcement technique for RE walls by using two vertical steel rods installed at two extremities of the walls. The discrete element method (DEM) was used to model unreinforced and reinforced RE walls. These walls were first loaded with a vertical stress on the top to simulate the vertical loads and then submitted to a horizontal loading on the top to simulate the seismic action. Two current cases of RE buildings were investigated: one-storey and two-storey buildings. The results showed that the reinforcement technique enhanced the maximum horizontal force about 25% and 10%, respectively, for the cases of one- and two-storey buildings. Higher effectiveness of this reinforcement technique is expected for RE materials having higher compressive strength, for example, stabilized RE.

1. Introduction

In the past decades, vernacular construction techniques (e.g., bhatar, gabion boxes, cator, and cribbage) have been objects of different scientific researches by academics and professional societies [1–4]. Among these techniques, rammed earth (RE) is also an object of several studies [5–15]. RE walls are made by compacting earth in vertical formworks (wooden or metal panels). The earth is compacted into layers having approximately 15 cm thick by using a manual or pneumatic rammer. With other techniques of earthen constructions, RE has a long and continuous history in several regions throughout the world [16, 17]. It is interesting to point out that RE constructions are adopted both in developed countries (such as France, Portugal, Spain, Austria, and other European countries) and in developing countries (such as Nepal, Vietnam, Asia, and Costa Rica and South America) where different forms of nonengineered building techniques may also present, and the seismic performance of these structures can be a serious concern. In the context of sustainable development and preserving the heritage of RE buildings, several studies have recently been conducted to investigate RE because this material possesses a low embodied energy and an interesting hygrothermal behavior [18, 19]. For the seismic performance of RE buildings, several recent studies have been conducted during the last years [20–26]; however, numerous aspects still need to be investigated, especially the solutions to enhance seismic performance of RE buildings. This paper presents an investigation assessing the relevancy of the seismic reinforcement technique which uses two vertical steel rods at two extremities of an RE wall. The study was carried out with a numerical model using the discrete element modelling (DEM) and the nonlinear pushover method. The numerical model used was successfully applied in the previous studies [22, 27]; therefore, the present study focuses on the numerical assessment on the reinforcement technique proposed to enhance the seismic performance of RE walls. Two configurations of RE buildings which correspond to the current cases of RE buildings in France [20, 28] were investigated: 1-storey building and 2-storey building; for two cases, in-plane and out-of-plane seismic performances were also investigated.

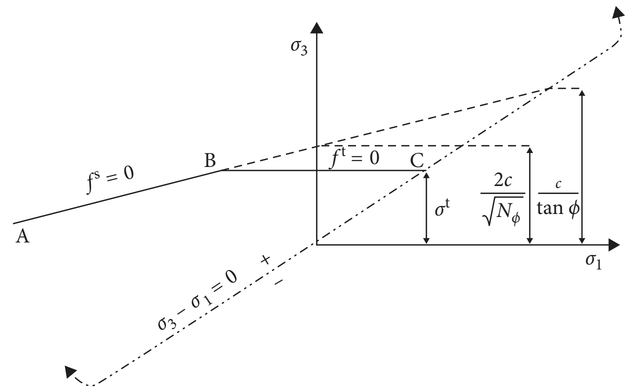


Figure 1: The Mohr–Coulomb failure criterion used for intralayers.

2. Numerical Investigation

2.1. Discrete Element Modelling. Due to the mode of manufacture, an RE wall is a superposition of different earthen layers (called "intralayers"). The discrete element modelling (DEM) is therefore a pertinent approach to simulate RE walls [22] because the behavior of the interfaces between the intralayers can be considered. In the present paper, the 3DEC code [29] was used for the DEM. The RE wall was modelled as an assemblage of discrete blocks (intralayers) where the interlayers (between the layers) were modelled by introducing an interface law.

RE layers, being assumed to be homogeneous and isotropic, were modelled by blocks that were further divided into a finite number of internal elements for stress, strain, and displacement calculations. The failure surface used in this study was the Mohr–Coulomb criterion with a tension cutoff behavior. The Mohr–Coulomb criterion is expressed in terms of the principal stresses σ_1, σ_2, and σ_3, which are the three components of the generalized stress vector for this model ($n = 3$). The components of the corresponding generalized strain vector are the principal strains ε_1, ε_2, and ε_3, in labelling the three principal stresses so that $\sigma_1 \leq \sigma_2 \leq \sigma_3$. This criterion may be represented in the plane (σ_1, σ_3), as illustrated in Figure 1 (where the compressive stresses are negative). The failure envelope $f(\sigma_1, \sigma_3) = 0$ is defined from point A to B by the Mohr–Coulomb shear failure criterion $f^s = 0$ with $f^s = \sigma_1 - \sigma_3 N_\varphi + 2c\sqrt{N_\varphi}$ and from B to C by a tensile failure criterion of the form $f^t = 0$ with $f^t = \sigma_3 - \sigma_t$ where φ is the friction angle, c is the cohesion, σ_t is the tensile strength, and $N_\varphi = (1 + \sin\varphi)/(1 - \sin\varphi)$.

Note that the tensile strength of the material cannot exceed the value of σ_3 corresponding to the intersection point of the straight lines $f^s = 0$ and $\sigma_1 = \sigma_3$ in the $f(\sigma_1, \sigma_3)$ plane. This maximum value is given by $\sigma^t_{\max} = c/\tan\varphi$.

The potential function, g^s, used to define shear plastic flow corresponds to a nonassociated law and has the form $g^s = \sigma_1 - \sigma_3 N_\psi$ where ψ is the dilation angle and $N_\Psi = (1 + \sin\psi)/(1 - \sin\psi)$.

If shear failure takes place, the stress point is placed on the curve $f^s = 0$ using a flow rule derived using the potential function g^s. If tensile failure is declared, the new stress point is simply reset to conform to $f^t = 0$; no flow rule is used in this case.

When the failure takes place in layers, there is no softening or hardening in the postpeak domain and the material corresponds to a perfectly plastic state. In the postpeak (failure) zone, only the strains continue to increase and the stress remains constant such as a plateau.

Interlayers were modelled by an interface law between the blocks according to the Mohr–Coulomb interface model with a tension cutoff [29]. This interface constitutive model considers both shear and tensile failures, and interface dilation is included. The parameters' values used are presented in the next section.

2.2. Principle of Reinforcement by Vertical Steel Rods. When an RE wall is subjected to an in-plane horizontal force, the base of the wall may suffer tractions. Since the tensile strength of RE material is low (about 0.1–0.3 MPa [9, 30]), the traction may lead to tensile damages or "rocking" failures where the base of the RE wall is unbonded from the foundation. Therefore, it is necessary to add reinforcements which enable to prevent the tensile stresses in RE walls. The technique investigated in the present paper is the installation of two vertical ties at the two ends of a RE wall (see Figure 2 for details). For practical applications, the vertical ties can be two steel rods which are placed inside of the RE wall, at two extremities [31]. These rods are also slightly tightened to ensure that the rods can support the tensile stresses appeared in the RE wall. The influences of this "prestress" are investigated in this paper.

However, the solution of reinforcement by vertical steel rods may be limited to new constructions; the insertion and fastening of vertical steel rods inside the vertical walls of existing buildings as retrofitting solution would be very arduous. For the existing buildings, the vertical steel rods can be used by adding these rods outside of the vertical walls and a top beam is used on the top of the wall. This solution was also adopted for a new RE construction recently constructed in France.

The vertical steel rods inserted inside of the RE walls can expose to corrosion risks when clay is in contact with RE material. Therefore, in practice, the vertical steel rods are placed in a plastic pipe. That is the reason why, in the present

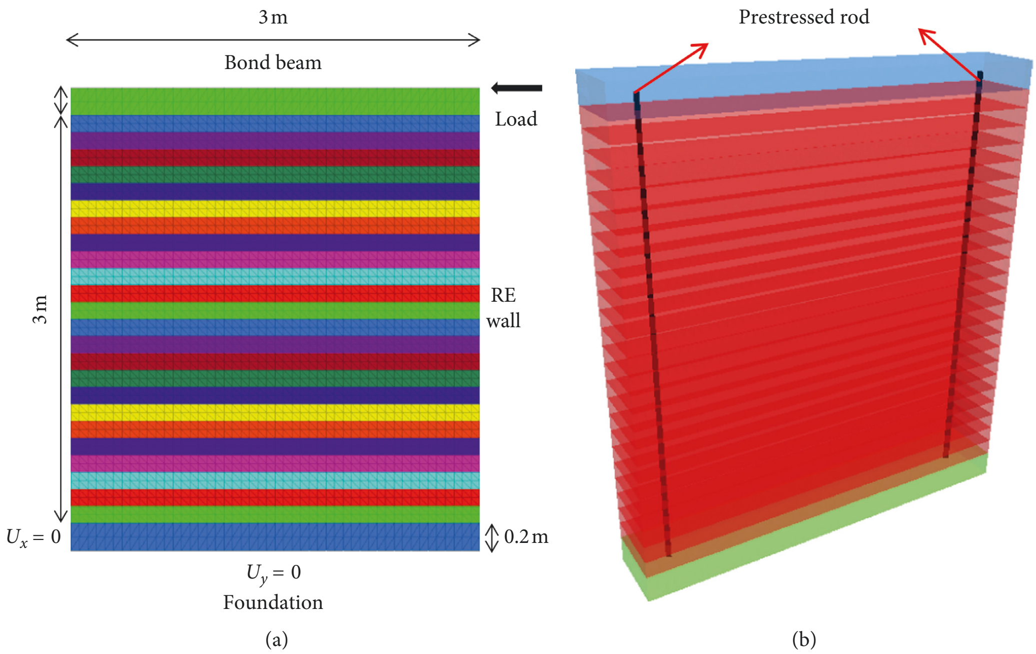


Figure 2: Mesh and boundary conditions of an RE wall.

study, the steel rods are considered unbonded to RE material.

For insitu constructions, the steel rods are slightly prestressed to maximize the effectiveness of this technique. It is worth mentioning that the prestress tension acting in the vertical steel rods used in this study is very small (discussed later), just to ensure that steel rods are in contact and work with RE wall. The prestress level can be verified by measuring the elongation of each steel rod. Indeed, from the elongation, the strain can be determined and therefore the stress is also determined because the steel is in the linear-elastic domain. The influences of the phenomena such as the creep of steel rod and the shrinkage of RE material are less important than the case of conventional prestressed structures; these phenomena are not investigated in the framework of this manuscript.

2.3. Case of One-Storey Building. Several aspects influence the seismic performance of an RE wall. Among these aspects, the most important ones are the dimensions (related to the stiffness) and the mass (due to the wall self-weight and the loads transferred from the upper floors). Rammed earth buildings in France currently have one or two storeys and the wall thickness is about 0.5 m [20]. The storey height is about 3 m; the length of RE walls is about 3–4.5 m. The spans between the load-bearing RE walls is about 6 m. The vertical elements between the walls and on the facades are light elements (wooden and glass infill, in order to benefit the thermal insulation or natural lighting from those materials).

The vertical loads at the top of the RE walls include the dead loads (self-weights of the floors, roof, and walls) and the live loads. The present study investigated an internal wall which had more permanent and live loads than the external walls (due to a higher area of the influence zones from the floor). In general, the vertical stress at the top of an internal RE wall at the ground storey is about 0.1 and 0.2 MPa, respectively, for the case of buildings with one or two storeys.

For the case of one-storey building, an RE wall of 3 m length × 3 m height × 0.5 m thickness was modelled which corresponded to the common cases of RE buildings in France [20, 28]. The numerical wall was constituted of 24 intralayers. A vertical stress of 0.1 MPa was applied on the top of the wall to simulate the dead loads and live loads from the roof transferred to the wall. This vertical stress was maintained constant, and then a horizontal force was applied on the top of the wall, until failure, following the principle of the pushover method which was described in Eurocode 8 [32]. The RE wall was installed on a concrete foundation of 3 m length × 0.2 m height × 0.5 m thickness (Figure 2). This foundation was blocked in vertical and horizontal directions. A second concrete beam of 3 m length × 0.2 m height × 0.5 m thickness was installed on the top of the RE wall.

The compressive strength, Young's modulus, density, and Poisson's ratio of the intralayers were of 2 MPa, 470 MPa, 20 kN/m^3, and 0.22, respectively. These values were taken from experimental results presented in previous studies [26, 33]. Other parameters used in the model followed experimental results and the recommendations presented in the previous studies [27, 34–36] which were successfully applied for numerical models in a previous study [22]. The summary of these parameters is presented in Table 1.

Two vertical steel rods were installed from the midheight of the foundation to the midheight of the top beam; at two extremities of the wall, with the distances of 0.25 m from the three surfaces of the wall. The diameter of the prestressed

Table 1: Parameters of intralayers and interlayers used in the DEM model.

	Tensile strength, f_t	Cohesion, c	Friction angle, φ	Normal stiffness, k_n	Shear stiffness, k_s
Intralayers	133 kPa	133 kPa	45°	NA	NA
Interlayers	113 kPa	113 kPa	38°	60 GPa/m	24.6 GPa/m

steel rods was chosen of 25 mm which is currently used for this technique of reinforcement and has been applied for several real constructions in France [37]. These steel rods were anchored in the foundation and the top beam, with a total length of 3.2 m [38]. As mentioned before, the steel rods were unbonded with the RE wall. The steel rod had an elastic modulus of 200 GPa with an elastic-perfectly plastic behavior and a yield strength of 500 MPa.

For practical reason, when the steel rods are placed within a RE wall, they are slightly put in tension during the fixation procedure to ensure an effectiveness of the reinforcement technique [37]. It is worth mentioning that the prestress in this reinforcement technique is just to ensure that steel rods are in contact and work with RE wall. The prestress level is very small and can vary following each construction. Therefore, three cases were investigated in this paper: Case 1: RE wall without steel rods, which corresponds to an unreinforced RE wall (UW); Case 2: RE wall with two steel rods prestressed at 0.05 MPa (RW-50, the prestress tension is expressed in kPa); Case 3: similar to Case 2, but the prestress in the rods was of 0.10 MPa (RW-100). These prestress levels are very low when compared to the yield strength of the steel rods (500 MPa). These values of prestress are considered as final, long-term values of the prestress acting on the two vertical steel rods.

2.4. Case of Two-Storey Building. For the case of two-storey RE building, a second RE wall of the second storey was directly built on the top of the first storey to create a total RE wall of 6 m height × 3 m length × 0.5 m thickness. No bond beam is present on the top of the first level (at the level of the first floor) because this kind of structure is currently observed in traditional RE houses in France. The interlayer between two walls had the same properties as other interlayers in the RE wall. This configuration is typical for RE buildings in France. A vertical stress of 0.1 MPa was applied on the top of the wall to simulate the dead and live loads from the roof. Another vertical stress of 0.1 MPa was also applied to the wall at the level of the floor to simulate the dead and live loads from the floor of the second storey (Figure 3). These vertical stresses were maintained constant. Then the wall was pushed in-plane at the top with a horizontal force until the failure. In fact, in the case of an earthquake, a second horizontal force could also apply at the level of the first floor, but in the present study, the main scope was to compare the cases with and without steel rods, so this second horizontal force was not simulated for simplification.

For reinforced cases, two steel rods of 6.2 m length × 25 mm diameter were installed from the midheight of the foundation to the midheight of the top beam. The same prestresses as the one-storey RE building (0.05 and 0.10 MPa) were used in this case.

3. Results Obtained

3.1. In-Plane Mechanism. In this section, the in-plane seismic performance of one-storey building and two-storey building is analyzed. With reference to the case of one-storey building and the case of two-storey building, under the hypothesis that disintegration of the RE wall will not occur before the activation of local collapse mechanisms (out-of-plane mechanism).

3.1.1. Case of One-Storey Building. The results obtained for the case of one-storey building is illustrated in Figure 4. The results show that the reinforcement enhances the first crack horizontal force and the maximum horizontal force. By comparing the cases between unreinforced and reinforced with vertical rods prestressed at 0.05 MPa and 0.10 MPa, the maximum horizontal force increased about 22% and 27% (122.8 kN compared to 150.2 kN and 156.4 kN, respectively). The cases reinforced RE walls enhanced considerably the first crack horizontal force compared to the unreinforced walls (increased about 63% and 132% for the case of 0.05 MPa and 0.10 MPa, respectively). From Figure 4, it is worth noting that, for unreinforced RE wall, after the elastic part, there is a nonlinear hardening part followed by a large plastic zone which is quasi-horizontal (with a slight hardening), and this behavior was observed during the experimental tests (see [26], for example). However, for reinforced RE walls, after the elastic and nonlinear hardening behaviors, the "peak" corresponding to the maximum horizontal load was observed; after the peak, a softening behavior was observed. So, the reinforcement technique increased the elastic limit of RE walls but also decreased the displacement corresponding to the maximum horizontal force. In other words, the ductility (ratio between the ultimate displacement and the yield displacement [38]) is decreased when the reinforcement technique is applied.

In the case of unreinforced RE wall for one-storey building, the typical shear failure was not clearly observed (Table 2), and the main failures were due to the slipping at the interlayers at the bottom and the top of the wall. This behavior is due to a small vertical stress (from the dead loads and live loads) in the case of a one-storey building. For the reinforced RE walls, the shear failure (with diagonal damages) was observed and this failure was clearer when the prestress increased; the slipping at the interlayers considerably decreased for the case of 0.10 MPa prestress. This behavior is comprehensible because according to the Mohr–Coulomb theory, when the normal stress increases, the shear strength of the interlayers increases. Thus, the

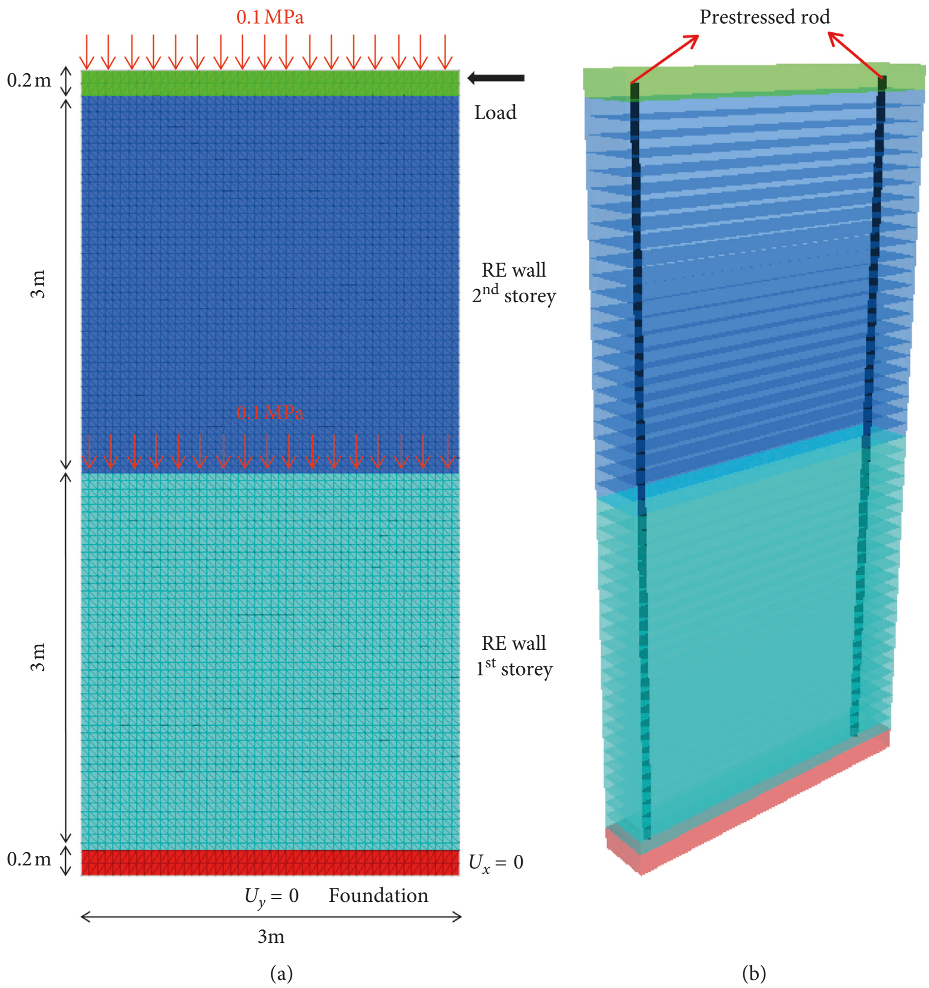


Figure 3: Mesh and boundary conditions of an RE wall.

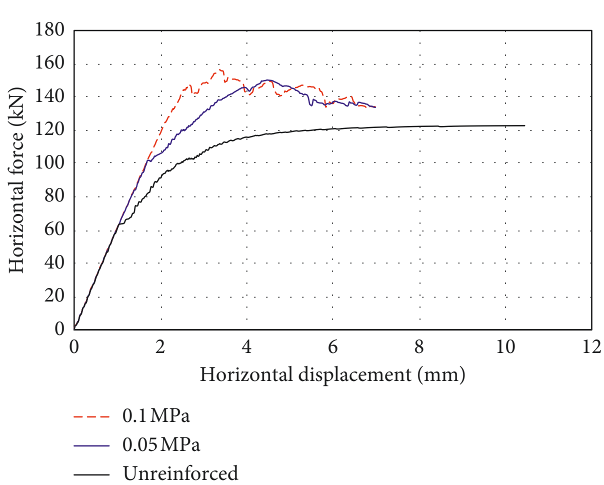


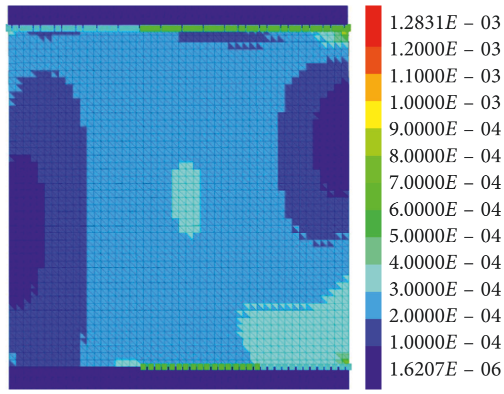
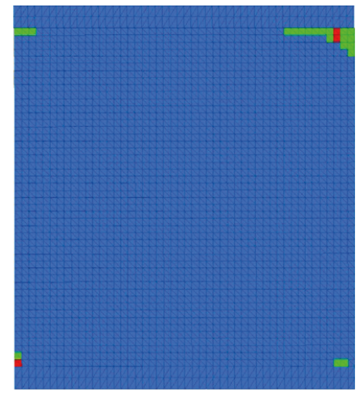
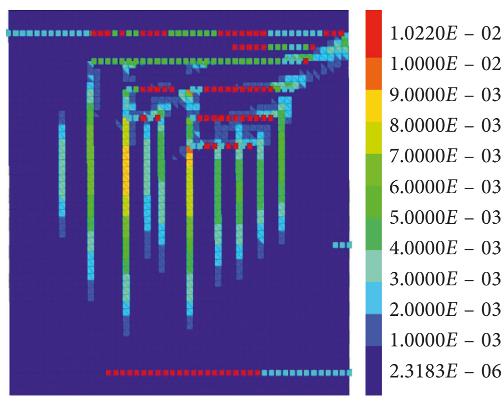
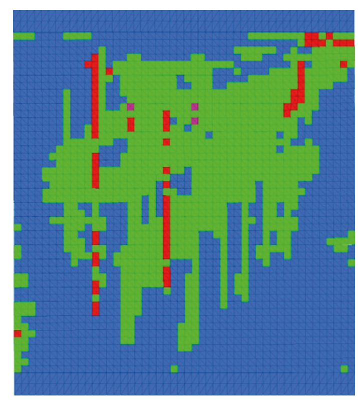
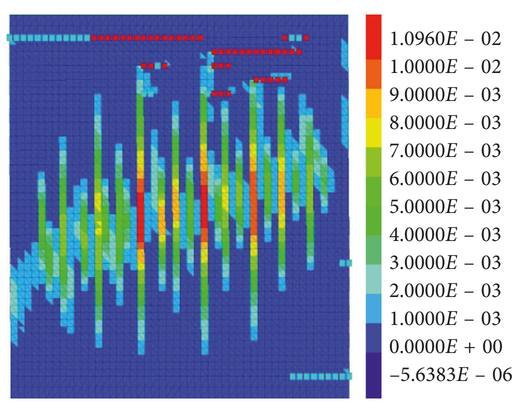
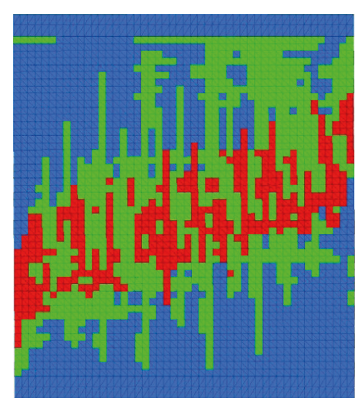
Figure 4: Relationship between horizontal force and horizontal displacement for the one-storey RE wall.

main failure observed in the case of reinforced RE walls was the damage due to the compression in the diagonal zone.

3.1.2. Case of Two-Storey Building. The results obtained for the case of two-storey building is presented in Figure 5 and Table 3. Some similar results obtained for the one-storey building are also noted in this case: the reinforcement enhanced the first crack horizontal force (increased about 49% and 177% for the case of 0.05 MPa and 0.10 MPa, respectively) and the maximum horizontal force, but the reinforcement also decreased the displacement corresponding to the maximum horizontal force. However, the improvement of the reinforcement in the maximum horizontal force was less clear than the previous case: the maximum horizontal forces were of 105.4 kN, 118.8 kN, and 113.8 kN, respectively, for the case of unreinforced RE wall and reinforced with vertical rods prestressed at 0.05 MPa and 0.10 MPa, which corresponds to increments of 13% and 7%, respectively. This phenomenon can be explained by the fact that higher prestress level induces an early compression failure of the wall material due to its limited compression strength.

It is also important to note that a prestress of 0.10 MPa did not increase the maximum horizontal force for the case of two-storey building when compared to a prestress of 0.05 MPa (113.8 kN and 118.8 kN, respectively); the

Table 2: Damages and force obtained (the pushing direction is the same as in Figure 2).

Walls	First crack force (kN)	Max. force (kN)	Max. principal strain and joint slip	Damage state in earthen block
Unreinforced	62	122.8		
Reinforced with 0.05 MPa prestress rods	101	150.2		
Reinforced with 0.1 MPa prestress rods	144	156.4		

difference was less than 5% for the maximum horizontal forces of these two cases. The behavior of a reinforced RE wall is analyzed in detail in Figure 6. From this figure, it is observed that the cracking starts at the corners of the diagonal strut (point A), then the damage developed in the inclined direction (points B and C) with an angle about 45–50° is compared to the horizontal axis; the reinforced wall reached the maximum force at point C when the inclined crack crossed the length of the wall. It is interesting to note that for the two-storey wall, the main crack was not diagonal but follows an angle of 45–50° compared to the horizontal direction. After a fall at point C, the curve had a slight hardening behavior (points D and E) with the apparition of vertical cracks.

3.2. Out-of-Plane Mechanism. In the structural design against horizontal loads (wind or earthquake), the out-of-plane capacity of the walls is usually neglected, and the horizontal loads are often assumed to be supported by the

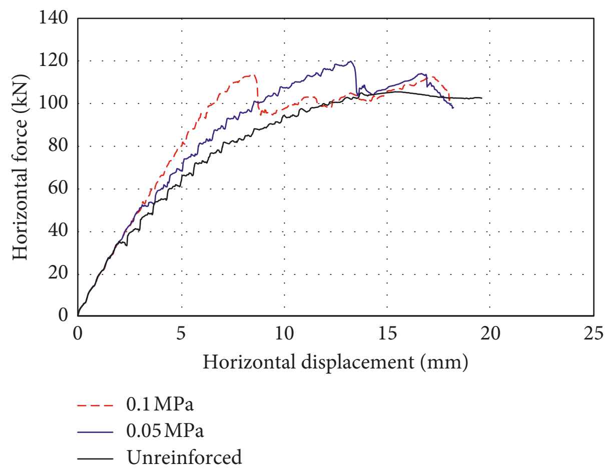


Figure 5: Relationship between horizontal force and horizontal displacements for the case of two-storey building.

walls in the direction of the load (in-plane behavior). In seismic design of the masonry buildings, a "closed-box" behavior of the building is recommended, which means the absence of the out-of-plane mechanism of the structural walls. In that case, the global seismic response of the building is analyzed by taking into account the in-plane mechanisms of the walls to resist the horizontal loads. In this regard, if torsional effects are negligible, the horizontal inertial load is supposed to be absorbed by the walls with the vertical plane parallel to the seismic load direction. Nevertheless, the out-of-plane is one of the modes of failures during earthquakes if the structure is not correctly designed. Therefore, it is interesting to assess the effects of the reinforcement studied on the out-of-plane seismic performance of RE walls. For this purpose, the out-of-plane loading was also investigated in this study but only for the case of one-storey wall.

The one-storey walls had the same properties as the case of in-plane horizontal loading presented above. A vertical stress of 0.10 MPa was applied on the top of the wall to simulate the dead loads and live loads from the roof transferred to the wall. This vertical stress was maintained constant. Then the wall was pushed out-of-plane at the top by a lateral force until the failure. This out-of-plane load can represent the wind or earthquake in the structural design. Three cases were also tested: without reinforcement and reinforced with vertical rods at 0.05 MPa and at 0.10 MPa. The results of the out-of-plane loading are illustrated in Figure 7 and Table 4.

From Figure 7(a), it is observed that the three cases tested had the same slope for the initial part, then the unreinforced RE wall presented a maximum load which was followed by a softening postpeak behavior. The reinforcement technique had clear effects where a hardening and ductile behavior were noted. A higher prestress level (0.10 MPa compared to 0.05 MPa) increased the maximum horizontal force.

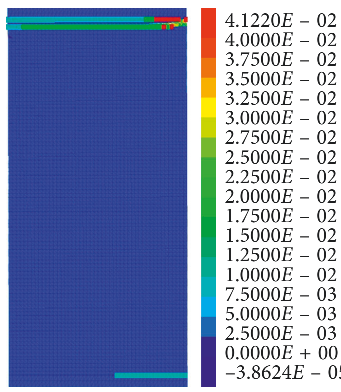
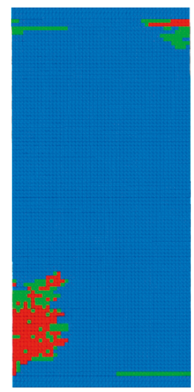
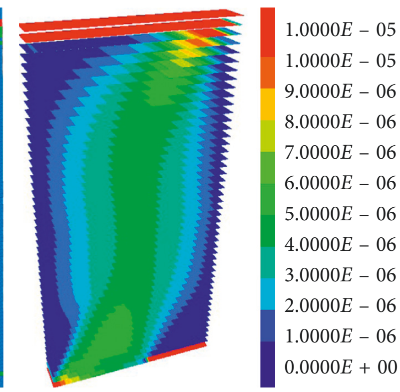
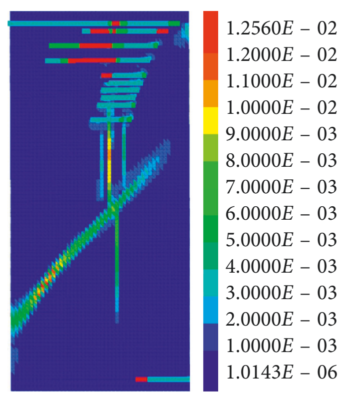
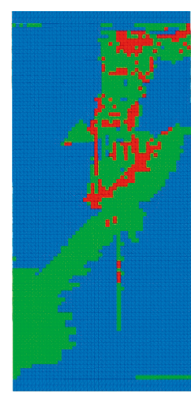
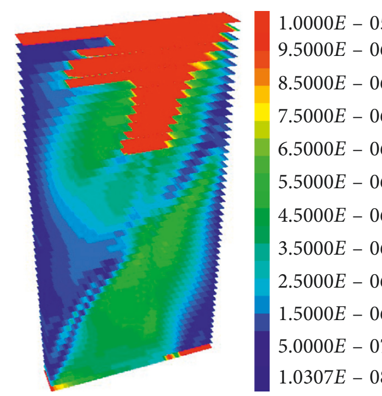
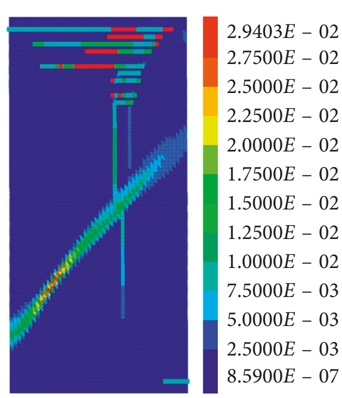
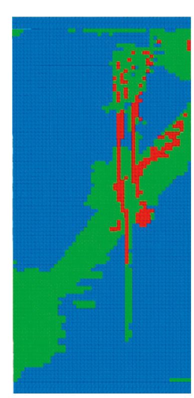
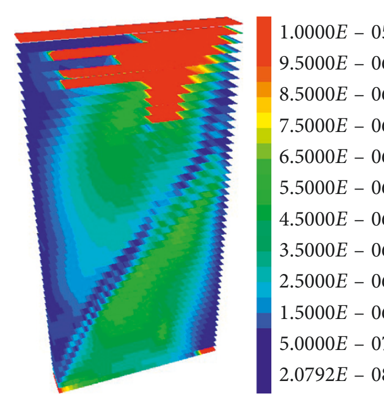
From Table 4, it is observed that the main failure of the out-of-plane loading case is related to the traction at the interlayers situating at bottom and top of the wall. Initially, the vertical steel rods which are placed at the neutral axis did not have considerable effects. Then, with the cracking due to the debonding of the interlayers at the bottom and the top, the neutral axis at these locations changed, which enabled the vertical steel rods to support the tensile stress. That was the reason why the reinforced walls had better performance than the unreinforced one, and the wall with the highest prestress level in the rods had the best result.

4. Conclusion and Outlook

This paper analyzes the numerical assessment of the effectiveness of a reinforcement technique adopting prestressed vertical steel rods on the maximum horizontal force of RE walls. The reinforcement technique consists of installing two vertical steel rods at two extremities of the wall. The in-plane seismic performance was investigated for the case of one-storey and two-storey walls, with three configurations: unreinforced RE wall, reinforced with vertical rods prestressed at 0.05 MPa, and reinforced with vertical rods prestressed at 0.10 MPa. The results showed that the reinforcement technique enhanced the elastic limit and the maximum horizontal force but also reduced the ductility of the RE walls. For the case of one-storey wall, the maximum horizontal force increased 22% and 27%, respectively, for the case of vertical rods prestressed at 0.05 and 0.10 MPa. The main damage observed was the failure in the compressive strut. Thus, it is expected that, with an RE material having a higher compressive strength, the robustness of this reinforcement technique can be improved.

In the case of two-storey wall (with a height/length ratio of 2), the increase of the maximum force by using the reinforcement technique was in the range of 7–13% which was lower than that of the one-storey case. The main reason was that the main damage was inclined an angle about 45–50° which crossed the length of the wall, and there was not a diagonal damage. If a height/length ratio is like the case of one-storey wall (with a height/length ratio of 2), it is expected that the reinforcement technique can

Table 3: Damages and the corresponding forces obtained (the pushing direction is the same as in Figure 3).

Walls	First crack force (kN)	Max. force (kN)	Max. principal strain and joint slip	Damage state in intralayers	Joint shear displacement (m)
Unreinforced	34.3 kN	105.4			
Reinforced with 0.05 MPa prestress rods	51.4 kN	118.8			
Reinforced with 0.1 MPa prestress rods	95.1 kN	113.8			

have more effects. For low compression strength of the wall material, the application of a certain long-term value of prestress on the vertical steel rods could produce a decrease in the performance of RE walls subjected to horizontal forces. It is observed that higher the compression strength of the wall material, the greater the effectiveness of such reinforcement solution. The application of this solution must not disregard about a mechanical characterization of the wall material (in particular, its compression strength once dried). Further studies on this subject could be interesting.

For the case of out-of-plane loading, the effect of the vertical rods has been found also by improving considerably the maximum horizontal force 67% and 80%, respectively, for the case of vertical rods prestressed at 0.05 and 0.10 MPa. In this study, the walls had a rectangular cross section; further studies on the walls with different cross sections would be interesting to investigate the interaction of in-plane and out-of-plane performances.

The application of vertical steel rods also increases the out-of-plane capacity of RE walls. However, it should be underlined that such solution, ideated to enhance in-plane

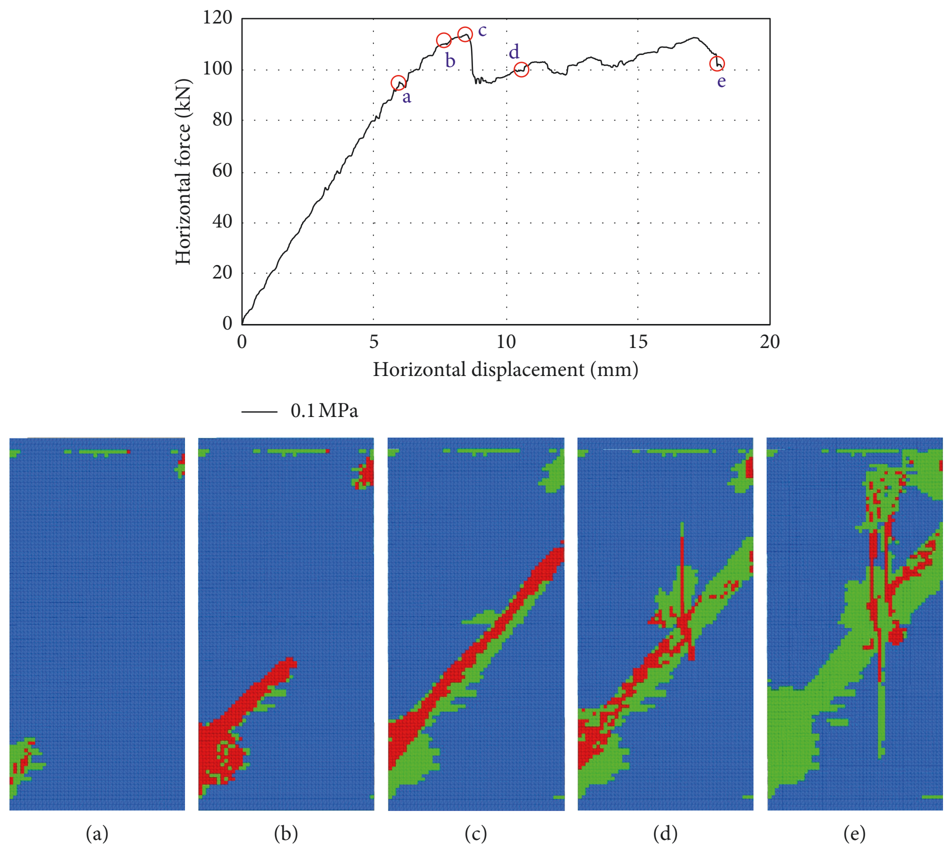


Figure 6: Evolution of damage state in the wall with 0.1 MPa prestress rods: Points (a) 95 kN, 5.96 mm; (b) 110 kN, 7.80 mm; (c) 113 kN, 8.50 mm; (d) 99 kN, 10.60 mm; (e) 101 kN, 18.20 mm.

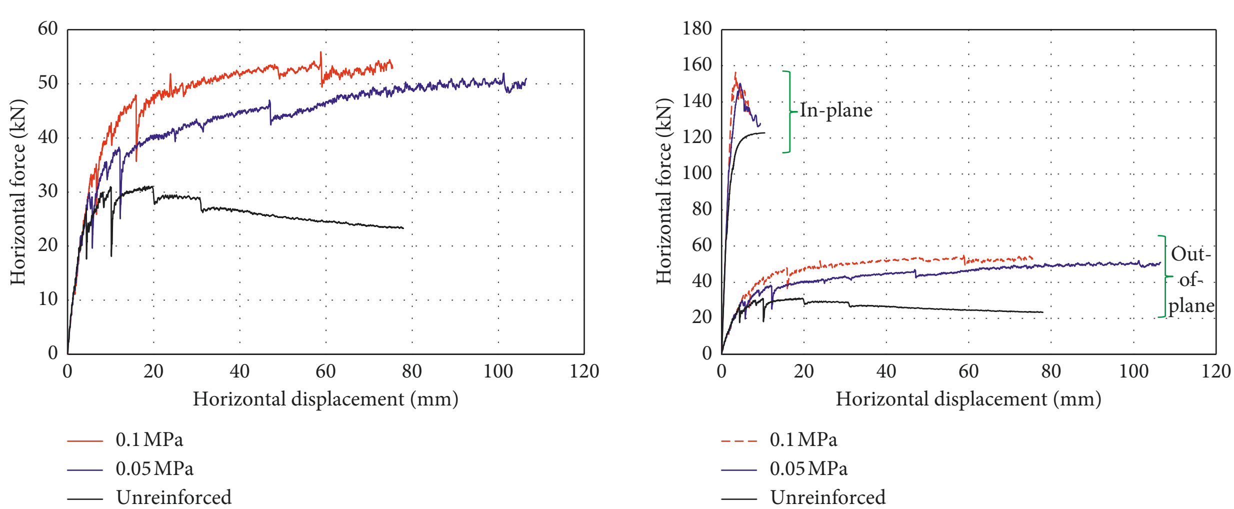
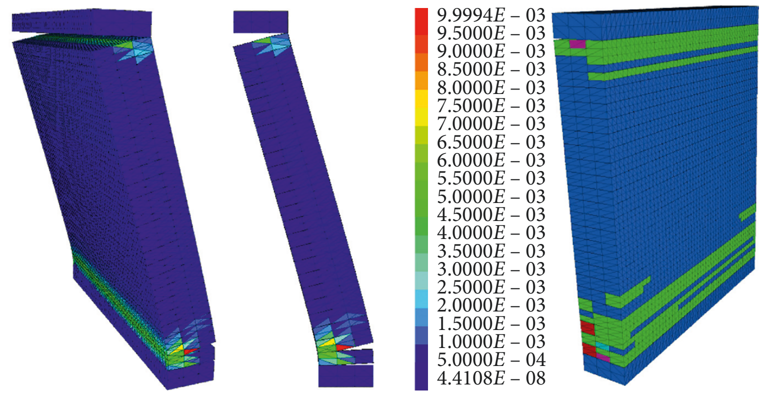
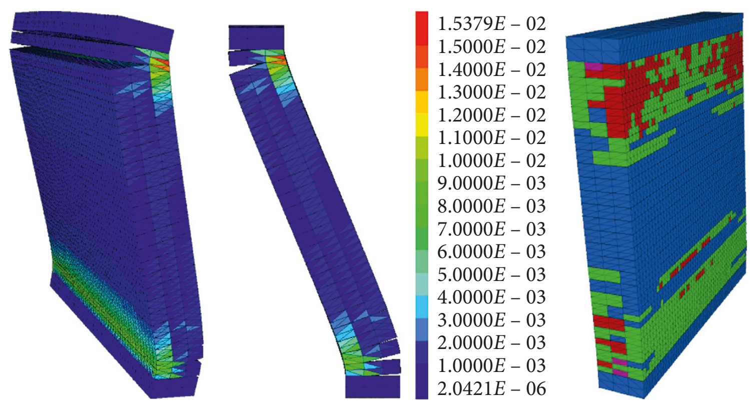
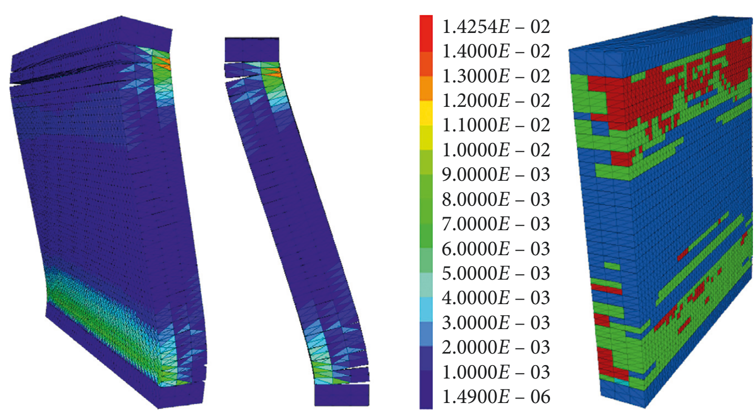


Figure 7: (a) Results obtained for the case of out-of-plane horizontal loading. (b) Comparison between the in-plane and out-of-plane horizontal loading cases.

seismic performance of RE walls, may result efficient only when the out-of-plane mechanism is fully inhibited by means of a proper seismic design of the building and when a "closed-box" behavior is guaranteed with respect to seismic loads (insertion of horizontal steel rods at each floor levels, proper detailing and realization of orthogonal wall intersection, infinitely stiff diaphragm behavior of each floor, etc.). This study focused on one-wall behavior and has not yet considered the overall 3D behavior of the building. Therefore, the influence of the floor has not been

Table 4: Damage evolution in the case of out-of-plane horizontal loading.

Walls	Max. force (kN)	Max. principal strain (deformed factor = 10)	Damage state in interlayers
Unreinforced	31.05		
Reinforced with 0.05 MPa prestress rods	51.95		
Reinforced with 0.1 MPa prestress rods	55.91		

evaluated in this study and can be investigated in further studies [39].

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

Acknowledgments

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