Dynamic multi-body behaviour of historic masonry buildings models

D. D’Ayala & Y. Shi
Department of Architecture and Civil Engineering, University of Bath, Bath UK

C. Stammers
Department of Mechanical Engineering, University of Bath, Bath UK

ABSTRACT: This paper analyses a series of shaking-table tests on three 1/10-scale 3 dimensional dry masonry models of same plan area and different height and total mass. Sinusoidal waves varying in frequencies based on both constant amplitudes and constant accelerations are used as input. The aim is to gain better insight into the behaviour of masonry historic structures subjected to seismic action by using the approach of multi rigid body dynamics. The models are relatively simple. Yet a sufficient number of blocks is used to ensure that the fundamental interaction among parts of masonry walls connected in 3D to form buildings is replicated to reasonable accuracy. Results are presented in terms of relative displacement, frequency and energy content of the motion of selected instrumented block. Global results in terms of deformed shape and crack pattern of the walls are also discussed.

1 INTRODUCTION

Shaking table test on masonry structures are relatively few in literature. Their results are also highly dependent on specific material characteristics and hence of modest generic utility. As it is seen in the brief literature review below somewhat contrasting results can be often obtained.

A series of shaking-table tests on 24 ½-scale two-storey masonry models (Benedetti et al., 1998) assessed the importance of the original quality of construction and the significant increase from even simple strengthening on the lateral resistance. In pseudodynamic tests conducted by Paquette et al., 2004, a full-scale one-story unreinforced brick masonry specimen with a flexible wood diaphragm and a gap at one corner was excited with selected accelerogrammes. The influence of discontinuous corners is stated to be negligible during high intensity seismic excitation. Stable combination of rocking and sliding mechanisms was found under large deformations without significant strength degradation. The influence of opening ratios on local and global rocking has been studied by Yi et al., 2006 with quasi-static tests on full-scale two-story unreinforced masonry model. It was observed that a large initial stiffness resulted in high initial damage, with stiffness decreasing rapidly with the increase of lateral drift. On the other hand, Adams (Adams, 1996) stated the difficulty in direct comparison of the results from scaled model with those from the full-scale system. Particularly he highlights the gaps between behaviour of models in labs and of real buildings because of differences in loading rates and component characterizations with scaled materials.

Shaking-table tests on masonry models can be basically classified into two groups. The first group uses only a few units of blocks (Casolo, 2000; Lourenço & Ramos, 2004), which is easy to carry out and gives clear behaviour of single rigid block motion. However, as it cannot simulate the interaction among large numbers of units in real masonry walls, the results and derived theoretical models are of limited use for real structures. The second group uses large-scale models(Benedetti et al., 1998; Griffith et al., 2004; Paquette et al., 2004; Lourenço et al., 2005). In this case, as the requirement of reasonable stress state at joints makes very stiff components and large extra loads, these models actually deviate largely from the original structures in the geometrical forms, loading conditions, stiffness and hence the dynamic characteristics. Thus, results obtained require substantial post analysis and interpretation to provide extrapolation to real structure. However, both groups of tests have shown that failure takes place in regular patterns whose arrangements depend on structural and geometrical characteristics of the wall layout both at material and structural element or assembly level (Anderson, 1976; Hendry, 1973; West et al., 1977). This brief review explains the rational for a series of test on 1/10 scale models of brick blocks 3D wall assemblies set in
dry work. The aim is to gain better insight into the behaviour of masonry historic structures subjected to seismic action by using the approach of multi rigid body dynamics. The models are relatively simple. Yet a sufficient number of blocks is used to ensure that the fundamental interaction among parts of masonry walls connected in 3D to form buildings is replicated to reasonable accuracy. In the following sections the background to this set of tests and the test set up are first presented, then the results discussed and a simple control approach is proposed to provide a simple and effective modelling.

2 SHAKING-TABLE TESTS

2.1 Background

The series of shaking-table tests presented here is part of an experimental program to identify dynamic behaviour of masonry structures. Previous theoretic and experimental studies conducted by D’Ayala & Speranza; 2003 Restrepo et al., 2004 and Shi & D’Ayala, 2006; Shi et al., 2008, have shown how using limit state analysis and pseudo-static tests is possible to derive a consistent model of behaviour of cracking and damage of historic masonry subjected to lateral action and how the behaviour can be correlated to a relatively small numbers of geometric and structural parameters, without relying on stress analysis. While results of these studies are also confirmed by in situ observation of damage to buildings subjected to earthquakes, their static nature fails to provide insight in the damaging process and hence fails to accurately quantify the strength and “ductility” resources that are available during the hysteretic behaviour. It is argued here that notwithstanding the fact that the constituent materials of masonry, bricks or blocks and mortar, are not ductile, substantial dissipation of energy can take place during the damaging process at the cracks interfaces due to sliding and rocking of portions relative to each other, and hence via friction and impact.

As the processes outlined above are post material capacity, both in tension and compression or shear, it is not essential to replicate the continuum nature of the masonry. This is the main reason for adopting models in dry masonry so that the emphasis is directly on the post-elastic behaviour. It may be argued that the lack of mortarred joint would increase the dissipating capacity of the scale model with respect to a real masonry structure, because of the possibility of each unit to randomly move with respect to the adjacent one. This indeed does not occur, as verified by the tests, because the gravity load distribution is proportional to the real case and hence actual relative movement does indeed occur only along main lines of crack pattern. Neglecting the elastic phase has two advantages: the first is that in scaling down the model the considerations on stresses need not to be addressed, the second is that results can be more easily generalised to masonry structures with a variety of materials.

2.2 Experimental setup

Three series of models were built and tested on the Instron Structural Testing Multi-Axis Shaking Table (IST-MAST) system with 6 degrees of freedom, sited at the Mechanical Engineering laboratory of the University of Bath, UK. The control system used is the Labtronics 8800 Control Electronics with a Basic 8800 Console, control cards for each axis position and SCM data acquisition boards-table accelerometer monitoring and customer feedback. Models tested have rectangular plan shape so as to collect information on façades with different height to length ratio. Panels perpendicular to y axis have 7 bricks in length, while the façades perpendicular to the x axis have 5 bricks. They are built without windows and at corners the superposition of bricks is equal to half the length resulting in competent connections. The bricks have dimensions (L × W × H) 100 × 50 × 35 mm, and they are regularly staggered to provide $s/h = 50/35 = 1.4$, which from previous experimental and analytical studies (D’Ayala, 2005, Shi et al., 2008) has shown to provide good equivalent shear strength to the masonry.

Model dimensions are shown in Figure 1. The first and second series of models were built with 9 courses

![Figure 1. Plan (a) and elevation (b) of the three models.](image-url)
of bricks, while the third series had 15 courses to investigate the difference in height to length ratio further and to study the effect of different area to mass ratio. The roof structure is simulated by five timber beams set a regular spacing on the top course with weights bolted at each end to simulate the actual roof mass and to ensure proper simulation of the constraining effect of floor structures onto the wall. Of course the issue of rigid or flexible diaphragm action is not addressed at all here. However, to obtain a more even distribution of mass and constraint, especially over the corners, series II models have extra weight applied to the corners for a total 20% more with respect to series I models (Figure 2).

Besides the accelerograms registering the six component of motion of the shaking table, each specimen is instrumented with 6 displacement transducers, 3 for each of the x and y alignment, set on the central brick and the corner bricks on each side. As the bricks of the top layer are less stable without mortar, in order to represent the global shaking, displacements of the 2nd top layer are measured. Additionally, there is one more transducer in the middle of the 8th layer in each direction for Model III. Details of the transducers layout are shown in Figure 3.

Using a simple sinusoidal input with 50-cycle duration, each of the models is subjected to different group of tests where only the amplitude to the acceleration of the signal was increased, as summarised in Table 1. One of the objectives of the tests is to clarify whether for a same energy input greater levels of damage are triggered by acceleration or increased amplitude of the motion. A second objective is to identify whether the response is frequency sensitive, in other words, notwithstanding the fact that the specimen is not a continuum, whether something akin to natural frequency or range of frequency for which the response is enhanced, can be identified. Hence the input frequency varies following two principles: (1) under constant amplitude. This is used on Model I to identify resonant frequencies. (2) Under constant peak acceleration. To ensure this, as the input wave is defined in terms of frequency and amplitude, the relationship between frequency and amplitude of two successive input series is \((f_1)^2/(f_2)^2 = A_b/A_a\), where \(f\) is the input frequency and \(A\) is the corresponding amplitude. For Model II Series, the effect of input

Figure 2. (a) Series I and II and (b) Series III set up model setup.

Figure 3. Displacement transducers setup.

Table 1. Group classification of shaking-table tests.

<table>
<thead>
<tr>
<th>Group</th>
<th>No. of tests</th>
<th>Direction</th>
<th>Constant</th>
<th>Series</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8</td>
<td>y</td>
<td>Amplitude</td>
<td>I</td>
</tr>
<tr>
<td>2</td>
<td>5</td>
<td></td>
<td>Acceleration</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>5</td>
<td>x</td>
<td>Amplitude</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td></td>
<td>Acceleration</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>y</td>
<td>Acceleration</td>
<td>II</td>
</tr>
<tr>
<td>6</td>
<td>5</td>
<td>x</td>
<td>Acceleration</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>x, y</td>
<td>Acceleration</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>9</td>
<td>y</td>
<td>Amplitude</td>
<td>III</td>
</tr>
<tr>
<td>9</td>
<td>3</td>
<td>y</td>
<td>Acceleration</td>
<td></td>
</tr>
</tbody>
</table>
non parallel to the structure’s walls was also considered by providing sinewaves of equal amplitude and displacement in the two coordinate directions.

3 EXPERIMENTAL RESULTS ANALYSIS

Both pseudo-static tests. (Shi & D’Ayala, 2006; Shi et al., 2008) and extensive on-site post earthquake observation have shown that under lateral action, masonry walls are more vulnerable to out of plane failure, even for shape ratio $H/L < 1$, and slenderness ratio $H/t < 11$, as it is the case for this series of tests. This was confirmed by the observation of both the recorded videos of the shaking and the comparison of overall displacement of in-plane and out-of-plane instrument for any given motion test. However differently from the monotonic static tests carried out on single walls with wings, the inversion of motion caused by the sinusoidal wave results in a much stronger interaction at the corner between parallel and orthogonal walls with effects on the two sets of walls which will be further discussed in detail in the following subsection where results are presented separately for each series of tests.

3.1 Analysis of results on Series I models

In Figure 4 maximum amplification of motion and dissipated energy for series of test with increased frequency and constant amplitude of 12 mm and constant acceleration of $1.53 \text{ m/s}^2$ are shown for the two coordinate directions of motion. The y direction relates to the out of plane motion of the longer façade. This shows a peak amplification of 2 for 2.0 Hz, with a distribution very similar to an amplification spectrum. For the shaking in x, peak response is associated to an input frequency of 1.9 Hz. and a value of 1.5, with a second peak at 3.6 Hz. Assuming a simple 1 degree of freedom oscillator as basic model for the behaviour of the walls would yield values of stiffness 29.5 KN/m and 15.7 KN/m respectively, contrary to expectations.

This can be explained by the fact that the floor structure afforded better friction restraint in the y direction shaking, resulting in overall greater stiffness.

In the case of constant input acceleration, and hence constant input energy, it is observed that for increasing input frequency damage and collapse occur for a smaller number of cycles. This phenomenon is summarised clearly in Figure 4b, where the total energy dissipated in each test is measured against the value of input frequency.

Figures 5 and 6 show that similar portion of the walls are excited in both directions for each peak amplification case. In the photos the panel parallel to the motion are shown. The thick lines at the edge of the panels shows the portion of this that participate to the out of plane motion of the orthogonal walls while the thinner line shows the relative sliding of blocks in the direction of motion. It should be noted that these two crack patterns relates closely to the theoretical assumptions taken in D’Ayala & Speranza, 2003 to derive out of
plane mechanisms collapse load factors for façades. Particularly as already outlined there the crack closer to the corner will occur before the wing crack if the connection is not particularly strong, as it is the case here for lack of overweight or restraint from the floor structure directly at the corner. This lack of restraint causes a torsional motion of the end blocks of each side and this eventually influences greatly the overall out-of-plane movement of the wall. This is illustrated in more detail by the motion profiles shown for different frequency inputs and different cycles in the y and x direction respectively in Figures 7 and 8.

In each graph the recorded displacement of each of the six instruments is plotted for a positive peak, 0 point, negative peak and subsequently positive peak of the input, providing basically the “modal shape” of the two orthogonal walls.

Figure 7 shows that the long façade has relatively stable cycles, and although the response lags with respect to the input, maximum positive and negative displacement are comparable and the overall deformed shape is stable. It is also evident that there is modest coupling of shaking with the orthogonal wall.

When shaking occurs in the x direction instead, lagging is much more substantial, there is no overlapping of the two subsequent cycles, but the phenomenon which is perhaps most apparent is that the movement at the corner is of the same order of magnitude than the movement at the centre of the façade, providing a clear measure of the torsional effect introduced earlier. This also results in substantial coupling for the longer orthogonal wall.

The superposition of effects of corner rotation and shaking of the façade has also the effect of accelerating the progression of damage. Thus, rotation at corners is more likely to influence narrower panels, and contrary to what expected the excitation in the x direction causes collapse for smaller energy input.

The difference between input and output phase angles $\varphi$ are listed in Tables 2 and 3 for the shaking in x and y. The further the input frequency deviates from the natural frequency, the larger the response deformation lags behind. Higher flexibility of the facade also causes larger lagging phase. In Table 2, for
Table 3. Difference in input – output phase angles of Model I shaking in y direction.

<table>
<thead>
<tr>
<th>Frequency (Hz)</th>
<th>Input Amplitude (mm)</th>
<th>Lagging (s)</th>
<th>Phase difference (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>9.7</td>
<td>0.16</td>
<td>18.33°</td>
</tr>
<tr>
<td>2.0</td>
<td>12</td>
<td>0.2</td>
<td>22.92°</td>
</tr>
<tr>
<td>2.2</td>
<td>12</td>
<td>0.2</td>
<td>25.21°</td>
</tr>
<tr>
<td>2.5</td>
<td>6</td>
<td>0.16</td>
<td>22.92°</td>
</tr>
<tr>
<td>2.5</td>
<td>12</td>
<td>0.18</td>
<td>25.78°</td>
</tr>
</tbody>
</table>

When the maximum response amplitude exceeds the input amplitude the top starts drifting more and more while the oscillations reduce in amplitude. When the frequency is closer to the resonance the first oscillation has the greatest amplitude and then the subsequent ones are slightly smaller but associated with substantial drifting, although overall slightly smaller than the previous case.

### 3.2 Analysis of results of Series II models

In order to further study the effect of corner restraint and decoupling of the two orthogonal walls, extra weights were placed directly at the edge of each wall. This addition resulted in a slight increase in natural frequency from 2.0 Hz to about 2.1 Hz in y direction and from 1.9 Hz to 2.0 Hz in x direction with respect to the Series I models with a substantially smaller amplification in x, but greater in y. This confirms the decoupling of the two motions when the corners are prevented from rotating. While in the present experiment this was simply provided by increasing the frictional restraint by way of a more even distribution of applied mass, it clearly points out the necessity of ensuring the 3d integrity and box behaviour for masonry structures. As seen from comparing Figures 4b and 10b, the overall dissipation of energy in the Series II models is slightly lower, in agreement with the restrained effects at the corners. The previous observation is also confirmed by the deformed shape plotted in Figure 11a and 11b, showing, especially for the shaking in y direction, more regular and lower magnitude deformations.

Further investigation of the coupling effect and the influence of rotation at corners, was investigated by subjecting Series II models to forcing sinusoidal waves in the direction of their plan diagonal. Maximum displacement was set at 9 mm for a total constant acceleration of 1.26 m/s² and a variable range of frequency. In all cases instruments at midspan panels record amplifications substantially smaller than 1 showing that out of plane movement is modest while instrument on corners show significant drift, in agreement with substantial shear deformation of the overall panel. As a result the energy dissipated in out of plane movement is very small compared with the previous two cases (see Figure 10b).

### 3.3 Analysis of Series III models

The Series III models have a H/L ratio of 1 for the small panel and of 0.72 for the larger panel. Condition of constraint and loading are unchanged from previous series. However smaller input amplitudes have been chosen. Contrary to the increased geometric slenderness the natural frequency increases to 2.5 Hz for shaking in the y direction, showing that the increased...
height also results in increased lateral stiffness due to better frictional behaviour among the blocks.

The shaking of the façade has still an arch shape involving only the upper 8 layers of the façade and the 6 top of the side panels. Rotation at corners is still visible but substantially reduced compared with previous models. The participating mass associated to the shaking is proportionally reduced when compared with previous series.

Finally in order to compare the overall behaviour of series II and III models their adimensionalised drift for the two groups is plotted in figure 13. The unit top drift is calculated as follows:

Unit drift = (Displacement of top layer/Input amplitude)/Height

in order to compare shakings for different amplitudes but overall same energy input. It can be seen that while for the second series the value of drift increases with frequency, for the third series this reduces in the post natural frequency zone.

Figure 10. (a) amplification and (b) energy dissipation response with different input frequencies for Series II models for constant acceleration.

Figure 11. Deformed shapes of Model II.

Figure 12. Façade view of the model at the end of the 2.5 Hz 50 cycles motion.
4 CONCLUSIONS

In the previous sections the results of 3 series of tests for a total of over 50 geometry/amplitude/frequency combinations have been presented in terms of global response parameters. From these, it would appear that notwithstanding the discontinuous nature of the masonry fabric, having bricks in simple dry contact, a behaviour that can be described in terms of natural frequencies and resonance can be identified, supported by observations in terms of both amplification and energy dissipation. Constraint conditions at corners are also critical to the behaviour and while they do not seem to influence either the façade portion taking part in the shaking or substantially the natural frequency. However the amplification are substantially reduced especially away form resonance. For frequencies closer to the resonance level, the apparent greater stiffness of the model and the fact that it dissipates less by relative rotation at the corners, means greater overall out of plane deformation at the centre of the façade and hence greater damage in this area.

In summary the behaviour and hence the collapse limits are due to the superposition of an overall motion of the walls panel that can be reduced to beam oscillation horizontally and cantilever oscillation vertically; however to this is superimposed a relative sliding of bricks of subsequent courses and a rotation of some of them around a vertical axis, mainly caused by the staggering. This rotation is initiated at corners and propagates. No relevant rocking of individual bricks was observed. On the basis of these observations a multi-body dynamic model is being developed.

ACKNOWLEDGEMENTS

This project is developed with support from the Dorothy Hodgkin Award scheme and from the Royal Society international collaboration scheme. Thanks go to the technicians of the Structures Laboratory of the Department of Architecture and Civil Engineering and of the IST MAST laboratory in the Mechanical Engineering Department of the University of Bath.

REFERENCES