BEHAVIOUR OF MASONRY INFILLED R/C FRAMES UNDER HORIZONTAL LOADING. EXPERIMENTAL RESULTS

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1. ABSTRACT

This paper reports some results of an experimental research programme carried out at the Centre for Studies and Equipment's in Earthquake Engineering (C3ES) of the National Laboratory for Civil Engineering of Portugal (LNEC), aiming at the behaviour analysis of masonry infilled R/C frames under horizontal loading.

Nine models, in a scale 2:3, were tested, of which seven consisted of one-storey, one-bay reinforced concrete frames infilled with brick masonry walls (three with a window, four fully infilled) and the remaining two consisted in just the bare frames. These bare frames were used as a reference for the analysis of the results obtained in the infilled frames tested. Two different horizontal histories of displacements (cyclic and monotonic, under identical velocities) were imposed at the level of the beam centreline, while constant vertical forces were applied at the top of the columns in order to reproduce the effect of the upper floors of a building. Materials and construction techniques normally used in Portugal were applied.

2. INTRODUCTION

It has been recognized in recent years that masonry infills have an important effect on the global seismic response of R/C frame structures [1]. Their effects can be positive because they generally increase considerably the global resistance to lateral loads and the energy dissipation capacity.

Keywords: Infilled Frames; Masonry; Static Test; Cyclic Loading; Clay Brick

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On the other hand, infills may also affect negatively the seismic performance of the R/C frame structures: they increase the lateral stiffness of the structures and, thus, the seismic forces which may not be counterbalanced by the increase in lateral resistance; they may affect the initial collapse mechanism of the bare frames (short column effect); they may originate torsional or soft storey effects due to irregular arrangements or panel failure at only one floor; the infills cracking and damage due to in-plane response increase their vulnerability to out-of-plane forces.

Seismic codes neglect completely or take into account to a very limited extent the effects of infills on the global response of reinforced concrete structures. These seismic codes must be changed in the direction of considering more realistically the positive and the negative effects of the infills.

Several experimental and analytical works have been carried out with the major aim of studying the phenomena of interaction between infill walls and frames, and the parameters likely to influence such phenomena, as well as developing analytical models to simulate the behaviour of infilled frames [2,3].

Before studying a complete structure, the structural behaviour of the infills (usually considered "non-structural" elements) has to be, in a first stage, better understood and, in a second stage, quantified. Because the structural characteristics of a single panel are very important and reasonably representative of the ultimate resistance of the buildings, an extensive experimental programme of tests is being performed at LNEC (Portugal) [4, 5, 6 and 7] aiming to study the behaviour of single structures made of one-storey, one-bay reinforced concrete frames infilled with masonry as usual in Portugal, under horizontal actions.

In this paper the results of three models tested with a cyclic horizontal history of displacements are presented.

3. MODELS DESCRIPTION

Nine models in a 2:3 scale were tested, named: two bare frames, four fully infilled and three with a window opening.

The geometric characteristics of the models and the loads applied reproduced a reinforced concrete frame infilled with brick masonry, located in the ground floor of an ordinary building.

The three models 14, 16 and 17 considered in this paper are constituted by one-storey, one-bay reinforced concrete frame infilled or not with brick masonry walls (one bare frame, one with a window opening and one fully infilled). The models had an height of 1.80 m and a length of 2.40 m. The columns and the beams cross sections have, respectively, 0.15 m x 0.15 m and 0.15 m x 0.20 m. The columns were reinforced with 8φ10 longitudinal bars and φ8//0.04 hooks. The beams were reinforced with 6φ8 longitudinal bars and φ6//0.05 stirrups. The infill was built with 0.30 m x 0.20 m x 0.15 m horizontally hollow bricks, usual in Portugal, bedded using mortars with the
proportions 1:4 in volume (cement: river sand). The materials used in the construction of the frame were a C20/25 concrete and a S400 steel.

The models were built on reinforced concrete blocks with a 3.24 m x 0.74 m x 0.35 m volume. These concrete blocks were used to fasten the models to the shaking table.

Figure 1 illustrates the characteristics of model 17.

![Fig. 1 - Model 17. Geometrical characteristics and reinforcements](image)

4. TEST SET-UP AND INSTRUMENTATION

It was decided to use the platform of a shaking table to move the base of the models under a quasi-static velocity of 5mm/s. In order to impose a relative alternate horizontal displacement between the base of each frame and its top beam centreline, this last was linked to a reaction wall using a connecting rod allowing the necessary rotations in the plane of the frame (the scheme of the test set-up is presented in Figure 2). A loading cell was installed to measure the force generated at the connecting rod during the tests. Vertical forces were also simultaneously applied at the top of both R/C columns by means of two single-acting servo-hydraulic actuators whose forces were duly monitored during the tests. In order to avoid out-of-plane deformations a guiding system was used.

Besides the devices for the measurement of forces already mentioned, three other types of instrumentation were used for the signals acquisition: optical displacement transducers (6), inductive displacement transducers (4 to 10) and accelerometers (2). The first ones, allowing a two channels in-plane measurement, were used to record the displacements effectively achieved with the shaking table and the ones produced along the R/C column. Six of the second ones were used to record masonry displacements, being used the remaining four for cross-checking the optical transducers responses. Finally, the accelerometers were used to determine the transfer functions, between the base and the top of the frames, before and after testing.
All the data from the 20 channels (bare frame - model 14), or 26 channels (infilled frames - models 16 and 17), were driven through different conditioning equipment to a data acquisition system allowing a quick preliminary interpretation and the necessary correction during the tests.

5. TESTING PROCEDURE

Previously to the input of the cyclic horizontal history of displacements, the impulsive response accelerations of the undamaged structures was obtained, obviously before the assemblage of the connecting rod, under two different conditions - without vertical load and under 100 kN forces simultaneously applied at the top of the columns. To generate those signals a stiff hammer shock was induced at the beam level.

After the assemblage of the connecting rod and under permanent 100 kN vertical loads, five stages of two complete sine waves (Figure 3) were successively imposed for progressive amplitudes (Table 1). In order to produce the quasi-static velocity already mentioned, the following frequencies were successively adopted: 1.326 Hz, 0.032 Hz, 0.016 Hz, 0.01 Hz and 0.008 Hz.

Finally, and after separation of the connecting rod, the acquisition of the accelerometer signals was, once more, obtained under and without the vertical load.
Table 1 - Peaks of displacement for the different stages of the tests

<table>
<thead>
<tr>
<th>Stage</th>
<th>Maximum displacements [mm]</th>
<th>Target</th>
<th>Measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.6</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>25</td>
<td>23</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>50</td>
<td>47</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>75</td>
<td>71</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>100</td>
<td>96</td>
<td></td>
</tr>
</tbody>
</table>

![Figure 3 - Sequence of imposed displacements.](image)

6. PRESENTATION OF RESULTS

In order to illustrate the behaviour of the tested models some results are presented in Figures 4 and 5 for specimens 14 (bare frame), 16 (infilled frame with window) and 17 (fully infilled frame). In Table 2 some summary results comparing the initial elastic stiffness and the maximum strength of the three models are also presented.

Table 2 - Comparison between parameters of infilled and bare frames

<table>
<thead>
<tr>
<th>Model</th>
<th>Maximum strength Fmáx [kN]</th>
<th>Initial elastic stiffness Ke [kN/m]</th>
<th>f máx In Fmáx 14</th>
<th>Ke In Ke 14</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>64</td>
<td>26 786</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>16</td>
<td>123</td>
<td>135 420</td>
<td>1.9</td>
<td>5.1</td>
</tr>
<tr>
<td>17</td>
<td>250</td>
<td>258 150</td>
<td>3.9</td>
<td>9.6</td>
</tr>
</tbody>
</table>

The photos presented in Figure 4 show the three models at the end of the final stage of the tests, illustrating the failure modes observed for each case. Figure 5 presents the horizontal force-top displacement diagrams and the corresponding hysteretic energy dissipation histories.
Fig. 4 - Models failure modes: a - model 14; b - model 16 and c - model 17.

Fig. 5 - Horizontal force-top displacement hysteretic loops and respective dissipated energy

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7. ANALYSIS OF RESULTS

Failure Mechanisms

The failure mechanisms observed in the three models can be deduced from the observation of Figure 4.

The bare frame, as it was expected, presented a hysteretic mechanism originated by the formation of hinges at the top and base of the columns. No significant damage was observed along the columns neither the spreading of the plastic hinges was verified.

In what concerns the infilled frames, a similar global failure mechanism was observed. In fact in both cases the behaviour was conditioned by the initial cracking pattern of the masonry panel, which began with the separation of the masonry from the reinforced concrete frame along their vertical and horizontal interfaces. Simultaneously, a full-length horizontal crack was formed approximately at the same level in both models, defining two different zones in the masonry panel. In the fully infilled frame an additional diagonal crack occurred at the upper block of the panel; this was not observed in the model with window since the upper part of this model was already completely separated from the reinforced concrete frame and naturally divided in two blocks, due to the existence of the window. At a latter stage the crushing of the masonry panel corners was observed in the fully infilled frame.

Under these circumstances, and as it is illustrated in Figure 4, for the level of imposed displacements in the tests, similar hysteretic mechanisms can be assumed for the reinforced concrete frames of the three models. This similarity has however to be carefully understood, as in the case of the infilled frames it is predictable that, for higher displacement demands, the inelastic deformation in the columns would be developed at a height of about 0.60 m and 0.40 m, respectively for the model with window and for the fully infilled one.

Evolution of the Hysteretic Response

As it can be seen in Figure 5 and in Table 2, the infilled frames presented an initial response with significantly higher stiffness and strength. The maximum strength of the fully infilled frame was about 4 times the maximum strength of the bare frame and about 2 times the one of the infilled frame with window. In what concerns the initial elastic stiffness, the fully infilled frame had a value of about 10 times the one of the bare frame and about 2 times the one of the infilled frame with window.

The high initial values of stiffness and strength observed in the infilled frames suffer an important decrease after the first hysteretic cycle. In the case of the model with window this decrease is such that the response after the third stage is coincident with the one of the bare frame model. This coincidence is also illustrated in the evolution of the dissipated energy.

The evolution of dissipated energy also shows that the bare frame was the only model that exhibited a considerable part of elastic response in the first cycles.
The above referred effects observed in the strength and stiffness evolution are also illustrated in Table 2 and depicted in Figure 6 where the envelopes of the hysteretic loops for the three models are presented. This figure shows clearly that the bare frame and the infilled frame with window have the same response after about 3% of drift and that the fully infilled frame presents a strength reserve of about 100% in relation to the other two models at about 6% of drift.

Damage Evaluation

Taking into account the above mentioned considerations, a tentative stiffness based damage index is proposed based on the following relation:

\[
D_K = 1 - \frac{K_s}{K_0}
\]

(1)

where \(K_s\) is the instantaneous secant stiffness of the hysteretic loops envelope and \(K_0\) the initial secant stiffness (yielding stiffness) obtained in the monotonic test of an identical bare frame model. The procedure to determine the value of \(K_0\) is schematised in Figure 7.

![Fig. 7 - Definition of the initial secant stiffness \(K_0\) for a bare frame model.](image1)

![Fig. 8 - Definition of the initial secant stiffness \(K_0\) for the bare frame model.](image2)
In Figure 8 the evolution of the proposed stiffness based damage index is shown for the three models. As it can be seen, for values over around 4% drift, corresponding to a damage index of about 0.6, the infilled frame with window and the bare frame present identical values of damage. On the contrary, for lower drifts, the evolution of damage for those two models indicates a more “brittle” behaviour of the infilled frame with window. In fact, for values of drift less than 3%, no damage occurred in the frame of this model.

A similar behaviour was observed in the fully infilled frame, which did not present any measurable damage for drifts below about 3%. However, from there on, the damage in this model increases rapidly up to values of 0.5 for drifts of 6%. For the fully infilled frame the damage index evolution over 3% of drift goes from about 0.05 to 0.5, much less than the values of about 0.45 to 0.75 observed in the other two models. This means that the purposed damage index is able to reflect the strength reserve of the structure relative to the bare frame. In fact all the strength degradation observed until such a drift is due to damage occurring in the infill panels.

8. CONCLUSIONS

The main conclusions that may be outlined from the present study are the following:

• Due to the type of cracking observed in the infills, similar hysteretic mechanisms can be assumed for the reinforced concrete frames of the three models after the maximum drift imposed in the tests.

• The infilled frame with window under drifts over about 3% presented a behaviour similar to the one of the bare frame.

• The fully infilled frame for drifts below 3% showed damage only in the masonry wall. For drifts from 3% up to 6% this models presents a considerable strength reserve.

• The proposed secant stiffness based damage index appears to be a satisfactory indicator of the state of damage of this type of structures, as it was able to reflect both the evolution of the degradation and the strength reserve of the models.

This paper presents only the results of the tests of three models from a set of nine, the results of the remaining are still under analysis. As part of a research program that is still in development, some of the tested models were retrofitted and are being tested under the same imposed displacement history.

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