31.—The Strength of Horizontally Loaded Prefabricated Brick Panel Walls

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ABSTRACT
The paper reports an experimental study of prefabricated brick panel walls 1.96 × 2.80 m (6.5 × 9.2 ft) laterally loaded to failure. The walls consist of two vertical layers of bricks with and without reinforcement. The load is applied by compressed air acting inside a plastic bag mounted between the test wall and a rigid panel. The influence of reinforcement mesh is studied and an attempt is made to calculate the ultimate load by the yield-line theory.

1. INTRODUCTION
The effect of horizontal loading on walls is a design problem of topical interest. This interest has been accentuated by the storm damage which occurred in Sweden during the autumn of 1969.

Different countries use different methods of calculating the strength of an unreinforced brick wall subject to lateral loading. The wall is usually regarded for purposes of calculation as an elastic plate. Load at failure is usually defined as the load which causes incipient tensile cracks in a joint.

A series of single-leaf brick walls measuring 2 × 3.5 m (6.5 × 11.5 ft) supported along all four edges and subjected to uniformly distributed horizontal loading, has been tested over the last few years at the Division of Structural Engineering at Chalmers University of Technology. The tests show that an increase in loading is possible after the first crack has formed. A crack pattern as shown in Figure 1 is obtained at final failure. This pattern bears a striking resemblance to crack formation obtained on testing concrete slabs which are reinforced by two layers of bars each parallel to the edges of the slab, which are usually analysed in accordance with the yield-line theory of JOHANSSON. It would seem natural therefore that an attempt should be made to calculate load at failure by means of the yield-like theory. Good agreement has been obtained with observed loads at failure. This work was presented elsewhere.

However, the method has been criticized on the grounds that all the conditions of the yield-line theory are not satisfied, the objection being that a moment cannot be transmitted along a joint that has cracked. In spite of this, some increase in loading has been observed after cracking has started. This may be due to arching action in the wall and to the fact that some moment can be absorbed in diagonal joints owing to friction in the connection.

There is a great need for experimental work in this field, in view of the very approximate rules used for the design of brickwork that is subjected to lateral loading.

2. TEST PROGRAMME
In order to study further the resistance of walls to wind loads, a new test series on horizontally-loaded walls, which is not yet fully completed, has been performed. The walls in this test series are prefabricated under factory conditions and are used as infill panels in dwelling houses and as facing walls in other buildings. They are

FIGURE 1—Single-leaf unreinforced brick wall after loading to failure in the test series according to LOSBERG and JOHANSSON. Load indications in kgf/m².
also used as load-bearing facing walls in, for example, detached houses in which the wall is complemented by heat insulation and an internal wall surface.

The walls are 2.80-m (9.2-ft) high and 1.96-m (6.5-ft) long overall, have a thickness of 14 cm (5.5 in.) and consist of two vertical brick leaves with mesh reinforcement in the middle, as shown in Figure 2. The facing brick is 6-cm (2.4-in.) thick and has the dimensions 23 x 5.2 cm (9 x 2 in.). The walls are prefabricated horizontally and vibrated, as a result of which the cement mortar fills the joints and the cavities in the bricks completely. The prefabricated panels have much lower permeability as a result and their bending strength is many times that of comparable brickwork constructed in the normal manner.

![Figure 2 - Sections through prefabricated brick wall panel.](image)

The tests were carried out about 8 months after manufacture with the walls simply supported along all four edges. Horizontal loading to failure was applied by compressed air acting inside a plastic bag of the same size as the wall, which was mounted between the test wall and a rigid panel. The rigid panel and the wall were firmly attached to a supporting frame. (Figures 3 and 4). The load was regulated by means of a compressed air valve placed on the inlet side. The pressure inside the bag was measured by means of a simple U-tube which recorded the static air pressure. Loading was increased in steps, the deflection at the centre being recorded at each value of the load.

3. OBSERVATIONS ON THE LOADING TEST

Figures 5–8 show some of the walls after test loading. The cracks have been painted in; the numbers adjacent to the cracks indicate the extent of crack formation at the respective pressure in kgf/m². Test results are summarized in Table 1.

Wall No. 1 was completely unreinforced. A vertical crack occurred right across the wall at 3000 kgf/m² (430 lbf/in²). This quickly spread out to assume the configuration shown in Figure 5. The maximum load occurred immediately prior to the first crack, contrary to the procedure in the case of the single-leaf brick walls described in the introduction. Failure in wall No. 1

![Figure 3 - Side view of test rig. From left to right, it consists of: (1) supporting frame UNP 28; (2) wooden insert plus soft fibreboard cut to the curvature of the wall; (3) test wall; (4) plastic bag (cannot be seen); (5) rigid panel affixed to supporting frame by large-diameter bolts.](image)

![Figure 4 - Front view of test rig, showing a test wall after it had been loaded to failure. Static air pressure was measured by the U-tube to the right of the illustration.](image)
Table 1—Test Results

<table>
<thead>
<tr>
<th>Wall No. 1</th>
<th>Wall No. 2</th>
<th>Wall No. 4</th>
<th>Wall No. 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>No reinforcement</td>
<td>One 6-mm horizontally + one 6-mm vert. in the middle of the wall</td>
<td>5-mm mesh reinforcement at 150-mm centres</td>
<td>5-mm mesh reinforcement at 150-mm centres</td>
</tr>
</tbody>
</table>

<table>
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<th>Test results:</th>
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<th>Wall No. 4</th>
<th>Wall No. 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cracking pressure (kgf/m²)</td>
<td>3000</td>
<td>2800</td>
<td>3100</td>
<td>4500</td>
</tr>
<tr>
<td>Failure pressure (kgf/m²)</td>
<td>3000</td>
<td>3200</td>
<td>4700</td>
<td>5600</td>
</tr>
</tbody>
</table>

| Calculated cracking pressure: | | | | |
| Theory of elasticity, v = 0 | 3250 | 3250 | 4060 | 5550 |

| Calculated failure pressure: | | | | |
| Yield-line theory (values of moments at failure according to detailed tests) | 3780 | 5210 | 5710 |

<table>
<thead>
<tr>
<th>Values of moments at failure from detailed tests:</th>
<th>Wall No. 1</th>
<th>Wall No. 2</th>
<th>Wall No. 4</th>
<th>Wall No. 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>( m_p ) (parallel to horizontal joint) (kgf/m)</td>
<td>638</td>
<td>638</td>
<td>983</td>
<td>750</td>
</tr>
<tr>
<td>( m_n ) (perpendicular to horizontal joint) (kgf/m)</td>
<td>756</td>
<td>756</td>
<td>937</td>
<td>1275</td>
</tr>
</tbody>
</table>

*1 mm = 0.03937 in; 1 kgf/m² = 0.14223 lbf/in²; 1 kgf/m = 0.67197 lbf/ft.

I. H. E. Nilsson and A. Losberg 193

1. H. E. Nilsson and A. Losberg

1. H. E. Nilsson and A. Losberg 193

was very sudden and occurred without prior warning.

Wall No. 2 was reinforced by only one 6-mm (0.24-in.) Ps50 bar (\( \sigma_{yield} = 6300 \) kgf/cm²; 89 600 lbf/in²) vertically, and one 6-mm (0.24-in.) Ps50 bar horizontally in the middle of the wall. The effect of the reinforcement was that the load could be increased to the failure pressure of 3200 kgf/m² (455 lbf/in²) after the cracking pressure of 2800 kgf/m² (398 lbf/in²) had been reached (Figure 6). Failure of this reinforced wall was thus not as sudden as that of the unreinforced one.

Wall No. 4 was mesh reinforced by 5-mm (0.20-in.) Ps50 bars at 150-mm (5.9-in.) centres (\( \sigma_{yield} = 7300 \) kgf/cm²; 103 828 lbf/in²). The first crack occurred at 3100 kgf/m² (440 lbf/in²). At a central deflection of 27 mm (1.06 in.), failure occurred gradually at 4700 kgf/m² (668 lbf/in²) (Figure 7).

Wall No. 6 had the same reinforcement as Wall No. 4 but it was turned with the facing side outwards. Testing thus corresponded to wind suction on the wall. The cracking pressure in this case is higher (4500 kgf/m²; 640 lbf/in²). Failure occurred gradually at 5600 kgf/m² (795 lbf/in²) at a central deflection of 38 mm (1.5 in.) (Figure 8). The increase in load compared with the previous wall is due to the larger effective depth of the reinforcement in this case.

Figure 9 shows the deflections in the walls for different loads. The unreinforced wall suffered sudden failure at an insignificant deflection. Reinforcement in wall No. 2, consisting only of one vertical and one horizontal reinforcing bar at right angles in the middle of the wall, conferred a certain amount of yielding capacity on the wall. Mesh reinforcement in walls Nos. 4 and 6 gave the walls considerable yielding capacity while the crack pattern was developing.

4. DETAILED TESTS TO DETERMINE VALUES OF THE MOMENT AT FAILURE

In order to be able to analyse the cracking pressures and failure pressures of the walls, the moments at failure parallel to and at right angles to the horizontal joints were determined by means of detailed tests.

Undamaged wall sections without reinforcement were extracted from walls Nos. 1 and 2 and were loaded to bending failure. Figure 10 shows the method of testing. The wall sections were loaded upright in order to eliminate the effect of the dead weight moment. The following mean values were obtained from each of five loading tests:

\[ m_p = 638 \text{ kgf/m (429 lbf/ft)} \] (failure parallel to the horizontal joint)

\[ m_n = 756 \text{ kgf/m (508 lbf/ft)} \] (failure perpendicular to the horizontal joint)

For walls reinforced with bars at right angles to one
The Strength of Horizontally Loaded Prefabricated Brick Panel Walls

FIGURE 5—Failure pattern of wall No. 1 (unreinforced).

FIGURE 6—Failure pattern of wall No. 2 which was reinforced by one 6-mm (0.24-in.) bar vertically and one 6-mm (0.24-in.) bar horizontally at the middle of the wall.

FIGURE 7—Failure pattern of wall No. 4 (mesh reinforced).

FIGURE 8—Failure pattern of wall No. 6 (mesh reinforced). The wall was loaded with the facing side outwards, corresponding to wind suction.
L. H. E. Nilsson and A. Losberg

Figure 9—Relationship between load and deflection at centre of wall.

Figure 10—Testing arrangement for bending tests on wall sections.

Another, the moment at failure $m_p$ was determined on a complete wall which was subjected to uniformly distributed loading. The wall was simply supported at the top and bottom. Figure 11 shows the failure pattern of the wall. The uncracked wall sections were again subjected to loading, values of $m_p$ being obtained together with further values of $m_p$. The mean values of the failure moments obtained are shown in Table 1.

The placing of the reinforcement in the middle of the wall means that the section resists a larger moment at Stage 1 (uncracked) than at Stage 2 (cracked). The way in which a slab spanning in one direction resists a bending moment is outlined in Figure 12. The moment is reduced by about 40% once cracking has occurred. The yield-line theory stipulates that bending moment can be increased up to the yield point of the reinforcement and then maintained there without reduction. This stipulation is therefore not satisfied for the tested walls reinforced by reinforcement crossing at right angles.

Figure 11—Cracking pattern of wall supported at the top and bottom and subjected to uniformly distributed load.

Figure 12—Relationship between load and deflection before and after cracking.
5. ANALYSIS OF TEST RESULTS

The analysis carried out aims to establish whether cracking loads of the walls can be calculated in accordance with the theory of elasticity and whether the failure loads of the reinforced walls can be estimated by means of the yield-line theory.

When calculating the cracking load according to the theory of elasticity, the wall is regarded as a thin isotropic plate of linearly elastic material simply supported on all four sides and subjected to transverse loading. Poisson's ratio \( \nu \) has been put equal to zero.

The principle of virtual work is made use of in calculating the failure load of a wall simply supported along all four edges, in accordance with the yield-line theory. The centre portion is assumed to have a central deflection \( y = 1 \). There is in this case, in connection with brick walls, a risk that there will be some constraint towards alignment of the diagonal yield-line cracks along the joint pattern. The tests show a complicated yield-line pattern. Failure calculations are, however, carried out according to a simplified yield-line pattern as shown in Figure 13, the work equation being

\[
q \left( x \cdot a \cdot \frac{1}{2} + (b - 2x) \cdot a \cdot \frac{1}{4} \right) = m_x \cdot a \cdot \frac{1}{2} + m_y \cdot b \cdot \frac{1}{2}
\]

The value of \( x \) is obtained by differentiation of the work equation. Calculated values of \( x \) produce acceptable agreement with the tests.

![Figure 13—Yield-line pattern.](image)

Calculated cracking and failure pressures are compared in Table 1 with those observed. Values of the moments on which the calculations have been based have been obtained in the detailed tests described above.

6. CONCLUSIONS

The unreinforced wall behaved as an elastic plate up to the cracking pressure. It may therefore be regarded as an unreinforced concrete slab with bricks cast in. The first crack gave rise to a very sudden failure without prior warning. Failure moments \( m_x \) and \( m_y \) were very high here, and diminution of moment along the first crack resulted therefore in sudden failure. The cracking pressure calculated in accordance with the theory of elasticity shows acceptable agreement with that observed.

Wall No. 2 was reinforced by only one vertical and one horizontal reinforcing bar in the middle of the wall. When performing calculations based on a yield-line as in Figure 13, the failure load obtained was somewhat high, the ratio observed failure pressure/calculated failure pressure being 0.85. The validity of the yield-line theory may be questioned in the case of this wall. The theory stipulates that the moment does not diminish after the wall had cracked. This cannot be satisfied with such a small amount of reinforcement. It has nevertheless been possible to record some increase in load and deflection after the cracking pressure had been reached, as will be seen in Figure 9.

Walls Nos. 4 and 6 were mesh reinforced. The failure pressures were calculated on the basis of a yield-line pattern according to Figure 13. Acceptable agreement with observation was obtained.

7. SUMMARY OF RESULTS OBTAINED

The prefabricated walls tested can resist much higher horizontal loads than single-leaf brick walls constructed in the conventional way.

For the unreinforced prefabricated wall panel, the agreement between the cracking load observed and that calculated in accordance with the theory of elasticity was good. For this wall, the cracking pressure was equal to the failure pressure.

A small quantity of reinforcement at right angles prevents sudden failure of the wall and raises the failure pressure a little in comparison with an unreinforced wall.

For mesh-reinforced walls, load can be raised appreciably after cracking pressure has been reached. Gradual failure occurs after a large deflection. For the walls tested, the failure pressures can be estimated by means of the yield-line theory.

REFERENCES
