Two techniques for repair and strengthening masonry constructions

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ABSTRACT: The paper refers to masonry based on burned clay bricks and lime mortars currently used to historical constructions. Theory of dislocations is used to explain the mechanism of damaging the masonry in structural members, but also to prevent any failures in the same members. There are two techniques for repair and strengthening masonry in historical constructions: reinforcing structural members with polymer grids and confining them with the same synthetic reinforcement embedded in lime or lime-cement mortars. Two numerical examples of designing shear walls are presented. The results show that polymer grids enhance the shear capacity of the weakest sections of walls, and economically the two techniques are extremely attractive.

1 INTRODUCTION

Romanian historical constructions like three-lobed churches, castles, palaces and walls of defence are made of genuine masonry. It is the original artificial stone based on burned clay bricks and lime mortars. This masonry was produced with the aid of gravity and remained dependent for all its service life on gravity. Solid porous bricks are elastic but brittle, while lime mortars by spontaneous plastic deformations protect them. When damaged by earthquakes, for instance, or decayed by long time actions of environmental factors this type of masonry can not be recovered but with the same or compatible materials. Cement mortars are not appropriate because their thermal coefficient of expansion is 2.5 times higher than that of masonry. If associated, under daily variations of temperature, the two materials are differently dilating or contracting, and after a while they are detaching one of another. On the other hand if reinforced concrete members like columns and beams are used for confining masonry on all four sides they are inducing structural non-homogeneity what is against of EC8 principles.

2 THEORY OF DISLOCATION

According to the well known hypothesis of Jacob Bernoulli if a continuous, homogeneous and isotropic structural member is submitted to a concentrated force then the resulting stresses in equilibrium state are uniformly distributed. The stresses are proportional with the ratio between the value of force $P$ and the area of cross-section $A$. This relation is currently used is design. However, if somewhere a structural fault occurs or is deliberately provided Bernoulli's hypothesis does not subsist. Around the fault a concentration of stresses develops. If the material of structural member is brittle then according to Lev Landau, Nobel prised for Physics, the stress concentrations could originate dislocations (Fig. 1).

Particularly, the structural members of masonry are essentially non-homogeneous. Masonry is far to be a continuous and isotropic material. From the point of view of Theory of dislocation the vertical joints
between bricks are like permanent faults. For long lasting actions like the gravitational ones the ductility of lime mortars protects the bricks against failure by the phenomenon known as adaptation. In the case of short time actions like those developed during earthquakes there is no time left for plastic deformations to redistribute the concentrated stresses. An efficient solution to prevent dislocations or failures and thus protect masonry is to superpose over the fault a grid. Due to its regular geometry and high strength to tension the stresses are redistributed among neighbouring sections avoiding masonry damage (Fig. 2).

Since such faults are everywhere in masonry the whole surfaces of structural members should be accordingly covered. Local interventions are less efficient (Coburn & Spence 2002).

3 PRACTICAL TECHNIQUES

There are many types of grids on the market but they are not suitable for such purposes. Steel grids, for instance, have apertures too large, comparing with masonry are too strong and for corrosion protection they should be embedded in cement mortar what is incompatible with masonry. The most appropriate reinforcement is based on polymer grids with solid integrated joints. They satisfy all the geometrical and mechanical requirements for such interventions.

There are two techniques for repair and strengthening masonry in historical constructions. One consists in reinforcing the vertical surfaces of structural members with sheets of polymer grids applied over masonry joints with great care for a good anchoring of render. The other technique consists in confining the structural members by wrapping them around with the same sheets of grids and also with great care for good anchoring. Both techniques are reversible and when necessary can be easily detached and replaced (Sofronie & Feodorov 1995).

4 DESIGN EXAMPLES

An existing masonry wall with the length l = 5.50 m, height \( h_o = 8.25 \) m and thickness \( t = 25 \) cm has the design compression strength \( f_c = 1.4 \) MPa and corresponding shear strength \( f_s = 0.2 \sigma_o \), unit weight \( \gamma_m = 18 \) kN/m\(^3\) and friction coefficient in horizontal joints \( \mu = 0.3 \). In first alternative the wall is reinforced on one side with a sheet of polymer grids RG20 with the design strength \( R = 20 \) kN/m and a reduction factor \( \gamma_r = 0.85 \), according to the recommendations contained in RG Manual (Sofronie 2004a). In the second alternative the wall is confined by wrapping around with the same type of reinforcement RG20 (Fig. 3).

Calculate in-plane design shear forces (DSF) according to capacity design principles at the levels of potential non-structural floors ±0.00, +2.75 and +5.50 when the wall is submitted to a vertical permanent load proportional with its own weight \( N = 1.788 \) G and emphasise the contribution of shear reinforcement. The approximate and simplified method adopted for this demonstrative example is currently
used in countries with seismic activity (Paulay & Priestley 1992).

DESIGN VERTICAL LOADS

Level ±0.00:
\[ N_0 = 1.788 \times 18 \times 5.50 \times 0.25 \times 8.25 = 365 \text{kN} \]

Level +2.75:
\[ N_1 = 1.788 \times 18 \times 5.50 \times 0.25 \times 5.50 = 243 \text{kN} \]

Level +5.50:
\[ N_2 = 1.788 \times 18 \times 5.50 \times 0.25 \times 2.75 = 122 \text{kN} \]

PLAIN MASONRY (PM)

1. Shear in eccentric compression

- Level ±0.00: \( N_0 = 365 \text{kN} \), aspect-ratio \( l/h_0 = 0.67 \).
- Area of compressed zone in the ultimate limit state (Fig. 4):
  \[ A_u = \frac{N_0}{f_t} = \frac{365}{1400} = 0.26 m^2 \] (1)
  \[ \xi = \frac{A_u}{t} = \frac{0.26}{0.25} = 1.04 m \] (2)
- Eccentricity of compression force
  \[ e = l/2 - \xi/2 = 0.5x(5.50 - 1.04) = 2.23 m \] (3)
- Moment of compression force
  \[ M_e = N_0 e = 365x2.23 = 813.95 kNm \] (4)
- Shear force afferent to moment
  \[ V_e = \frac{3M_e}{2h_0} = 1.5x\frac{813.95}{8.25} = 148 kN \] (5)
- Level +2.75: \( N_1 = 243 \text{kN} \), aspect-ratio \( l/h_1 = 1.00 \).
- Area of compressed zone in the ultimate limit state (Fig. 5):
  \[ A_u = \frac{N_1}{f_d} = \frac{243}{1400} = 0.174 m^2 \] (6)

2. Level +5.50: \( N_2 = 122 \text{kN} \), aspect-ratio \( l/h_2 = 2.00 \).
- Area of compressed zone in the ultimate limit state (Fig. 6):
  \[ A_u = \frac{N_2}{f_d} = \frac{122}{1400} = 0.087 m^2 \] (11)
- Length of compressed zone
  \[ L = \frac{N_2}{f_d} = \frac{122}{1400} = 0.087 m \] (7)
- Eccentricity of compression force
  \[ e = l/2 - \xi/2 = 0.5x(5.50 - 0.69) = 2.41 m \] (8)
- Moment of compression force
  \[ M_e = N_2 e = 243x2.41 = 585.63 kNm \] (9)
- Shear force afferent to moment
  \[ V_e = \frac{3M_e}{2h_1} = 1.5x\frac{585.63}{5.5} = 160 kN \] (10)
- Level +5.50: \( N_2 = 122 \text{kN} \), aspect-ratio \( l/h_2 = 2.00 \).
- Area of compressed zone in the ultimate limit state (Fig. 6):
  \[ A_u = \frac{N_2}{f_d} = \frac{122}{1400} = 0.087 m^2 \] (11)
- Length of compressed zone
  \[ L = \frac{N_2}{f_d} = \frac{122}{1400} = 0.087 m \] (7)
- Eccentricity of compression force
  \[ e = l/2 - \xi/2 = 0.5x(5.50 - 0.35) = 2.58 m \] (13)
- Moment of compression force
  \[ M_e = N_2 e = 122x2.58 = 314.76 kNm \] (14)
- Shear force afferent to moment
  \[ V_e = \frac{3M_e}{2h_2} = 1.5x\frac{314.76}{2.75} = 172 kN \] (15)
2. Shear in inclined sections

Level ±0.00: \( N_o = 365 \text{kN} \)

Compression stress in masonry cross-section
\[
\sigma_o = \frac{N_o}{A_w} = \frac{365}{5.50 \times 0.25} = 265.45 \text{kPa}
\] (16)

Shear strength
\[
f_v = 0.2 \sigma_o = 0.2 \times 265.45 = 53 \text{kPa}
\] (17)

Shear force in plain masonry
\[
V_i = f_v A_w = 53 \times 5.5 \times 0.25 = 73 \text{kN}
\] (18)

Level +2.75: \( N_1 = 243 \text{kN} \)

Compression stress in masonry cross-section
\[
\sigma_o = \frac{N_1}{A_w} = \frac{243}{5.50 \times 0.25} = 176.73 \text{kPa}
\] (19)

Shear strength
\[
f_v = 0.2 \sigma_o = 0.2 \times 176.73 = 35.35 \text{kPa}
\] (20)

Shear force in plain masonry
\[
V_i = f_v A_w = 35.35 \times 5.5 \times 0.25 = 49 \text{kN}
\] (21)

Level +5.50: \( N_2 = 122 \text{kN} \)

Compression stress in masonry cross-section
\[
\sigma_o = \frac{N_2}{A_w} = \frac{122}{5.50 \times 0.25} = 88.73 \text{kPa}
\] (22)

Shear strength
\[
f_v = 0.2 \sigma_o = 0.2 \times 88.73 = 17.75 \text{kPa}
\] (23)

Shear force in plain masonry
\[
V_i = f_v A_w = 17.75 \times 5.5 \times 0.25 = 24 \text{kN}
\] (24)

3. Shear in horizontal joints

Level ±0.00: \( N_o = 365 \text{kN} \)

Shear force in plain masonry
\[
V_j = \mu N_o = 0.3 \times 365 = 110 \text{kN}
\] (25)

Level +2.75: \( N_1 = 243 \text{kN} \)

Compression stress in masonry cross-section
\[
\sigma_o = \frac{N_1}{A_w} = \frac{243}{5.50 \times 0.25} = 176.73 \text{kPa}
\] (16)

Shear strength
\[
f_v = 0.2 \sigma_o = 0.2 \times 176.73 = 35.35 \text{kPa}
\] (20)

Shear force in plain masonry
\[
V_i = f_v A_w = 35.35 \times 5.5 \times 0.25 = 49 \text{kN}
\] (21)

Level +5.50: \( N_2 = 122 \text{kN} \)

Compression stress in masonry cross-section
\[
\sigma_o = \frac{N_2}{A_w} = \frac{122}{5.50 \times 0.25} = 88.73 \text{kPa}
\] (22)

Shear strength
\[
f_v = 0.2 \sigma_o = 0.2 \times 88.73 = 17.75 \text{kPa}
\] (23)

Shear force in plain masonry
\[
V_i = f_v A_w = 17.75 \times 5.5 \times 0.25 = 24 \text{kN}
\] (24)

The results of shear analysis are comparatively presented in Table 1.

<table>
<thead>
<tr>
<th>Level</th>
<th>Eccentric compression ( V_i ) (kN)</th>
<th>Inclined sections ( V_i ) (kN)</th>
<th>Horizontal joints ( V_j ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>±0.00</td>
<td>148</td>
<td>73</td>
<td>110</td>
</tr>
<tr>
<td>+2.75</td>
<td>160</td>
<td>49</td>
<td>73</td>
</tr>
<tr>
<td>+5.50</td>
<td>172</td>
<td>24</td>
<td>37</td>
</tr>
</tbody>
</table>

REINFORCED MASONRY (RM)

1. Shear in eccentric compression

Level ±0.00: \( N_o = 365 \text{kN}, \) aspect-ratio \( l/h_o = 0.67. \)

Forces on cross-section (Fig. 7):
\[
C_m = (0.25 \xi) \times 1400 = 350 \xi
\]
\[
C_r = 20 \xi
\]
\[
T_r = (5.5 - \xi) \times 20 = -20 \xi + 110
\] (28)

Equilibrium equation
\[
\sum X = 0; \ C_m + C_r - T_r = N_o;
\]
\[
350 \xi + 20 \xi - (-20 \xi + 110) = 365;
\] (29)

Comments:
1) Shear capacity in eccentric compression is strongly influenced by wall aspect-ratio. Sub-unitary aspect-ratios mean column-effect while supra-unitary ones wall-effect.
2) Shear capacity in inclined sections and horizontal joints is not influenced by aspect-ratio modification.
3) The weakest shear capacity of wall is in inclined sections. That means when submitted to seismic actions the wall is in danger to crack in form of X-shape at all levels. The shear capacity of the upper level is only one third of that of ground level.
Neutral axis
\[ \xi = \frac{365 + 110}{350 + 20 + 20} = \frac{475}{390} = 1.22m \]  

Values of forces
\[ C_m = 350 \times 1.22 = 427kN \]
\[ C_r = 20 \times 1.22 = 24.4kN \]
\[ T_r = -24.4 + 110 = 85.6kN \]

Eccentricities of forces
\[ e_m = 0.5 \times (5.50 - 1.22) = 2.14m \]
\[ e_r = e_m = 2.14m \]
\[ e_r' = 0.5 \times 1.22 = 0.61m \]

Moment in the ultimate limit state of eccentric compression
\[ M_e = (427 + 24.4) \times 2.14 + 85.6 \times 0.61 = 1018.21kNm \]

Shear force afferent to moment
\[ V_r = \frac{3M_e}{2h_o} = \frac{1.5 \times 1018.21}{8.25} = 185kN \]

Level +2.75: \( N_1 = 243 \) kN, aspect-ratio \( l/h_1 = 1.00 \).

Forces on cross-section (Fig. 8):
\[ C_m = (0.25\xi) \times 1400 = 350\xi \]
\[ C_r = 20\xi \]
\[ T_r = (5.5 - \xi) \times 20 = -20\xi + 110 \]

Equilibrium equation
\[ \sum X = 0; \quad C_m + C_r - T_r = N_1; \]
\[ 350\xi + 20\xi - (-20\xi + 110) = 243; \]  

Neutral axis
\[ \xi = \frac{243 + 110}{350 + 20 + 20} = \frac{353}{390} = 0.90m \]

Values of forces
\[ C_m = 350 \times 0.90 = 315kN \]
\[ C_r = 20 \times 0.90 = 18kN \]
\[ T_r = -18 + 110 = 92kN \]

Eccentricities of forces
\[ e_m = 0.5 \times (5.50 - 0.90) = 2.30m \]
\[ e_r = e_m = 2.30m \]
\[ e_r' = 0.5 \times 0.90 = 0.45m \]

Moment in the ultimate limit state of eccentric compression
\[ M_e = (315 + 18) \times 2.30 + 92 \times 0.45 = 807.3kNm \]

Shear force afferent to moment
\[ V_r = \frac{3M_e}{2h_2} = \frac{1.5 \times 807.3}{5.50} = 220kN \]

Level +5.50: \( N_2 = 122 \) kN, aspect-ratio \( l/h_2 = 2.00 \).

Forces on cross-section (Fig. 9):
\[ C_m = (0.25\xi) \times 1400 = 350\xi \]
\[ C_r = 20\xi \]
\[ T_r = (5.5 - \xi) \times 20 = -20\xi + 110 \]

Equilibrium equation
\[ \sum X = 0; \quad C_m + C_r - T_r = N_2; \]
\[ 350\xi + 20\xi - (-20\xi + 110) = 122; \]
Neutral axis

\[ \xi = \frac{122 + 110}{350 + 20 + 20} = \frac{232}{390} = 0.60m \]  

Values of forces

\[ C_m = 350 \times 0.60 = 210 kN \]  
\[ C_r = 20 \times 0.60 = 12 kN \]  
\[ T_r = -12 + 110 = 98 kN \]  

Eccentricities of forces

\[ e_m = 0.5 \times (5.50 - 0.60) = 2.45m \]  
\[ e_r = e_m = 2.45m \]  
\[ e_r' = 0.5 \times 0.60 = 0.30m \]  

Moment in the ultimate limit state of eccentric compression

\[ M_e = (210 + 12) \times 2.45 + 98 \times 0.30 = 573.3 kNm \]  

Shear force afferent to moment

\[ V_r = \frac{3M_e}{2h_2} = \frac{1.5 \times 573.3}{2.75} = 313 kN \]  

2. Shear in inclined sections (Fig. 10)

Level \( \pm 0.00: N_o = 365 kN \)

Shear force from reinforcement

\[ V_1 = \gamma \cdot Rl = 0.85 \times 20 \times 5.50 = 86 kN \]  

Level +2.75: \( N_1 = 243 kN \)

Shear force from reinforcement

\[ V_1 = \gamma \cdot Rl = 0.85 \times 20 \times 5.50 = 86 kN \]  

Level +5.50: \( N_2 = 122 kN \)

Shear force from reinforcement

\[ V_2 = \gamma \cdot Rl = 0.85 \times 20 \times 2.75 = 43 kN \]  

The results of shear analysis are comparatively presented in Table 2.

Table 2. DSFs in the wall of reinforced masonry.

<table>
<thead>
<tr>
<th>Level</th>
<th>Eccentric compression ( V_s ) (kN)</th>
<th>Inclined sections ( V_i ) (kN)</th>
<th>Horizontal joints ( V_j ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \pm 0.00 )</td>
<td>185</td>
<td>159</td>
<td>196</td>
</tr>
<tr>
<td>+2.75</td>
<td>220</td>
<td>135</td>
<td>159</td>
</tr>
<tr>
<td>+5.50</td>
<td>313</td>
<td>67</td>
<td>123</td>
</tr>
</tbody>
</table>

3. Shear in horizontal joints

Level \( \pm 0.00: N_o = 365 kN \)

Shear force from reinforcement

\[ V_j = \gamma \cdot Rl = 0.85 \times 20 \times 5.50 = 86 kN \]  

Level +2.75: \( N_1 = 243 kN \)

Shear force from reinforcement

\[ V_j = \gamma \cdot Rl = 0.85 \times 20 \times 5.50 = 86 kN \]  

Level +5.50: \( N_2 = 122 kN \)

Shear force from reinforcement

\[ V_j = \gamma \cdot Rl = 0.85 \times 20 \times 5.50 = 86 kN \]  

The comments:

1) Reinforcement enhances the shear capacity in all three limit states of failure, but mostly the weakest state of wall, namely that of principal stresses in inclined sections.
Table 3. Ratio RM/PM x 100 due to one sheet RG20.

<table>
<thead>
<tr>
<th>Level</th>
<th>Eccentric compression (%)</th>
<th>Inclined sections (%)</th>
<th>Horizontal joints (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>±0.00</td>
<td>125</td>
<td>218</td>
<td>178</td>
</tr>
<tr>
<td>+2.75</td>
<td>138</td>
<td>276</td>
<td>218</td>
</tr>
<tr>
<td>+5.50</td>
<td>182</td>
<td>279</td>
<td>332</td>
</tr>
</tbody>
</table>

2) Mortar in which reinforcement is embedded does not appear explicitly in any analysis. Its role of matrix is only to provide the mechanism of stress transfer from masonry to polymer grids. For this reason lime mortar is recommended because the strength comes from reinforcement. The thickness of render can be as thin as to cover the reinforcement, for instance 20 mm.

3) The wall is uniformly reinforced from bottom to top with the same polymer grids RG20 providing continuity and homogeneity as requested by EC8 principles (Sofronie 2002).

The ratios between shear forces for reinforced masonry and plain masonry are comparatively presented in Table 3.

Comments:

1) The effect of reinforcement is high exactly in the needed sections where pure shear could occur. The cost of reinforced masonry compared with plain masonry is 119%.

2) Higher mechanical performances can be obtained either by using stronger grids like RG30 or by reinforcing both sides of wall with the same grids RG20. However, the proportion between shear capacity of masonry and that of reinforcement is recommended to remain between 1/3 and 3/1.

3) The above described method of seismic protection can be similarly applied to flanged cross-sections like L, I, T, U profiles with care for fixing the grids along the inner corners.

CONFINED MASONRY (CM)

As a consequence of confining by wrapping the wall around the compression strength of masonry increases with 25% to \( f_c = 1.25 \times 1.4 = 1.75 \) MPa, while the shear strength and friction coefficient in horizontal joints became \( f_s = 0.25\sigma_o \) and \( \mu_s = 0.4 \), respectively. The reduction factor \( y_f \) is also increasing but for sake of simplicity in analysis was used the same value.

1. Shear in eccentric compression

Level ±0.00: \( N_o = 365 \) kN, aspect-ratio \( l/h_o = 0.67 \).
Figure 12. Cross section at +2.75.

Level +2.75: $N_1 = 243 \text{kN}$, aspect-ratio $l/h_1 = 1.00$.

Forces on cross-section (Fig. 12):

\[
C_m = (0.25\xi)x1750 = 437.5\xi \quad \text{(62)}
\]

\[
C_r = 2x20\xi = 40\xi
\]

\[
T_r = (5.5 - \xi)x2x20 = -40\xi + 220
\]

Equilibrium equation

\[
\sum X = 0; \quad C_m + C_r - T_r = N_1;
\]

\[
437.5\xi + 40\xi - (-40\xi + 220) = 243; \quad \text{(63)}
\]

Neutral axis

\[
\xi = \frac{243 + 220}{437.5 + 40 + 40} = \frac{463}{517.5} = 0.90m \quad \text{(64)}
\]

Values of forces

\[
C_m = 437.5x0.90 = 394kN
\]

\[
C_r = 40x0.90 = 36kN \quad \text{(65)}
\]

\[
T_r = -36 + 220 = 184kN
\]

Eccentricities of forces

\[
e_m = 0.5x(5.50 - 0.90) = 2.30m
\]

\[
e_r = e_m = 2.30m
\]

\[
e_r' = 0.5x0.90 = 0.45m \quad \text{(66)}
\]

Moment in the ultimate limit state of eccentric compression

\[
M_e = (394 + 36)x2.30 + 184x0.45 = 1071.8kNm \quad \text{(67)}
\]

Shear force afferent to moment

\[
V_e = \frac{3M_e}{2h_o} = 1.5x\frac{1071.8}{5.50} = 292kN \quad \text{(68)}
\]

Figure 13. Cross section at +5.50.

Level +5.50: $N_2 = 122 \text{kN}$, aspect-ratio $l/h_2 = 2.00$.

Forces on cross-section (Fig. 13):

\[
C_m = (0.25\xi)x1750 = 437.5\xi \quad \text{(69)}
\]

\[
C_r = 2x20\xi = 40\xi
\]

\[
T_r = (5.5 - \xi)x2x20 = -40\xi + 220
\]

Equilibrium equation

\[
\sum X = 0; \quad C_m + C_r - T_r = N_2;
\]

\[
437.5\xi + 40\xi - (-40\xi + 220) = 122; \quad \text{(70)}
\]

Neutral axis

\[
\xi = \frac{122 + 220}{437.5 + 40 + 40} = \frac{342}{517.5} = 0.66m \quad \text{(71)}
\]

Values of forces

\[
C_m = 437.5x0.66 = 289kN
\]

\[
C_r = 40x0.66 = 26.4kN \quad \text{(72)}
\]

\[
T_r = -26.4 + 220 = 193.6kN
\]

Eccentricities of forces

\[
e_m = 0.5x(5.50 - 0.66) = 2.42m
\]

\[
e_r = e_m = 2.42m
\]

\[
e_r' = 0.5x0.90 = 0.33m \quad \text{(73)}
\]

Moment in the ultimate limit state of eccentric compression

\[
M_e = (289 + 26.4)x2.42 + 193.6x0.33 = 827.16kNm \quad \text{(74)}
\]

Shear force afferent to moment

\[
V_e = \frac{3M_e}{2h_o} = 1.5x\frac{827.16}{2.75} = 451kN \quad \text{(75)}
\]
2. Shear in inclined sections

Level ±0.00: \( N_0 = 365 \, kN \)

Shear force in confined masonry

\[
V_i = 0.25N_o + 2\gamma, RI = 0.25\times365 + 2\times0.85\times20\times5.50 = 278 \, kN
\]

Level +2.75: \( N_1 = 243 \, kN \)

Shear force in confined masonry

\[
V_i = 0.25N_1 + 2\gamma, RI = 0.25\times243 + 2\times0.85\times20\times5.50 = 248 \, kN
\]

Level +5.50: \( N_2 = 122 \, kN \)

Shear force in confined masonry

\[
V_i = 0.25N_2 + 2\gamma, RI = 0.25\times122 + 2\times0.85\times20\times2.75 = 124 \, kN
\]

3. Shear in horizontal joints

Level ±0.00: \( N_0 = 365 \, kN \)

Shear force in confined masonry

\[
V_j = \mu N_o + 2\gamma, RI = 0.4\times365 + 2\times0.85\times20\times5.50 = 333 \, kN
\]

Level +2.75: \( N_1 = 243 \, kN \)

Shear force in confined masonry

\[
V_j = \mu N_1 + 2\gamma, RI = 0.4\times243 + 2\times0.85\times20\times5.50 = 284 \, kN
\]

Level +5.50: \( N_2 = 122 \, kN \)

Shear force in confined masonry

\[
V_j = \mu N_2 + 2\gamma, RI = 0.4\times122 + 2\times0.85\times20\times5.50 = 236 \, kN
\]

Table 4. DSFs in the wall of confined masonry.

<table>
<thead>
<tr>
<th>Level</th>
<th>Eccentric compression</th>
<th>Inclined sections</th>
<th>Horizontal joints</th>
</tr>
</thead>
<tbody>
<tr>
<td>±0.00</td>
<td>233</td>
<td>278</td>
<td>333</td>
</tr>
<tr>
<td>+2.75</td>
<td>292</td>
<td>248</td>
<td>284</td>
</tr>
<tr>
<td>+5.50</td>
<td>451</td>
<td>124</td>
<td>236</td>
</tr>
</tbody>
</table>

Table 5. Ratios CM/RM \times 100 and CM/PM \times 100.

<table>
<thead>
<tr>
<th>Level</th>
<th>Eccentric compression</th>
<th>Inclined sections</th>
<th>Horizontal joints</th>
</tr>
</thead>
<tbody>
<tr>
<td>±0.00</td>
<td>126% 157%</td>
<td>175% 381%</td>
<td>170% 303%</td>
</tr>
<tr>
<td>+2.75</td>
<td>133% 183%</td>
<td>184% 506%</td>
<td>179% 389%</td>
</tr>
<tr>
<td>+5.50</td>
<td>144% 262%</td>
<td>185% 517%</td>
<td>192% 638%</td>
</tr>
</tbody>
</table>

3) Confined masonry is a true composite material. The cost of confined masonry compared with that of plain masonry is 138%.

The ratios between design shear forces for confined and reinforced masonry as well as confined and plain masonry are comparatively presented Table 5.

Comments:

1) The effect of confinement is large exactly in the sections it is necessary, namely where pure shear can occur.
2) Comparing with reinforced masonry the confinement enhances the shear capacity almost from simple to double.
3) Comparing with plain masonry the confinement enhances the shear capacity from almost four to six times.
4) In confined walls the openings for doors and windows should be avoided. If that is not possible the corners of the openings should be additionally reinforced according to Manual provisions (Sofronie 2004b,c).
5) The sheets of reinforcement are joined by superposition over a length of minimum 400 mm.

5 CONCLUSIONS

1. Polymer grids used for reinforcing or confining masonry enhances the shear capacity of walls in buildings and structures. The merit of grids is first due to the fact they are acting on the whole surface of walls not locally only. Secondly by their regular geometry the grids contribute to standardize the stresses and prevent stress concentrations around faults, which originate dislocations. Finally,

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due to high strengths of grids the reduced tensile resistances of mortars are substituted and lime or lime-cements can be used for plastering.

2. The remarkable quality of RichterGard System is reversibility. Indeed, in case of damage or changing the construction approach the synthetic reinforcement can be easily detached and replaced with a new one in accordance with the new requirements.

3. The relation geometry-matter is not limited to pure shear only. In the sections with high aspect-ratios where the compression force is diminishing the friction is reduced and consequently the shear capacity in horizontal joints decreases. The thickness of plaster loses its significance comparing with the strength of grids. The relative cost of reinforcing or confining also depends by the ratio between the surface on which the grids are applied and the volume of masonry. It is more convenient to reinforce massive masonry members than slender walls or columns. However, the most important effect of RichterGard System remains the uniform distribution of stresses on wall surfaces with the aid of polymer grids used as reinforcement.

4. The comparative economic analysis presented in the paper was limited only to the cost of materials, namely of masonry and reinforcement. A comprehensive analysis should include the low cost and high productivity of labour as well as the short duration of construction.

5. There are also other several applications of the two techniques for repair and strengthening masonry in historical constructions. One is to support thermal isolation like foam glass plates or others. Then seismic protection of brick veneers and curtain walls. Finally, due to their capacity of avoiding stress concentrations, the walls reinforced or confined with polymer grids are well protected against terror attacks. There is not at all much trouble to apply the method to the existing constructions.

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REFERENCES