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CALCULATION OF FERROCEMENT LAYERS USED TO STRENGTHEN THE BRICK MASONRY WALLS

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Abstract: The paper presents a calculation model for the ferrocement layers used to strengthen the brick masonry walls.

Keywords: ferrocement, brick, walls, consolidation

1. GENERAL CONSIDERATIONS

Although ferrocement as a material has been known for quite a long time, being used in a wide variety of buildings, from boats to sheeting lost casing and from tanks different liquids to architectural masterpieces, such as the Sydney Opera House, its use for the rehabilitation of buildings damaged from different causes has been insufficiently researched and applied.

2. DESCRIPTION OF THE CONSOLIDATION SOLUTION BY MEANS OF FERROCEMENT COATING

In what follows, reference will be made to the solution of coating consolidation using ferrocement. This solution is indicated for old structures which are strongly deteriorated and which have a significantly diminished portant capacity of the structural walls.

As a result of the before mentioned facts, the authors of this study aim at presenting some aspects regarding the calculation of the elements which were consolidated using ferrocement.

The masonry walls (4 panels) have been loaded upon fracture, after which they were consolidated with ferrocement.

The masonry walls have been built from 240x115x63mm filled ceramic bricks. The masonry mortar was M50. The outline framing of the masonry has been accomplished with 24x10cm pillars and 24x20cm belts made of concrete class C8/10 (B150), according to the standard P2-85. The vertical reinforcement with pillars was 4Ø8 PC52, the superior belt being also embedded in the foundation. The transversal reinforcement of the pillars was accomplished by means of black wire bars Ø5mm, having a transversal reinforcement percentage of 0.157%. The final dimensions of the masonry panels were: h=190cm (including the 20cm belt), l=180cm and the thickness of 24cm.

The wall models have been submitted to cyclical lateral trials in the presence of a constant vertical load which produced a 0,4 N/mm^2 axial stress at their foot (including its own weight).

As a result of the trials performed to the brick masonry walls, it was found that the panels made of masonry confined with pillars surrendered under the shear sliding force on the route of inclined fissures and the shearing of the pillars.

In order to consolidate the respective walls, which were initially tried until they broke down, the following technology was used:

- the affected surfaces were polished with a chisel in order to remove the deteriorated material (exfoliated, broken, etc.);
- the continuity of the reinforcement was restored using pillars with welded fishplates;
- the surface was cleaned using a water jet and a wire brush to remove the small pieces of material and the dust;
- on the clean and rugged surface, nails were hammered in the mortar empty spaces. The distance between the nails was 20….25cm, and the remaining free length of 10….15mm. at the same time, plastic disks were fitted, playing as distance pieces;
- a layer of mixture was applied on the surface, made of cement milk;
- two layers of zinc-coated steel wire mesh were positioned, with a diameter of 1mm and the distance between the bars of 10x10mm. In order to fix the meshes, the previously positioned nails were used;
- the mortar was applied by an average 3,5 cm injection with concrete, the covering layer being at least of 0,5cm; the mortar was prepared according to the following recipe: cement Pa40 - 500kg/mc, sand 0-3mm 1700 kg/mc, water 250kg/mc. From the trials on the test pieces made of this mortar, the following values resulted regarding the resistance to compression: 7,5N/mm^2 after 7 days and 39N/mm^2 after 28 days. The values for the stretching resistance from bending were 1,12N/mm^2 after 7 days and 7,75N/mm^2 after 28 days.
- the sides of the element were finished off through the application of a polishing plaster coat and of a white painting which makes fissures more visible;

The trial of the consolidated elements was accomplished in the same conditions as the unconsolidated ones, respectively by applying alternating lateral forces by means of hydraulic jack in the presence of a vertical force which created an stress of 0,4N/mm^2 at the foot of the masonry panel.

The two layers of concrete-injected mortar together with the broken masonry formed such a rigid unitary whole that, under lateral forces equal with those which made unconsolidated panels to break down, respectively 224KN, the panel rotated as a rigid solid in rapport with the contact point between the wall and the foundation, without fissures to be produced on the sides of the panel (it tended to fall down).

In order to mobilize the concrete-injected mortar, it was necessary to block the rotation by introducing two additional steel tie bars of Ø = 24 mm each, with leak resistance of 331,8 N/mm^2; they were placed at the edge of the consolidated masonry panel.

In this new situation, the alternating forces were applied in an increasing pace, in stages of 40kN. The breaking down took place when the value of the lateral force was around 460 KN, at the VI cycle, when the foundation broke down.

3. THE COMPUTATIONS OF PANELS CONSOLIDATED WITH FERROCEMENT

The calculations were accomplished using the theoretical model which was put into practice, both for ultimate limit states (SLU) for determining normal stress from eccentric compression and tangent stress from shearing, and for limit states of normal exploitation (SLEN), respectively fissuring and deformations.

Taking into account the specific way of yielding, both for consolidated and unconsolidated elements, respectively through the shearing of the masonry and of the pillars and then, the appearance of diagonal fissures in the coating, in what follows we will consider the calculation of the stress in inclined sections resulted from the shear forces applied to the research element.

The calculations were accomplished according to two methods, the one present in the Code of ferrocement elements design drawn up by the Technical University of Cluj Napoca in 1999 and the stipulations EUROCODE EC2; in what follows, we will give them due consideration.

3.1 Calculation scheme

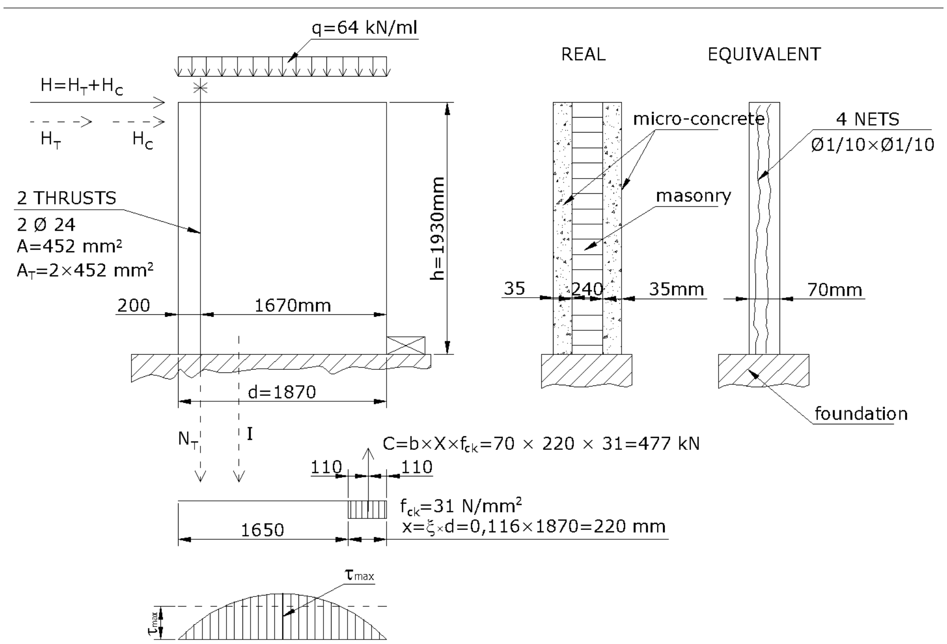

Fig. 1 Calculation scheme

3.2 Calculation according to the model from „Code of ferrocement design" – drawn up by the Technical University of Cluj Napoca

Wire mesh is used for reinforcement, Ø = 1 mm thin, having a distance of 10 mm, with the following peculiarities:

- number of meshes: $n = 4$ meshes
- weight of the mesh: $G_a = 1{,}138\ \text{kg/m}^2$
- area of the mesh: $A = 1{,}93 \times 1{,}87 = 3{,}61\ \text{m}^2$
- steel density: $\rho_0 = 7850\ \text{kg/m}^3$

The volumetric reinforcement percentage is found: Vf

$V_f = \dfrac{V_p}{V_s}$, where:

$$V_p = \frac{n \times G_a \times A}{\rho_0} = \frac{4 \times 1{,}138 \times 3{,}61}{7850} = 0{,}0021\ \text{m}^3$$ – volume of the meshes

$V_s = h \times A = 2 \times 0{,}035 \times 3{,}61 = 0{,}25\ \text{m}^3$ – volume of micro-concrete

$V_f = \dfrac{0{,}0021}{0{,}25} = 0{,}00837$; it results:

$V_{fx} = 0{,}5 \times V_f = 0{,}0042$ – on one direction.

The unitary stress capable of shearing at fissuring is determined with the formula:

$\tau_{cr} = f_{bt} + 450 \times V_{fx}$ [N/mm^2]

$f_{bt} = m_{bt} \times f^*_{ct}$

$f^*_{ct} = \dfrac{f_{tk}}{\gamma_{bt}}$; $m_{bt} = 1{,}0$; $\gamma_{bt} = 1{,}5$

$f_{tk} = 0{,}22 \times (f_{ck})^{\frac{2}{3}}$ [N/mm^2]

$f_{ck} = (0{,}87 - 0{,}002 \times f_{bk}) \times f_{bk}$ [N/mm^2]

$f_{bk} = 39$ N/mm^2 –experimentally obtained through submitting to trial the test pieces drawn from the micro-concrete used

$f_{ck} = (0{,}87 - 0{,}002 \times 39) \times 39 = 31$ [N/mm^2]

$f_{tk} = 0{,}22 \times (31)^{\frac{2}{3}} = 2{,}17$ [N/mm^2]

$f^*_{ct} = \dfrac{f_{tk}}{1{,}5} = \dfrac{2{,}17}{1{,}5} = 1{,}45$ [N/mm2]

$f_{bt} = m_{bt} \times f^*_{ct} = 1 \times 1{,}45 = 1{,}45$ [N/mm^2]

$\tau_{cr} = f_{bt} + 450 \times V_{fx} = 1{,}45 + 450 \times 0{,}0042 = 3{,}34$ [N/mm^2]

The (average) experimental unitary shearing stress

$\tau_{max} = \dfrac{3}{2} \times \dfrac{H_F}{b \times d} = \dfrac{3}{2} \times \dfrac{440 \times 10^3}{70 \times 1870} = 5{,}04$ [N/mm^2], where:

$H_F = 440$ kN – force under which the fissure appeared

$\tau_{med} = \dfrac{2}{3} \times \tau_{max} = \dfrac{2}{3} \times 5{,}04 = 3{,}36$ [N/mm^2]

From the afore mentioned data it results that the unitary tangent calculation stress $\tau_{cr} = 3{,}34$ N/mm^2 is approximately equal with the experimental unitary stress $\tau_{med} = 3{,}36$ N/mm^2, which shows a good concordance between the calculation formulae used in the Code of ferrocement design" and the experimentally obtained real scale findings.

3.3. Calculation according to the stipulations EUROCOD EC2

The shear force is determined, which could be taken over by the micro-concrete: V_{Rdc}

$$V_{Rdc} = [\frac{0{,}18}{\gamma_c} \times k \times (100 \times \rho_1 \times f_{ck})^{\frac{1}{3}} + 0{,}15 \times \sigma_{cp}] \times b_w \times d$$

where:

$\gamma_c = 1{,}5$ – safety coefficient from the table 2.3 from EUROCODE EC2

$k = 1 + \sqrt{\dfrac{200}{d}} = 1 + \sqrt{\dfrac{200}{1870}} = 1{,}327 < 2$

$\rho_i = \dfrac{A_{si}}{b_w \times d} = \dfrac{1422{,}1}{70 \times 1870} = 0{,}011 < 0{,}02$ – longitudinal reinforcement coefficient

$A_{si} = A_{plasa} + A_{tiranti} = 518{,}1 + 904 = 1422{,}1$ [mm^2]

$A_{palsa} = (\dfrac{d - x}{s}) \times n \times \dfrac{\pi \times \phi^2}{4} = (10) \times 4 \times \dfrac{\pi \times 1^2}{4} = 518{,}1$ [mm^2]

$A_{tiranti} = 2 \times 452 = 904$ [mm^2]

n = 4 – number of meshes

s = 10 mm – size of the spaces between the bars

Ø = 1 mm – diameter of mesh wire

$b_w = 2 \times 35 = 70$ mm – total thickness of the concrete layer

$f_{ck} = 31$ N/mm^2 – characteristic resistance of concrete (experimentally found)

$\sigma_{cp} = \dfrac{N_{Ed}}{A_c} = \dfrac{116000}{152600} = 0{,}76$, where:

$N_{Ed} = q \times d = 64 \times 1{,}87 = 116$ [kN], where:

q = 64 kN/m – vertical load

d = 1,87 m – width of the element

$A_c = 2 \times (1870 \times 35 + 310 \times 35) = 152.600$ [mm^2] – total compressed area (at the foot of the element)

It results:

$$V_{Rdc} = [\frac{0{,}18}{1{,}5} \times 1{,}327 \times (100 \times 0{,}011 \times 31)^{\frac{1}{3}} + 0{,}15 \times 0{,}76] \times 70 \times 1870 = 82{,}7 \quad [kN]$$

$$V_{min} = 0{,}035 \times k^{\frac{3}{2}} \times f_{ck}^{\frac{1}{2}} = 0{,}035 \times (1{,}327)^{\frac{3}{2}} \times (31)^{\frac{1}{2}} = 0{,}297 < 0{,}517$$

It results that V_{Rdc} is well calculated.

It is found that V_{Rdc} = 82,7 kN is less than the lateral force H = 460 kN under which the element was broken: $V_{Rdc} << H = 460$ [kN] – lateral force

We determine the maximum force shear force which can be taken over without the micro-concrete to be broken: VRdmax

$$V_{Rdmax} = \frac{\alpha_c \times b_w \times z \times \upsilon \times f_{cd}}{\cot g\theta + tg\theta} = \frac{1 \times 70 \times 1683 \times 0{,}5 \times 20{,}66}{1+1} = 609 \quad [kN]$$

where:

α_c = 1 – coefficient for unpretensioned structures

$\upsilon = 0{,}6 \times (1 - \dfrac{f_{ck}}{250}) = 0{,}60 \times (1 - \dfrac{31}{250}) = 0{,}525$we adopt: $\upsilon = 0{,}5$

z = 0,9 × d = 0,9 × 1870 = 1683 – lever arm

$f_{cd} = \alpha_{cc} \times \dfrac{f_{ck}}{\gamma_c} = 1 \times \dfrac{31}{1{,}5} = 20{,}66$, where:

α_{cc} = 1 – coefficient which takes into account lasting effects

$\theta = 45°$ - angle of inclination of compressed diagonals

γ_c = 1,5 – safety coefficient from table 2.3 from EUROCODE EC2

We determine the maximum shear force taken over by the transversal reinforcement (assimilated with reinforcement only with cradle stirrups): V_{Rds}

$$V_{Rds} = \frac{A_{sw}}{S} \times z \times f_{ywd} \times ctg\theta = \frac{3{,}14}{10} \times 1683 \times 270 \times 1 = 142684 \quad [N] = 142{,}7 \quad [kN]$$

where:

$A_{sw} = 4 \times \dfrac{\pi \times \phi^2}{4} = 4 \times \dfrac{\pi \times 1^2}{4} = 3{,}14$ [mm^2] – area of transversal wires

$f_{ywd} = \dfrac{f_{yk}}{\gamma_s} = \dfrac{310}{1{,}15} = 270$ [N/mm^2] – calculation of leak resistance for steel

f_{yk} = 310 N/mm^2 – characteristic leak limit (from table 2.14 – Code ferrocement design)

Remarks:

- The value V_{Rds} = 142,7 kN shows how much wire meshes can take from the lateral force H=460kN;

  The value V_{Rds} = 142,7 kN is also confirmed by the experiment when the trial was performed without tie bars when the lateral force attained was H = 155 ÷ 159 kN;
- The difference $\Delta V_{Rds} = H - V_{Rds}$ = 460 – 142,7 = 317,3 kN is considered to be taken over by the two tie bars;

  $V_T = A_T \times f_{ywT} = 2 \times 452 \times 331 \approx 300.000$ N = 300 kN

$A_T = 2 \times 452 = 904 \quad [mm^2]$

$f_{ywT} = \dfrac{f_{yT}}{\gamma_s} = \dfrac{381}{1{,}15} = 331 \quad [N/mm^2]$

$f_{yT} = 381\ N/mm^2$ – stress after which TER no longer recorded (entered the leak stage)

- I this hypothesis, the value of the shear force taken over by the entire reinforcement (meshes + tie bars) becomes:
  $V_{Rds}^{T} = V_{Rds} + V_T = 142{,}7 + 300 = 442{,}7\ kN \approx H = 460\ kN$
- The difference between the calculated value and the one experimentally obtained is:
  $\Delta V = 460 - 442{,}7 = 17{,}3\ kN$
- This difference can be ascribed to the punch effect created by the reinforcement with pillars and calculated with the formula:

$$V_{dorn} = N \times 4{,}12 \times \phi_{bare}^{\frac{2}{3}} \times b_w \times \sqrt[3]{f_{ck}} = 4 \times 4{,}12 \times 8^{\frac{2}{3}} \times 70 \times \sqrt[3]{31} = 14{,}5 \quad [kN]$$

in which::

$N = 4$ – number of bars made of pillars

$Ø_{bare} = 8$ mm – diameter of the bars made of pillars

The other dimensions were previously defined.

It results that that total lateral force that the consolidated assembly can take over is:

$V_{total} = V_{Rds} + V_T + V_{dorn} = 142{,}7 + 300 + 14{,}5 = 457{,}2\ kN \approx H = 460\ kN$

It is found that, by applying this method, we have the experimental confirmation of the theoretic model.

4. CONCLUSIONS

As a result of the trials on elements and of the calculations made on the theoretic model, we can conclude that, irrespective of the calculation method used, the values of the stress (or of the shear forces) are confirmed by the measurements accomplished when trials were performed on real scale models.

Together with other consolidation solutions using the coating of the element (ex. Reinforcement with polymeric meshes, micro-concrete reinforced with wire mesh 4-6mm thick with the distance between the bars of 100x100mm, etc), the solution proposed and checked by the authors of this study represents a viable solution, with a good theoretical and experimental support.

Figures 2 shows the trial scheme of masonry panels.

Figures 3 shows the way the panel under trial was equipped with measuring apparata.

In fig.4 (photo) we presented the panel which underwent the trial using the above mentioned equipment.

Figure 5 shows the fissures for the consolidated panel under trial, during the breaking stage.

Figure 5 shows the diagram stress – deformation when tie bars are fitted which prevent the panel from falling down.

5. ADVANTAGES OF THE SOLUTION

The quality of ferrocement to have a very good resistance to fissuring gives it a great advantage over the use of reinforced concrete. The increased resistance to fissuring, combined with the easiness of applying it, as well as its relatively small weight and the reduced cost, make ferrocement an ideal system for rehabilitating structures.

The main objective envisaged for the consolidation and rehabilitation of masonry structures is represented by the recovery of their portant capacity with as reasonable costs as possible. Between the advantages of the solution we may list:

- the accomplished consolidations allow changes and subsequent reparations;
- the relatively reduced weight resulted from the consolidation systems does not require changes in the structure support system;
- facility to oppose to temperature changes;
- easiness to purchase necessary materials;
- does not require special technological equipment;
- flexibility to subsequent changes;
- possibility not to alter the architectural concept of the structure and, implicitly, of the building as a result of the consolidation.

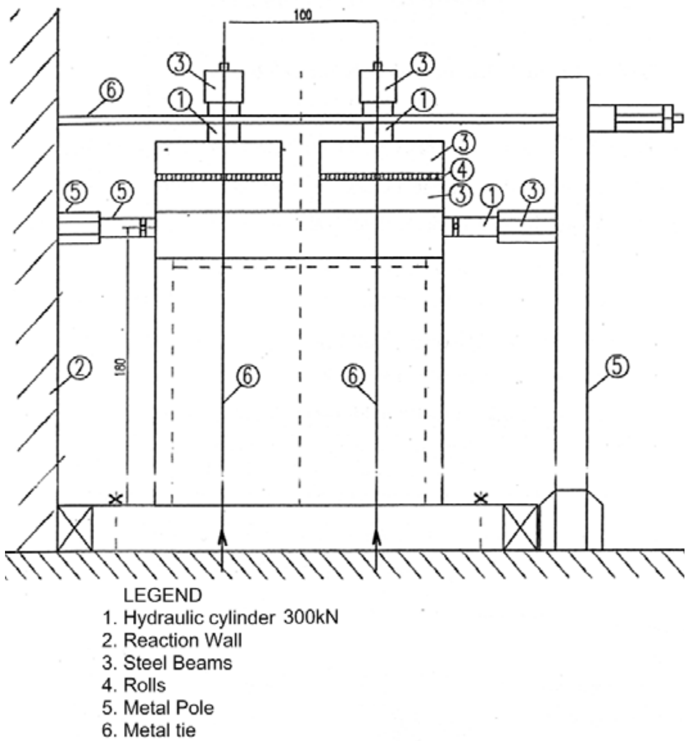

Fig. 2 Trail scheme

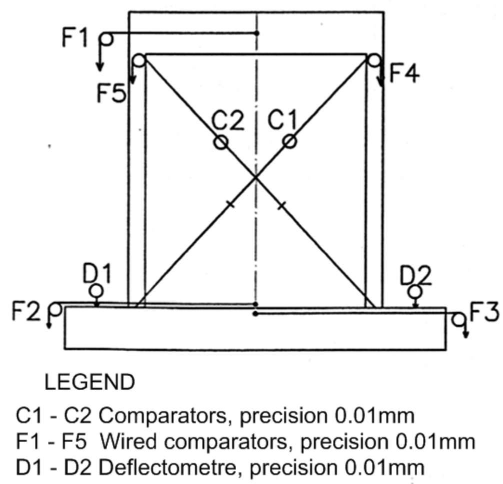

Fig. 3 Scheme of measuring equipment

Fig. 4 Equipping of the trial stall for the consolidated masonry element

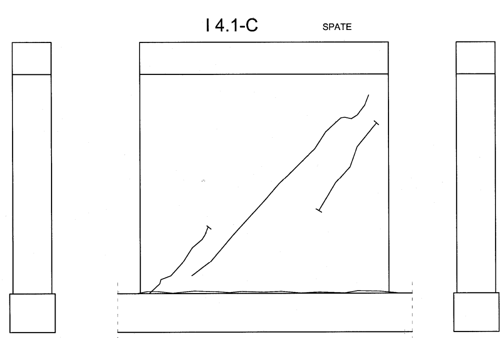

Fig. 5 The fissures for the consolidated element

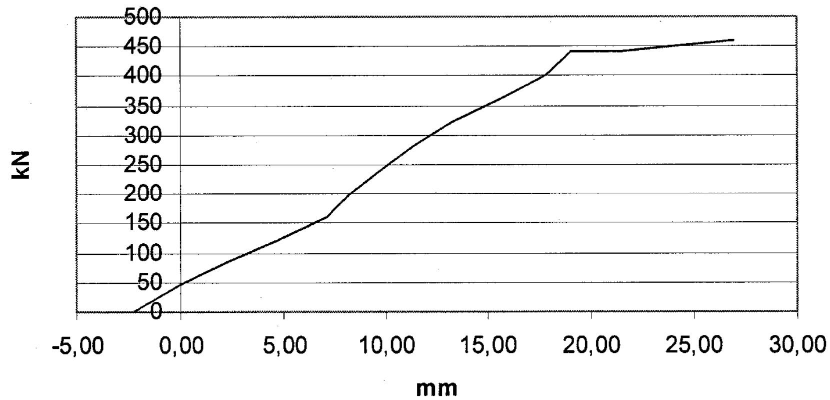

Fig. 6 Diagram force – movement for the consolidated element equipped with tie bars

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