Strength Performance of the Connection between Brick and SPF Lumber

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Abstract: There are many ways to strengthen an unreinforced brick masonry wall. One of the strengthening methods applies timber, which is a lightweight and easy installation. The connection between the timber and the brick masonry wall plays an important role. In this study, the experimental and theoretical methods were investigated, and the pullout and shear strengths of the chemical anchor were employed as the connection between clay brick and SPF (spruce, pine, fir) lumber. Three bolt diameters (ø8, ø10, ø12 mm) were utilized in nine pullout tests and two types of eighteen shear tests. Four types of prediction models estimated the pullout strength of the chemical anchor. The European Yield Theory (EYT) expected the shear strength of the chemical anchor, involving six failure modes. The results of all experiments were discussed as failure mode, strength, and stiffness under monotonic load. Finally, the effective result of this study highlights A12 specimens using ø12 mm bolt and employing the epoxy resin in brick and lumber. The maximum pullout load of A12 specimens is 12.1 kN, and the yield shear load is 11.2 kN.

Keywords: pullout strength; shear strength; chemical adhesive; bolt connection

1. Introduction

The unreinforced masonry structures have a high risk of earthquakes, evidenced by postearthquake damages and research. The seismic evaluation of three to five-story buildings constructed in the 1970s was assessed because of the 2019 earthquake in Albania [1]. A push-over analysis was performed for each building, and the buildings showed a nonlinear response, and diagonal in-plane cracks appeared in most masonry buildings. The damage and failure of masonry structures were studied after the 2017 earthquake in Greece [2] and highlight the vulnerabilities that cause damage and the prevention factors. The study [2] also inspected the unreinforced and timber-reinforced masonry structures, and failure modes were observed in the in-plane, out-of-plane, and combined directions. After the Croatia earthquake in 2020 [3], site investigations of residential buildings were conducted, and they inspected postearthquake assessments, damage classifications, and failure patterns in the masonry buildings. There is research material on the damage to buildings and structures caused by strong earthquakes in other countries [4,5]. Based on the above earthquake lessons, it is important to predict a seismic assessment of the masonry buildings, determine the failure mechanism, and consider the possibility of the strengthening method. Severe damage to URM buildings due to seismic load is related to the composite material characteristics of masonry structures, including brick and mortar. Masonry material has high compressive strength and low shear and tensile strength. Masonry walls can be easily broken based on the weak strength under horizontal loads.
Therefore, it is necessary to increase the strength and stiffness of the masonry structures. Researchers have investigated and carried out various strengthening methods of masonry structure, depending on material properties [6–8].

There has been an increasing number of studies on the strengthening technique of using timber frames and panels for unreinforced masonry structures [9–11]. Timber stud [10] is used on seismic retrofit of masonry walls. In this study [10], 90 mm × 45 mm timber studs were located in the unreinforced masonry wall to increase the out-of-plane capacity. Mechanical screws (D12/L230 mm) were used to secure the timber stud to the masonry wall. The timber frames and boards [9] were employed to strengthen the masonry piers. When fixing the wooden frame to the masonry wall, a 90 mm × 50 mm metal angle is connected to timber with a 70 mm long screw, and a 10 mm threaded rod with chemical adhesive is connected to the masonry wall [9]. The two-story full-size unreinforced masonry building with an innovative timber retrofit was tested on the shake table [11]. Connecting the timber to the masonry structure, ø10 mm threaded rods were installed at a depth of 50 mm and fastened with epoxy resin. An on-site experiment [12] was conducted to strengthen the masonry structure, and two types of screws (M12/L180 and M10/L230) were tested to attach the cross laminated timber (CLT) panel to the masonry wall. The method of strengthening was able to increase the in-plane shear strength of the wall by 40 percent [12]. The study [13] carried out OSB board threaded dry rod, and the chemical anchor was connected to the masonry wall. The bolts were fixed to the masonry wall only, and the installation depth was 50 mm. The cross-laminated timber panel was used to strengthen the unreinforced masonry wall [14].

One of the most important aspects of timber structures is the connection. The bolt or screw fasteners were utilized to secure the unreinforced masonry wall to timber. There are two types of anchors, mechanical and chemical. The chemical anchor uses a threaded rod or reinforcing bar set in predrilled holes with adhesive compounds [15]. In a mechanical anchor system, forces on the heavily loaded bonded anchor are generally transferred uniformly to the concrete along the length of the embedded portion of the anchor [16]. When estimating the strength of an anchor, two fundamental strengths act on the fastener. These include the pullout strength acting along with the bolt and the shear strength perpendicular to the bolt. The mechanical properties of metal anchors [17] were tested on historical stone masonry. The fastener types of metal, chemical, and mortar were inspected on tension and shear loads and compared with theoretical assumptions. The experimental result [17] concluded pullout strength of the chemical anchor showed more values than the other two. The experimental study [18] identified the shear strength of five types of screw anchors, including ø12 mm, ø12.5, and ø16.6 mm threaded diameters for connecting masonry structures to wood. Dry screw anchors [18] have been a possible solution for connecting masonry and wood materials. It has been concluded that it may be better to use chemical adhesives for irregular stone masonry. The study [19] determined anchors’ tensile and shear strength for limestone structures. Three different diameter bolts were tested with two bond agents (epoxy adhesive and cement-based grout) [19]. The tensile strength of the chemical anchor is greater than cement-based grout, and it was concluded that the shear strength showed similar values. An experimental study of pullout strength [20] was conducted on masonry walls, and the bolts were bonded with epoxy resin and were placed in the head and bed-joint and brick in the brick masonry wall. The test result [20] concluded that the bolt’s pullout strength placed in the mortar was greater than the bricks’.

As shown in the result of the above studies, timber may be beneficial to use as a simple strengthening approach to withstand seismic loads and increase the strength of the unreinforced masonry wall. Applying epoxy resin in the retrofitting method improves the strength of the connection. Most of the studies have investigated using the screw rather than bolt securing to the masonry wall. There are few studies where epoxy resin has been used to connect the timber and bolt when the epoxy resin bonds the bolt in the masonry wall. Various retrofitting methods have been proposed in the investigations. Our study considered that the strengthening method of the brick construction, which can be
composed of wood, bolts, and adhesive, could be proposed as a simple method. Firstly, this study evaluated the strength performance of the joint brick and wood and proposed the calculating method of their performance.

2. Materials and Setup

This section describes the material properties, descriptions of specimen preparation, experimental setup of the pullout test, the dowel-bearing test, and the shear test setup. Test specimens were prepared and tested following applicable standards.

2.1. Materials

Brick is the substrate material in this study. To predict the pullout strength and shear strength of composite material (brick-to-lumber) in the analytical estimation, the compressive strength of brick is one of the important parameters. Experimentation was used to determine the material properties of the brick. The applicable standard and recommendations are used to determine the mechanical qualities of SPF lumber, bolts, and epoxy resin.

2.1.1. Brick

The nominal brick size used in Japan is 210 mm × 100 mm × 60 mm. The actual dimension, density, and absorption of brick [21] are tabulated in Table 1 and value in the bracket is the coefficient of variation. The compressive and flexural strength of brick is determined by experiment according to ASTM C67-20 standard [21]. The mean flexural strength of the ten specimens is obtained from the three-point bending test in Figure 1a. The mean compressive strength of the six specimens is obtained from the uniaxial test in Figure 1b.

Table 1. The dimension, density, and absorption of brick (mean of 10 bricks).

| Brick Size (mm) | Length | Width  | Depth  | Density (g/cm³) | Absorption |
|----------------|--------|--------|--------|----------------|------------|
|                | 210.3  | 97.4   | 61.8   | 1.9            | 0.3        |

![Figure 1](image1.png) (a) Flexural test; (b) Compression test.

The mean flexural strength of brick is 3.9 N/mm², and the variational coefficient percent is 27.5. The mean compressive strength is 31.4 N/mm², and the coefficient of variation percent is 11.5. The material properties of brick are shown in Table 2. The transducers were put on the front and rear sides of the specimen to measure the vertical displacement during the compression test.

Table 2. Material properties of brick.

| Properties                | Compressive Strength | Young’s Modulus (Measured by Transducers) | Flexural Strength |
|---------------------------|----------------------|------------------------------------------|------------------|
| Mean value (N/mm²)        | 31.4                 | 1201.9                                   | 3.9              |
| St.Dev (N/mm²)            | 3.64                 | 298.8                                    | 1.1              |
| CV, %                     | 11.5                 | 24.8                                     | 27.5             |
2.1.2. SPF Lumber

An abbreviation of SPF is three species of spruce, pine, and fir. The size of SPF lumber used in this study is $2 \times 4$ (38 mm $\times$ 89 mm). The SPF lumber material is lightweight, has a clear appearance, has high strength, and has good working properties. The Standard for Structural Design of Timber Structures [22] was used to determine the material parameters of SPF lumber in visual grading class No.2. Table 3 lists the standard material properties.

Table 3. Standard material properties of SPF lumber.

| Material | Compressive Strength | Tensile Strength | Bending Strength N/mm² | Shear Strength | Elastic Modulus | Bearing Strength |
|----------|----------------------|------------------|------------------------|---------------|-----------------|-----------------|
| SPF      | 17.4                 | 11.4             | 21.6                   | 1.8           | 9600            | 25.8            |

2.1.3. Bolt

The SS400 bolts (threaded steel rods), including M8, M10, and M12, were employed for the pullout and shear test. The ultimate tensile strength of the SS400 bolt is 400 N/mm², and the yield tensile strength is 240 N/mm². The yield strength in bending for the M10 and M12 bolts is assumed by 310 N/mm² and 413 N/mm² for the M8 bolt [23].

2.1.4. Epoxy Resin

The chemical adhesive (epoxy resin) was adopted in this study to bond the brick to bolt and the wood to bolt. The chemical adhesive (NIT-RE 500 V3, Hilti, Inc., TX, USA) performed better in shorter embedment depths, had the fastest curing time, and had a safer installation in the drilled holes. After injecting the epoxy into the drilled hole for the brick or wooden part, the curing period was 24 h. Table 4 shows the chemical adhesive material parameters obtained from HILTI’s product technical handbook [24].

Table 4. Material properties of the chemical adhesive.

| Properties | Bond Strength (2 Days Curing) | Compressive Strength N/mm² | Compressive Modulus | Tensile Strength (7 Days Curing) |
|------------|-------------------------------|-----------------------------|---------------------|----------------------------------|
| Adhesive   | 10.8                          | 82.7                        | 2600                | 49.3                             |

2.2. Pull–Out Test

This study carried out experiments to define the pullout strength acting on the adhesive-bonded anchor. The pullout test was performed on 9 specimens with epoxy resin injected into the brick.

2.2.1. Specimen

Test specimen providing the pullout strength includes bricks, bolts, and epoxy resin. Three types of specimens with different bolt diameters, including M8, M10, and M12, were prepared for the pullout test. The bolt position and dimension are illustrated in Figure 2. A vibration drill bored a predrilled hole in the brick. The diameter of the hole was 2 mm larger than the bolt’s diameter. The holes in the bricks were meticulously cleaned to improve the epoxy resin’s adherence. The length of the bolt was 100 mm. The hole was filled to two-thirds with chemical adhesive, and the bolt was slowly poured into it. According to the guideline of the epoxy resin [24], the requirement of hole diameter was 2 mm larger than the bolt’s diameter, and the minimum depth for concrete was 60 mm in the M8 bolt [24]. In this test, the depth of the bolt was assumed by 50 mm based on the symmetry condition.
Figure 2. Bolt position and dimension on the brick. (a) Specimen using M8 bolt; (b) Specimen using M10 bolt; (c) Specimen using M12 bolt.

2.2.2. Setup

Figure 3 shows the setup used in the pullout experiment. The experiments were carried out on the universal testing machine (Instron 4204), and the setup follows ASTM E754–80 [25]. In this experiment, a jig was prepared by pulling the bolts upwards. The jig was square and was attached to the load cell at the top side. The specimen’s bolt was linked to the bottom of the jig. The top and bottom connections inside the jig were tightened with a washer and nut. The steel plate is placed perpendicular to the brick part, as indicated in Figure 3. Four bolts secure the steel plate to the base of the test machine.

Figure 3. Setup of the pullout test. (a) Schematic setup; (b) Photo of setup.

The steel plate sustained the brick from moving upwards when the specimen was pulled up. The speed of the load was 2 mm/min. Monotonic loading was during the test, and the specimen was loaded until a failure. Three displacement gauges were used to measure the deformation. Transducer 1 was positioned in the cross-plane connecting the load cell. Figure 3b shows the placement of transducers 2 and 3 in front of and behind the specimen. The displacement was defined as the mean of the displacements measured by transducers 2 and 3 and is referred to as the relative displacement.

2.3. Dowel-Bearing Test

The test that determines the dowel-bearing strength of wood is called the dowel-bearing test [26]. The reference value of the dowel-bearing strength of wood, metal, aluminum, and concrete is shown in NDS standard. In the case of composite materials (brick-to-lumber), the European Yield Theory (EYT) is often used to predict the shear strength of the bolted connection in the calculation of timber structures. European Yield Theory determines the failure mode and the minimum load causing damage. In the estimation of EYT, one of the most important parameters is the dowel-bearing strength of the main material (brick). The dowel-bearing strength of the main material should
be three times the compressive strength of concrete or masonry, according to theoretical modeling in the NDS standard. The dowel-bearing strength of the brick material was defined experimentally in this study.

2.3.1. Specimen

According to the ASTM D5467 standard [26], which defines the dowel-bearing strength of a wooden structure, specimen preparation and test instruction was carried out in this study. The size of the brick specimen was 100 × 60 × 60. The specimen was initially drilled in the center of the brick as part of the preparation process. The hole was then sliced in half, and the hole diameter was the same as the bolt diameter. To determine the bearing strength of bricks, three types of bolts (M8, M10, and M12) were examined.

2.3.2. Setup

Figure 4 shows the half-hole testing setup of the bearing strength of brick. During the experiment, the vertical load and displacement were measured. The specimen was placed on the steel plate, and then the uniform compressive load was applied to the bolt along the length. Transducer 1 measures the vertical displacement during loading. The load was conducted until a failure.

![Figure 4. Schematic setup of the dowel-bearing test.](image)

2.4. Shear Test

It is assumed that strengthening an unreinforced masonry structure with the wooden material will increase the shear strength of the masonry wall subjected to the horizontal load. The dowel fasteners are often bonded to the masonry wall with chemical adhesive. In addition, chemical adhesive was used to improve the adhesion of bolts and wood materials. Our study examined the difference between when the chemical adhesive is used and not used for the wood-to-bolt section when the chemical anchor is utilized in the masonry wall.

2.4.1. Specimen

We prepared two types of shear specimens (N-type and A-type) in Table 5. One is indexed by N-type, which is chemical epoxy not used in the lumber-to-bolt. The nut and washer were used to tighten the end of the bolt on the lumber side. As for the A-type specimen, epoxy resin was utilized in the lumber-to-bolt. Both types of specimens have the same chemical anchor for the brick. Specimens for the shear test consist of brick, SPF lumber, bolt with nut and washer, and chemical adhesive. Two lumber material was placed on both sides of the brick. The brick sides indicate the “B” side and “A” side shown in Figure 5a. Because the bolt positions are varied, it is necessary to assemble brick and lumber material. The “A” side of the brick was fastened to the “A” side of the lumber, and the “B” sides were the same. Three alternative bolt sizes (M8, M10, and M12) are included in both N-type and A-type specimens. Specimen name, the density of lumber material, bolt diameter, and the number of specimens are shown in Table 5.
The 60 mm side of the brick was drilled with two holes for connecting bolts with chemical adhesive. The hole positions in the brick are indicated in Figure 5a. As for the brick, the nominal hole diameter was 2 mm larger than the bolt diameter, and the depth was 50 mm for both types of specimens. For the SPF lumber, the hole diameter was 1 mm larger than the bolt diameter for N-type specimens and 2 mm larger for A-type specimens. Figure 5b depicts the dimension and hole position in the SPF lumber. According to the Japanese standard for timber building, the minimum edge distance of a bolt hole in the lumber is seven times the diameter of the bolt (7d) [22]. The edge distance will prevent the lumber material from cracking before loading. The average density of lumber was 480 kg/m$^3$ in this experiment. The lumber length depends on bolt diameters, such as
220 mm, 230 mm, and 250 mm in the shear test. Two SPF lumber for one shear specimen has the same density. The width of lumber was 89 mm. The bolt was placed across the width of the lumber. The total length of the bolt was 200 mm. The necessary nuts and washers for M8, M10, and M12 were employed.

The first step of the assembly process is that the epoxy resin was gradually poured into the “A” side or “B” side’s brick hole, and then the bolt was slowly inserted. After the bolt reached the bottom of the hole, the bolt was moved up and down several times. For the second step of the N-type specimen, the brick specimens with bolts were attached to lumbers using nuts and washers. For the A-type specimen, epoxy resin was poured into the hole of the lumber, and then the brick specimen with bolt was attached with the lumber material instantly tightened by nuts and washers.

2.4.2. Setup

The shear test setup shows in Figure 6, and the experiments were carried out on the universal testing machine (Instron 4204). The load cell capacity is 50 kN. During the test, the speed of load was 2 mm/min. Transducers 1 recorded the movement of the crosshead. Under the load cell, transducer 2 measured the down lift of the steel plate. To measure a relative displacement, transducers 3 and 4 were placed on the opposite side of the brick. To establish a flat surface before subjecting the load, nonshrink cement was applied to the top of the brick.

![Image of test setup](image)

**Figure 6.** Specimen of shear test (a) schematic setup; (b) scene of the test setup.

3. The Prediction Method

3.1. Pullout Strength

Theoretically, there are four types of prediction pullout failures, depending on the type of damage. The four prediction failure modes in Figure 7 include steel failure, cone failure, bond failure, and combined cone–bone failure [15,16,20]. Figure 7 illustrates the
predictive states which may show such failure mode for the single brick material. When the bolt’s tensile strength is less than the adhesive anchor’s strength, the steel failure mode occurs \[19\]. This is frequently the case for long-depth anchors, according to studies. The first formula defines the steel failure formula in Table 6. Cone failure predicted by the second formula in Table 6 occurs when the depth of bolt installation is shallow \[15\].

![Figure 7. Predicted failure modes of pullout](image)

**Figure 7.** Predicted failure modes of pullout (a) Steel; (b) Bond; (c) Cone; (d) Combined cone–bond. Where; \(N\) is pullout load (kN); \(N_{\text{cone}}\) is cone pullout load for combined cone–bond failure (kN); \(N_{\text{bond}}\) is a bond pullout load for combined cone–bond failure (kN); \(d_0\) is hole diameter (mm); \(h_\text{ef}\) —effective depth of chemical adhesive (mm); \(h_{\text{cone}}\) is the chemical adhesive depth of cone for combined cone–bond failure (mm).

| №  | Failure Mode                          | Formula                                                                 |
|----|--------------------------------------|-------------------------------------------------------------------------|
| (1) | Steel                                | \(N_a = A_s \cdot f_y\)                                                 |
| (2) | Cone                                 | \(N_{a,m} = 0.92 \cdot h_{\text{ef}} \cdot \sqrt{f_c}\)                 |
| (3) | Bond (Uniform bond stress)           | \(N_{a,m} = \tau_0 \cdot \pi d_0 \cdot h_{\text{ef}}\)                 |
|     | Bond (Elastic bond stress)           | \(N_{a,m} = \tau_{\text{max}} \cdot \pi d_0 \cdot \left( \frac{\sqrt{\frac{h_0}{d_0}}}{f_c} \right)\) |
| (4) | Combine (Uniform bond stress)        | \(N_{a,m} = 0.92 \cdot h_{\text{cone}}^2 \cdot \sqrt{f_c} + \tau_0 \cdot \pi d_0 \cdot \left( h_{\text{ef}} - h_{\text{cone}} \right)\) |
|     | Combine (Elastic bond stress)        | \(N_{a,m} = 0.92 \cdot h_{\text{cone}}^2 \cdot \sqrt{f_c} + \tau_{\text{max}} \cdot \pi d_0 \cdot \left( \frac{\sqrt{\frac{h_0}{d_0}}}{f_c} \right)\) |

\(A_s\) is the cross-section area of bolt (mm\(^2\)); \(f_y\) is yielding strength of bolt (N/mm\(^2\)); \(h_{\text{ef}}\) is effective depth of chemical adhesive (mm); \(h_{\text{cone}}\) is the chemical adhesive depth of cone for combined cone–bond failure.

The cone failure formula depends on the compressive strength of the material and the installation depth. When the effective height of the bolt is at least equal to the depth of the bolt, this formula should be employed. Shear stress on the embedded surface area of the anchor exceeds the adhesive bond strength before any mode of failure \[19\]. In the literature review, there are two bond models: the uniform bond stress model and the elastic bond stress model \[15,27\]. Studies have shown that the values of these two models give the same value up to an installation depth of \(40\sqrt{d_0}\) \[16\]. The prediction calculations for bond failure are based on the results of concrete shear tests. According to Mansur’s formula \[28\], the pure shear stress of the concrete is considered by Equation (5). The concrete’s compressive
strength is directly related to the equation parameter. Concrete and brick both have material features that are similar.

\[ \tau_0 = 0.56 f_c^{0.615} \] (5)

By substituting the compressive strength of the brick into Equation (5), the shear stress is considered by 4.77 N/mm².

The combined cone–bond failure mode is given by Equation (4) in Table 6. According to the study of Ronald A. Cook, the conditions for the location of one anchor are specified. The depth of the bolt is at least equal to the anchor site. Splitting cracking is expected if the anchor location is smaller than the distance criteria. In the splitting fracture, the flexural strength of brick is estimated to be 3.9 N/mm², (Table 2). As the anchor at a distance is similar to the depth of the bolt, the maximum load capacity will develop as per Equation (3) in Table 6 [16].

### 3.2. Shear Strength

The bolted connection is often used to connect wood to brick. For the case of bolted connections, they receive horizontal loads in most cases for timber design. The European Yield Theory (EYT) was used to estimate the bolt’s horizontal load strength. The NDS standard for estimating shear resistance of the single shear model [29] is relevant to this research. To calculate the shear resistance of the brick-to-lumber specimen, the single shear model was used. The single shear model consists of the main element, side element, and bolt. The main element in our case was brick, and the side element was SPF lumber. Failure mode, diameter, bending strength, length, and dowel-bearing strength are all factors that affect EYT [30]. There are six modes for determining the type of failure modes. The state of the load operating on the element determines the six modes. Both elastic and plastic states are included in the EYT. The elastic state depends on the dowel-bearing capacity of the main and side materials. The plastic state depends not only on the bearing capacity but also on the dowel-bending moment. In the elastic state, there is no deformation of the bolt. Only the main or side material is damaged. The fastener deforms in the plastic state due to dowel-bending, suggesting the creation of a plastic hinge. Figure 8 depicts the single shear model’s likely failure condition as well as the loads occurring on the elements.

Mode I failure is illustrated in Figure 8a,b, and the dowel-bearing strength can cause damage to the main or side member. The bolt did not deform, and it indicates the elastic state. Mode II failure is shown in Figure 8c. The fastener rotated but did not deform. The local crush occurred in the main or side member. Mode II failure indicates the elastic state. Mode III is shown in Figure 8d,e. The bolt was deformed by the dowel-bending strength. In the case of Mode III, the bolt section in the main member was moving down because of the bearing strength of the main member. However, the bolt section in the side member did not deform. This indicates that the dowel-bending moment takes place in this segment. The bolt belonging to the side member makes a plastic hinge. Because a plastic hinge was formed, Mode III denotes a plastic state. Figure 8f depicts Mode IV. Near the shear plane, a local crush occurred. In that situation, both members had two bending moments. Two plastic hinges were formed by two dowel-bending moments. Therefore, Mode IV is related to the level of plasticity.

In the EYT prediction calculation, it is assumed that the length of the main member is 50 mm, length of the side member is 89 mm. The dowel-bearing strength of the side member is 25.8 MPa. For theoretical modeling, the concrete dowel-bearing strength is 51 N/mm² [29]. We conducted a brick dowel-bearing experiment. The dowel diameter was identical to the bolt diameter, assuming chemical adhesive for the N-type specimen. The bolt diameter was 2 mm greater than the dowel diameter for the A-type specimen.
Figure 8. Yield modes and formula. (a) Mode I\textsubscript{m}; (b) Mode I\textsubscript{s}; (c) Mode II; (d) Mode III\textsubscript{m}; (e) Mode III\textsubscript{s}; (f) Mode IV where \( V \) is the shear force of the dowel type fastener (N), \( q_m \) (\( F_m \cdot l_m \)) is the main member dowel-bearing resistance (N/mm), \( q_s \) (\( F_s \cdot l_s \)) is the side member dowel-bearing resistance (N/mm), \( M_m \) is the maximum moment in the main member (N-mm), and \( M_s \) is the maximum moment in the side member (N-mm).

4. Experimental Result

4.1. Pullout Performance

The result of the pullout test is illustrated in Table 7, and Figure 9 shows the experimental failures of three types of specimens. The chemical adhesive attaches completely to

\[ V = q_m \cdot l_m \]  
(a)

\[ V = q_s \cdot l_s \]  
(b)

\[ V^2 \left( \frac{1}{4q_m} + \frac{1}{4q_s} \right) + V \left( \frac{l_m}{2} + \frac{l_s}{2} \right) - \left( q_m \cdot \frac{l_m^2}{4} + q_s \cdot \frac{l_s^2}{4} \right) = 0 \]  
(c)

\[ V^2 \left( \frac{1}{4q_m} + \frac{1}{2q_s} \right) + V \left( \frac{l_m}{2} \right) - \left( M_m + q_m \cdot \frac{l_m^2}{4} \right) = 0 \]  
(d)

\[ V^2 \left( \frac{1}{2q_m} + \frac{1}{4q_s} \right) + V \left( \frac{l_s}{2} \right) - \left( M_s + q_s \cdot \frac{l_s^2}{4} \right) = 0 \]  
(e)

\[ V^2 \left( \frac{1}{2q_m} + \frac{1}{2q_s} \right) - (M_m + M_s) = 0 \]  
(f)
the bolt in each experiment shown in Figure 9. Table 7’s coefficient of variation is indicated by the value in brackets.

Table 7. The result of the pullout experiment.

| Specimen № | The Number of Specimens | Bolt Diameter (mm) | The Max. Pullout Strength (kN) | Displacement at Max. Load (mm) | Stiffness (kN/mm) |
|------------|-------------------------|-------------------|-------------------------------|-------------------------------|------------------|
| P-8        | 3                       | 8                 | 8.8 (29.7)                    | 0.9 (22.9)                    | 15.7 (32.7)      |
| P-10       | 3                       | 10                | 9.5 (37.5)                    | 0.7 (37.4)                    | 26.6 (14.1)      |
| P-12       | 3                       | 12                | 12.1 (42.7)                   | 0.4 (39.2)                    | 46.1 (34.6)      |

Figure 9. Pullout failure on the bricks. (a) P-8-1 specimen; (b) P-10-1 specimen; (c) P-12-3 specimen.

Figure 10 illustrates the load-displacement diagram of the pullout test, including the 5th percentile value in the graphs. The load-displacement diagrams in Figure 10 show that the load goes linearly to reach its maximum load and then decreases. Failure modes in Figure 9 illustrate the state of the combined cone–bond failure mode until the pullout load approaches its maximum value during the test. In addition, the microcracks created around the bolt on the top of the brick can be seen in Figure 9. Then the bricks specimens were split into two parts as the failure extended rapidly to the short side of the upper edge when the maximum load decreased. As shown in Figure 10d, the mean maximum pullout strength increases with rising bolt diameter. As seen in Table 7, the displacement at maximum load progresses with declining bolt diameter. Figure 10d shows the highest pullout strength for the brick specimen with the M12 bolt specimen. The maximum stiffness likewise indicates the brick with an M12 bolt in Table 7. The maximum load of the M8 bolt specimen is 8% lower than that of the M10 bolt specimen and 37.5 percent lower than that of the M12 bolt specimen. The maximum pullout strength of M10 is 27.6% lower than M12 specimens. The stiffness of M12 is the highest result, and it is 2.9 times higher than the M8 specimen and 1.7 times for M10.

4.2. Dowel-Bearing Performance

Figure 11 illustrates the failure of the dowel-bearing test of the brick. During the test, the load was directed evenly downwards until the specimen was broken.

Figure 11b shows that the cracks in specimens initiated from the top side of the brick and propagated in the vertical direction. Table 8 shows the experimental result of the three types of specimens. The value in the bracket indicates the coefficient of variation in Table 8. The mean dowel-bearing strengths of the three types of specimens are similar.

Table 8. The result of the dowel-bearing experiment.

| Bolt | The Number of Specimens | Load (kN) | Dowel-Bearing Strength (N/mm²) |
|------|-------------------------|-----------|-------------------------------|
| M8   | 3                       | 14.6      | 40.9 (66)                     |
| M10  | 3                       | 19.3      | 46.2 (44)                     |
| M12  | 3                       | 26.7      | 43.0 (14)                     |
| Mean |                         |           | 43.4 (6.2)                    |
Figure 10. The load-displacement diagram of the pullout test. (a) Specimen with M8 bolt; (b) specimen with M10 bolt; (c) specimen with M12 bolt; (d) comparison between three types of specimens.

Figure 11. Photo of the dowel-bearing test of brick. (a) Specimen before experiment; (b) failure of the specimen.

The mean strength of the dowel-bearing test (43.4 N/mm²) is slightly reduced to 42 kN/mm² in the prediction calculation (EYT) for the shear resistance of the composite element.

4.3. Shear Performance

The experimental load-displacement diagrams of shear specimens are illustrated in Figure 12. We estimated the yield load for the dowel type connection of composite element obtained from the 5% offset method by McLain 1993 [31]. The yield and maximum load in Table 9 and Figure 12 are two times lower than the experimental load because the computation model is a single shear model. The yield and maximum strength, slip modulus, and ductility ratio are shown in Table 9. The ductility ratio was derived by the relationship between the displacement at yield and maximum load.
Figure 12. The load—displacement diagram of the shear test. (a) N8-type; (b) A8-type; (c) N10-type; (d) A10-type; (e) N12-type; (f) A12-type.

Table 9. The experimental result of the shear test.

| Nº   | Yield Load (kN) | Max. Load (kN) | Displacement at Yield Load (mm) | Displacement at Max. Load (mm) | Slip Modulus (kN/mm) | Ductility Ratio |
|------|-----------------|----------------|-------------------------------|-------------------------------|----------------------|----------------|
| N8   | 4.3             | 7.8            | 2.2                           | 15.5                          | 1.9                  | 7.0            |
| N10  | 4.3             | 7.4            | 1.5                           | 3.7                           | 2.8                  | 2.4            |
| N12  | 6.2             | 6.2            | -                             | 1.6                           | 2.2                  | -              |
| A8   | 6.2             | 9.5            | 0.5                           | 9.2                           | 8.7                  | 17.2           |
| A10  | 7.0             | 9.0            | 0.5                           | 1.3                           | 24.2                 | 3.1            |
| A12  | 11.2            | 12.6           | 0.9                           | 2.1                           | 30.5                 | 1.9            |

The red line highlighted in Figure 12 is the mean value of the three specimens. The displacement in the plastic region of one specimen in Figure 12a–d and the load in Figure 12c are less than the other two specimens. In that situation, the average value of three specimens is estimated until the yield load of the lowest result. Beyond the yield load, the mean of two specimens is calculated in the plastic region. The load-displacement diagram in Figure 12a,b demonstrates the ductility characteristics. The A8 specimens had a ductility
ratio of 2.45 times that of the N8 type. The yield and maximum load of A8 specimens are higher than N8 specimens by approximately 1.8 kN.

Figure 12c,d illustrate the test result of N10 and A10 specimens. For N10 and A10 specimens, the experimental load-displacement diagram presents different results. The dimensions of the brick holes firstly checked the differences in the results of shear tests. It was concluded that the preparation of the brick hole was good. In the second, it is predicted due to excessive tightening of the bolts. During the test specimen preparation, the tightening strength of the bolt was not predetermined and measured. It is assumed that a constant tightening force gives the specimen. This reason may rely on the different results. Table 9 shows that the mean yield load of N10 and N8 specimens are the same. The N10 specimens have a ductility ratio that is similar to the A10 specimen. N10 specimens have a larger slip modulus than N8 specimens, although it is 12 times lower than A10 specimens. A8 and N8 specimens are approximate values for maximum shear stresses, although there is a significant discrepancy in slip modulus.

As seen from the load-displacement diagram of the N12 specimen, the load is linearly going to the maximum load and then suddenly broken. The N12 specimen shows brittle behavior. For A12 specimens, the ductility ratio is close to the result of N10 and A10 specimens. The slip modulus of A12 is 14 times greater than N12 specimens. The maximum shear load of A12 is twice higher than N12.

Figure 13 shows the crack pattern of the shear test specimen. The cracks in A-type and N-type specimens were the approximate failures for all specimens, and the cracks appeared in the main member (brick). The crack started at point “1” and extended to points “2”, “3”, and “4” in Figure 13. The direction of the crack was horizontal or diagonal.

A visual inspection is presented in Figure 14. For N8 and A8 specimens, the bolt is deformed, and it is indicated by a red dashed line in Figure 14a,b. The wood part had no cracks, and the shape of the hole transferred to oval. Figure 14c–f show no deformation on M10 and M12 bolts. Between the bolt-to-brick and the bolt-to-lumber, the chemical adhesive is well attached. It was not easy to separate lumber and bolt in Figure 14b,d,f.
The crack started at point “1” and extended to points “2”, “3”, and “4” in Figure 13. The direction of the crack was horizontal or diagonal. A visual inspection is presented in Figure 14. For N8 and A8 specimens, the bolt is deformed, and it is indicated by a red dashed line in Figure 14a,b. The wood part had no cracks, and the shape

Figure 13. Crack on the brick (main member). (a) N8-1 specimen; (b) A8-1 specimen; (c) N10-2 specimen; (d) A10-2 specimen; (e) N12-3 specimen; (f) A12-2 specimen.
5. Discussion

5.1. Pullout

Figure 7 depicts the prediction failure modes, whereas Table 6 highlights the calculations. Figure 15 represents the comparison result between the experimental and predicted pullout resistances. The predicted pullout failure comprises steel, cones, bonds, and combined bond–cone failure. The mean maximum load of the pullout test is compared with the predicted failure loads.

![Figure 14](image_url). Deformation of bolt and hole of wood (side member). (a) N8-1 specimen; (b) A8-1 specimen; (c) N10-2 specimen; (d) A10-2 specimen; (e) N12-3 specimen; (f) A12-2 specimen.

![Figure 15](image_url). The comparison results of the pullout test and the prediction estimation.
Steel failure prediction load differs by more than 40% from the test load. There was no damage to the bolt. The steel failure is less likely to occur due to the shallowness of the bolt installation.

The predicted cone failure load indicates 12.9 kN for three types of specimens. Since cone failure is directly related to the compressive strength of the brick and the installation depth, the same assumptions are given for the three types of pullout specimens. For specimens with M8 bolts, the experiment and theoretical resistance difference is 48%, 35% for specimens with M10 bolts, and 7% for specimens with M12 bolts. The formula for predicting cone failure is that the anchor distance from the concrete edge is equal to the bolt’s installation depth. In our case, the anchor distance from the brick edge around 25 mm is less than the bolt installation depth (50 mm). This type of failure mode does not occur in the brick because the requirements of the cone failure formula cannot provide for a single brick.

Bond failure [17] can occur between the bolt and the chemical adhesive or the chemical adhesive and substrate element (brick). The adhesion between bolt–epoxy and epoxy–brick has illustrated good results for our experiment. There was no failure in their connection. However, the prediction load of bond failure is close to the test load in Figure 15.

The combined cone–bond failure exhibits both bond and cone failure properties. There is a discrepancy of about 20% in the three types of specimens when the test load is compared to the predicted combined cone–bond failure. The experimental failure modes are both the combined cone–bond failure and splitting failure.

As shown from the four failure mode assumptions shown in Figure 7, it is assumed that the combined cone–bond failure and splitting failure occurred because of the experiment in Figure 9. Failure theories indicate the minimum load value of all possible failure modes. The minimum loads of the prediction calculations are related to the combined cone–bond failure and splitting failure. It is suggested that the estimations of the predicted failure modes and loads are considered reasonable in this study. No case showed damage to the threaded rod during the pullout test. The failure occurred in the weak resistance surface with the high-stress concentration. Our pullout experimental result showed that the brick surface had low resistance, and then the connections between brick to epoxy and epoxy to bolt showed the brick material’s damage. The chemical anchor in the brick wall can show sliding failure. To attach the timber to the existing brick wall, the main role of timber is related to reducing the movement of brick and chemical anchors in a parallel direction to the load. The load acting on the out-of-plane wall concentrates the tensile force on the chemical anchor. In the case of only brick failure, it is necessary to withstand the tensile load for a thick threaded rod having high stiffness and strength. Therefore, the strengthened brick walls with wood are important to reduce out-of-plane damage.

5.2. Shear

Table 10 highlights the comparison results between the experimental and theoretical shear loads. It can be a reasonable estimation value, as seen in the comparison result. The minimal load of EYT modes determines the predicted yield load. For all specimens in the estimation, the minimum yield load of EYT modes is Mode IV. Mode IV failures were found in N8 and A8 specimens, but failures in other specimens were a different mode. As the bolt diameter increases, the yield load is the same for N8 and N10 test results in Table 10. As for the expected yield load of N8 and N10, it increases depending on the diameter of the bolt. For the A-type specimen, the expected and experimental shear load grows by raising the diameter of the bolt. The effect of bolt diameter for N-type specimens is low for test yield load.

Figure 16 illustrates the load-displacement curve of the shear test and the minimum load of EYT mode. In Figure 16, a narrow cross-section fastener (M8 bolt) can show the highest plasticity. In Figure 16a, the maximum shear load decreases as the bolt diameter increases.
The mean slip modulus of the N-type specimens is 2.3 kN/mm in Table 9. However, the slip modulus of the A8 specimen has the lowest value compared with A10 and A12 specimens, but it is four times higher than the slip modulus of the N-type specimen. It suggests that applying chemical adhesives to both the main and side elements increases the strength and slip modulus of the specimen. The use of chemical adhesives in the strengthening process can provide high strength between brick-to-lumber material, according to experimental and theoretical results.

### 6. Conclusions

This study determined and compared the pullout and shear strengths of brick and brick-to-lumber material experimentally and theoretically. The prediction formulas of the pullout and shear strength could give a suitable result.

The pullout strength depends on the bolt diameter. The predicted failure load illustrates the combined cone–bond failure load. The experimental failure mode is also the combined cone–bond failure until the load reaches the maximum value. Then splitting failure occurs when the maximum load gradually decreases. For the single brick, the probability of steel and cone failure occurring is small.

The yield shear strength of the composite specimen (brick-to-wood) on the chemical anchor is reasonable to expect in the European Yield Theory (EYT). However, it is complex to determine the failure mode. When it comes to increasing the strength of masonry construction, the A12 type is recommended since it provides excellent strength and stiffness.

The limitation of this study is that the substrate material is only brick, and the bolt depth is shallow. The number of specimens can be at least three. However, if high deviations are shown, the number of tests should be increased, and the specimens should be well prepared. However, this study is applicable to investigate the strengthening technique of the unreinforced brick wall using timber material. The future contribution of this
research will consider the in-plane shear behavior of the unreinforced brick wall with a wooden panel.

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