Behaviour of an in-plane steel stiffening solution for timber floors
Experimental and Numerical Study
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Abstract: The seismic vulnerability of masonry buildings is strongly dependent on the characteristics of timber floors, on their in-plane stiffness and on the quality of connections between the floors and the walls. It is well-recognized that adequate connections and in-plane stiffness can improve the three-dimensional response of buildings. In this work a steel strengthening solution is proposed to improve the in-plane stiffness. For that purpose, five types of connections (steel bar-beam angle connection) were tested and a cycle test was carried out on a real scale strengthened timber floor. Furthermore, a numerical modelling of the strengthening solution was developed. All types of connections tested had screws. The connection types differed in the addition of epoxy resin, the steel surface preparation and the surface protection. The PM connection (screws and surface pickled and protected by metallization) was chosen to be used in the strengthening solution because of its better behaviour. The strengthening solution proposed was numerically modelled with SAP2000. Only linear models were developed as it is expected that the reinforcing structure remains elastic when the building is subjected to seismic action. The numerical model with the PM connection’s stiffness and without the steel bars under compression provided the results that best fit the experimental ones. The experimental cycle test, showed that the strengthened timber floor performed well. In fact, the strengthening solution improved the floor resistance and its in-plane stiffness. The loss of strength was not significant in the linear range. The strengthened floor deformation was essentially caused by shear deformation.

Key-words: Old Buildings; Timber floors; In-plane strengthening technique; Seismic behaviour; In-plane stiffness

1 Introduction

In historical cities, such as Lisbon, the percentage of masonry buildings with timber floors is high. These buildings are very important as evidence of a vast patrimonial, constructive and historical legacy. Vertical elements are essentially responsible for providing building seismic resistance, with timber floor contribution being almost irrelevant. When submitted to horizontal actions, such as seismic activity, the timber floors show a flexible behaviour failing to properly distribute and transfer forces to the lateral load resisting walls. The large floor deformations do not prevent overturning of the walls and, consequently, the building’s collapse (Figure 1) (Piazza et al., 2008). It is generally well-recognized that an adequate in-plane stiffness and proper connections can significantly improve the three-dimensional response of these buildings.

In Lisbon’s housing stock the construction typologies with timber floors are: buildings built before 1755; “Pombalino” buildings; “Gaioleiro” buildings and mixed masonry-reinforced concrete buildings (also known as “placa” buildings). The percentage of existing masonry buildings with timber floors is high, therefore, it is important to propose a retrofit intervention to the floors to reduce the seismic vulnerability of this types of buildings. The study of some strengthening solutions has been conducted to increase the in-plane stiffness, i.e. reduce the in-plane deformations. These solutions have the aim of improving the floors contributions to obtain a better distribution and transference of forces to the lateral load resisting walls and, consequently, a better seismic behaviour of buildings.
In this work, a strengthening solution in steel is proposed to improve the in-plane stiffness for timber floors. For such, five types of connections (steel bar-beam angle connection) were tested and a cycle test was carried out on the real scale strengthened timber floor. Furthermore, a numerical modelling of the strengthening solution was developed.

2 Previous experimental studies to improve the in-plane stiffness

Experimental studies of different kind of strengthening solutions have been conducted in order to increase the in-plane stiffness. Fragomeli (2015) studied the nails disposition, in which the nails were placed along a zigzag line for each beam, with two nails per plank. The purpose was to maximize the distance between the nails on the same board, to increase the torque produced by the rotation of the board in relation to the joist. Corradi et al. (2006), studied the stiffness enhancement by increasing the number of the nails. Both solutions were unsuccessful and did not result in the in-plane stiffness increase.

Piazza et al. (2008), Valluzzi et al. (2010) and Corradi et al. (2006) studied the addition of extra layers of planks with different angles to the original floor. According to Piazza et al. (2008), adding a layer of planks at an angle of 45° to the original flooring increases the in-plane stiffness around eleven times more than the original floor. Valluzzi et al. (2010) verified that the addition of a layer of planks improves the stiffness, such as Piazza et al. (2008). Valluzzi et al. (2010) verified a higher improvement with the addition of two layers of planks, but this increase was not proportional to the total number of layers added. According to Corradi et al. (2006), the addition of one layer in perpendicular disposition as reinforcement attained best results by including GFRP strips glued to the planks with epoxy resin between this layer and the original flooring.

Piazza et al. (2008) tested as strengthening solutions the application of diagonal bracing resorting to light gauge steel plates screwed to the boards and wide strips of CFRP glued to the planks with epoxy resin. Both bracing systems were displayed in grid with the same layout. Among the solutions tested by this author, the system with CFRP strips displayed the best results when in relation to the steel strips and the addition of a layer of planks.

Further to the strengthening solutions with the addition of one and two layers of planks studied by Valluzzi et al. (2010), he also studied the application of a single diagonal punched metal strip, a single diagonal plank placed at 45° with the original boarding and a double diagonal plank placed at ±45° with the original boarding. Of all the solutions tested by Valluzzi et al. (2010), the solution with the addition of two layers of planks showed the best results in increasing the stiffness, followed by the solution with the addition of a layer of planks, then the solution which placed a double diagonal planks positioned at ±45°, then the single diagonal punched metal strips and lastly the single diagonal plank positioned at 45°.

The addition of a plywood panel, addition of steel elements around the diaphragm panel, application of metal sheet-blocking stapled between the flooring boards or between the plywood panels and shear connectors to improve flexible lateral support was studied by Brignola et al. (2012). The experimental studies concluded that the application of metal sheet-blocking, the plywood panels and the steel chord increase the in-plane stiffness. The addition of shear connectors decreases the stiffness.

A reinforced concrete slab connected to the timber beams and the application of three layers of plywood panels are the strengthening techniques which provide the higher in-plane stiffness (Piazza et al., 2008).

3 Strengthening solution specimen

The timber floor specimen used to build and to test the strengthening solution in this study is the specimen studied by Fragomeli (2015). The timber floor specimen was at real scale. The strengthening solution was a reticulated steel structure and the joints were done by Hilti S-MD 55 GZ 5,5x52 screws and Hilti S-MP 63 S 6,5x100 screws (Figure 2). In the steel bar-beam angle connections Hilti S-MD 55 GZ 5,5x52
screws were used and in the timber floor-steel strengthening connection Hilti S-MP 63 S 6,5x100 screws were used. For each corner connection, a M24 headless screw and a steel plate (220x80 mm²) with 10 mm thickness was used. The steel plate was welded to the beam angle with smaller length. The steel strengthening solution was placed under the timber floor (the original board flooring will be exposed) and connected to timber beams and to the lateral walls. Assuming that the rooms will have plasterboard ceiling, which is common in rehabilitation works of this sort of old masonry buildings, the strengthening solution will not be visible.

The steel structure has an element around the diaphragm perimeter (tie ring) to transmit the shear forces to the lateral walls. This ring tie is composed by a steel beam angle with unequal legs. The big leg allows the bar-beam angle connections and the short one the lateral walls connection. The steel used for the beam angles (100x50x8 mm²) and for the bars (100x5mm²) was S275 JR steel. The steel structure does not increase the floor and building weight significantly and also provides a good structural behaviour. The steel bars are placed at 45º with the tie ring and the bars have the same spacing between them (566mm). The steel bars exhibited 100x5 mm of transversal section and the beam angle 100x50x8 mm of transversal section.

4 Numerical analysis

The strengthening solution proposed was numerically modelled with SAP 2000. This intends to simulate the steel structure behaviour integrated in a building when it is submitted to seismic actions. Only linear analysis was developed as it is expected that the reinforcing structure remains linear when the building is subjected to seismic action.

4.1 Model description

4.1.1 Geometrical properties and mechanical properties of materials

The beam angle and the bars were modelled with bars elements. Table 1 presents the mechanical properties of the steel. The connection of the strengthening solution to the lateral walls is considered at boundary conditions with the masonry walls parameters presented in Table 2.

<table>
<thead>
<tr>
<th>Steel</th>
<th>γ (kN/m³)</th>
<th>E (GPa)</th>
<th>α</th>
<th>G (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S275</td>
<td>1.00</td>
<td>21</td>
<td>0.3</td>
<td>80769.231</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Masonry</th>
<th>E (GPa)</th>
<th>Thickness (m)</th>
<th>Influence area (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry</td>
<td>1</td>
<td>0.6</td>
<td>0.6 x 0.8</td>
</tr>
</tbody>
</table>

The timber floor-steel strengthening connection and the corner connections were modelled with swivel joint behaviour. The bar-beam angle connections were defined according to the force and displacement of experimental results of connection type PM (chapter 5.1). The modulation of these connections consisted...
in changing the section proprieties of a bar section with the equivalent stiffness of type PM.

4.1.2 Support conditions (boundary conditions)

The displacements outwards the plane, according to Z-axis, were restrained in the ring tie (beam angle) knots (Figure 3). In one side of the strengthening solution, alignment A, the translation movements according the Y-axis were restrained. The rotation movements of all the knots were allowed. The connections between the lateral walls to the strengthening solution were modelled, in alignment A and K, by resorting to springs with the same axial stiffness of a masonry walls piece, according to X-axis. The probability of there being masonry walls in both alignments, A and K, in a building is reduced. However, there is always the beam angle of the compartmentation room on the other side whereby was considered the same stiffness for both alignments. The axial stiffness determination of the masonry walls piece \( k_{\text{mason}} \) corresponded to the influence area and half of the thickness \( L_{\text{mason}} \) previously indicated (Equation 1).

\[
k_{\text{mason}} = \frac{EA}{L_{\text{mason}}} = 1600000 \text{ kN/m} \quad (1)
\]

4.1.3 Loading

The loading was applied along the beam angle, present in alignment K, according to the Y-axis and with positive direction. In a first stage, the loading applied was defined according to the response spectrum of seismic action, recommended by EC8 (2009). For such, buildings representative of construction typologies were analysed: “Pombalino” building (Ponte, 2017), “Gaioleiro” building (Frazão, 2013) and “Placa” building (Ferreira, 2014 e Monteiro et al., 2012, 2013). In a second stage, the quantification of loading depended on model analyses with the secant stiffness of bar-beam angle connections.

To determine the loading, according with the EC8 (2009) and National Annex (2009), the masses of elements of the buildings in study were estimated. Afterwards, the base shear force \( F_b \) in each building (according to the axis parallel to the sidewall) and the corresponding horizontal forces equivalent of each floor were calculated \( F_i \). It was considered, in a simplified way, that sidewall mobilize the total efforts. To calculate the base shear force, the frequencies of the buildings \( f_{\text{ou,Y}} \) obtained by dynamic characterization tests were used. Lastly, the loading \( F_{\text{sidewall}} \) is given by half of the equivalent force without taking into consideration the mobilization of the sidewall’s mass of the most conditioning floor (Equation 2) (Figure 4). The loading value determinates \( F_{\text{sidewall}} \) was to be divided by the length of the sidewall to be introduced in the model \( f_i \) (Table 3).

\[
F_{\text{sidewall}} = F_i \times \frac{\text{perimeter sidewalls area}}{m} \quad (2)
\]

4.2 Stress and displacement analysis

4.2.1 Loading according to the building case studies

In a first stage, the analyses of the model that considered the most conditioning loading, determined in section the 4.1.3 \( (f_i=32.6 \text{ kN/m}) \) taking as a variable the contributions of the axial resistance capacity of the bars in compression. Three cases of resistance contribution of compression force bars were held, which are: 100% resistance, null resistance and 4.3% resistance. The percentage of 4.3 was determined by the ratio between the value of resistance force of bar in compression and the resistance force of bar in tension. The model analyses had as goal to estimate the number of screws to be applied to bar-beam angle connections and to evaluate the secant stiffness influence of these connections in the stiffening solution behaviour.

To define the bar-beam angle connections behaviour, the correspondence area of yielding secant stiffness for these connections was determined. The stiffness values (Equation 3) refer to the medium values of

\[
k_{\text{secant}} = \frac{\text{length of the bar}}{\text{section bar}} \quad (3)
\]
The axial force of the more solicited bar in tension is 

\( T = \frac{F_{PM}}{\delta_{PM}} \) (Equation 3).

\[ k_{\text{axial}} = \frac{F_{PM}}{\delta_{PM}} \] (3)

\[ k_{\text{axial}} = \frac{E A}{L} \] (4)

Figure 5 - Illustration of the length (L) of the connection (measures in mm).

The displacement, the force value of condition bar and stiffness \( k \) (loading divided by displacement) are presented in Table 4, for two different model analyses, one without the influence of yielding secant stiffness connections (definition of connections) and the other with it (evaluation of connections influence). For both model analyses, different resistance contribution of the compression bars was considered. The displacement was measured at the k alignment.

Table 4 - Displacement, axial force of condition bar and stiffness (k) of the strengthening solution.

<table>
<thead>
<tr>
<th>Definition of connections</th>
<th>Evaluation of connections influence</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression bars contribution</td>
<td>100% 4,3% 0% 100% 4,3% 0%</td>
</tr>
<tr>
<td>Displacement (mm)</td>
<td>0,8 2,5 3,1 1,1 2,8 4,6</td>
</tr>
<tr>
<td>Axial force of condition bar (kN)</td>
<td>20,62 56,25 69,23 21,02 51,52 66,30</td>
</tr>
<tr>
<td>K (kN/mm)</td>
<td>138,91 44,45 35,85 101,03 39,69 24,16</td>
</tr>
</tbody>
</table>

The axial force of the more solicited bar in tension is 69,23 kN and the project shear resistance of each screw was 5,78 kN (ETA-10/0182, 2013). Therefore, twelve screws were necessary for the bar-beam angle connection in order for them to hold the axial force of the more condition bar. The model with zero contribution of the compression bar was chosen to determine the number of screws because the analysis is more conditioning, thus, the design was more conservative.

With the introduction of yielding secant stiffness of bars-beam angle connections, it was verified that the displacement increased and the stiffness of the model decreased. This fact is due to the stiffness of the bars-beam angle connections being less than the stiffness of the bar corresponding to the same section.

4.2.2 Loading according to the connection behavior

In this section, the intent was to quantify the necessary loading for the yielding and rupture points of connections to happen. This loading was estimated according to the axial force in the more solicited bar. When this bar attains, the stresses corresponding to the yielding (75,6 kN) and rupture (98,66 kN) connection, it is assumed that the stiffness model enters in yielding and rupture, respectively. The simulation of the bars-beam angle connections was the same as previously described. In the case of the reinforcement rupture, the connections area was 7,69 mm². The displacement, the force value of condition bar, the loading and the stiffness \( k \) (loading divided by displacement) are presented in Table 5.

Table 5 - Displacement, axial force of condition bar and stiffness (k) of the strengthening solution.

<table>
<thead>
<tr>
<th></th>
<th>Strengthening solution yielding</th>
<th>Strengthening solution rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression bars contribution</td>
<td>4,3 % 0 % 4,3 % 0 %</td>
<td></td>
</tr>
<tr>
<td>Displacement (mm)</td>
<td>4,2 5,3 6,8 16,9</td>
<td></td>
</tr>
<tr>
<td>Axial force of condition bar (kN)</td>
<td>75,63 75,64 98,66 98,66</td>
<td></td>
</tr>
<tr>
<td>Loading (kN/m)</td>
<td>47,84 37,18 66,63 53,62</td>
<td></td>
</tr>
<tr>
<td>Loading (kN)</td>
<td>163,13 126,78 227,21 182,84</td>
<td></td>
</tr>
<tr>
<td>K (kN/mm)</td>
<td>38,84 23,92 33,41 10,82</td>
<td></td>
</tr>
</tbody>
</table>

According with the analyses, it was verified that the inclusion of compression bars in the modelling confers a lot of stiffness to the reinforcement, although the contribution is reduced, 4,3%. The model analyses for rupture loading is not reliable, because this one is in a nonlinear range and the modulation only works in a linear range.

5 Experimental Campaign

The experimental campaign was divided in two stages. In the first one, different types of bar-beam angle connections were tested under tension. In the second stage, a cycle test was carried out on a real scale strengthened timber floor, with the bar-beam angle connections type that provided the best behaviour in the previously experimental tests.

5.1 Connections experimental study

To characterize the bar-beam angle connections behaviour, five types of these connections were tested under tension, resorting to three samples of each type. The tests were carried out according to the procedure described in ASTM D 1002-10 (2010). The samples
had the real dimensions of this connection (Figure 6). The bonding materials used to fix the steel elements (bars) were the Hilti HIT-RE 500 epoxy resin and the Hilti S-MD 55 GZ 5,5x52 screws.

The types of connections tested were:
- Connection with screws (P);
- Connection with screws and epoxy resin, in which the steel surface was pickled by sandblasting (PCA);
- Connection with screws and epoxy resin, in which the steel surface was pickled by shot peening steel shots (PCG);
- Connection with screws and epoxy resin, in which the steel surface was pickled by shot peening steel shot and the surface protected by metallization (PCM);
- Connection with screws, in which the steel surface was pickled by shot peening steel shot and the surface protected by metallization (PM).

Taking into consideration that the same materials were used in the experimental displacement determination for connections types PM (0.71 mm) and the results were extrapolated, Equation 5 is proposed. This equation defines the connection displacement ($\delta_{\text{connection}}$) regarding the screw radius ($r_{sc}$) and the bar thickness ($e_{\text{bar}}$). The equation considered that the connection has an elastic linear behaviour and the stresses are proportional to the displacement. It is considered that the stresses around each screw in the bars present an uniform distribution in all the bar thickness ($e_{\text{bar}}$) with the length equal to the screw diameter.

$$\delta_{\text{connection}} = 0.71 \times \frac{r_{sc}}{e_{\text{bar}}} \times \frac{5.4}{5}$$

With this equation and the project shear resistance of the connection (ETA-10/0182) it is possible to determine the connection secant stiffness without doing any experimental work.

### 5.2 Timber floor strengthened experimental study

#### 5.2.1 Specimen Properties, test setup, instrumentation and loading protocol

A cycle test was carried out on a real scale strengthened timber floor. The timber floor specimen used to build and test the strengthening solution was the same studied by Fragomeli (2015). The specimen test is represented in Figure 2. Because of economical reasons, the steel surfaces were not pickled and metallized, therefore, the type of connections used was the type P instead of type PM, as expected. The P connections showed values of resistance and displacement similar to the PM connections, therefore no disadvantages is noticed.

The specimen and test set-up is represented in Figure 8 a) and b). Figure 8 c) shows a drawing of the anchorage system onto the horizontal reaction beam.
This system is very stiff and avoids the body motion of the specimen while loaded. The specimen is anchored to the horizontal reaction beam with screws (M24, M14 and M12) and a welding system (UPN 80 and both steel plates). The out-of-plane movement of the specimen was restricted by a system of lateral roller bearings supported in a metallic frame. The force transmission system is composed by an UPN 300 connected by means of M14 screws to the timber beams and “L” steel shapes connected to the top of reinforced solution.

Seventeen displacement transducers (LVDT) were used in the full-scale experimental tests in order to verify the boundary conditions, the lateral displacement and the rotation of the specimen. The cyclic tests were conducted under displacement control by means of the horizontal LVDT connected to the left side of the specimen. The loading was applied on the right-hand side top corner of the specimen, using a 1000 kN capacity actuator with a 400 mm stroke.

Cyclic tests were performed in accordance with EN 12512 (2006) and each cycle was repeated three times. The historic displacement was estimated (Figure 8 d)), based in the modelling yielding displacement (0.53 mm). The modelling with 5.3% compression bars resistance contribution and the equivalent secant bars resistance of the steel bar-beam angle connection was the modelling considered.

5.2.2 Experimental results

The hysteresis curve presented in Figure 9 (horizontal force-displacement relationship) represents the strengthening timber floor. Table 7 presents the relevant values of the force and displacement related to the first shear rupture of screw, the moment when the buckling of compression bars was visible, the maximum lateral load and the failure load.

Figure 8 – a) and b) the specimen and test set-up; c) anchorage system; d) displacement history.

Figure 9 – Hysteresis curve.
The hysteresis curve exhibit an asymmetry between the positive and negative displacements, in which the higher resistance values are found on the side of the positive displacements. This asymmetry difference could be due to the fact that the superficial bars can suffer buckling easier than the deeper ones, and this may cause a loss of resistance when these bars are under tension in the following cycle. Another reason could be the “protection” of the deeper bars compared to the superficial ones because it might require more force to get the same displacement of the deeper bars under tension compared to the loading in the opposite direction in which the superficial bars are under tension. Up until the 8.48 mm of displacement there was no significant loss of energy. At this point, the shear rupture of a screw occurs and the dissipated energy highly increases along the following displacements. Figure 10 shows the evolution of the dissipated energy for each cycle. The dissipated energy was determined by the calculation of the area defined by the hysteresis cycles.

<table>
<thead>
<tr>
<th>Table 7 – Relevant values of force and displacement.</th>
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<tbody>
<tr>
<td>Negative displacements</td>
</tr>
<tr>
<td>Buckling of compression bars</td>
</tr>
<tr>
<td>First shear rupture of screw</td>
</tr>
<tr>
<td>Maximum lateral load</td>
</tr>
<tr>
<td>Failure load</td>
</tr>
<tr>
<td>Positive displacements</td>
</tr>
<tr>
<td>Maximum lateral load</td>
</tr>
<tr>
<td>Failure load</td>
</tr>
</tbody>
</table>

For each amplitude of displacement, the cycle was repeated three times. Up until the 6.36 mm of displacement, the loss of resistance between these three cycles was low (less than 6 %), however, for bigger amplitude of displacements, the loss of resistance increased considerably (±10-15 %). This can be associated with the rupture of some screws and the hardening steel structure.

The global deformation (d) of the strengthening timber floor was caused by shear deformation (δshear) and flexural deformation (δflexural). In order to determine the flexural deformation of the panel rotation was evaluated and an approximation of the behaviour panel to the cantilever behaviour was made. The shear deformation is much more important than the flexural deformation in the strengthen timber floor (Figure 11). For small displacement, the flexural deformation presents slightly higher values although the shear deformation always remains higher. The flexural component is higher in the strengthen timber floor than in the timber floor without strengthening solution because the steel structure has the ability to bend.

![Figure 11 – Ratios of shear and flexural components to the total displacement.](image)

Figure 12 depicts the envelope of the load-displacement curve and the values of the envelope stiffness. Table 8 shows the values of stiffness and equivalent yielding point of the strengthen timber floor and the timber floor.

The strengthen timber floor revealed a considerable in-plane stiffness. The displacement was 36.1 mm and the force was 144.81 kN for the failure load. In the case of the timber floor without the strengthening solution (Fragomeli, 2015), the displacement was 110 mm and the force was 22.35 kN. Up until the amplitude displacement of 10 mm, the hysteresis curve of the strengthen timber floor exhibited an in-pane stiffness that is much higher than the registered by Fragomeli (2015).
The values of the elastic stiffness, the equivalent elastic stiffness \(\left( G_{eq} \right) \) and the yielding point of the strengthened timber floor were much higher than the values of the original timber floor. The stiffness increase reveals that the strengthening solution meets the purpose for which it was designed, to increase the in-plane stiffness with a better diaphragm behaviour. Comparing the results of this case study with the strengthening solutions, described in section 2.4, the strengthening solutions proposed revealed an excellent performance. Between the different analytic model options, studied in section 4.2.2, the model which presented the secant yielding stiffness of bar-beam angle connections and did not take into consideration the compression bars was the model that provided the most similar results when comparing with the experimental results.

6 Conclusions and future developments

6.1 Conclusions

For the numerical analyses of the strengthening solution in the linear range, three buildings of different typologies were studied. Then, the base shear force and the equivalent horizontal forces were determined for each one according to EC8 (2009). By using the numerical modelling of the strengthening solution with the conditional loading (Table 3) the number of Hilti S-MD 55 GZ 5,5x52 screws was calculated. Twelve Hilti S-MD 55 GZ 5,5x52 screws were necessary in each bar-beam angle connection to resist the axial force in the most conditional bar.

Five types of connections (steel bar-beam angle connection) were tested each with twelve screws. The difference between the connections types was the addition of epoxy resin, the steel surface preparation and the surface protection. The connections with the best behaviour were the ones without epoxy resin (P and PM). The connection type PM was the one chosen to use in the steel strengthening solution proposed, because it was more effective, compared with type P, and the steel surface protection was already done (metallization).

With the experimental results of type PM, an equation was suggested to determine the bar-beam angle connection displacement in the elastic range, regarding the screw radius and the bar thickness. With this equation and the design resistance of the connection (according with the shear resistance of screws in ETA-10/0182) it is possible to estimate the connection stiffness without realizing experimental tests.

A cyclic test was performed on a real scale strengthened timber floor at real scale with the type connection P. Through the experimental test results it was possible to conclude:

- The strengthening solution offers a significant increase of in-plane stiffness to the timber floor. The in-plane stiffness of the timber floor was almost null;
- The hysteresis curve exhibited an asymmetry between the two displacements direction. In order to avoid this, the chess disposition between rooms in a building is suggested. The chess disposition is suggested when applied the strengthening solution in a building. There exist two layers of bars, the deeper one and the superficial one. Therefore, the idea is alternating the direction of the deeper bars and the directions of the superficial bars, from room to room.
- The displacement history does not influence the efficiency of the strengthening solution significantly in linear range. For this range the loss of resistance between cycles of the same amplitude of displacement was reduced;
- The shear component of displacement is much higher than the flexural component in the strengthened timber floor;
• The values of loading, displacement and stiffness results from the analytic model which do not have the compressions bars represented and which considered the yielding stiffness of the bar-beam angles connections was similar to the experimental results. For the plastic range the developed analytic models do not show a good representation of the strengthening solution.

6.2 Future developments

Even though satisfactory results were obtained from this study further recommendations will be suggested for future studies.

• Due to the satisfactory results obtained in this study and the possibility of implementation of this strengthening solution in buildings, it is suggested the study of the strengthening solution in larger areas of timber floor;
• The study of the connections between the strengthening solution and lateral walls, masonry walls and interior walls, is suggested;
• It is proposed the calibration of the displacement equation of bar-beam angle connections, through experimental tests and numerical analysis;
• For the practical application of the strengthening solution it is proposed the realization of more analytical models for better calibration of the floor stiffness.

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