Cracks in historic masonry walls due to settlement – experiments and mechanical modelling

H. Wigger, (1) F.S. Rostásy (1)

(1) Testing Institute for Building Materials, D-38106 Braunschweig, Germany
EMail: h.wigger@tu-bs.de
EMail: f.rostasy@tu-bs.de

Abstract

Many walls – of brick or natural stone – of historic buildings exhibit cracks through the entire thickness of wall. Such cracks are often caused by “historic” and also by recent differential settlement along the base of wall. Thereby, restraint actions arise in the wall, which are the response to the interaction of the wall with the foundation and ground.

In many cases, the causes of historic cracks must be clarified in order to assess the load carrying capacity of the damaged wall and to decide on eventual structural repair. On the other hand, the possibility of future cracks must be known if additional settlement is to be expected, if repair of foundations and consolidation of ground are to be planned etc. For such tasks, the development of a predictive mechanical model of the interaction is necessary. This report presents first results of this problem.

Wall strips of single and triple leaf natural stone masonry were built for experiments (L:H:D = 4.5 m:1.0 m:0.45 m). In a heavy test set-up, a hollow with a bending radius of R = 400 m was forced upon the specimen. Forces, deformations and cracks were measured, fracture phenomena were observed.

The mechanical model includes the description of the critical settlements and the leading to cracks structural behaviour of masonry walls in bending and shear. Furthermore, strengthening measures and their effects on the behaviour are considered in the model.
1 Introduction

Subsistence or consolidation of ground as well as load changes may lead to differential settlement. Restraint occurs in rising masonry. Due to small tensile failure strain of masonry, shear and bending shear cracks may already result for small settlement differences. These differences absorbable freely from a building structure are dependent on the construction, the stiffness and the used materials.

TERZAGHI (1935) started the discussion about settlement differences. They have to be considered for building structures (except for foundations on rock). Damage criteria were determined by observation of existing buildings. The following parameters must be known to determine the restraint stresses caused by differential settlement:

- distribution of settlement below foundation,
- time dependence of settlements and
- location of soil layers sensitive to settlement.

The settlement distributions can show different forms. The basic form represents the settlement hollow. It appears in case of a soft building structure and a homogenous soil layer. Since such case hardly occurs, different deformation shapes will appear. The temporal process of settlement is dependent on the ground. Buildings on sand or gravel layers already reach their maximum settlement after few months. On the other hand, foundations on cohesive soil reach their maximum settlement only after longer periods or a continuing creep can be recognized. Settlements can also occur unexpectedly. Sinks, mining industry activities etc. may lead to sudden settlement of building.

Boundary values for settlement differences Δ can be defined by the angle β, by the curvature κ and by absolute values of maximum settlement s^max. The deformations due to settlement which can lead to cracks are shown in Figure 1 for a hollow without change of curvature. Approved boundary values are derived from geodesic measurements of existing buildings. Table 1 gives some damage boundaries.

The radius of curvature represents an essential criterion to assess safety against cracking. Regarding the geometrical values of Figure 1 and maximum settlement difference Δ of a beam on two supports at midspan the curvature κ and the radius of curvature R is determined by:

\[ \kappa \approx \frac{1}{R} \approx \frac{8 \cdot \Delta}{L^2} \]  

(1)
2 Experiments

To analyse the behavior of deformation, cracking and breaking of single- and multi-layer stone walls a large test set-up was built at our institute.
In course of the experiment, a hollow with a uniform curvature was forced on the walls. The initial situation was the wall with its non-uniform load at the wall crown. During the deformation controlled test, the wall is put into the desired hollow by means of controlled redistribution of the base reactions. The corresponding base reactions were adjusted for a nearly constant curvature over a large area of the wall’s bottom edge. The load was increased adequately to obtain an increase deflection bending of one millimetre per load step.
The investigations were carried out on different kinds of walls which varied in stone material, thickness, number of layers, and stone bond. Several load stages were run. At first a bending load was put to the wall. After the first cracks, the load was increased to 75% of maximum test bed capacity. Then the wall was prestressed across its front sides while retaining the current bending conditions. The bending load was increased up to maximum load capacity after prestress was set up totally.

The experiments began with single-layer walls made of sand-lime stone \((L:H:D = 4.5 \text{ m}:1.0 \text{ m}:0.24 \text{ m})\) to check the reliability of the testing and measuring equipment. Subsequently test runs on single- and multi-layer walls made of local limestone were carried out.

### 3 Results

The analysis of the experiments was made in regard to curvature and tensile failure strain. The curvature was derived by means of eqn. (1) and the measured differential settlement \(\Delta\). The distribution of moments within length \(L\) was simplified to a constant moment. Figure 4 gives an example of a deflection line.

![Deflection line of ashlar masonry wall (M = 144 MNm)](image)

The modulus of elasticity was determined by the curvature, the constant moment, and the wall’s second moment of area. Adequate conformity was found with the secant E-modulus of accompanying compression tests.

Table 2 shows the results. The higher bending stiffness of sand-lime stone and ashlar masonry leads already at smaller differential settlement to cracking within the walls. Masonry with higher mortar content has a lower bending stiffness allowing bigger differential settlement before cracking.

A change in load-bearing behaviour of the masonry wall after cracking is clearly recognisable (Figure 4). In the upper part of the wall load-bearing takes place by compressive stresses, whereas in the lower part of the wall tensile forces are diverted into the ground by friction (Figure 5).
Table 2. Results of experiments at first cracking

<table>
<thead>
<tr>
<th>Masonry</th>
<th>Stone</th>
<th>$E$ [MN/m$^2$]</th>
<th>$\varepsilon_U$ [%]</th>
<th>$M_U$ [MNm]</th>
<th>$\Delta L$ $[10^{-3}]$</th>
<th>$\kappa$ $[1/m\cdot 10^{-3}]$</th>
<th>$R$ [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>stretcher bond</td>
<td>sand-lime</td>
<td>8150</td>
<td>0,11</td>
<td>106</td>
<td>0,11</td>
<td>0,23</td>
<td>4444</td>
</tr>
<tr>
<td>regular coursed</td>
<td>Elmkalk</td>
<td>2950</td>
<td>0,69</td>
<td>98</td>
<td>0,35</td>
<td>0,70</td>
<td>1429</td>
</tr>
<tr>
<td>irregular coursed</td>
<td>Elmkalk</td>
<td>3700</td>
<td>0,53</td>
<td>99</td>
<td>0,45</td>
<td>0,90</td>
<td>1111</td>
</tr>
<tr>
<td>block-in-course</td>
<td>Elmkalk</td>
<td>3850</td>
<td>0,17</td>
<td>92</td>
<td>0,48</td>
<td>0,95</td>
<td>1053</td>
</tr>
<tr>
<td>ashlar</td>
<td>Elmkalk</td>
<td>9100</td>
<td>0,44</td>
<td>26</td>
<td>0,11</td>
<td>0,21</td>
<td>4762</td>
</tr>
</tbody>
</table>

Hollow formation may continue until an exceeding of friction forces $F_H$ occurs or until the masonry fails due to exceeding of compressive strength in horizontal joint direction in the upper part.

Further, it was shown by experiments that the initial load-bearing stiffness of masonry was able to be restored by prestressing the wall in its lower part (Figure 6).
Additionally the deformation was recorded by photogrammetry using 300 points on the wall to measure the movement in direction of x, y and z (Figure 2). By means of those data strain values can be computed (DOLD, REINKING & WARNECKE (1993)) and local deformations, curvatures and distortions can be determined. Hereby the mechanical behaviour due to cracking can be described more exactly. Figure 7c shows the principal strain of ashlar masonry deduced from the results of the photogrammetry.

Figure 7. Crack map and strain of ashlar masonry (M = 144 MNm)

Regarding those results a mechanical model for masonry under differential settlement can be suggested. According to it, the load-bearing behaviour of masonry under settlement compulsion varies in three cases:
1. Intact masonry (without cracks)
   As a criterion for permissible deformations through settlements, the bending
   radius is prevail. For this, the permissible tensile failure strain must be deter-
   mined. The simplified model by BURLAND & WROTH (1974) can be used.

2. Masonry with cracks
   If a crack occurs in the masonry, a new bearing system will appear. In the
   lower part of the wall the horizontal forces are diverted into the ground by
   friction. In the upper part of the wall load-bearing takes place by compressive
   strain. The failure occurs by exceeding the compressive stresses in masonry
   or the frictional resistance between ground and wall.

3. Prestressed masonry with cracks
   The horizontal forces within the wall are diverted into the ground by friction
   and are also absorbed by prestressing. The criterion is based on the permissi-
   ble compressive stresses in the upper part as well as in the force application
   area.

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