Challenges for Civil Construction

BRIDGING SCIENCE AND APPLICATIONS WITH ENGINEERING TOWARDS INNOVATIVE SOLUTIONS FOR CONSTRUCTION

Safety, Sustainability and Rehabilitation with Innovative Solutions

U. PORTO FEUP FACULDADE DE ENGENHARIA UNIVERSIDADE DO PORTO

home

Originally conceived in the Faculty of Engineering of the University of Porto (Portugal) in 2001, the CCC Composites in Construction Conference moved to Rende (Italy) in 2003 and to Lyon (France) in 2005. Thinking about the new challenges that are increasingly being put forward in civil construction, namely to provide innovative and sustainable materials and technical solutions, it was decided to redefine symbol CCC to mean **Challenges for Civil Construction**, as it appears in the present edition of the **CCC2008** International Conference.

The Organizing Committee is convinced that the **CCC2008**, with the aim of "Bridging Science and Engineering Applications Towards Innovative Solutions for Construction", will provide a forum for dissemination of new design and construction solutions for Civil Engineering Structures, including issues of great actuality such as rehabilitation of built heritage and health monitoring.

It is the central aim of **CCC2008** to bring together engineers, researchers and companies concerned with the new challenges in Civil Construction. For this purpose a set of topics such as Advanced Monitoring Systems, New Cement-Based Materials, Rehabilitation and Durability, Innovative Applications, New Construction Techniques and Systems, Guidelines and Codes and Numerical Modelling was selected to fit the new Conference scope, which received 90 papers, reviewed by an International Scientific Committee. Furthermore, four worldwide recognized researchers were invited to present the following keynote lectures:

- **Frieder Seible** (University of California, USA):
  "Safety of the New San Francisco-Oakland Bay Bridge"

- **Christian U. Grosse** (University of Stuttgart, Germany):
  "Monitoring of Structures Using Wireless Sensors and Acoustic Emission Techniques"

- **Pedro Pacheco** (University of Porto, Portugal):
  "Movable Scaffolding Systems Strengthened with Organic Prestressing"

- **Michael Edén** (Chalmers University of Technology, Sweden):
  "Design for Sustainable Building. A Swedish Perspective"

Finally, a special session in **CCC2008** was dedicated to PhD students, providing them an opportunity to present underway R&D activities. The best contribution will be awarded with a prize.

The present Proceeding includes printed versions of the extended abstracts and keynote lectures, and the CD-Rom includes electronic versions of the full-length papers, published as prepared by and under the responsibility of the authors.

The Organizing Committee of **CCC2008** wishes that this will be the basis of a long series of Conferences on updated topics related with Challenges for Civil Construction.

Porto, FEUP, April 2008
A. Torres Marques
(Chairman of the Organizing Committee)

Challenges for Civil Construction
Safety, Sustainability and Rehabilitation with Innovative Solutions
U.PORTO
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2. NUMERICAL MODELLING
3. NEW CEMENT-BASED MATERIALS
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REHABILITATION OF BRICK MASONRY

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Keywords: Masonry, Cracked walls, Serviceability limit states, Real scale tests, Rehabilitation technique.

Summary: *This paper presents the behaviour evaluation of perforated clay brick masonry walls related to serviceability limit state. In order to compare the deformation of the perforated bricks simple leaf walls real scale tests were performed. A rehabilitation technique for the cracked area is proposed. An experimental verification of its behaviour after the intervention was made. The tests results allowed concluding that the deformation values presented on the existing standards for reinforced concrete structures are not compatible with the masonry walls deformability capacity. The solution used on the walls rehabilitation gave a better behaviour, in terms of strength capacity and deformation, relatively to the reference walls.*

1 INTRODUCTION

Due to the technological advance of reinforced concrete and steel structures, and the consequent increase of the span of beams and slabs, as well as fast building execution, special care with the design of masonry walls become essential. In this process two construction subsystems, structure and masonry, need to be matched. For this, several points need to be focus, namely the deformation level. This aspect is particularly relevant because the values currently used for the structures deflection generally interfere with the masonries, adversely affecting its proper functioning or appearance.

Beams and slabs bend due to their own weight, permanent and accidental loads and the concrete's shrinkage and creep. The structural components admit deflections that do not compromise either their own appearance or the construction's stability or strength. However, these deflections may not be compatible with the deformation capacity of the walls or other components that are part of buildings [1].

In buildings, it is commonly accepted that the masonry is the most sensitive component to cracking due to the deformation of their support. The studies made by Pfeffermann [2, 3] with clay bricks masonry (walls of 7.50 m length and 2.50 m height) reported the appearance of the first crack on the masonry when the deflection of the support beam was just 6.54 mm, which is L/1150. These studies [2,3] still mention the appearance of cracks in the masonry even with deflections of about L/1500.

The maximum allowable deflection (for long time) imposed by the REBAP [4] is the minimum of two values: L/400 or 15.0 mm, if the deflection affects division walls in below levels. According to the more severe Belgium prescription, the relative and instantaneous deflection of slabs, in which walls are supported, should not pass L/2500., recommends that The maximum deflection of floor slabs recommended by Mathez cited in Pfeffermann [5] should not pass L/1000.

There is no agreement about the admissible deflection value for beams and slabs where masonry is supported. Nevertheless, the values shown before are very inferior to the ones admitted by the REBAP [4]. In fact, experimental studies are needed to evaluate which deformations of structures could become compatible with the deformations of other components of buildings.

2 EXPERIMENTAL EVALUATION

The aim of this experimental evaluation was to analyze the behavior of simple leaf masonry walls, constructed with perforated clay brick, under the point of view of the maxim deformation until the appearance of the first crack. After, these deformations were compared with the maximum deformations allowed by the structural component of support. Another objective was the proposal of a rehabilitation technique for the cracked area and the experimentally verification of the behaviour after the intervention.

Two types of clay brick masonry walls were considered:

- "PD11MO1" in masonry with the components in perforated brick 30x20x11 cm and mortar type "MO1", with cement and sand in a proportion by volume of 1:5;
- "PD11MO2" in masonry with the components in perforated brick 30x20x11 cm and mixed mortar type "MO2", cement, hydrated lime and sand in a proportion by volume of 1:2:9.

2.1 Laboratory's experimental evaluation

The characteristics of the components of the masonry, as well as the masonry's specimens, were evaluated by laboratory tests:

- **Fresh mortars**: consistence and air content [6,7];
- **Hardened mortars**: compressive and flexural tensile strength [8];
- **Bricks**: dimensions, water absorption and compressive strength [9,10,11];
- **Masonry specimens**; compressive strength and Young's modulus [12].

A theoretical–experimental evaluation was also made according to EC6 [13] expressions, following the experimental knowledge of mechanical characteristics of wall components. Tables 1 and 2 summarize the results obtained.

Table 1 : Laboratory characterization

CHARACTERISTIC	UNITS	MORTARS		BRICKS	BRICKWORK	
		MO1	MO2	30x20x11	PM11MO1	PM11MO2
Consistence	mm	141	150			
Air content	%	30.4	20.4			
Flexural tensile strength, f	N/mm²	1.3	0.5			
Compressive strength: fm; fb and fk	N/mm²	3.5	1.6	2.1	0.8	0.5
Water absorption	%			14.8		
Young's modulus, E	N/mm²				2006	879

Table 2 : Theoretical-experimental mechanical characterization

MECHANIC CHARACTERISTIC	UNITS	Type	BRICKWORK	
			PM11MO1	PM11MO2
Compressive strength, fk	(N/mm²)	E	0.8	0.5
		T	0.8	0.6
Young's modulus, E	(N/mm²)	E	2006	879
		T	800	600
Shear strength, fkv	(N/mm²)	E		
		T	0.1	0.1

Note:
E – Experimental values
T – Theoretical-experimental values

2.2 "In situ" experimental evaluation

The tests took place in a building site located in the North of Portugal, in external environment conditions. The clay brick masonry walls considered for this research were built with 4.00 m length and 2.00 m height with an estimated self-weight of 8.0 kN. The constituent components were of two types:

"PD11MO1" and "PD11MO2".

2.2.1 General Considerations

As mentioned before, the REBAP [4] establishes for the structural components a long term deflection of:

$$f_{lp} = \frac{L}{400} \tag{1}$$

Admitting that, the long term standard deflection is given by:

$$f_{lp} = 2.5 \times f_{inst} = \frac{L}{400} \tag{2}$$

where:

f_{lp}- long term deflection of the structural component
f_{inst}- instantaneous deflection of the structural component
L - span

Resulting:

$$f_{inst} = \frac{L}{1000} \tag{3}$$

When the wall was built, the support and the above structural components were already deformed. So, the wall settlement will follow, along its plan, the development of the deformation already introduced on the adjacent components. These initial deformations, f_{cpr}, are a result of the own weight of the structural elements, and in current cases they correspond at about 45% of the total loading application. Consequently, they correspond to a 45 % mobilisation of the instantaneous deflection of the support. When applied the total load of the building the remaining part of the support instantaneous deflection will be mobilised. Therefore, in this phase, 55% of the instantaneous deflection will occur which correspond to a deflection f_{cr} (Figure 1).

So:

$$f_{inst} = f_{cpr} + f_{cr} \tag{4}$$

Where:

$$f_{cpr} = 0.45 \times f_{inst} = \frac{L}{2222} \tag{5}$$

$$f_{cr} = 0.55 \times f_{inst} = \frac{L}{1818} \tag{6}$$

The remaining value, f_{cr}, that is going to be mobilised by the support, after the mobilisation of the deflection inherent to the own weight of the structural elements, is perceptibly the same as the value obtained for the deflection on the bottom of the wall, f_{pi}. This means that, probably, the anomalies will take place after the conclusion of the work, with the appearance in the structural components, of the creep phenomena.

According to the EC6 acceptance [13], the creep coefficient at an infinite time, for the ceramics masonry units, is equal to 1. In order to eliminate the remaining anomalies of the support deformations, the deflection at long time, f_{lp}, of the support should be considered, with the following value:

$$f_{lp} \leq f_{cpr} + f_{pi} \tag{7}$$

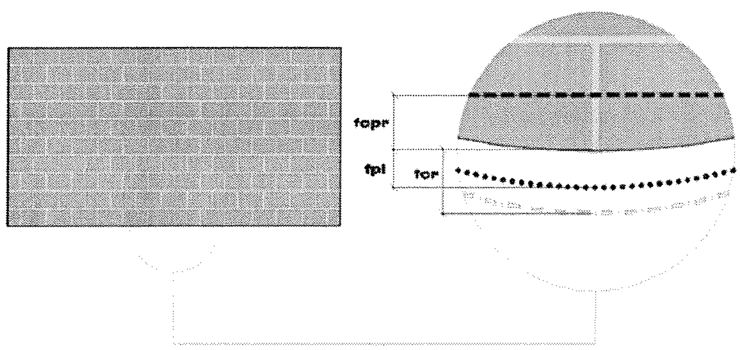


Figure 1: Deformation evolution

2.2.2 Equipments

In order to realise the experimental "in situ" study the following equipments were used:

- One frame (Figure 2a) to assembly of wall panel;
- One hydraulic jack with manometer to read the applied pressure;
- Deformations measuring equipment (Figure 2b);
- Equipment acquisition and treatment of data.

As noticed before, the frame was designed in a way that was possible to build the wall and gave it the desired support conditions. The frame was built by:

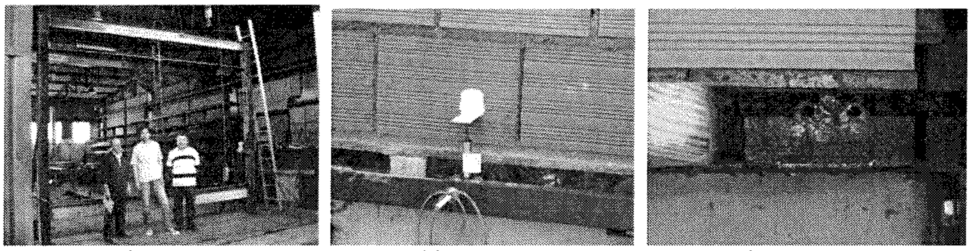
- Lateral columns with ground connection to the reinforced concrete spread foundations;
- Lower beam used to the indirect support of the wall during the time between the building of the wall and the test day, as well as, to the secondary structure support used to the deformation reading equipment installation (Figure 2c);
- Wall assembly plate supported upon wooden wedge and these ones supported upon the inferior beam. This plate guarantees the support continuity conditions. The plate was taken out before the test in order to support the wall on the extremes.
- Metallic supports to assimilate the structural modelling that was previewed (Figure 2c).
- Superior beam used to mobilize the hydraulic jack reaction in order to allow the load application to the wall. In the hydraulic jack application area, the beam was connected to a reinforced concrete spread foundation by four tie steel rods (Figure 3a).
- Secondary structure used to support the deformation reading equipment (Figure 3b).

The measure of deformations was done by linear voltage displacement transducers (LVDT) connected to a data acquisition system and was placed on the secondary structure. Its position was defined in order to allow mid-span and quarters-span readings, in the top and in the bottom of the wall.

2.2.3 Test description

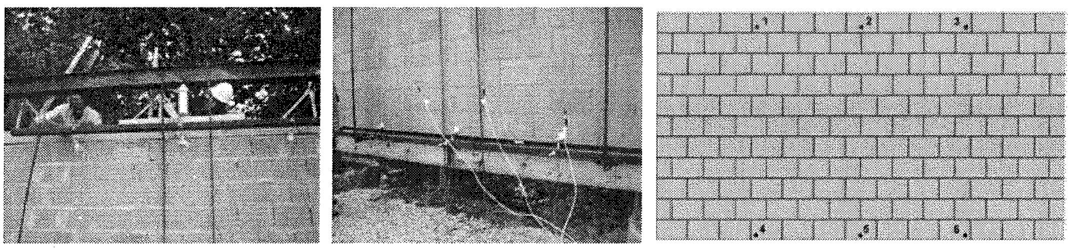
The wall was built upon a steel plate that was supported on wooden wedges and these ones on the lower beam of the frame. All the appropriated measures were taken in order to stop the wall's desiccation during the first three days since their construction. For this propose they were covered with a polyethylene film, and after they stay uncovered subjected to exterior environmental conditions.

The superior surface of the wall was regularised using a mortar similar to the masonry mortar. The wall was tested at the 28th day. For this intent, two wooden beams were placed on the top of the wall. The objective was to obtain a vertical variable load upon the wall with the maximum at mid-span. The hydraulic jack was interposed between these wooden beams and the superior beam of the frame (Figure 3a).



a) b) c)

Figure 2: a) General view of the frame; b) Deformation reading equipment c) Lower beam used to the indirect support of the wall, its steel plate and steel support.



a) b) c)

Figure 3: a) Superior beam used to mobilize the hydraulic jack reaction; b) Secondary structure used to hold the LVDT's; c) Reading positions.

The first stage of the test consisted in loading the wall with its own weight action, considering in this stage that the support was already excessively deformed and the wall was supported on its extremes. This was achieved by taken out the assembly plate and measuring the deformation. At this stage of the test, the only deformation that was measured was from the 5th position (Figure 3c). These measures were achieved by reading the distance of two specific points, before and after this first stage of the test, corresponding at mid-span of the bottom of the wall. This procedure was taken to avoid the damaged of the electronic reading equipment caused by the excessive vibration on the "plate assembly" removal.

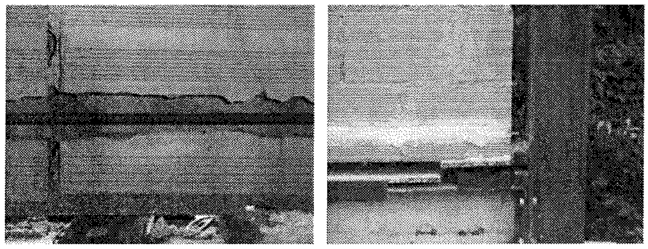
The next step of the test was the loading of the wall, considering in this stage, as well as on the previous stage, that the support was already excessively deformed. This simulation was made using a hydraulic jack. The load was submitted in a continuous way upon the wood beams. Due to the stiffness of the beams was allowed to distribute the charge in a variable way, with a maximum at mid-span and a minimum on the extremes. The following registrations were made:

- Dimension of the wall that was subjected to the load;
- Levels of the several deflections points of lecture;
- Evolution of load with time;
- Evolution of deflections of the six measurement points with time.

After the reference wall has been tested, it was wedged and rehabilitated considering the following items:

- Complete removal of the vertical shell of one of the brick holes of the first course;
- Placement on the hole space of a 10 mm diameter bar in A400NR class steel (Figure 4a);
- Application of "MO1" mortar type to which an set accelerator was added, involving the rod and completely closing the intervened space (Figure 4b);
- In the next day, an identical operation on the opposite side was made.

The loading test was made after 28 days using the operation previously described.



a) b)

Figure 4: a) Complete renovation of the exterior vertical septum to apply the reinforcement; b) External view of the bricks after reinforcement placement

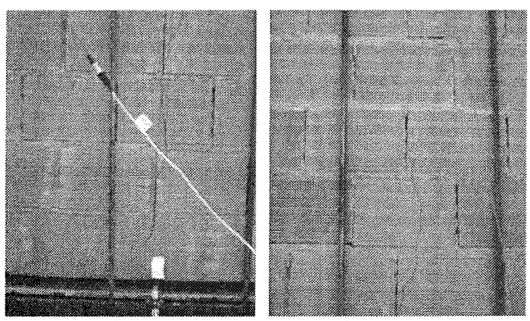
2.2.4 Reference wall type "PD11MO1"

2.2.4.1 Load test

The test procedure was the same as described in 2.2.2, realising for this purpose two loading phases. In the first one – self-weight load – it did not occur any type of cracks. In the second phase, the test was interrupted after the first crack appeared.

2.2.4.2 Results

After the withdrawal of the setting plate the wall stayed charged to its own weight on the conditions determined by its structural model. The deformation obtained on the fifth position was 0.3 mm. In the second phase of the test the first crack occurred at about 55kN of applied load. This crack was a typical flexural one, located in the wall mid-span and with a vertical development from the bottom to the top of the wall (Figure 5).



a) b)

Figure 5: a) Crack opened at half span between the first and the fourth row of bricks; b) Crack opened at half span, between the fourth and the eighth row of bricks

The first crack in the wall appeared for a load that conducted to a maximum deformation on the fifth position of 1.622 mm. The most significant results are indicated on Table 3. Figure 6 shows the displacement on each one of the positions considering the different applied loads.

2.2.4.3 Results evaluation

In the first loading phase the wall did not suffer any visual anomaly. In the second one, the load was increased until failure. A maximum concentrated load of 55.0 kN was applied on the wall.

Table 3 : Load-displacement results – PD11MO1

TIME	LOAD	DISPLACEMENTS (mm)						OBS.
(s)	(kN)	1	2	3	4	5	6	
0	0	0.000	0.000	0.000	0.000	0.000	0.000	
30	13.25	0.188	0.446	0.163	0.132	0.168	0.141	
60	22.97	0.285	0.649	0.350	0.233	0.317	0.227	
90	32.25	0.304	0.780	0.424	0.282	0.385	0.275	
120	44.18	0.370	1.002	0.553	0.356	0.540	0.439	
150	55.23	0.724	1.868	0.935	0.650	1.622	0.636	First crack

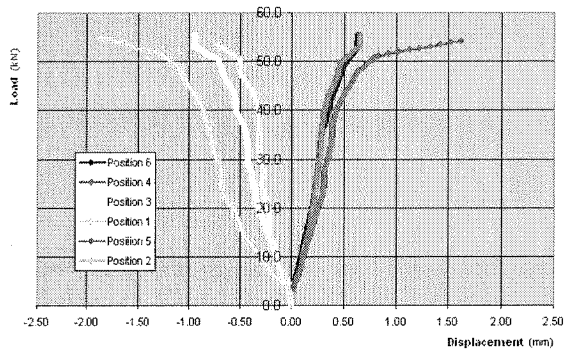


Figure 6: Load-displacement graphic. PD11M01

The wall deformation on the fifth position was 1.622 mm. Adding this displacement to the one obtained in the first phase results in a total deformation of 1.922 mm. This value corresponds to the deflection, f_{pi}, given by:

$$f_{pi} = \frac{L}{2081} = \frac{4000}{2081} = 1.922 \tag{8}$$

This is the maximum deformation value that the wall can support without anomalies.

For the considerations presented in 2.2.1., relative to the evolution of the deformations in the period of building of the components and to the effect of creep, the long time deflection, f_{lp}, from the support should be considered, in case of "PD11MO1" type of walls, with the following value:

$$f_{lp} \le \frac{L}{2222} + \frac{L}{2081} = \frac{L}{1073} \tag{9}$$

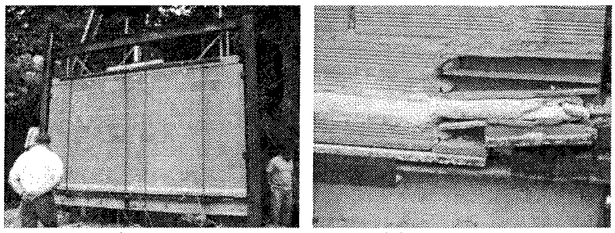
2.2.5 Rehabilitated wall type "PD11MO1"

2.2.5.1 Load test

The test was made as described in 2.2.2. Figure7a shows the general aspect of the wall that was settled on the test frame. The second phase of the loading was only considerer because was not possible to guarantee a support that would not deform between the rehabilitation and the test period of time.

2.2.5.2 Results

The test was interrupted after failure by compression of the vertical bordering shell of the first level of holes from the brick, situated on the right support, being the wall rested on the inferior beam of the frame (Figure7b). This failure occurred for a load close to 93.0 kN.



a) b)

Figure 7: a) General view of the wall before the test; b) Smashing rupture of the vertical septum on the first level of bricks

The first crack in the wall occurred for a load that caused a maximum deformation of 0.812 mm on the fifth position. The most significant results are indicated on Table 4. The displacements in each position and the corresponding measured load are represented in Figure 8.

Table 4 : Load-displacement results – PD11MO1 - Rehabilitated wall

TIME	LOAD	DISPLACEMENTS (mm)						OBS.
(s)	(kN)	1	2	3	4	5	6	
0	0	0.000	0.000	0.000	0.000	0.000	0.000	
30	17.67	0.099	0.207	0.188	0.106	0.112	0.166	
60	35.34	0.255	0.429	0.273	0.200	0.231	0.261	
90	53.02	0.346	0.565	0.357	0.245	0.316	0.342	
120	70.69	0.441	0.699	0.488	0.350	0.478	0.449	
150	77.32	0.466	0.784	0.566	0.412	0.550	0.540	
180	83.94	0.521	0.880	0.640	0.486	0.654	0.634	
210	89.24	0.597	1.025	0.741	0.547	0.749	0.734	
224	92.78	0.635	1.121	0.782	0.631	0.812	0.834	First crack

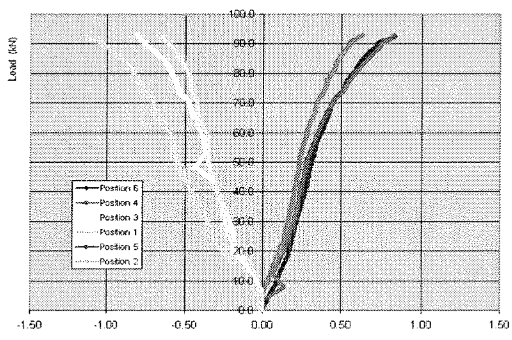


Figure 8: Load-displacement graphic. Rehabilitated wall PD11M01

2.2.5.3 Results evaluation

The introduction of the reinforced steel bars, as a way to rehabilitate the wall gave:

- A superior load capacity, since the failure load changed from 55.0 kN to 93.0 kN;
- The rehabilitated wall deformations, in the reference positions, were inferior to the ones

achieved on the reference wall.

For this type of loading the deep beam section is characterized for being superiorly compressed and subjected to tensile stresses in the bottom. It is obvious, that the increase of the load capacity is due especially to the introduced reinforcement steel which has greater tensile strength. This tensile resistance gave to the rehabilitated wall a better performance in this mechanical characteristic.

The deformation reduction, even for a superior loading, can also be associated to the reinforced steel introduction. Assimilating the wall to a trussed frame, composed by two diagonals connected on the extremes to the inferior chord. For a vertical load, the diagonals will be compressed and the inferior chord will be subjected to tensile stresses. The increase of the Young's modulus and tensile strength of the inferior chord will lead to a decrease on the vertical displacement of the superior vertex of the trussed frame. By analogy, the statement that an identical phenomenon occurred with the reinforcement is possible.

2.2.6 Reference wall type "PD11MO2"

2.2.6.1 Load test

The test procedure was the same as described in 2.2.2. The first stage of the test was only made because the first crack occurred with the own weight application.

2.2.6.2 Results

After the load withdrawal the wall was subjected only to its own weight. The first crack occurred instantly as a result of disconnecting the first and the second rows of the wall bricks (Figure 9). The own weight of the wall corresponded to about 8.0 kN loading. The maximum deformation of 3.4 mm, was obtained in the fifth position. The most significant results of this test are presented on Table 5.

Table 5 : Load-displacement results – PD11MO2

TIME	LOAD	DISPLACEMENTS (mm)						OBS.
(s)	(kN)	1	2	3	4	5	6	
0	0	0.0	0.0	0.0	0.0	0.0	0.0	
1	8.00	-	-	-	3.0	3.4	2.6	First crack

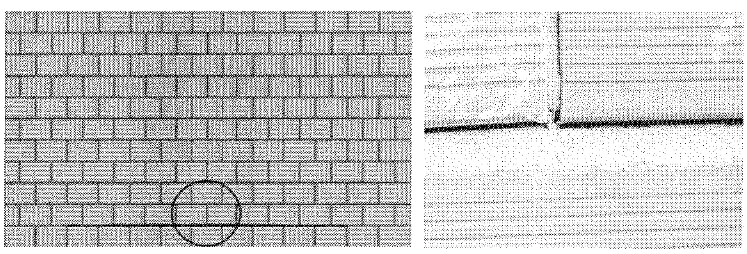


Figure 9: Cracked area

2.2.6.3 Results evaluation

The deformation in the fifth position, was 3.4 mm, corresponding to a deflection, f_{pi}, given by:

$$f_{pi} = \frac{L}{1176} = \frac{4000}{1176} = 3.4\,\text{mm} \tag{10}$$

For the considerations presented in 2.2.1., relative to the evolution of the deformations in the period of building of the components and to the effect of creep, the long time deflection, f_{lp}, from the support should be considered, in case of "PD11MO2" type of walls, with the following value:

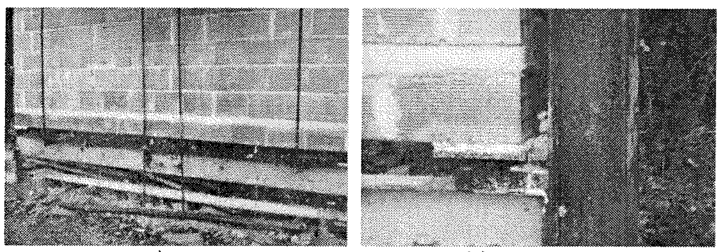
$$f_{lp} \leq \frac{L}{2222} + \frac{L}{1176} = \frac{L}{769} \tag{11}$$

2.2.7 Rehabilitated wall type "PD11MO2"

2.2.7.1 Load test

The rehabilitation was made using a procedure similar to the one shown in 2.2.2, with the exception of the position where the reinforced steel was placed. In this case, according to the type and local of the crack, the vertical shells were removed from the superior row of the brick holes. This turned possible the reestablishment of the connection between the first and the second row of the brick holes and also the placement of the reinforcement.

Figure 10a shows wall area where the rehabilitation was made. In Figure 10b there is a detail of the intervention area near the support. The second phase of the loading was only considered because was not possible to ensure a support that would not deform, in the period of time between the rehabilitation and the test.



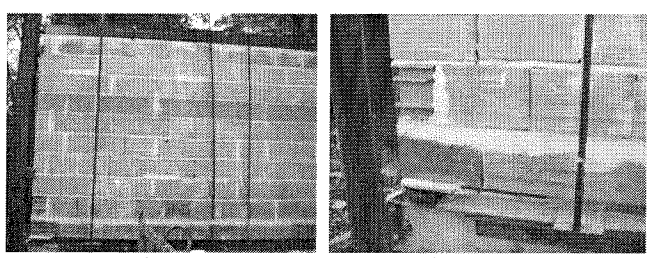
a) b)

Figure 10: a) General view of the rehabilitated wall; b) Rehabilitated area next to the support

2.2.7.2 Results

Once the loading was made the test was interrupted after the appearance of the first crack which occurred for a 53.0 kN load. This crack stayed above the line that was defined by application of the reinforcement. The crack was located upon the defined line by the reinforced steel application and with a 45 degrees development from the left support, as showed in Figure 11. In some areas the crack's thickness was about 3 or 4 mm.

The first crack in the wall occurred for a load that caused a maximum deformation of 0.485 mm on the fifth position. The most significant results are indicated on Table 6. The displacements in each position and the corresponding measured load are represented in Figure 12.



a) b)

Figure 11: a) General view of the 45 degrees fissures on the left area of the wall; b) Cracked area next to the left support

Table 6 : Load-displacement results – PD11MO2 - Rehabilitated wall

TIME	LOAD	DISPLACEMENTS (mm)						OBS.
(s)	(kN)	1	2	3	4	5	6	
0	0	0.000	0.000	0.000	0.000	0.000	0.000	
30	17.7	-0.026	0.182	-0.041	0.050	0.048	0.023	
60	28.7	-0.014	-0.134	0.039	0.169	0.170	0.107	
90	44.2	-0.327	-0.025	0.262	0.447	0.400	0.316	
120	53.0	-0.308	0.180	0.399	0.571	0.485	0.375	First crack

2.2.7.3 Results evaluation

The reinforcement with steel bars, as a way to rehabilitate the wall, just like it had already happened with the "PD11MO1" type of wall rehabilitation, gave:

- A superior load capacity, since the failure load changed from 8 kN to 53 kN;
- The rehabilitated wall deformations, on the known reference positions, were inferior to the ones obtained on the initially considered wall.

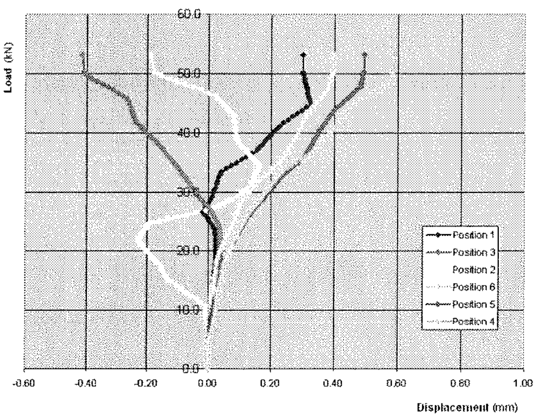


Figure 12: Load-displacement graphic. Rehabilitated wall PD11M02

The results obtained with the rehabilitation technique used on this wall were similar to the ones described in 2.2.5.3. However the existence of 45 degree's cracks developing from the left support indicates that the shear strength of the wall was achieved.

3 CONCLUSIONS

In order to harmonize the deformations of the reinforced concrete structures and the simple leaf masonry wall, the long term deflection of the support should not overpass:

- $\frac{L}{1073}$ with cement mortar (cement and sand in a proportion by volume of 1:5);
- $\frac{L}{769}$ with mixed mortar (cement, hydrated lime and sand in a proportion by volume of 1:2:9).

This last kind of walls has a better ability to accommodate movement, as we can see trough the Young's modulus obtained in the laboratory tests. The long term deflections admissible for the supports or for the superior components of brick walls are inferior to the values specified by the national rules.

The solution used for the wall's rehabilitation gave a structural working behaviour similar to a reinforced deep beam. This solution gave to the walls a better behaviour in what concerns the strength and deformation, relatively to the reference walls.

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