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Anchors to Solid Clay Brick Masonry in Tension: Behavior under Monotonic and Repeated Loading for Constant Embedment Depth

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Abstract: This paper presents a part of an extensive experimental campaign performed at the National Technical University of Athens (NTUA) with the purpose of investigating the behavior of chemical anchors embedded in solid brick masonry. Anchors are tested in tension under monotonic or repeated loading. All tests are performed under displacement-controlled conditions. The experimental setup and the instrumentation are presented, along with the investigated parameters and the rationale for the selection of the values of those parameters. In this part of the experimental work, comprising fifty-six (56) tests, the examined parameters are the anchor locations (in mortar joints and in the center of bricks), the state of the substrate (cracked or uncracked), the width of the crack crossing the anchor or at its vicinity (up to 1.20 mm), as well as the loading history (monotonic or repeated). In the tests presented herein, the embedment length of the anchors is equal to 100 mm. The anchors are embedded in solid brick masonry wallettes, and subjected to a normal compressive stress equal to 0.20 MPa. The observed failure modes are explained, and the overall behavior of anchors subjected to tension is presented and commented upon, along with the effect of the investigated parameters on the measured tension resistance, and on the corresponding displacement.

Keywords: anchors; steel; injection; masonry; solid clay brick; tension; cracking; seismic; repeated loading

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1. Introduction

Chemical anchors with steel elements are frequently used to connect structural elements to masonry. Several applications, such as fixing a steel corbel to a masonry wall, improving the connection between two perpendicular masonry walls, connecting a (timber or RC or steel or composite) floor or roof to masonry walls, etc., require the installation of anchors to masonry. In earthquake-prone areas, anchors are expected to be subjected either to repeated or cyclic loading (tension, shear, or both) and they should be adequately designed to sustain those actions.

The topic is quite complex for several reasons: Masonry is a general term including a vast variety of constituent materials and construction typologies. Furthermore, the bearing capacity of an anchor depends on numerous parameters, namely, the location of the anchor (masonry unit, bed, or vertical mortar joint), its diameter and embedment depth, its distance from adjacent anchors and from the edges of the element, the type of loading and the loading history and, of course, on whether there are cracks occurring due to a seismic event in the vicinity of an anchor or crossing the anchor location.

Therefore, it is quite understandable that one cannot find in current codes rules of general applicability for the design of post-installed anchors to masonry. Thus, e.g., TR

54 [1] requires specific basic data for anchors to masonry elements made of each specific brick and masonry mortar, via qualification following a European Assessment Documents (EAD). It is noted that an EAD was recently issued [2] covering the case of metal injection anchors embedded in solid brick masonry under seismic conditions. In this technical document, laboratory test procedures are defined for the determination of the resistances, in tension and shear, of anchors installed in masonry members that are subject to seismic loading. These include anchor tests under pulsating tension or alternating shear load in cracked masonry substrate for the simulation of seismic actions and the evaluation of the performance corresponding to the specific type of anchor and masonry substrate tested.

To contribute to this field and, indeed, to the development of adequate specifications, e.g., the above-mentioned EAD, an experimental campaign was planned and executed at the Laboratory of Reinforced Concrete, National Technical University of Athens (NTUA). The presentation of this experimental campaign, counting more than three hundred tests up to now, is articulated in a series of papers. The current paper focuses on the behavior of single metal injection anchors (thread size: M12; embedment length: 100 mm) installed in solid clay brick masonry and subjected to tension loading, either monotonic or repeated. It is noted that the NTUA research comprises the investigation of further significant parameters for anchors in tension, e.g., the embedment depth of anchors, while the shear behavior (monotonic and cyclic) of injection anchors is also investigated. This extensive and systematic work will be completed with guidelines for the design of injected anchors under monotonic and cyclic conditions.

2. Literature Review

The behavior of chemical or grouted anchors in solid clay brick masonry under tension loading was investigated in several experimental campaigns. In most cases, tests were monotonic and performed under load-controlled conditions. Arifovic and Nielsen [3] report the results of seventy-four (74) monotonic pull-out tests, performed under unconfined conditions, in anchors embedded in wallettes free of cracks and made of solid bricks. Three locations were examined for anchors located away from edges and corners, namely, in stretcher, header, and mortar joints (horizontal or intersection with vertical). In the case of anchors close to corners and edges, two locations were examined, namely, in stretcher and in header bricks, with varying distances from the edges. Anchors 10 mm to 16 mm in diameter were tested, whereas the embedment length varied between h_{ef} = 90 mm and h_{ef} = 230 mm. The experimental results have shown that for anchors installed in uncracked bricks (both stretcher and header) and depending on the embedded length, the observed failure modes were pull-out of the anchor, the (cone-shaped) brick breakout, brick pull-out, and a mixed mode involving pull-out of one brick followed by pull-out or pull-through of the anchor. For anchors installed in mortar joints, in most cases, pull-out of the anchor occurred. The edge effect consisted of the splitting of the brick or of the wall.

Dizhur et al. [4], based on field observations of anchor connections (damage surveys of clay brick URM buildings following the 2010/2011 Canterbury earthquakes), have studied the mechanical behavior of horizontal and inclined (22.5° downwards to the horizontal) anchors under monotonic tension loading. Approximately four hundred (400) in situ tests were performed to study the behavior of chemical or grouted anchors installed in existing solid brick masonry buildings. The anchors were installed in the middle of stretcher bricks and the test setup was designed to allow for the formation of an assumed 45o-failure cone. Six epoxy adhesive products from different manufacturers were used and were injected using a proprietary dispensing gun for each product. Cementitious grouts from two different manufacturers were used and were mixed onsite. The anchors had a nominal diameter of 12 mm, 16 mm, and 20 mm and they were examined and installed in various embedment lengths (between h_{ef} = 100 mm and h_{ef} = 400 mm). The results have shown that, in most cases, the anchors exceeded the design capacity assessed by the authors according to existing codes, namely, NZSEE [5] and FEMA [6].

Giserini et al. [7] investigated the tension behavior of grouted anchors installed in masonry wallettes made with solid clay bricks (compressive strength: 62 MPa), autoclaved aerated concrete blocks, and hollow bricks laid in cement-based mortar, M5 in the case of clay bricks, or M10 for walls made with autoclaved aerated concrete (AAC). One hundred eight (108) unconfined monotonic tension tests were performed on anchors, 10 mm in diameter, that were embedded at 90 mm to 160 mm in header or stretcher bricks or in uncracked mortar joints. The experimental results showed that as expected, the resistance was increasing for increasing embedment depths.

Algeri et al. [8] have performed laboratory and in situ monotonic pull-out tests on grouted anchors installed in different types of masonry. Two injection mortar mixes were tested, a cement-based one (compressive strength: 55 MPa) and a lime-based one (compressive strength: 12 MPa). Reinforcing steel bars ($d \approx 20$ mm) were used as anchors and they were installed with a sock into a drilled borehole of $d = 60$ mm to at least two thirds of the wall thickness. Brick and mortar joint locations in uncracked substrate were tested. Within the testing program, one of the investigated parameters, namely, the compressive strength of the mortar, was proven to significantly affect the performance of the anchors. A result of significance, especially for historical masonry structures, is the observed satisfying behavior of anchors bonded to masonry through lime-based grout, compatible with the original materials.

Investigations performed by Muñoz et al. [9] and Ramirez et al. [10] focused on the behavior of different types of anchoring systems (chemical, cementitious grout, and mechanical) installed in stone and solid clay brick masonry. The aim of the work was to identify the most efficient system for use in masonry strengthening applications. Unconfined monotonic pull-out tests were performed on anchors installed in uncracked wall specimens made with granite stones (compressive strength 68.7 MPa) and lime-based mortar (f_{mc} = 1.63 MPa) or solid clay brick walls (brick compressive strength 19.9 MPa and mortar f_{mc} = 1.4 MPa) under a compressive strength of 0.2 MPa. Metallic fasteners (diameter d = 10 mm) were used with an embedment depth of 150 mm (for chemical and grouted anchoring systems) or 85 mm (for mechanical anchorage). The anchors were installed in the central area of the wall specimens, away from the edges, in stone units, or in the middle of stretcher brick (200 mm $\times$ 65 mm $\times$ 95 mm), as appropriate. The cases of anchors perpendicular or inclined with respect to the wall face were investigated. The anchors bonded to masonry exhibited better behavior than those mechanically anchored to the substrate. Finally, anchors embedded in stones exhibited slightly higher tension resistances than their counterparts installed in clay bricks.

Several experimental campaigns focused on anchor applications, such as strengthening the connection of walls at corners of heritage buildings, using steel anchors. In this framework, Paganoni and D'Ayala [11] studied the behavior of grouted anchors under monotonic and repeated tension loading. The monotonic tests were performed on anchors installed in masonry wallettes, having an L-shaped cross-section, while the repeated tests were performed on anchors to masonry panels of a T-shaped cross-section. In both cases, the walls were made of solid clay bricks and lime-based mortar. For the installation of the 16 mm anchors at a depth equal to 350 mm, a hole of 80 mm in diameter was drilled, and a cement-based injection mortar was used in combination with a fabric sleeve. The anchor locations were randomly selected and at least one mortar joint was included in the drilled core. The purpose was to involve more than one brick, thus avoiding the pull-out of a single brick unit. Six unconfined monotonic pull-out tests were performed with a simultaneous application of compressive stress on the wall. In the case of L-shaped walls, this stress was equal to 0.07 MPa (three tests) or 0.70 MPa (three tests), while in the three cyclic tests on T-shaped walls, the compressive stress was equal to 0.08 MPa. The measured tension resistances of the anchors were almost 70% higher than anticipated, based on calculations, while they seemed to be insensitive to the value of the compressive stress applied on the wall. This was mainly attributed to the development of mechanical locking between the anchor and the wall due to the presence of the fabric sock.

Silveri et al. [12] have studied the behavior of grouted anchors under monotonic or repeated tension load. The tests aimed at acquiring data on the use of grouted anchors in applications, namely, the connection of masonry panels (T and L connections). The wall specimens were made of solid clay bricks and lime-based mortar of low compressive strength (0.42 MPa), representative of historic masonry. The anchors were installed in masonry including brick and joint areas using three injection mortars, namely, two cement-based mortars (compressive strength: 49.2 MPa and 59.1 MPa, respectively) and one lime-based (compressive strength: 9.3 MPa), all of them in combination with a polyester fabric sleeve. The anchors were of nominal element diameter d = 20 mm (drillhole equal to 60 mm), whereas two embedment lengths were examined, namely, h_{ef} = 400 mm in monotonic tests and h_{ef} = 400 mm and h_{ef} = 900 mm in tests under repeated loading. The tension tests were carried out with the masonry wall subjected to a vertical compressive stress. Several values of this parameter were examined (between 0.05 MPa and 0.20 MPa), to simulate the conditions of masonry walls at various levels within a building. The findings show that there is no significant dependence of the tension resistance values of the injected anchor for low vertical stress values, varying between 0.05 and 0.10 MPa. On the contrary, for larger compressive stresses (up to 0.20 MPa), the tension capacity of the anchors was increased. As described by the authors of [12], the failure was due to the extraction of a limited portion of bricks sliding along weak mortar joints. It is noted that due to the large diameter of the borehole and the location of the anchors (including both brick and mortar joint areas), a large area around the anchors was affected. To reach conservative results, in the case of repeated tests, the adverse case of low normal stress was chosen. The quasi-static repeated load-controlled tests have shown a reduction of the tension resistance by 25–35%, compared to that measured under monotonic loading.

In the experimental campaign by Cattaneo and Vafa [13], the purpose was to investigate the tension behavior of anchors embedded in masonry, as part of the strengthening of walls using reinforced plaster. Within their campaign, they have tested solid clay brick masonry wallettes in in-plane cyclic shear until severe damage occurs. The anchors, 8 mm diameter reinforcing bars of 200 mm embedment depth (12 mm borehole diameter) were installed to the walls prior to their in-plane loading as part of the strengthening method. The anchors were installed in several locations (brick and joints), where the damage due to in-plane loading of the walls varied. Thus, several anchors had at their vicinity cracks of width ranging between 0.10 mm and 2.50 mm. After the in-plane testing phase, the anchors were tested in tension, and the effect of cracking, as well as that of their location on the wall, was evaluated. The results show a decrease in the tension resistance as the crack width increases. However, no clear correlation of the crack width to the displacement of the ultimate resistance was observed. Moreover, as expected, anchors along the diagonals exhibited the lowest capacity, accompanied by a large scatter of tension resistances and corresponding displacements. On the contrary, anchors that were installed far from the diagonals showed high-tension resistance, and in cases where the crack width was limited (<0.50 mm) (e.g., the bottom of the wall), the scatter of resistance values was also limited. Thus, the authors conclude that the location of the anchor in the wall is a critical parameter, as good performance was observed in the areas of limited damage.

Mirra et al. [14] investigated timber-to-masonry connections under monotonic, cyclic, and dynamic loading. Seven types of connections were investigated. The authors focus on the loading protocol, with the purpose of yielding conservative experimental results. They have investigated the effect of short-duration earthquake signals, long-duration ones, and quasi-static cyclic loading. On the basis of their experimental results, they have concluded that quasi-static cyclic tests are adequate for the assessment of connections to masonry under seismic conditions.

Burton et al. [15] have conducted monotonic, repeated, and impact tension tests on anchors installed in single brick units. Although this testing procedure allows for a large number of tests to be performed within a limited amount of time and with reduced cost, the experimental results cannot be directly compared to those obtained from testing anchors installed in masonry walls and conducted under unconfined conditions. Indeed, under unconfined conditions, the field of stresses around the tested anchor is not affected by the setup, and testing of anchors embedded in walls allows for the occurrence of all possible failure modes. Furthermore, the numerous parameters affecting the behavior in tension are investigated under more realistic conditions.

In most of the aforementioned investigations, the experimental results are used to assess and calibrate equations aiming at predicting the resistance of injection anchors in tension. Thus, Arifovic and Nielsen [3] have compared their experimental results to resistances calculated based on the upper-bound theorem of the theory of plasticity. This comparison was found quite satisfactory. On the other hand, Dizhur et al. [4] have proposed an empirical equation for the calculation of the pull-out capacity of anchors installed in unreinforced masonry. The comparison between calculated and experimental resistances (obtained from 230 tests compiled in a database) has shown that 93% of the calculated resistances were below the obtained test values, with an overall safety factor approximately equal to 2.29. On the contrary, Ceroni and Ludovico [16], assessed equations available in the literature against the database of experimental results on the pull-out resistance of injection anchors, performed by the authors and available in the literature. Their conclusion is that the examined equations are not suitable to safely calculate the tension resistance of injection anchors. Giserini et al. [7] have proposed an equation for the prediction of the tension strength of anchors by defining a stress model of the masonry solid surrounding the anchor. The evaluation of formulae of the literature by Ramirez et al. [10] has proven the inadequacy of those formulae to predict the ultimate tension capacity of anchors in brick masonry, the predictions in some cases overestimating the anchor resistances. Finally, Cattaneo and Vafa [13] have assessed the equations included in the 2016 edition of TR 054 [1] and those included in the literature [17,18]. The authors have confirmed that the available formulations fail to predict the tension resistance of the tested anchors. Moreover, cracking or damage to the wall are parameters that affect the anchor resistance but are not taken into account in the theoretical calculations.

This brief survey of literature related to the behavior of injection and grouted anchors to masonry subjected to tension shows that (a) due to the importance of the subject, there is a significant corpus of experimental results. Several parameters, such as the diameter of the anchor, location in masonry, embedment length, and bonding material are examined, although not necessarily all of them in each of the experimental campaigns. Furthermore, (b) the results from testing anchors under repeated tension loading, although limited in number, clearly show the effect of this adverse loading condition on the behavior of anchors. This is an aspect that needs to be further investigated, in combination with parameters affecting the behavior of anchors in tension (e.g., location and embedment depth of the anchor to masonry, mechanical properties of the base materials, etc.). On the other hand, (c) the effect of cracks in masonry on the behavior of anchors is not systematically investigated. Nonetheless, the occurrence of cracks (either in the vicinity of an anchor or crossing it) is imminent at least in the case of moderate and strong earthquakes. It has to be admitted that (d) even if the behavior of anchors in tension were exhaustively investigated, there would still be the need to design and perform tests simulating real applications, adequate either for new buildings or for existing constructions that need to be repaired or strengthened. Finally, (e) there is a need for adequate models allowing for the safe design of single anchors or anchors being part of an application. The purpose of the entire NTUA experimental campaign, part of which is presented herein, is to acquire data on the tension and shear behavior of injection anchors to masonry, based on systematic investigation of all influential parameters and combinations thereof, to support the development of testing and assessment procedures as the basis for an appropriate anchor qualification, and to propose formulae for the design of anchors in seismic conditions.

3. The Experimental Campaign

3.1. Materials

The selection of masonry units to be used in tests was based on common European building practices. Due to the vast variety of available blocks (solid, horizontally, or vertically perforated, of various geometries of the perforations, and a wide range of mechanical properties), the decision was made to start the experimental program using solid clay bricks. Another reason for this selection was the long European tradition of using ceramic blocks, also reflected in a significant portion of heritage buildings on the continent. A solid clay brick typically used in Greece, one of the most earthquake-prone countries in Europe, was chosen (Figure 1). The brick is 210 mm long, 100 mm wide, and 50 mm thick (nominal values), whereas the mean compressive strength of the brick, f_{bc}, is equal to 36.7 MPa.

Figure 1. Solid clay brick, f_{bc} = 36.7 MPa.

To reach results representative of low-strength masonry and, hence, conservative values of bearing capacity of the anchors, a low-strength mortar (mean compressive strength, f_{mc} = 3.20 MPa, Table 1a) was used in most specimens. Nonetheless, to evaluate the effect of mortar strength on the anchor behavior, a strong mortar (mean compressive strength, f_{mc} = 20.4 MPa, Table 1b) was used in a few cases. The mortar compressive strengths were measured by testing conventional specimens taken during the construction of masonry wallettes. The wallettes were constructed in batches, due to space limitations. Despite the use of the same mix proportions for the mortar, there were differences in the compressive strength of the mortar among batches of wallettes. The respective measured f_{mc} values are listed in Table 1a,b, and accounted for in Section 5.3.1, where the experimental results per test series are presented. It should also be noted that there are inevitable differences between the strength measured on conventional mortar specimens and that in bed joints or in vertical joints of a real wall.

Table 1. (**a**) Mean values of compressive strength of weak mortar per batch of wallettes (construction phases). (**b**) Mean values of compressive strength of strong mortar per batch of wallettes (construction phases).

(a)

Construction Phase	Phase 1	Phase 2.1	Phase 2.2	Phase 2.3	Phase 3.1	Phase 3.2
$f_{mc,\,min}$ (MPa)	-	2.81	1.88	1.75	2.01	2.25
$f_{mc,\,max}$ (MPa)	-	7.63	3.48	2.72	5.12	3.54
$f_{mc,\,mean}$ (MPa)	3.42	4.53	2.84	2.29	3.41	2.89
CoV (%)	16.91	36.27	11.79	13.45	26.92	13.34

Table 1. *Cont.*

(b)			
Construction Phase	**Phase 1**	**Phase 2.1**	**Phase 2.2**
$f_{mc, min}$ (MPa)	-	21.34	10.11
$f_{mc, max}$ (MPa)	-	29.13	27.26
$f_{mc, mean}$ (MPa)	17.78	26.31	17.01
CoV (%)	24.93	6.93	35.25

In the (a), mean compressive strength of all construction phases, f_{mc} = 3.2 MPa. In the (b), mean compressive strength of all construction phases, f_{mc} = 20.4 MPa.

3.2. The Specimen

To simulate, as closely as possible, the real conditions, wallettes of almost square shape (Figure 2) were used for the installation of the anchors. The wall specimens are 1.09 m long, 0.95 m to 1.00 m high, and 0.21 m thick. The nominal thickness of the mortar joints is 10 mm.

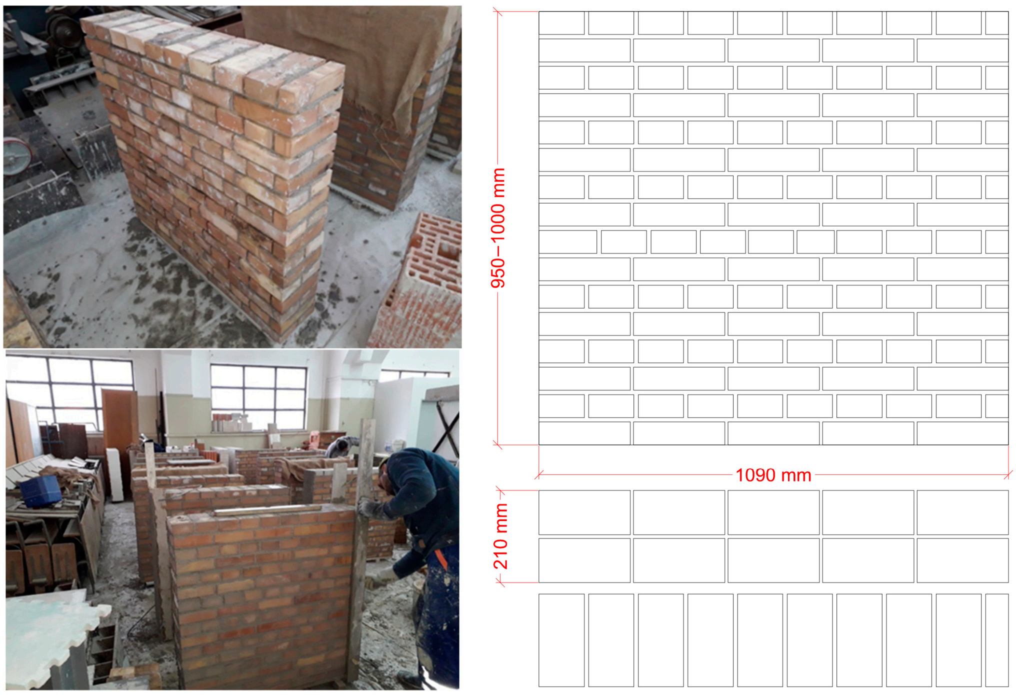


Figure 2. Layout of two-wythe wallettes.

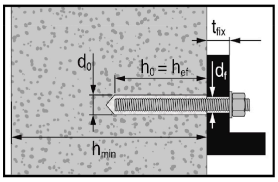
The dimensions of the wallettes were dictated by the decision made to perform unconfined tests in tension. Thus, the reaction ring sitting on the wall (see Section 4.1) had to be of a diameter at least four times the embedment depth of the anchor, to allow for unobstructed development of all possible failure modes.

3.3. Investigated Parameters

To limit the large number of parameters affecting the behavior of anchors to masonry and to, thus, keep the required tests to a reasonable number, some of the characteristics were kept constant throughout the experimental program. Thus, (a) a unique thickness of mortar joints (10 mm) was investigated. It is known that this is a significant parameter, especially in historic masonries, where—quite frequently—the mortar joints are thick. (b) Exclusively 12 mm chemical anchors were used in a drill hole of a nominal diameter equal to 14 mm. The anchors consisted of threaded steel rods of 8.8 strength class (f_{uk} = 800 MPa, f_{yk} = 640 MPa) bonded to masonry using the Hilti HIT-HY 270 [19] injection system. The main installation parameters are listed in Table 2. (c) A unique value of embedment depth, equal to 100 mm, is investigated in this part of the project, whereas a quite wide range of values for this major parameter is examined in further parts. Furthermore, (d) the decision was made to exclude edge effects and, thus, all tested anchors were installed away from the edges of the wallettes (at least 1.5 times the anchor embedment depth, h_{ef}). Finally, (e) one single value (0.2 MPa) of normal compressive stress was applied to all tested wallettes. The selected value roughly represents the compressive stress at the base of the walls of a two- to three-story building and it is equal to the value recommended by EAD 330076-01-0604 [2].

Table 2. Installation parameters of threaded rods in solid bricks ETA19/0160 [19].

Threaded Rod M12	
Nominal diameter of drill bit, d_0 (mm)	14
Drill hole depth = embedment depth, $h_0 = h_{ef}$ (mm)	50. . .300
Maximum diameter of clearance hole in the fixture, d_r (mm)	14
Minimum wall thickness, h_{min} (mm)	h_0 + 30
Brush HIT-RB	14
Maximum torque moment, Tmax (Nm)	10



3.3.1. Anchor Location

The overall behavior of an anchor installed in masonry, in terms of maximum capacity, stiffness, deformation at failure, mode of failure, etc., is expected to depend on the exact location of the anchor. Differences in behavior are expected between anchors installed in bricks and those installed in masonry joints. Whether the anchor is installed in a header or in a stretcher may also affect its behavior. It is also reasonable to anticipate that the typically lower quality of the mortar in vertical joints will lead to worse behavior than for anchors installed in a bed mortar joint, presumably of better quality and benefiting from the normal compressive stress on the wall. Nonetheless, for the reasons explained in Section 3.3.2 and after preliminary tests, the decision was made to test at the T-joint location, i.e., at the intersection of a vertical and a horizontal joint. It should also be noted that cracks expected to occur due to a seismic event, typically, pass through mortar joints, as in most cases the mechanical properties of the mortars are significantly lower than those of (solid) masonry units. Thus, cracks in horizontal mortar joints were formed in wallettes, where anchors were to be installed either to bricks or mortar joints.

For the design of the experimental campaign, the following was considered: (a) In practice, one cannot expect and cannot impose the application of chemical anchors to the best possible location of masonry. Therefore, both brick and mortar joint locations were investigated within the program. (b) Although in practice, an anchor may be installed at any point of a brick (along its height or its length), the case of anchors installed in the middle of a brick is examined in this part of the experimental work. Other locations within a brick are examined in subsequent parts of the research. (c) For anchors installed in mortar joints, the most adverse location (after that of vertical joints was excluded) was selected for investigation, namely the intersection of a bed and a vertical mortar joint. Furthermore, (d) the case of cracks occurring due to a seismic event in the vicinity of anchors was given special attention, as described in more detail herein.

The anchor locations accounted for within the work presented in this paper are shown in Figure 3.

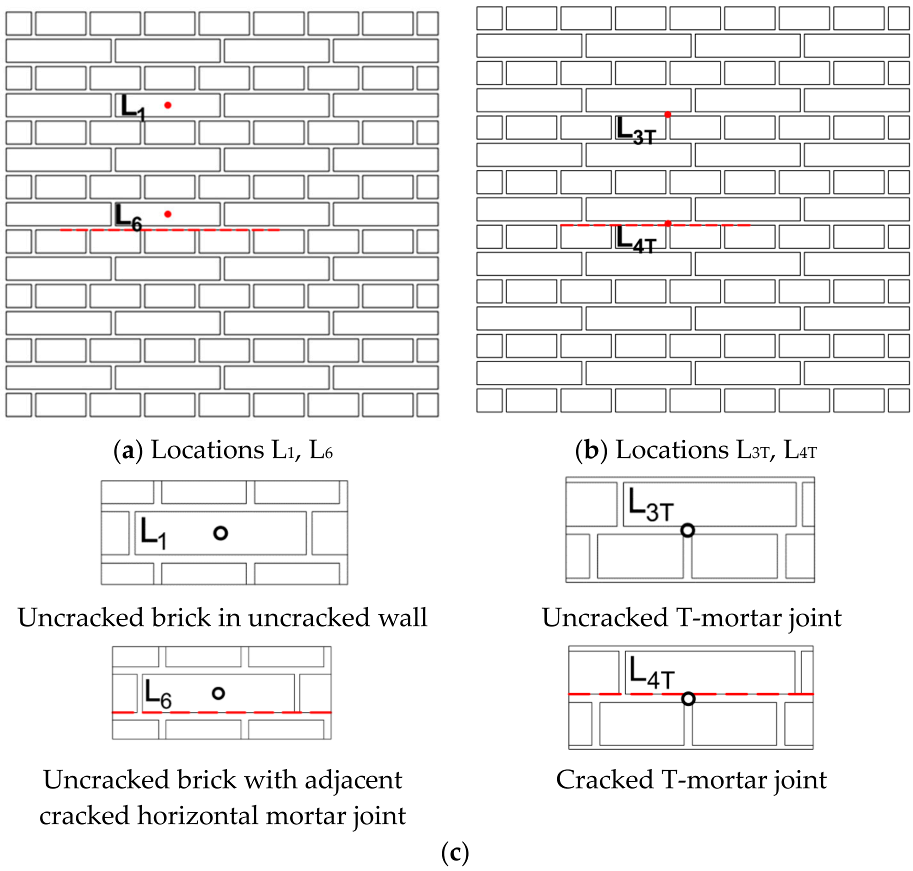


Figure 3. Examined anchor locations: (**a**) in bricks; (**b**) in mortar joints; (**c**) index of adopted locations.

Among the possible anchor locations, L_1 (anchor installed in the middle of a brick, in a masonry element free of cracks) is expected to exhibit the highest resistance. Thus, within the entire campaign, this location serves as a reference for the assessment of the behavior in other locations. An alternative to L_1 is location L_6, where the anchor is installed in the middle of an uncracked brick, but there is a horizontal crack through one of the bed joints adjacent to the brick.

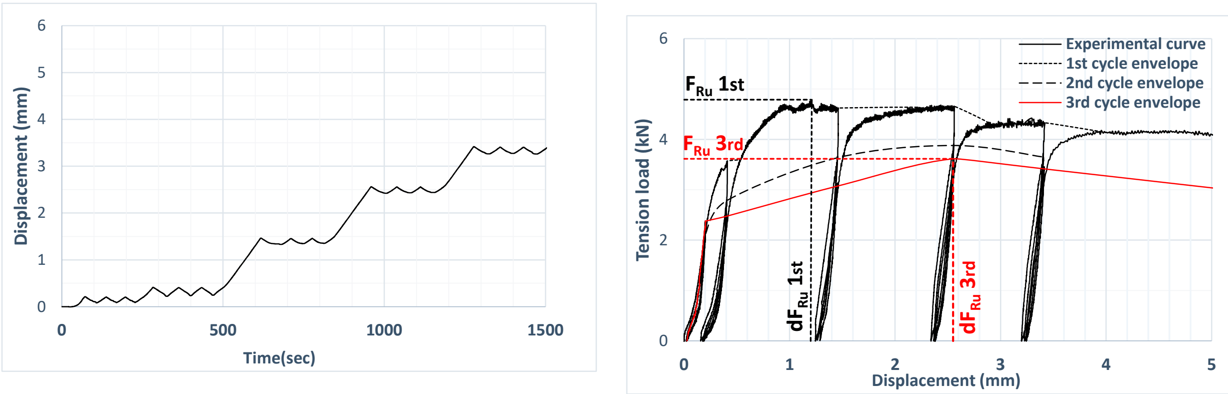
For anchors installed in mortar joints, the reference location is L_{3T}, i.e., the anchor is installed at the intersection of the horizontal and vertical joints, both of them being uncracked. Location L_{3H}, where the anchor is installed in the horizontal joint between two brick units, was also investigated (not shown in Figure 3). Nonetheless, as location L_{3H} was proven to be more favorable than L_{3T}, the latter was exhaustively investigated within the project, as well as its equivalent under cracked conditions, L_{4T}.

3.3.2. Crack Opening

A crack in a masonry wall, either in the vicinity of an anchor or crossing it, is expected to affect the anchor behavior in a negative way. This is more so under seismic conditions, where the effect of cracking is combined with resistance and stiffness degradation due to repeated or cyclic loading. To assess the effect of cracking, several crack width values were investigated, namely, 0.30 mm, 0.50 mm, 0.80 mm, and 1.20 mm. Those tests have yielded the expected results regarding the negative effect of larger cracks on the behavior of anchors. However, the question arose as to whether a limiting value could be adopted for the crack width. It is noted that this is a parameter that also needs to be defined in the framework of the qualification of an anchor (compare qualification of fasteners in concrete, e.g., EAD 330499 [20]). Of course, such a limiting value should be related to the expected crack pattern of a masonry wall subjected either to in-plane or out-of-plane actions during a seismic event, to the reliability of such a prediction by the designer and—as a result—to the avoidance of the installation of anchors to regions where extensive and severe damage of the wall is expected to occur. Those issues are treated in Vintzileou et al. [21], based on the available literature related to tests on bearing masonry walls subjected to cyclic in-plane shear or out-of-plane bending. It is noted that, although numerous relevant tests are published, the authors of the respective works do focus on the crack pattern and its development, rather than on measuring the respective crack widths. Thus, the evaluation of the published results was in several cases qualitative. Nonetheless, the adoption of conceptually sound criteria has allowed for the deduction of a conservative, still realistic crack width value for testing anchors, within the research and qualification process. The two adopted criteria are the following: (a) The designer will avoid (following adequate guidelines) installing anchors in regions expected to undergo significant cracking or even disintegration during the design earthquake, e.g., the region along the diagonals of a wall subjected to in-plane shear. (b) Even if those vulnerable regions are avoided, one cannot exclude the occurrence of large cracks for large deformations imposed on the structural member bearing the anchors. However, it is reasonable to accept that the anchor reaches its maximum resistance simultaneously with the structural masonry element. In light of the qualitative and quantitative assessment of the experimental data, a crack width of 0.50 mm was adopted in most of the experiments presented herein. This is also the value recommended by the authors for use within an anchor qualification test program. Nevertheless, in this paper, the effects of crack widths other than 0.50 mm are presented and commented upon. Finally, (c) after the occurrence of oblique cracks to walls subjected to in-plane shear deformations, the vertical portions of the cracks (passing through vertical mortar joints) increase at a much higher rate than the horizontal portions of the oblique cracks. Nonetheless, it seems that one masonry unit away from the diagonal, even the opening of vertical cracks is not excessive [21]. However, due to the limited relevant quantitative experimental data, and considering the lower quality of vertical mortar joints, it may be wise to encourage the designers to avoid as much as possible the installation of anchors in vertical mortar joints.

3.3.3. Loading History

The anchors were tested either under monotonic or under repeated imposed displacements. The loading history adopted by numerous researchers worldwide for the investigation of the seismic behavior of elements and subassemblies, as well as by ASCE/SEI 41-171 [22], SEAOC [23], FEMA356 [24], and described in guidelines (e.g., ACI 374.2R-13 [25]), has served as a basis for this experimental work. The chosen loading history, schematically shown in Figure 4a for tension, prescribes (a) the execution of displacement-controlled tests, with the purpose of reliably recording the post-peak behavior of the specimens as well; (b) the selection of displacement/deformation steps (based on monotonic test results) and the execution of three cycles at each step before moving to the next step, until failure or until significant response degradation is recorded. In this way, the relevant properties under seismic actions, namely, the force response and stiffness degradation due to cycling, the falling branch of behavior diagrams, etc., can be reliably recorded and assessed. Indicatively, typical hysteresis loops obtained within this work, following the aforementioned rules, are presented in Figure 4b. The same figure shows that during unloading, the tension in the anchor drops to zero and, thus, there is a residual displacement between the anchor and the substrate, increasing as the displacement steps increase in magnitude. Therefore, the anchors are exclusively under tension throughout testing.



(a) (b)

Figure 4. (**a**) Loading pattern adopted for repeated tension tests. (**b**) Typical load vs. displacement curve obtained according to the load protocol of (**a**). Anchor in location L_{4T}, weak mortar.

4. Test Setup, Instrumentation, and Crack Formation

4.1. Experimental Setup and Instrumentation

The test setup is shown in Figure 5a. The wall is placed in the horizontal position on a steel table, between two stiff steel beams (HEB180), interconnected through four steel rods 27 mm in diameter (quality S275 steel). An actuator (1), fixed on a steel column (A), imposes the compressive stress of 0.20 MPa on the wall, held constant throughout testing. The compression load is uniformly distributed to the wall, thanks to the rigid steel beam between the wall and the actuator. After installation of the anchor and opening of the crack (for locations L_6 and L_{4T}, according to 4.2), a steel ring (Figure 5b) is positioned on the wall. The steel ring is designed with an internal diameter of 400 mm to allow for unconfined testing of the anchors. Actuator 2 (max. capacity equal to 100 kN) is fixed at the top of the ring in a perfectly vertical position. Its axis coincides with the longitudinal axis of the ring, allowing for the alignment of the ring with the axis of the anchor to be tested. The actuator is connected to the anchor by means of an adaptor device shown in Figure 5b. Displacements are imposed at a rate of 0.5 mm/min approximately. The displacement of the tested anchor is measured by two LVDTs (Figure 5b), of maximum measuring length equal to 50 mm (accuracy 0.05 mm), placed symmetrically to the anchor axis. The crack opening was measured using two LVDTs (Figure 5a) of measuring length 25 mm (accuracy 0.025 mm), attached to the wall surface symmetrically to the anchor axis and orthogonal to the crack plane.

As the walls were constructed in a vertical position, an adequate procedure should be followed to ensure their safe transportation and rotation to a horizontal testing position, to avoid their premature uncontrollable cracking. To this purpose, each wallette, before movement to the testing position, was subjected to vertical compressive stress using threaded steel rods. Subsequently, using the bridge crane, the wallette was transported in a vertical position on the testing table and very slowly rotated. Once the wallette was safely positioned, the compressive stress was removed and the wallette was thoroughly inspected to confirm that it was free of cracks. This procedure proved to be efficient.

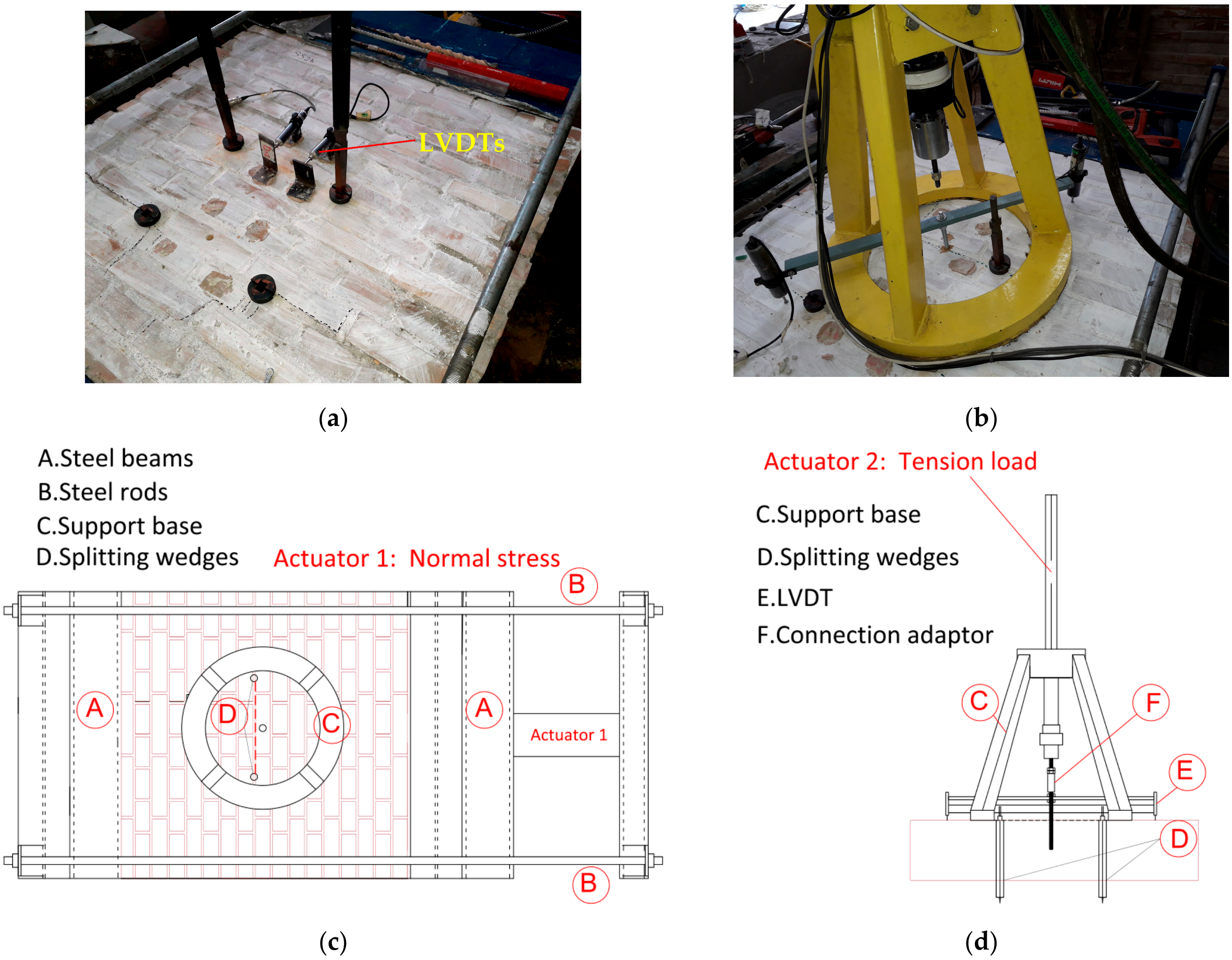


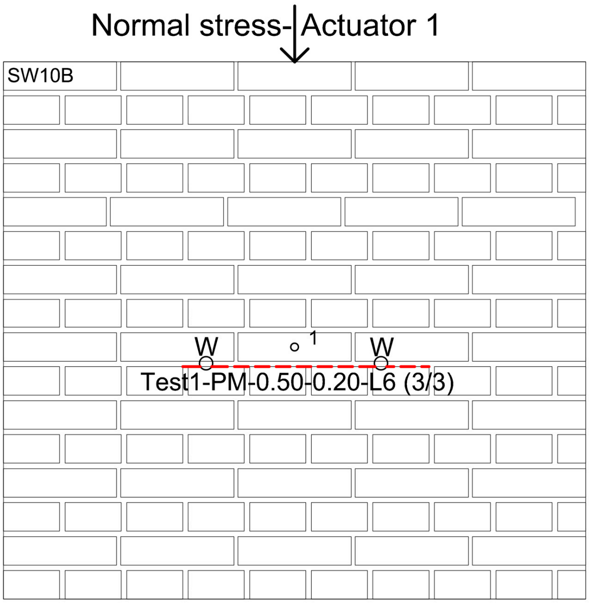
Figure 5. Test setup for anchors subjected to tension: (**a**) a wall in testing position; (**b**) reaction ring and LVDTs; (**c**) drawing of the setup; (**d**) detail of the connection between the anchor and Actuator 2.

4.2. Crack Formation

For the generation of cracks, the methodology described in EAD 330499 [20] for concrete was followed as far as possible, and it was adequately modified for masonry. The crack formation was steered via two sets of steel wedges (Figure 6a). The half-shells of the set were inserted into holes pre-drilled in the wall and the wedges were driven into them through hammering, thus generating a crack and, subsequently, setting it to the desired value (Figure 6b). The wedges with their half-shells are installed along the horizontal mortar joint to be cracked, according to the testing program, at a distance of at least 150 mm on either side of the selected location of the anchor (both for L_{4T} and L_6, Figure 6c). Prior to the opening of the crack, the normal stress equal to 0.20 MPa was imposed on the wall (Figure 5). For tests in cracked mortar joint (L_{4T}), prior to anchor installation a hairline crack was created. After the installation of the anchor and hardening of the injection mortar, the crack was set to the desired width according to the testing program, and the tension test was performed. For tests in L_6, the crack was formed and set to the predefined value in one step, prior to the installation of the anchor to the brick. It is noted that the deliberately created cracks extended through the entire thickness of the wall. This was confirmed by LVDTs located on purpose at the bottom of the wall.

(**a**)

(**b**)



(**c**)

Figure 6. (**a**) Set of steel wedges; (**b**) installed wedge set (W); (**c**) example of wedge locations and respective crack.

5. Experimental Results and Discussion

5.1. Combination of Parameters

The denomination of the tests is codified based on the values of the investigated parameters, as shown in Table 3. At least three tests were performed for each combination of parameters.

Table 3. Test denomination.

Type of wallette–Number of wall specimens and phase within the program *–Number of tests on the same wallette–Type of loading and loading protocol–Crack width (mm)–Value of normal stress (MPa)–Anchor location (Number of test/total number of identical tests)
Type of Wallette: SS: Solid bricks with strong mortar SW: Solid bricks with weak mortar
Protocol: PM: Monotonic test PS: Repeated test according to the displacement-based loading protocol
Crack width: 0.30–0.50–0.60–0.80–1.20 mm

Table 3. *Cont.*

Anchor location: L_1: uncracked brick/uncracked mortar joint L_{3T}: uncracked T-mortar joint L_{4T}: cracked T-mortar joint (intersection of horizontal and vertical mortar joint) L_6: uncracked brick with an adjacent cracked horizontal mortar joint
Example: SS4b-Test1-PM-0.50-0.20-L_{4T}(1/3) The test was performed on an anchor installed in a wall made with solid bricks and strong mortar (SS). It was the fourth (4) wallette of the second (b) phase of the program. It was the first test on the wall (Test 1), performed under monotonic loading (PM). The crack width was equal to 0.50 mm and the normal stress applied on the wall was equal to 0.20 MPa. The location was cracked T-joint (L_{4T}). It was the first out of three identical tests (1/3).

* The serial number of the testing phase within the program is noted for the second and the third phases (using (b) and (c), respectively).

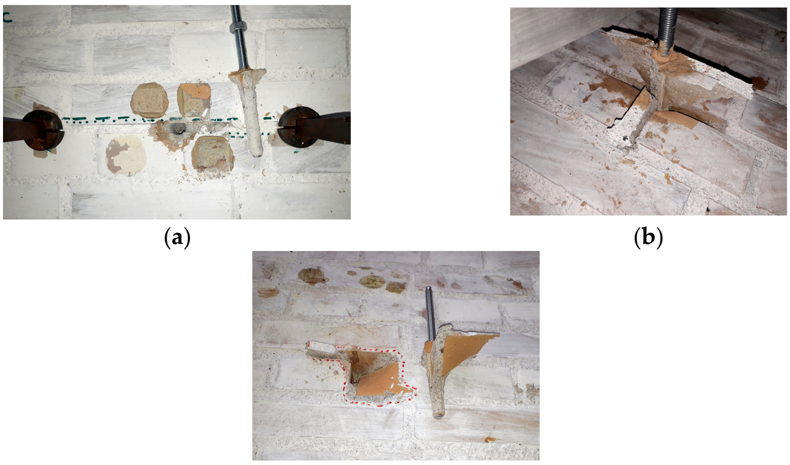
5.2. Failure Modes

5.2.1. Mortar Joint Locations

Independently of the compressive strength of the mortar, the failure observed in (uncracked and cracked) mortar joint locations (Table 4) was typically due to the pull-out of the anchor, surrounded by the injection mortar (Figure 7a). In most cases, in addition to the anchor, a portion of mortar from the adjacent joints was simultaneously pulled out, as shown in Figure 7b. Moreover, in approximately 13% of tests (4 out of 31), a small brick breakout cone occurred in one of the adjacent bricks (stretcher or header), while the anchor was pulled out (Figure 7c). This secondary mechanism led to more extended damage of the area around the anchor, and, in some cases, to increased tension resistance.

Table 4. Typical failure modes observed during tension tests in mortar joint locations.

Failure Mode	Description	Percentage of Occurrence (%)
AP	Anchor pull-out	10
APM	Anchor pull-out with parts of the adjacent mortar joints	77
APB	Anchor pull-out with parts of the adjacent mortar joints and breakout cone of the adjacent brick	13



(**a**) (**b**) (**c**)

Figure 7. Photos of typical failure modes observed during tension tests in mortar joint locations: (**a**) anchor pull-out (AP); (**b**) anchor pull-out with a part of the mortar joint (APM); (**c**) anchor pull-out with breakout cone of the adjacent brick (APB).

5.2.2. Brick Locations

For anchors installed in bricks, a variety of failure modes was observed (Table 5). As shown in Figure 8, the failure modes include the formation of a brick breakout cone, more or less extended, deeper, or shallower (Figure 8a,b), always associated with pull-out or pull-through anchor failure. In many cases, brick pull-out was observed (Figure 8c), whereas in other cases a splitting crack occurred within the brick, accompanied by anchor pull-through failure (Figure 8d). Combinations of failure modes were also observed.

Table 5. Typical failure modes observed during tension tests in solid bricks locations.

Failure Mode	Description	Percentage of Occurrence (%)
BC	Brick breakout cone	11
SBC	Shallow brick breakout cone with anchor pull-out and pull-through failure	3
BP	Brick pull-out failure	56
BS	Brick splitting followed by anchor pull-through	30

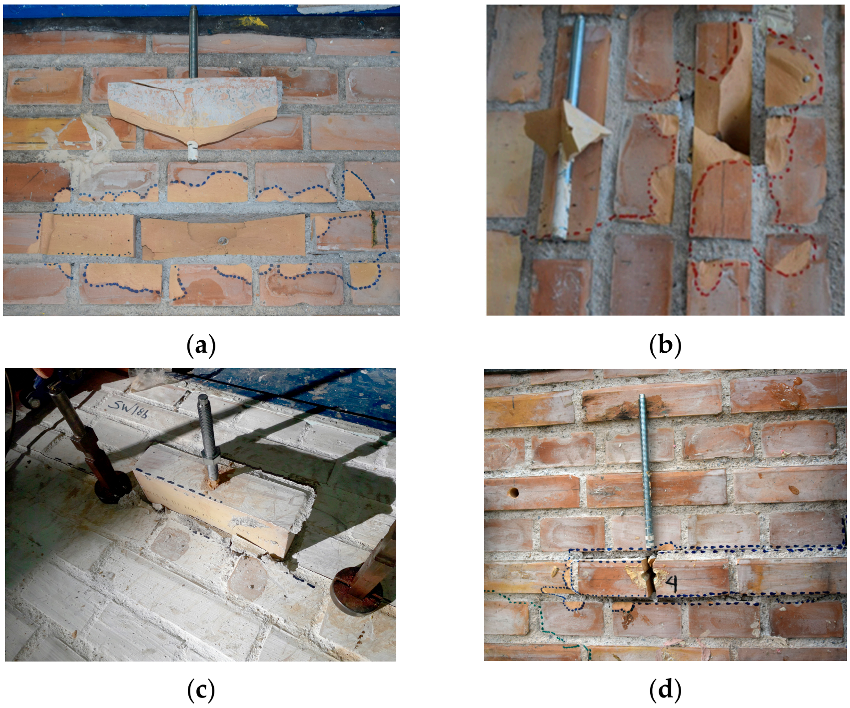


Figure 8. Photos of typical failure modes observed during tension tests in solid bricks in locations: (**a**) brick breakout cone (BC); (**b**) shallow brick breakout cone combined with anchor pull-out and pull-through failure (SBC); (**c**) brick pull-out failure (BP); (**d**) brick splitting followed by anchor pull-through (BS).

Especially in walls made with strong mortar, anchors installed in bricks have exhibited a large variety of failure modes, involving a limited or an extended area of masonry around the anchor, mainly in the uncracked location L_1. This variability in failure modes has led to a rather large scatter of the experimental results, in terms of tension resistance and corresponding displacement, as discussed in Section 5.3.1.

On the contrary, in walls made with weak mortar, local failure modes prevailed. The weak mortar joints enclosing the bricks allowed for brick pull-out to occur (Figure 8c), thus preventing the formation of failure modes affecting a larger area around the tested anchor. Moreover, brick pull-out was the dominant failure mode observed in the case of testing under cracked conditions.

5.3. The Effects of Investigated Parameters on the Behavior of Anchors

A summary of key experimental results obtained from monotonic tests is presented in Table 6.

Table 6. Summary of results for anchors subjected to monotonic loading.

Test Name/Location	Mortar $f_{mc,mean}$ (MPa)	Crack Width (mm)	F_{Ru} (kN)	Mean Value per Series	CoV (%)	d_{FRu} (mm)	Mean Value per Series	CoV (%)	Failure Mode
SS1-Test2-PM-0-0.20-L_1(1/6)			26.70			2.22			BS/BP
SS2-Test3-PM-0-0.20-L_1(2/6)			55.80			0.90			BC
SS2-Test4-PM-0-0.20-L_1(3/6)	17.78	Uncracked	46.85	33.84	42.78	0.60	1.19	77.72	BC/BS
SS3-Test3-PM-0-0.20-L_1(4/6)			21.95			2.49			BS/BP
SS3-Test4-PM-0-0.20-L_1(5/6)			19.67			0.55			BP
SS4-Test1-PM-0-0.20-L_1(6/6)			32.06			0.37			BC
SS1b-Test1-PM-0-0.20-L_{3T}(1/3)			39.79			0.81			APB
SS1b-Test2-PM-0-0.20-L_{3T}(2/3)	26.31	Uncracked	44.49	41.08	7.25	0.62	0.64	26.02	APM
SS1b-Test3-PM-0-0.20-L_{3T}(3/3)			38.97			0.48			APB
SS4b-Test1-PM-0.50-0.20-L_{4T}(1/3)			16.06			4.88			AP
SS4b-Test2-PM-0.50-0.20-L_{4T}(2/3)	26.31	0.50	17.30	18.86	20.32	0.46	2.46	90.89	APM
SS4b-Test3-PM-0.50-0.20-L_{4T}(3/3)			23.23			2.05			APB
SS3b-Test1-PM-0.50-0.20-L_6(1/3)			18.11			0.34			BP
SS3b-Test2-PM-0.50-0.20-L_6(2/3)	26.31	0.50	16.85	17.98	5.96	7.41	2.76	145.95	BP
SS3b-Test3-PM-0.50-0.20-L_6(3/3)			18.98			0.53			BP
SW1-Test3-PM-0-0.20-L_1(1/4)			19.65			1.4			BS/BP
SW1-Test4-PM-0-0.20-L_1(2/4)	3.42	Uncracked	23.37	23.28	22.01	0.19	0.53	110.49	BS
SW2-Test4-PM-0-0.20-L_1(3/4)			19.61			0.24			BS/BP
SW3-Test1-PM-0-0.20-L_1(4/4)			30.50			0.28			BS
SW1b-Test1-PM-0-0.20-L_{3T}(1/3)			21.52			0.25			APB
SW1b-Test2-PM-0-0.20-L_{3T}(2/3)	4.53	Uncracked	19.45	19.75	8.31	1.17	1.66	103.05	APM
SW1b-Test3-PM-0-0.20-L_{3T}(3/3)			18.28			3.57			APM
SW8b-Test2-PM-0.30-0.20-L_{4T}(1/3)			19.22			13.18			AP
SW10b-Test2-PM-0.30-0.20-L_{4T}(2/3)	4.53	0.30	10.44	13.42	37.46	0.60	4.86	148.11	APM
SW10b-Test3-PM-0.30-0.20-L_{4T}(3/3)			10.59			0.81			APM
SW6b-Test1-PM-0.50-0.20-L_{4T}(1/3)			13.48			1.83			APM
SW6b-Test2-PM-0.50-0.20-L_{4T}(2/3)	4.53	0.50	17.64	14.14	22.74	2.57	4.27	84.48	APM
SW6b-Test3-PM-0.50-0.20-L_{4T}(3/3)			11.31			8.42			APM
SW18c-Test1-PM-0.50-0.20-L_{4T}(1/3)			9.17			2.08			APM
SW18c-Test2-PM-0.50-0.20-L_{4T}(2/3)	2.89	0.50	6.44	7.21	23.72	2.66	2.78	27.43	APM
SW18c-Test3-PM-0.50-0.20-L_{4T}(3/3)			6.02			3.59			APM
SCW14-Test1-PM-0.30-0.20-L_{4T}(1/3)			13.59			1.78			APM
SW2b-Test2-PM-0.30-0.20-L_{4T}(2/3)	4.53	0.60	10.68	11.09	20.94	0.62	1.82	67.22	APM
SW2b-Test3-PM-0.30-0.20-L_{4T}(3/3)			9.00			3.07			APM
SW11b-Test1-PM-0.80-0.20-L_{4T}(1/4)			7.78			0.87			APM
SW12b-Test1-PM-0.80-0.20-L_{4T}(2/4)	2.84	0.80	8.39	8.34	46.68	0.88	2.16	78.20	APM
SW12b-Test3-PM-0.80-0.20-L_{4T}(3/4)			3.85			2.45			APM
SW12b-Test3-PM-0.80-0.20-L_{4T}(4/4)			13.34			4.43			APM
SW19b-Test2-PM-1.20-0.20-L_{4T}(1/3)			8.41			0.21			APM
SW19b-Test3-PM-1.20-0.20-L_{4T}(2/3)	2.29	1.20	9.71	7.63	33.47	0.32	0.24	31.15	APM
SW20b-Test1-PM-1.20-0.20-L_{4T}(3/3)			4.78			0.18			APM
SW4b-Test1-PM-0.30-0.20-L_6(1/3)			19.38			0.72			BP
SW4b-Test2-PM-0.30-0.20-L_6(2/3)	4.53	0.30	24.52	20.60	16.89	1.24	0.88	35.01	BP
SW4b-Test3-PM-0.30-0.20-L_6(3/3)			17.89			0.69			BP
SW8b-Test1-PM-0.50-0.20-L_6(1/3)			16.62			1.15			BP
SW8b-Test3-PM-0.50-0.20-L_6(2/3)	4.53	0.50	8.18	12.31	34.32	1.37	1.24	9.30	BP
SW9b-Test1-PM-0.50-0.20-L_6(3/3)			12.12			1.20			BP
SW15b-Test1-PM-0.80-0.20-L_6(1/3)			9.01			2.97			BP
SW15b-Test2-PM-0.80-0.20-L_6(2/3)	2.84	0.80	6.81	8.86	14.94	1.22	1.48	93.66	BP
SW16b-Test1-PM-0.80-0.20-L_6(3/3)			8.86			0.24			BP

Notation: $f_{mc,mean}$: mean mortar compressive strength. F_{Ru}: tension resistance. d_{FRu}: displacement corresponding to tension resistance.

5.3.1. The Effect of Mortar Strength

The strength of the masonry mortar is expected to affect the behavior of anchors in tension, independent of whether the anchors are installed in brick or mortar locations. Indeed, as shown in Section 5.2, the mortar strength affects the failure mode and, by way of consequence, both the anchor resistance and the corresponding displacements. The effect of mortar strength is evaluated based on the average compressive strength of the weak and the strong mortar (3.2 MPa and 20.4 MPa, respectively).

Figure 9 shows the load–displacement curves for two sets of tests performed on anchors installed in an uncracked mortar joint (L_{3T} location). The only difference between the two sets was the compressive strength of the mortar. The data show that (a) on average, the tension resistance of anchors installed in mortar joints of average compressive strength = 20.4 MPa was approximately twice as high as the tension resistance of those installed in mortar with low compressive strength (=3.2 MPa). Furthermore, (b) this higher resistance was activated for significantly smaller displacement. It is noted though that (c) despite the variability of mortar strength from wall to wall and, expectedly, also from joint to joint, the scatter of the obtained tension resistances is limited. However, (d) this is not the case for the displacement corresponding to the ultimate tension resistance; this property presents quite a large scatter, which is more pronounced for low-quality mortar. Finally, (e) it is interesting to observe that if one assumes that the tensile strength of mortars (as is the case for concrete) is proportional to the square root of their compressive strength, the ratio $\sqrt{3.2}/\sqrt{20.4}$ is equal to 0.40, i.e., rather close to the ratio obtained for the corresponding tension resistances (0.48 on average). This result is attributed to the fact that the tension resistance of anchors, in case of the governing pull-out failure, depends on the bond properties between the anchor and the masonry mortar. It is known that the tensile strength of the substrate (i.e., the mortar of the joint) is an estimator of the bond resistance.

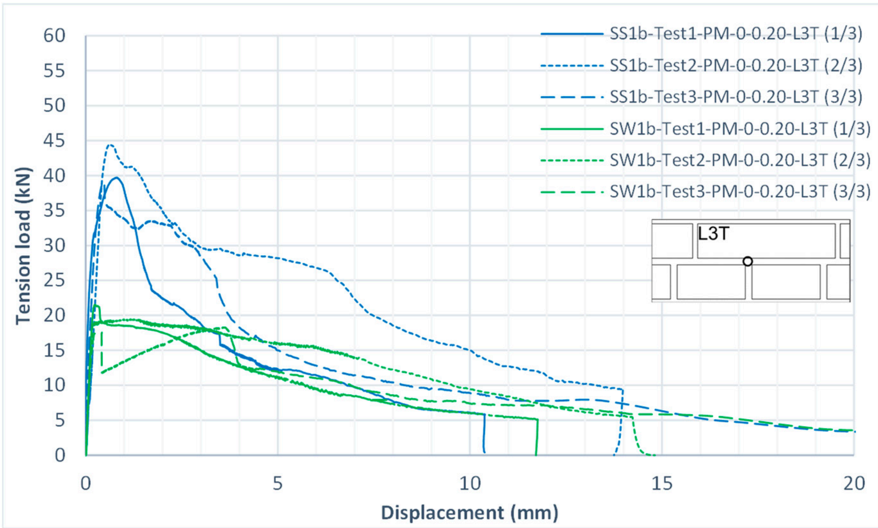


Figure 9. Load–displacement curves for anchors installed in an uncracked T-mortar joint (L_{3T}) made with strong or weak mortar.

To evaluate the effect of the mortar strength in case a crack crosses the anchor installed in a mortar joint (L_{4T} location), relevant load–displacement curves are presented in Figure 10. The comparison of Figures 9 and 10 shows the significant effect of the 0.50 mm crack on the tension resistance of the anchors. This effect (further discussed in Section 5.3.2) is governing and, thus, the influence of mortar strength is attenuated (the ratio of tension resistances for weak and strong mortar is equal to 0.75 on average). Furthermore, the crack through the mortar joint leads to a significantly larger scatter of the experimental results in terms of both tension resistance and respective displacement (Table 6).

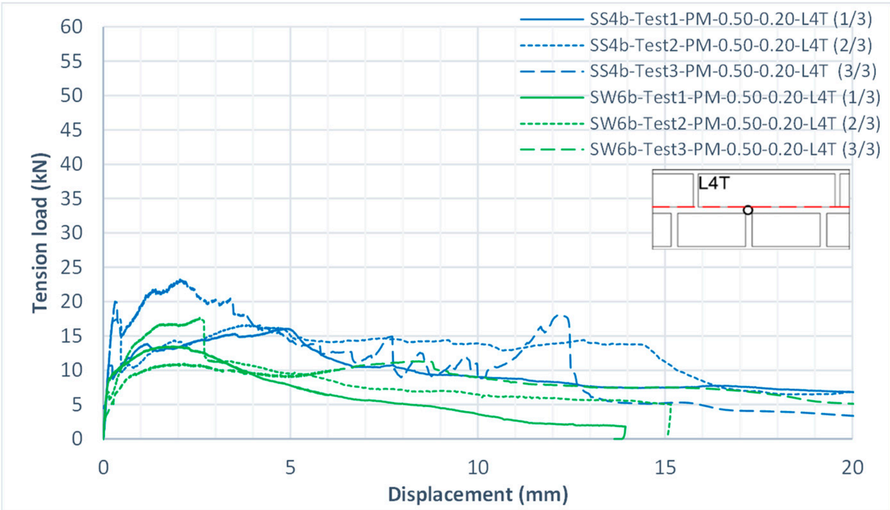


Figure 10. Load–displacement curves for anchors installed in a cracked (c_w = 0.50 mm) T-mortar joint (L_{4T}) made with strong or weak mortar.

The effect of mortar strength is also investigated for brick locations, in uncracked (L_1) and cracked (L_6) conditions (Figures 11 and 12). When anchors are installed in bricks within a wallette free of cracks, the mortar compressive strength affects the tension resistance, but this effect is less pronounced than for anchors installed in mortar joints. The curves in Figure 11 and the data in Table 6 show that for location L_1 the reduction of the tension capacity is on average equal to 30% for weak mortar joints, as compared to an average reduction of almost 50% observed in mortar joint location L_{3T}. The presence of a 0.50 mm wide crack at the vicinity of the anchor embedded in the middle of the brick (L_6) leads to a substantial reduction of the tension resistance both for strong and weak mortar, with the tension resistance in case of weak mortar remaining by almost 30% lower than for strong mortar (Figure 12 and Table 6).

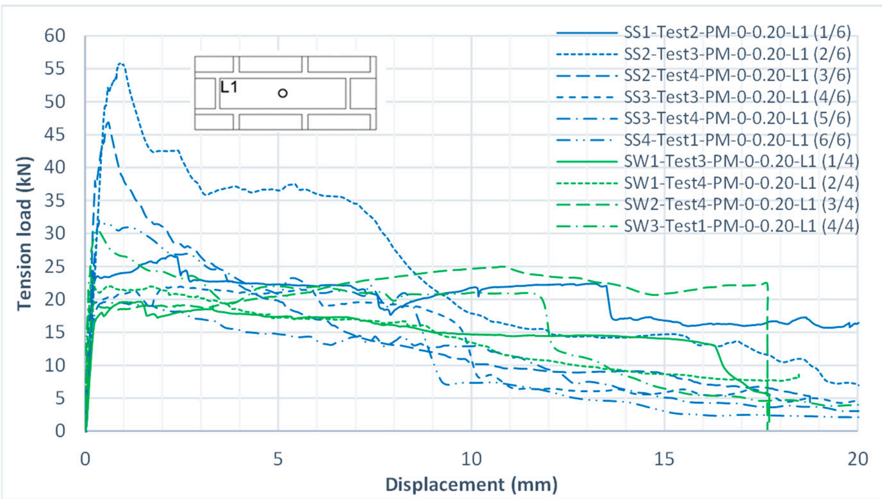


Figure 11. Load–displacement curves for anchors installed in uncracked bricks with an adjacent uncracked joint (L_1) made with strong or weak mortar.

The reduction coefficients calculated based on the test results are valid for the materials used within this experimental campaign and, hence, they cannot be of general applicability. Nonetheless, the test data prove the preponderance of brick against mortar locations for the installation of anchors, at least for the short embedment depth tested within this part of the campaign. Further caution is needed in the design of anchors in mortar locations, due to the inevitable scatter of the mortar properties. Even under laboratory conditions, with continuous control of mortar mix proportions, when curing the walls and the conventional mortar specimens, there was quite a scatter of the measured compressive values (Table 1). The effect of this variability on the compressive strength of mortar is illustrated in Figure 13. This figure presents sets of tension load vs. displacement curves for anchors differing only in the compressive strength of mortar (4.53 MPa vs. 2.89 MPa). Site conditions are not expected to be more favorable in this respect. Moreover, in the case of existing masonry structures, where the mortar strength is estimated based on a limited number of in situ and/or laboratory tests, a more pronounced variability of mortar properties is expected. This issue may be reduced by direct anchor testing on site, provided that specific care is taken in the determination of representative testing locations.

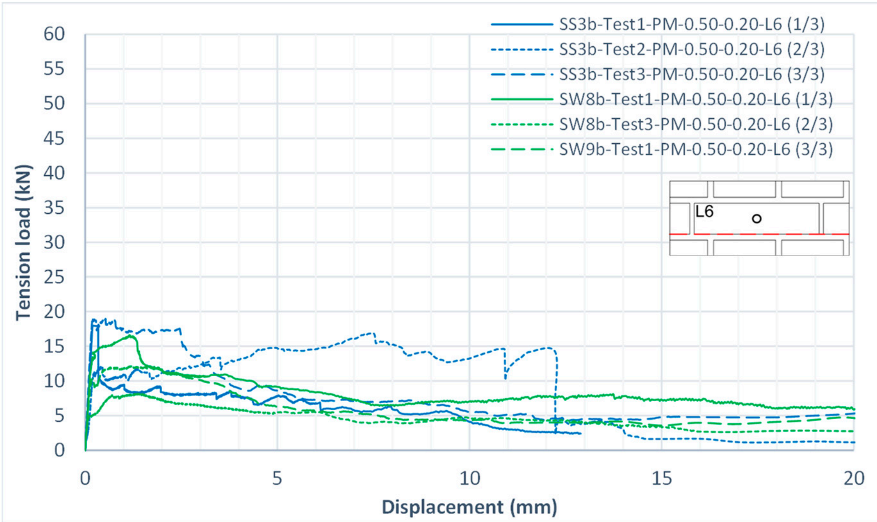


Figure 12. Load–displacement curves for anchors installed in uncracked brick with an adjacent cracked (c_w = 0.50 mm) joint (L_6) made with strong or weak mortar.

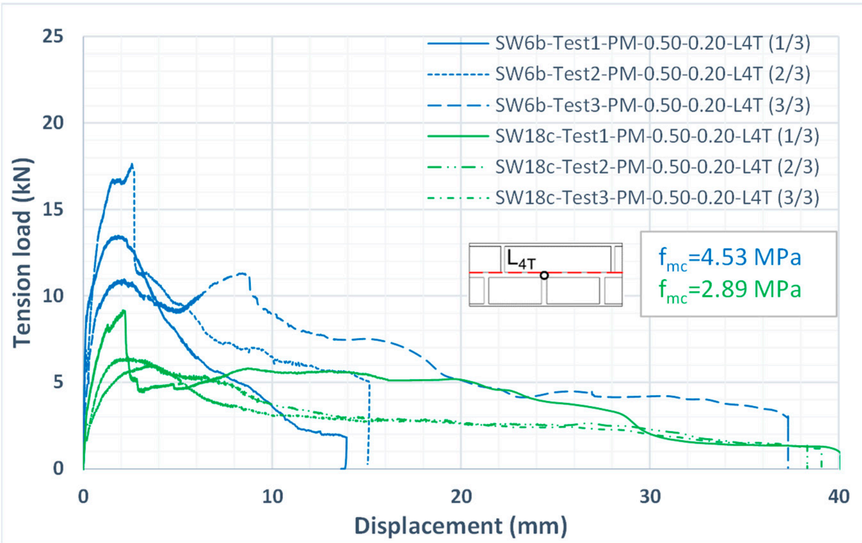


Figure 13. Load–displacement curves for anchors installed in a cracked (c_w = 0.50 mm) T-mortar joint (L_{4T}) made with weak mortar of different mean compressive strength (4.53 MPa vs. 2.89 MPa).

It is noted that significant is the effect of the mechanical properties of the mortar on the tension capacity of anchors embedded in bricks as well. Although it seems that for brick locations this effect is less pronounced than for mortar joint locations, the sensible preference to install anchors exclusively in brick locations is not possible in practice. Indeed, quite often, some wall regions are of limited accessibility, thus not permitting the installation of anchors. In other cases, anchor arrangements dictated by each specific application may lead to the application of anchors to various locations. It is, therefore, believed that, although it is necessary to investigate the behavior of individual anchors installed in various locations, application tests, with the use of multiple anchors (anchor groups), are also needed. This is more so because the tension resistance of anchors installed in various locations is not activated for the same value of imposed displacement.

5.3.2. The Effect of Cracking for Various Anchor Locations

Although most of the experimental results of this section were already presented in the previous Section 5.3.1, they are repeated here grouped by location (brick or mortar), with the purpose of more thoroughly assessing the effect of a 0.50 mm wide crack. Thus, the plots of Figures 14 and 15 show monotonic curves for uncracked and cracked mortar locations, separately for strong and weak mortar.

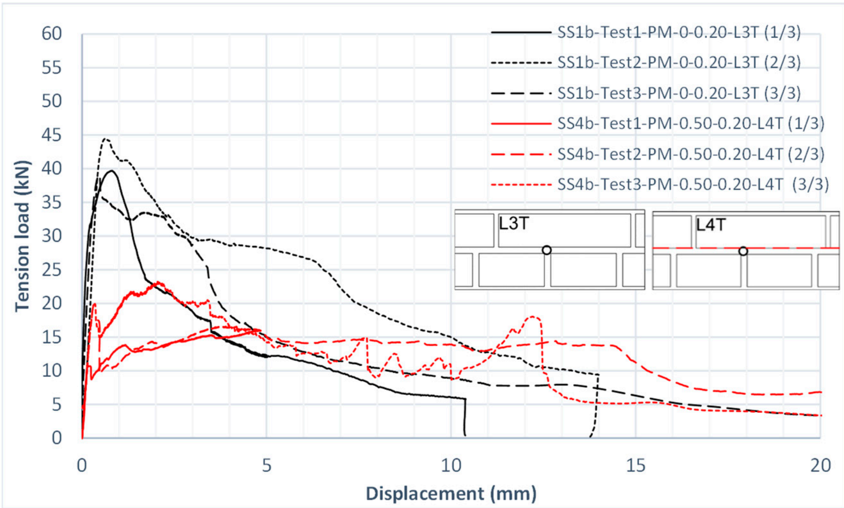


Figure 14. Load–displacement curves for anchors installed in an uncracked vs. cracked (c_w = 0.50 mm) T-mortar joint (L_{4T}) made with strong mortar.

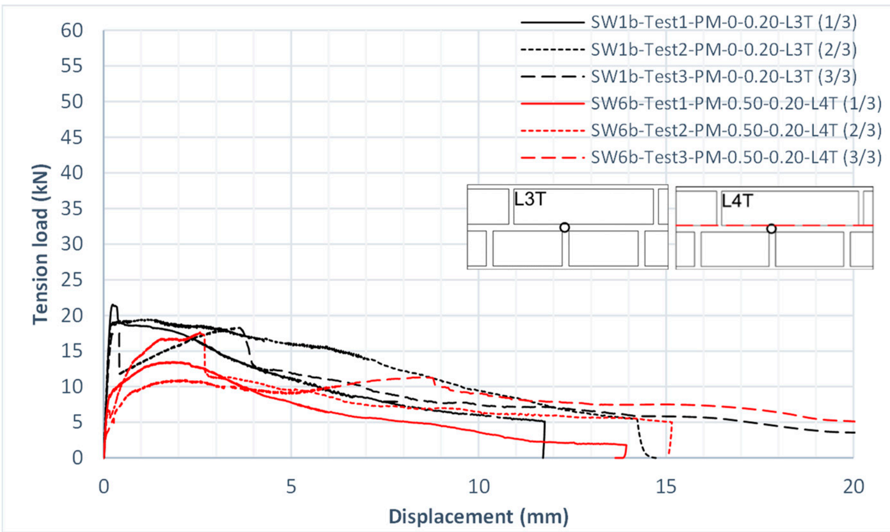


Figure 15. Load–displacement curves for anchors installed in an uncracked vs. cracked (c_w = 0.50 mm) T-mortar joint (L_{4T}) made with weak mortar.

The effect of cracking on the anchor behavior in a T-joint can be summarized as follows: (a) It reduces the mean tension resistance, for both tested mortar strengths. A reduction factor of 0.47 applies to anchors installed in strong mortar joints, while this factor is equal to 0.72 for weak mortar. (b) It increases the displacement at which the tension resistance is activated, i.e., it reduces the stiffness of the bond mechanism. (c) It renders the curves smoother than in the absence of a crack. (d) It increases the scatter of both the tension resistance values and the respective displacements. For example, the data in Table 6 show that in the case of strong mortar, the CoV of the average tension resistance is equal to 7.3% for the uncracked location L_{3T}. This coefficient rises to 20.3% for the cracked location L_{4T}. The respective figures for the displacement at tension resistance are 26.0% and 90.9%.

Similar is the effect of cracking on the tension behavior of anchors embedded in bricks (Figures 16 and 17). However, as pointed out in the previous Section 5.3.1, in this case, the reduction factor of the tension resistance due to the 0.50 mm wide crack is approximately equal to 0.50, independent of the mortar compressive strength.

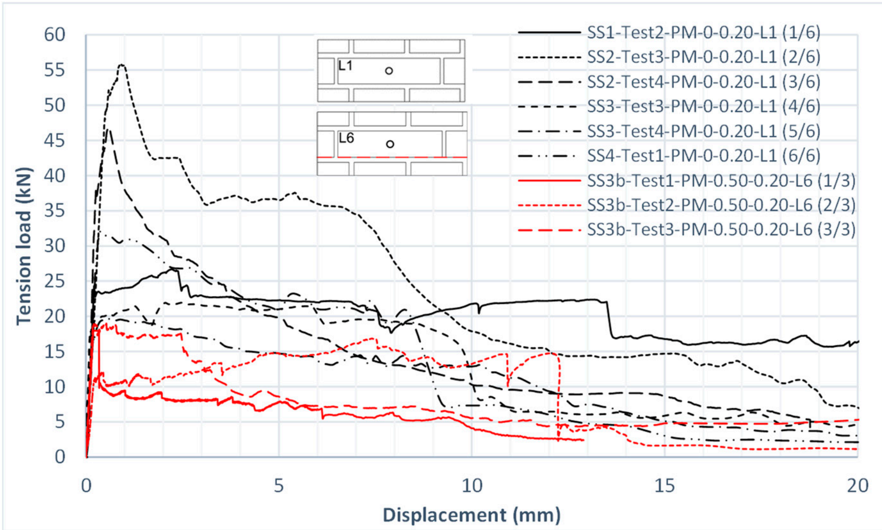


Figure 16. Load–displacement curves for anchors installed in an uncracked brick with uncracked vs. adjacent cracked (c_w = 0.50 mm) mortar joint (L_6) made with strong mortar.

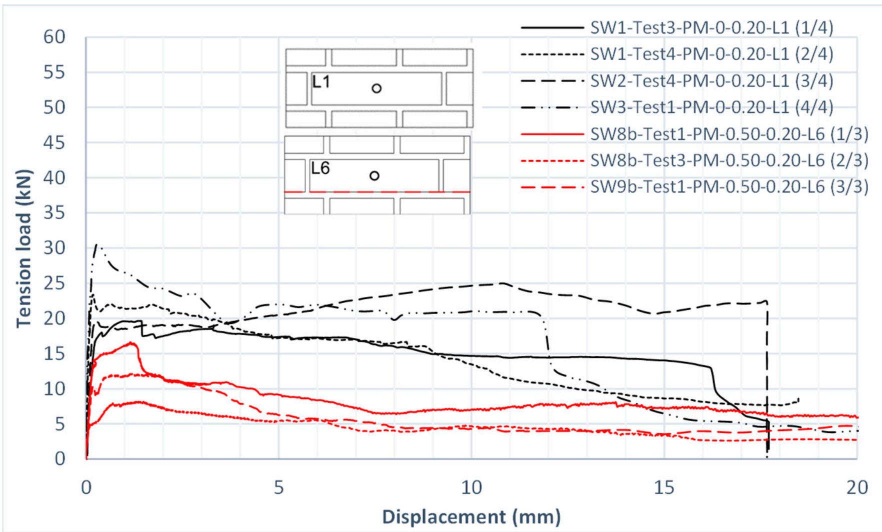


Figure 17. Load–displacement curves for anchors installed in an uncracked brick with uncracked vs. adjacent cracked (c_w = 0.50 mm) mortar joint (L_6) made with weak mortar.

A summary of the experimental data illustrating the effect of anchor location (brick or weak mortar joint), as well as the effect of cracking on the tension resistance and on the corresponding displacement is presented in Figures 18 and 19, respectively. The reduction of tension resistance is apparent, as the anchor "moves" from brick to mortar and from uncracked to cracked conditions (Figure 18). The opposite holds true for displacements, which increase when the anchor moves from brick to mortar and from uncracked to cracked locations (Figure 19). It is also noted that brick locations are characterized by a smaller scatter of displacement values than mortar joint locations.

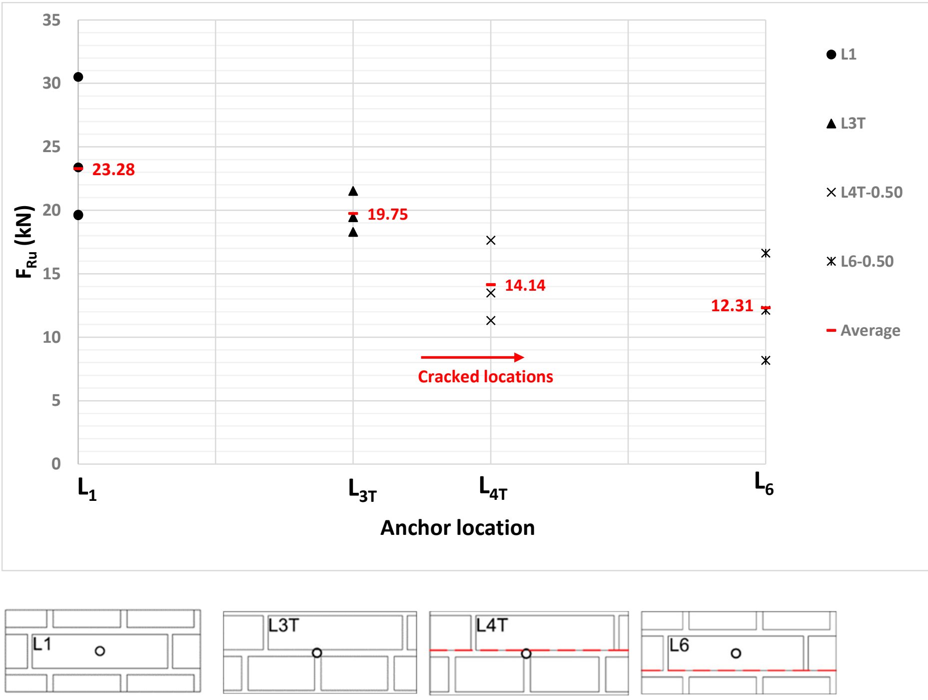


Figure 18. Tension resistance (F_{Ru}) vs. location for anchors installed in walls made with weak mortar.

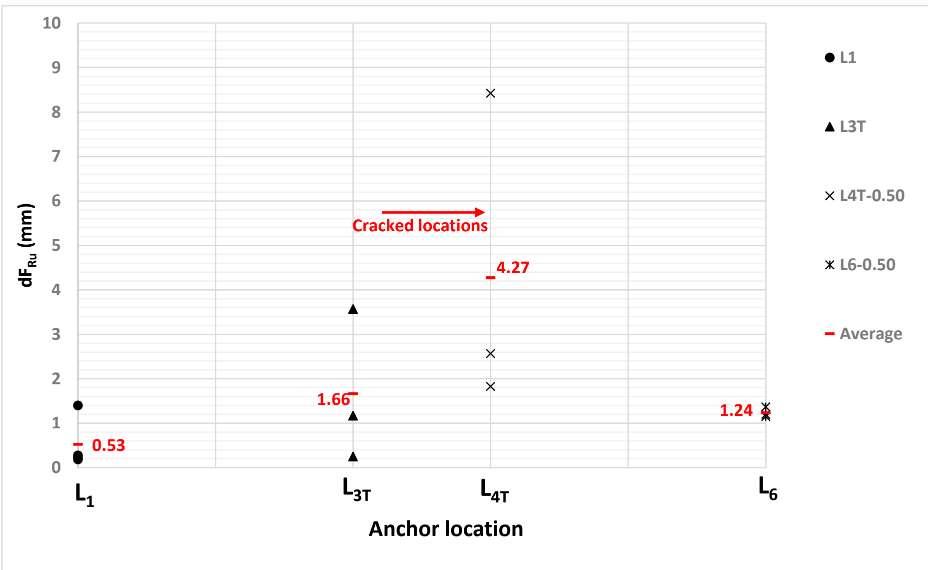


Figure 19. Displacements at tension resistance (d_{FRu}) vs. location for anchors installed in walls made with weak mortar.

5.3.3. The Effect of Crack Width

The effect of the crack width was investigated for anchors installed in weak mortar joint and brick locations (L_{4T} and L_6, respectively).

For the cracked T-joint location L_{4T}, several target values of crack width were investigated, namely, 0.30 mm, 0.50 mm, 0.60 mm, 0.80 mm, and 1.20 mm. It is noted that the targeted crack width values were not always reached with accuracy. Figure 20 shows the reduction of the tension resistance for increasing crack width. It also indicates that for extreme crack widths, a certain threshold is reached, and the resistance does not drop to zero. This effect can be attributed to the fact that, although the anchor in the drill hole is crossed by the crack, a substantial portion of the injection mortar remains in contact with the base material and, thus, it can transfer load. It is also observed that the scatter of the tension resistances is larger when the anchor is installed in a cracked mortar joint, compared to anchors in an uncracked substrate. Part of the scatter can be attributed to the scatter of the measured mortar compressive strengths. Similar to tension resistances, the respective displacement values present significant scatter (Figure 21), with the exception of the anchors installed in a 1.20 mm wide crack. In that case, premature pull-out of the anchor from the mortar joint occurs at a small displacement value with limited scatter. In general, the displacement values are so scattered that it is not possible to draw a trend line between the displacement at tension resistance and the crack width.

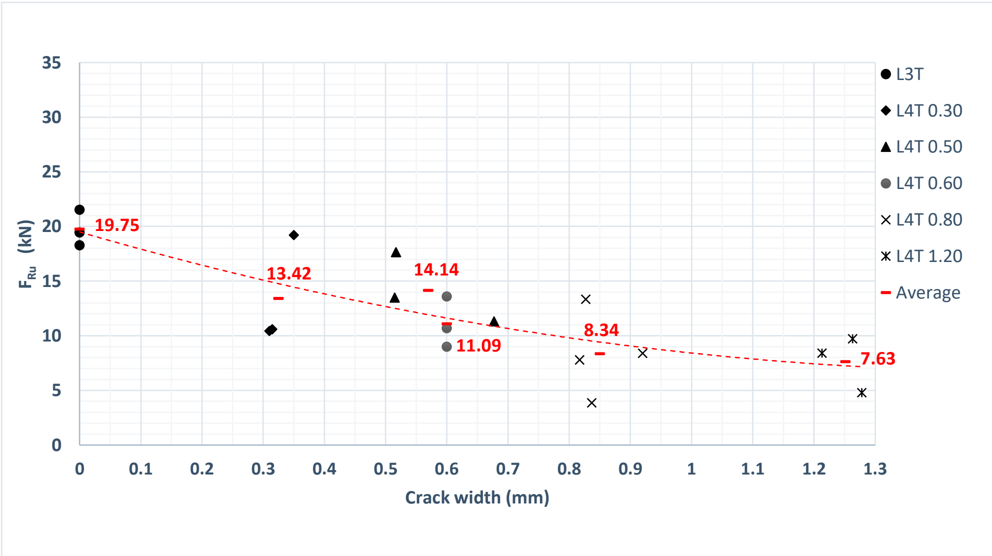


Figure 20. Tension resistance (F_{Ru}) vs. crack width (w_{tot}) value for anchors installed in T-mortar joint locations made with weak mortar.

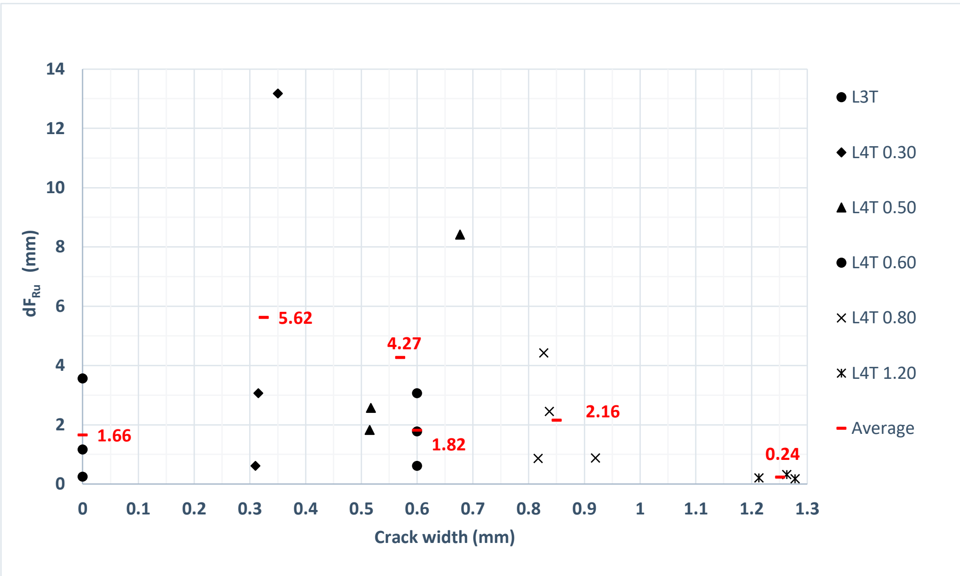


Figure 21. Displacement at tension resistance (d_{FRu}) vs. crack width (w_{tot}) value: Anchors installed in T-mortar joints made with weak mortar.

For anchors installed in the middle of an uncracked brick with adjacent cracked joint (L_6), the effect of cracks was investigated for widths equal to 0.30 mm, 0.50 mm, and 0.80 mm. In this case too, the increase in the crack width leads to reduced tension resistance (Figure 22). Nonetheless, for anchors embedded in bricks, the displacements at which the tension resistance is activated show a clear tendency to increase with increasing crack width (Figure 23). They are, as a rule, smaller than for anchors embedded in mortar joints.

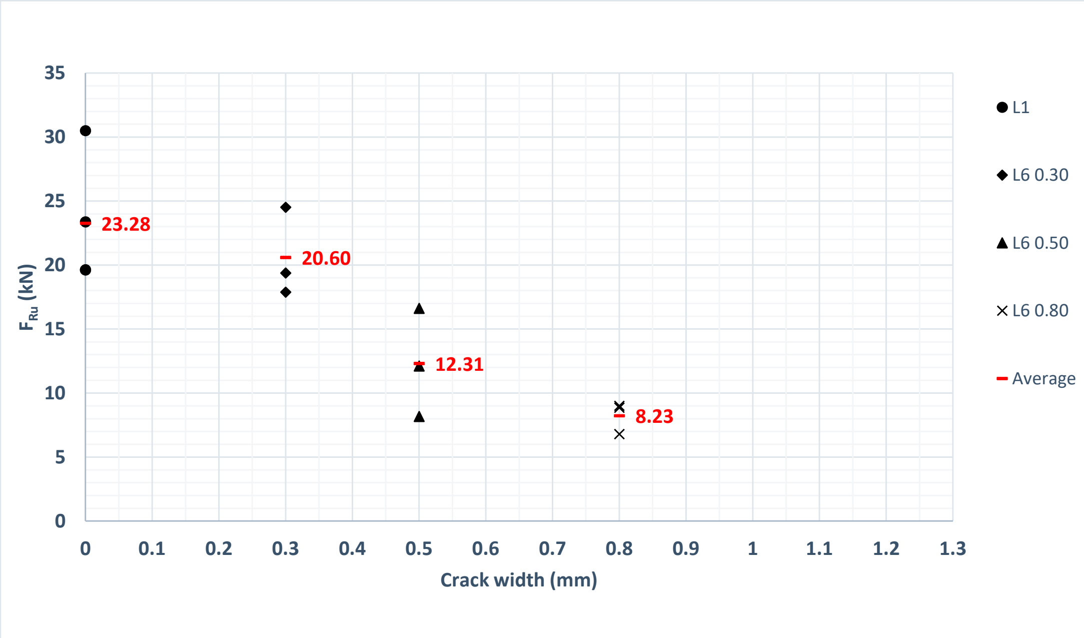


Figure 22. Tension resistance vs. crack width value for anchors installed in uncracked brick joints and uncracked bricks with cracked adjacent horizontal mortar joints made with weak mortar.

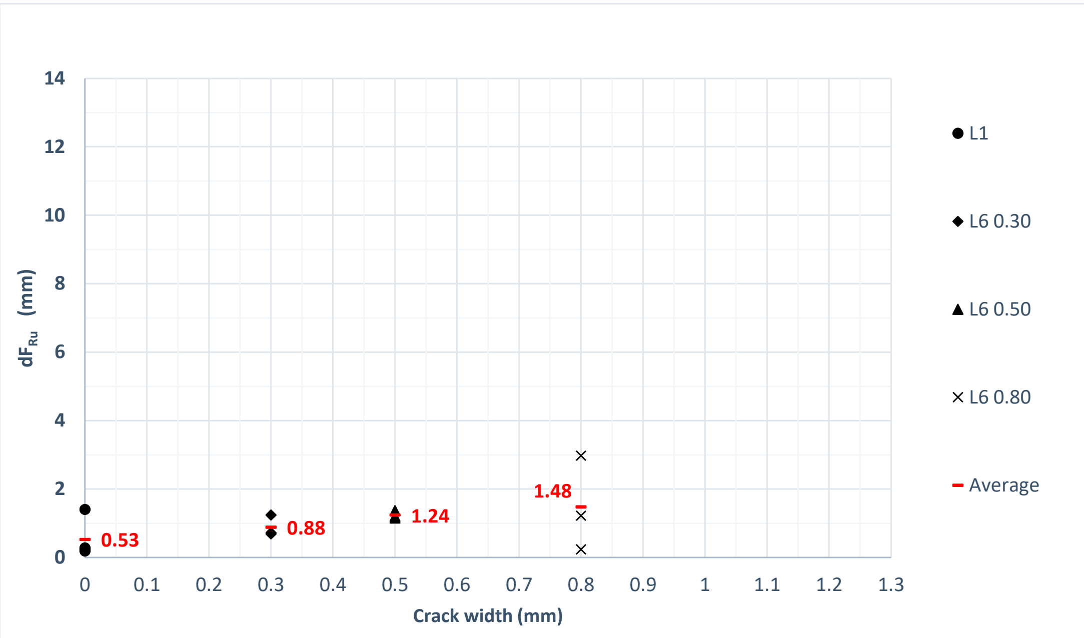


Figure 23. Displacement at tension resistance vs. crack width value for anchors installed in uncracked brick joints and uncracked bricks with cracked adjacent horizontal mortar joints made with weak mortar.

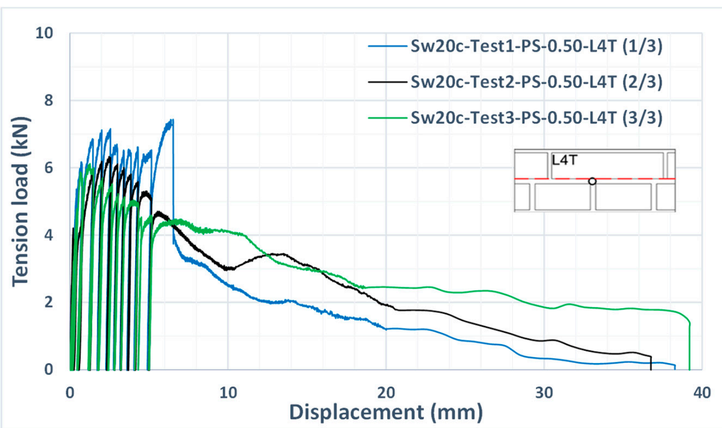
The effect of crack opening has shown that the design of anchors must be based on criteria, one of them being a limiting value of the expected crack width. Indeed, the design of anchors should follow a realistic scenario and avoid adopting overconservative tension resistances corresponding to excessively large cracks. Thus, the study presented in [21] was undertaken. This study, although in several cases of qualitative nature due to the lack of published data on the width of cracks occurring to masonry walls during seismic events, has led to the following results: (i) the opening of the cracks in unreinforced masonry walls under in-plane cyclic load remains limited, up to the attainment of the maximum shear resistance of the wall; (ii) typically, larger crack openings occur in the vertical rather than the horizontal portions of diagonal cracks; (iii) depending on the failure mode of the wall (shear, flexural, hybrid), areas that remain slightly damaged can be distinguished as adequate for the installation of connection elements.

Based on this evaluation of the literature, the decision was made to set the limit for the crack width to 0.50 mm within the experimental campaign. Furthermore, considering the usual case of masonry, where the units are of significantly higher mechanical properties than the mortar, the crack was generated exclusively in the bed joints. For the investigated anchor locations under crack influence (L_{4T}, L_6), this option in combination with the chosen crack width is regarded to deliver conservative yet realistic resistance values.

5.3.4. The Effect of Repeated Loading

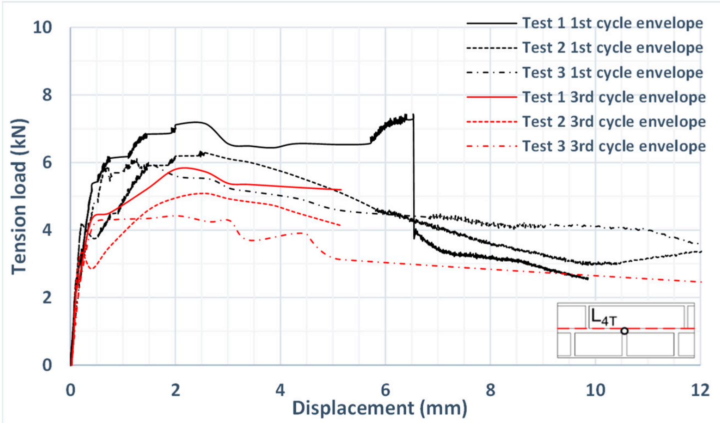
The effect of repeated loading on the behavior of anchors in tension was investigated for anchors installed in wallettes made with weak mortar, in two locations: (i) L_{4T}, i.e., cracked mortar T-joint (crack width = 0.50 mm); (ii) L_1, anchor installed in uncracked brick. As aforementioned, T-joint, location, was selected as the most adverse in terms of expected resistance, while L_1 is the most favorable location. For the displacement-controlled tests of this series, the displacement steps were defined based on the monotonic curves obtained for the same location. Two basic rules are applied for the definition of displacement steps, namely, the interval between consecutive steps should allow for a smooth hysteresis-loop envelope to be recorded, while at least three cycles should be performed after the attainment of the maximum resistance and before failure occurs. On the other hand, it is desirable to limit the displacement steps to an adequate minimum number, to avoid unnecessary testing effort. A summary of relevant experimental results is presented in Table 7.

After the completion of each test, as shown in Figure 4b, hysteresis-loop envelopes are drawn for the three cycles per selected imposed displacement value (Figures 24a and 25a). For the envelope curve of each cycle, the value of tension resistance, Fu,n, n being the number of the cycle (1, 2, or 3), is determined and the effect of repeated loading is calculated as the ratio (reduction factor) between the maximum resistance obtained during the third and that activated during the first cycle (Table 7, Figures 24b and 25b).



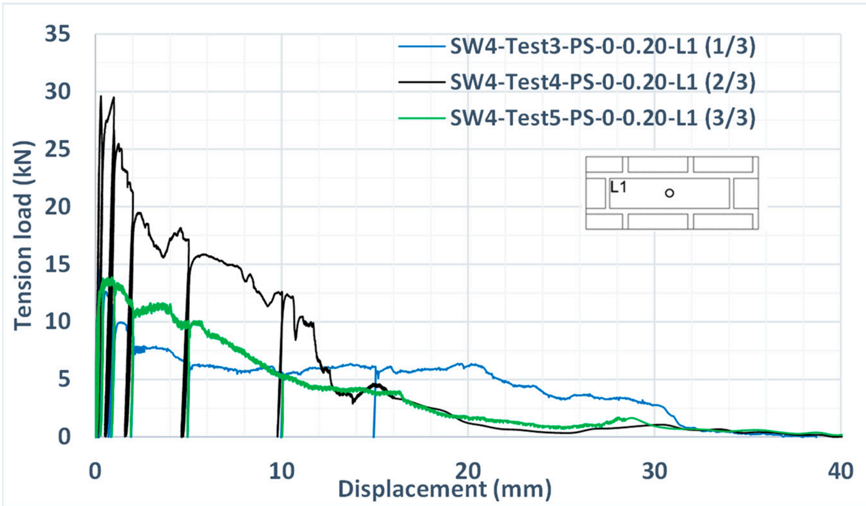
(**a**)

Figure 24. *Cont.*

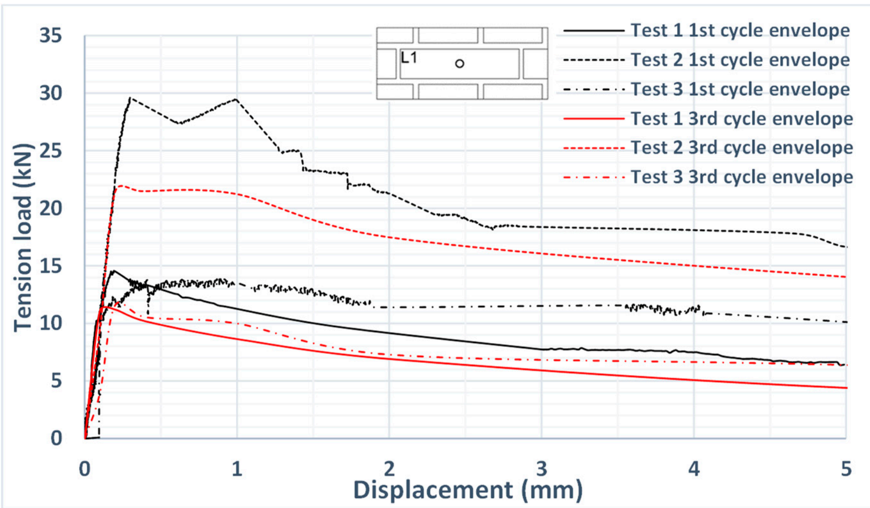


(**b**)

Figure 24. (**a**) Tests (1, 2, 3), experimental curves in repeated tension tests; (**b**) summary of envelope curves of 1st and 3rd cycle in the case of repeated tension tests in L_{4T}.



(**a**)



(**b**)

Figure 25. (**a**) Tests (1, 2, 3), experimental curves in repeated tension tests; (**b**) summary of envelope curves of 1st and 3rd cycle in the case of repeated tension tests in L_1.

The experimental results show that the ratio between the tension resistance at the third loading cycle and that of the first cycle is practically the same for both examined locations, L_{4T} and L_1 (0.77 and 0.78, respectively).

Table 7. Summary of results for anchors subjected to repeated loading. Anchors were installed in walls made with weak mortar.

Test Name/Location	Mortar $f_{mc,mean}$ (MPa)	Crack Width (mm)	$F_{Ru,}$ 1st $_{cycle}$ (kN)	$d_{FRu,}$ 1st $_{cycle}$ (mm)	$F_{Ru,}$ 3rd $_{cycle}$ (kN)	$d_{FRu,}$ 3rd $_{cycle}$ (mm)	$F_{Ru,3}/F_{Ru,1}$	Failure Mode
SW20c-Test1-PS-0.50-0.20-L_{4T}(1/3)	2.89	0.50	7.43	6.53	5.80	2.01	0.78	APM
SW20c-Test2-PS-0.50-0.20-L_{4T}(2/3)			6.33	2.47	5.09	2.56	0.80	AP
SW20c-Test3-PS-0.50-0.20-L_{4T}(3/3)			6.12	1.27	4.42	2.06	0.72	APM
Mean value			6.63	3.42	5.10	2.21	0.77	
CoV (%)			10.62	80.52	13.52	13.76		
SW4-Test3-PM-0-0.20-L_1(1/3)	3.41	Uncracked	14.57	0.19	11.11	0.104	0.76	BP
SW4-Test4-PM-0-0.20-L_1(2/3)			29.60	0.30	21.49	0.37	0.73	BS/SBC
SW4-Test1-PM-0-0.20-L_1(3/3)			13.90	0.90	11.59	0.21	0.85	BS/BP
Mean value			19.36	0.46	14.73	0.23	0.78	
CoV (%)			45.85	82.48	39.78	58.73		

Notation: $f_{mc,mean}$: mean mortar compressive strength. F_{Ru}: tension resistance. d_{FRu}: displacement corresponding to tension resistance. 1st/3rd cycle: first/third cycle of envelope curves.

6. Conclusions

The experimental results presented in this paper investigate the effect of several parameters on the behavior of anchors embedded in solid brick masonry and subjected to tension. The parameters that are investigated are the following: (i) compressive strength of mortar; (ii) anchor location (middle of brick or mortar joint); (iii) absence or presence of crack close to or crossing the anchor, as well as the value of the crack width; (iv) the type of loading protocol, monotonic vs. repeated. It is noted that although the conclusions related to the effect of the investigated parameters are of general validity, the arithmetic values expressing this effect are dependent on the specific characteristics of the materials used in this research, as well as on the values of the investigated parameters.

The main conclusions of this investigation are as follows:

1. The mortar quality, and more specifically, its compressive strength, affects the behavior of anchors in tension. Indeed, anchors installed in masonry walls made with weak mortar exhibited reduced capacity compared to the ones installed in walls constructed with strong mortar. This reduction is not proportional to the compressive strength of the mortar; it is more likely proportional to its square root, i.e., proportional to the tensile strength of the mortar. On the other hand, the effect of masonry mortar quality depends on the anchor location (brick or mortar) and on the occurring failure modes. More specifically, the reduction of the tension resistance is more pronounced in the case of anchors installed in mortar joints than for anchors installed in bricks (average reduction factor equal to 0.50 and 0.70 for mortar joint and brick location, respectively). It was observed, however, that the difference between the two examined locations, in terms of the effect of the mortar quality seems to be reduced, when there is a crack either crossing the anchor or at its vicinity (reduction factor equal to 0.75 and 0.70 for mortar joint and brick location respectively). This shows that the effect of cracks governs the behavior of the anchors in tension.
2. The presence of a crack, 0.50 mm wide, has led to a substantial reduction of the tension capacity of anchors for both tested locations (between 30% and 50%, depending on the anchor location and on the compressive strength of the masonry mortar), accompanied by an increase in the displacement corresponding to the attainment of the resistance. In parallel, the scatter of both resistance and corresponding displacement values is increased when the masonry is cracked.
3. Before making the decision to limit the crack width to 0.50 mm, based on an assessment of the relevant literature, the effects of larger crack values were also investigated. As expected, regardless of the anchor location, the increase in the crack width leads to a

decrease in the tension resistance, combined with a more pronounced scatter of the experimental tension resistance values.
4. Anchors subjected to repeated loading have exhibited a moderate reduction of their tension resistance (by 20% approximately), both for brick and mortar locations.
5. A significant observation is that the reduction of the tension capacity due to various adverse parameters, the increase in the displacement at tension resistance, as well as the effect of the investigated parameters on the scatter of the experimental results are closely related to the failure mode that takes place. It was observed that there is a variety of possible failure modes. Their effect can be interpreted, after testing, but their occurrence cannot be reliably predicted. Indeed, in addition to some basic failure modes (such as anchor or brick pull-out), further failure modes are observed, involving a smaller or larger portion of the mortar joint or the adjacent brick. Those failures seem to depend on the local conditions and, thus, they cannot be predicted on the basis of average characteristics of the materials or based on the construction typology of the wall.

Therefore, all those aspects should be accounted for, when design values of tension resistance are set, by providing adequate factors. Those factors should cover not only the variability of material properties, the presence of cracks in the substrate, and the type of loading. They should also allow for consideration of the inherent large scatter of the data.

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