CHARACTERIZATION OF MASONRY BEHAVIOR UNDER HIGH STRAIN RATES

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Abstract. The study of the dynamic properties of materials has increased in recent times. It is well established that the strain rate influences most of the materials used in construction. This effect on materials such as concrete or steel has been intensively investigated. However, similar studies on masonry materials such as clay bricks cannot be found in the literature easily. Understanding the strain rate effect on masonry materials is important for proper modelling and design of masonry structures under high velocity impacts, such as masonry parapets in bridges, or historic masonry subjected to blast loads, due to deliberate or negligent actions. This work aims at studying the behavior of masonry and its components in compression at different strain rates. A Drop Weight Impact Machine is used on historic clay brick, mortar and masonry specimens at different heights and weights introducing different levels of strain rate, ranging from 4 s\(^{-1}\) to 199 s\(^{-1}\). The strain rate effect on the ultimate strength, Young’s modulus and strain at peak strength are determined from the experimental results. Empirical relations of dynamic increase factors (DIF) for these material properties are also presented. Finally, the study of a case study regarding the analysis of a historical structure in Portugal subjected to blast is presented and discussed for different scenarios.
1 INTRODUCTION

Different loading conditions might lead to different strain rates. Quasi-static loading produces strain rates of around $10^{-5}\text{ s}^{-1}$, while impacts and blast loading produce strain rates of well over 100 s$^{-1}$. When subjected to dynamic loading conditions, materials can have a much different behavior when compared with their static behavior [1, 2]. Most research work on structural response and damage under impact and blast loading assumes typically static material properties, e.g. [3]. This can lead to an inaccurate prediction of structural damage and fragmentation.

Very few studies can be found in the literature on masonry materials, such as clay bricks or mortar. Recently, Hao and Tarasov [4] conducted an experimental study under dynamic uniaxial compression using a Triaxial Static-Dynamic Testing Machine. These authors reported a DIF for the compressive strength of around 2.3 and 1.12 for the DIF of the ultimate strain, for a strain rate of 150 s$^{-1}$. For the Young’s modulus a DIF of 1.95 was reported for the same level of strain rate. Burnett et al. [5] presented results from dynamic tensile experiments on a mortar joint using a specially designed Split-Hopkinson pressure bar. Their results showed that there is a dynamic enhancement of the joint strength at strain rate of 1 s$^{-1}$, with a DIF of 3.1. Asprone et al. [6] studied this effect on a specific Italian stone using a Hydro-Pneumatic Machine and a modified Hopkinson bar for tensile tests. The tensile strength of this type of stone showed an increase of three times the static reference.

In this paper, an experimental campaign on the influence of the strain rate on the mechanical properties of masonry and its components is described. The tests were conducted with a Drop Weight (DW) tower available at the Mechanical Engineering Department in the University of Minho. This equipment consists of a “hammer” with a given mass being released at a chosen height. Authors like [7,8] have used this kind of testing apparatus to investigate the influence of the strain rate effect on different materials.

As an example of the relevance of considering the dynamic effects in the material properties, the Rossio train station in Lisbon is presented, which is one of the most used train stations in Portugal.

2 TESTING DETAILS

2.1 Equipment

A Drop Weight Impact Machine (DW) was used to perform the compression tests at different strain rates. This equipment is available in the Mechanical Engineering Department at the University of Minho (Figure 1). It allows drop heights up to 9 m and the weight of the hammer ranges from 40 kg to 150 kg. The load profile was measured at the base of the test specimen using a load cell specifically for dynamic applications (Figure 1). The data acquisition system limits the sampling speed to 200 kS/s (200 samples per millisecond), which was found to be enough even at a later stage where 5 channels where used at the same time, allowing an acquisition frequency of 40 kHz per channel. The deformation behavior of the specimen was measured in two different ways. First, a FastCam video camera was used, with a maximum frame rate of 250 000 frames per second. This equipment allowed the visualization of the test in slow motion and the measuring of the strain in only one face of the test specimen. This strain measurement was possible using targets in the specimen at a specific location and performing a tracking sweep of those targets in the video. With the relative position of the targets, the strain at each instance was calculated. The second methodology used to obtain the strain time history was using strain gauges placed in each face of the specimen. In the quasi-
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static tests, strain gauges were used to measure the strains in the specimen. In the dynamic tests, the FastCam was used to measure the strains, for cost-efficiency reasons. In order to validate and compare the results, some dynamic tests were also performed using strain gauges.

2.2 Specimens

When testing at high strain rates, using dynamic loading, it is difficult to avoid resonances and inertial effects. Specimens are often kept small in order to retain the assumption of stress equilibrium within the deforming specimen. Lateral friction may also provide some confinement at the two ends of the specimen under compression tests. Therefore, the dimensions of specimens must be a compromise between maximizing the size to ensure a complete representation of the material, proper height to base width (or thickness) ratio to reduce the friction effects at the ends, and minimizing the sample size to reduce the effect of inertia and non-uniform stress and strain in the specimen.

With the objective of reproducing old Portuguese masonry construction, solid handmade bricks were used. The brick measured 20x10x5 cm in dimensions and the test specimen (Figure 2) measured 7x3x3 cm. From each brick, five specimens were prepared. The test specimens were cut from the original brick by means of a disk cutting machine. After cutting the specimens, their edges were ensured to be intact and the loadbearing surfaces at both ends of the specimen were ground flat and parallel to each other. A commercial ready-mix mortar was used and the test specimens’ dimensions for the mortar were kept the same as the brick specimens, in order not to adapt the testing rig. After the mixture was placed in the molds (Figure 2), these were placed in a climatic chamber at 25°C and 65% humidity for five days. After these five days, the specimens were taken out of their molds and tested in that same day. The compressive strength of mortar was intended to be similar to the values of old mortars and, after five days of curing the compressive strength of this mortar is about 3 MPa.

The masonry specimens were composed of four clay bricks and three mortar joints with one centimeter thickness, arranged in a stacked pattern. Test setup limitations lead to the final dimensions of 23x8x8 cm for the masonry specimens. Again, each brick was cut to match the required dimensions. The specimens were built in a flat surface, but because the brick faces on top and bottom are irregular, on top of the specimens a layer of self-levelling mortar was used (Figure 2). The treatment regarding the brick units used in the masonry specimens were the same as the specimens for the brick specimens, and the time of curing for the masonry specimens was the same as the mortar specimens.
3 TEST RESULTS

3.1 Quasi-static regime

In order to have the quasi-static reference for comparison with the results subjected to the dynamic regime, an experimental campaign on the behavior of these materials under quasi-static uniaxial compression was performed. The mechanical properties under study were: a) compressive strength ($\sigma_{\text{max}}$); b) strain at peak strength ($\epsilon_u$); c) Young’s modulus ($E$); and d) compressive fracture energy ($G_c$). Figure 3 shows the typical stress-strain relation and stress-displacement relation with information on how to determine these mechanical properties. The compressive strength is the maximum value of the stress-strain curve. At that point the corresponding strain is the strain at peak strength. The Young’s modulus is calculated as the slope of the stress-strain diagram between 30% and 80% of the maximum stress. The compressive fracture energy is determined from the stress-displacement curve and corresponds to the marked area in Figure 3. This procedure, to calculate the post-peak fracture energy in compression has been used before by [9].

A total of 5 (five) specimens of handmade clay brick, 9 (nine) specimens of mortar and 4 (four) specimens of masonry were tested under quasi-static uniaxial compression and the results are presented in Table 1. The value between brackets is the coefficient of variation (CoV). The brick specimens have the highest compressive strength with 13.59 MPa, whereas the mortar has strength of 4.46 MPa. This resulted in masonry specimens with 7.94 MPa of compressive strength. Also important, the Young’s modulus is 2.32 GPa for the brick specimens and 0.80 GPa for the masonry specimens. The fracture energy in compression is similar for brick and mortar specimens, with 1.56 N/mm and 1.43 N/mm respectively, while for ma-
sonry the fracture energy is much larger, reaching 7.64 N/mm. The reason for this is that the masonry specimens showed higher deformation capacity due to the interaction between the masonry components. The CoV for the handmade clay brick under quasi-static compression is slightly higher than the values found in the other tests performed. This is a material made by hand, where the quality control and the manufacturing procedure is not as high as those from industrialized materials.

Table 1: Reference (quasi-static) mechanical properties, with CoV inside brackets.

<table>
<thead>
<tr>
<th></th>
<th>Brick</th>
<th>Mortar</th>
<th>Masonry</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{\text{max}}$ [MPa]</td>
<td>13.59 (14%)</td>
<td>4.46 (10%)</td>
<td>7.94 (9%)</td>
</tr>
<tr>
<td>$\varepsilon_u$ [mm/m]</td>
<td>6.95 (12%)</td>
<td>6.36 (6%)</td>
<td>10.93 (15%)</td>
</tr>
<tr>
<td>$E$ [GPa]</td>
<td>2.32 (20%)</td>
<td>1.06 (11%)</td>
<td>0.80 (14%)</td>
</tr>
<tr>
<td>$G_c$ [N/mm]</td>
<td>1.56 (31%)</td>
<td>1.43 (9%)</td>
<td>7.64 (3%)</td>
</tr>
</tbody>
</table>

3.2 Dynamic regime

For the dynamic regime testing, several impact tests under uniaxial compression were performed. The objective is to obtain the stress-strain relations from the data recorded at the load cell acquisition and the fastcam video. To facilitate the treatment of the data obtained, the stress-time curve was approximated to a second degree polynomial, while the strain-time curve was approximated to a linear function. As a result, the final shape of the stress-strain curves is a second degree polynomial, as shown in Figure 4. With the stress-strain relations for each test, the mechanical properties could be determined and the dynamic increase factor (DIF) could be calculated as a function of the strain rate, given by

$$DIF = \frac{\text{Property (Dynamic)}}{\text{Property (Static)}} \cdot f(\dot{\varepsilon}) \quad (1)$$

These results were used to establish the relations between the mechanical properties and the strain rate. These relations are usually described as bi-log-linear relations, meaning that they can be written with two log-linear functions, low slope for the quasi-static regime and high slope for the dynamic regime. In order to simplify these relations, the first log-linear function for the quasi-static regime was considered constant and set as DIF equals to 1 (one) until the point where the regime changes to dynamic.

![Figure 4: Examples of final stress-strain curves at different strain rates (clay brick shown).](image)
A total of 58 specimens of handmade clay brick were successfully tested under uniaxial compression, with strain rate ranging from $4 \text{ s}^{-1}$ to $199 \text{ s}^{-1}$. Figure 5 shows a typical image sequence of the impact test recorded with the fast video equipment. Five additional tests using strain gauges confirmed the results obtained with the targets and video equipment. These five tests actually represent 20 equivalent tests, meaning that strain gauges were placed in all four faces of the test specimen and the average was taken for each test. In the video equipment, only one face of the test specimen was measured, meaning that rotation of the specimen is not captured and the single strain value has to be considered carefully. Figure 6 shows the relation between the dynamic increase factor and the strain rate for the compressive strength of clay brick. As expected the strain rate influences the compressive strength of this material and the dynamic increase factor for a strain rate of $200 \text{ s}^{-1}$ is 2.5, meaning that the compressive strength is two and a half times the static value at that strain rate. The log-linear trend-line has a coefficient of determination $R^2$ higher than 70%, which for this material was considered good. The results obtained with the strain gauges also showed a good agreement with the results from the targets. Comparing the results with [4], who tested clay brick with different equipment, there is also a good agreement.

Similar results were obtained the Young’s modulus, the strain at peak strength and the compressive fracture energy of clay brick. Figure 7 shows a summary of the results obtained for the handmade clay brick specimens. As it can be seen, the strain at peak strength appears
to be the least affected by the strain rates. The compressive strength and the Young’s modulus have similar behavior, which is consistent with the fact that the strain at peak strength is lightly affected. The fracture energy has a high increase with the strain rate, more than twice the increase of the compressive strength, meaning that the post-peak behavior of this material is greatly influenced by the increase of strain rate. The best-fitted equations of DIF as a function of strain rate were the following:

![Graph showing typical results for DIF for different mechanical properties (clay brick shown).](image)

At a strain rate of about 200 s\(^{-1}\) the DIF regarding the compressive strength for brick, mortar and masonry are 2.54, 4.13 and 2.17 respectively. The results show that the behavior of the masonry specimens is similar to the behavior of the clay brick. For the mortar specimens this influence is more pronounced with a DIF for mortar of around double of the brick. This difference could be explained with the existence of synthetic fibers in the mortar composition. These observations are similar for the Young’s modulus. For a strain rate of 200 s\(^{-1}\) the DIF for Young’s modulus for brick, mortar and masonry are 2.43, 3.02 and 2.15 respectively. At a strain rate of around 200 s\(^{-1}\) the DIF regarding the strain at peak strength for brick, mortar and masonry are 1.31, 1.11 and 1.26 respectively. Regarding the compressive fracture energy, the results obtained for mortar and masonry are closer with DIF of 2.73 and 3.10 respectively, for a strain rate of 200 s\(^{-1}\). The DIF for brick at the same strain rate is 5.95. These results suggest that the post-peak behavior of the handmade clay brick is more influenced by the strain rate than mortar. One possible explanation is related to the porosity of both materials. The porosity of the clay bricks used (23 %) is higher than the porosity of the mortar (9 %).

4 PROTECTING CRITICAL INFRASTRUCTURE: A CASE STUDY

Rossio station was selected for this study. Formerly known as Central Station, this building was design between 1886 and 1887 by the Portuguese architect José Luís Monteiro. This building is classified since 1971 as Property of Public Interest. The three-story building is constructed in limestone stonework. The building “L” shape is a high value element in the Public Transportation Operator Network, not only because of its effect on public opinion but due to its very high passenger flow.

Two different scenarios (Figure 8) were studied in this analysis: (a) Scenario A – corresponds to an explosion at a square on the South side of the building (at 4 meters from the
building). The luggage size Improvised Explosive Device (IED) (about 20 kg TNT) was the selected delivery system for this scenario; (b) Scenario B – corresponds to an explosion at the East façade of the building (at 5 meters from the center of the façade). The vehicle size IED (about 1500 kg TNT) was the selected delivery system for this scenario. Another situation was analyzed – Scenario B’ – where the access to vehicles up until 25 meters from the East façade was closed. The same delivery system (1500 kg TNT) would still be possible but only at 25 meters from the center of the façade.

![Image: Different explosion scenarios: scenario A on the South side and scenario B on the East side.](image)

**Figure 8: Different explosion scenarios: scenario A on the South side and scenario B on the East side.**

### 4.1 FEM Model

The FEM model was built in the ABAQUS software, where the Explicit solver was used. The walls were modelled as shell elements and the columns were modelled as beam elements. The walls are discretized with quadrilateral 4 nodes (S4R) and 3 nodes (S3R) shell elements. These are three-dimensional, iso-parametric, doubly curved thin or thick shell element. These elements have five degrees of freedom at each node, reduced integration, hourglass control, and finite membrane strain. The columns are discretized with 2-node linear beam elements (B31). The final mesh has 27968 nodes and 24491 elements.

The material model adopted is de CDP (Concrete Damaged Plasticity) model available in ABAQUS software and it is a modification of the Drucker-Prager model. The mechanical properties for the masonry were collected from literature with DIFs from UFC-3-340-02. Due to the lack of information about the connections to the floors, two different situations were considered: a) neglecting the contribution of the pavements, meaning that the masonry panels are only constrained at ground level and at the connections with the other panels; and b) considering a generic pavement assuming perfect connections to the walls, introducing intermediate constrains at the masonry panels.

In order to keep this problem as a pure Lagrangian formulation, the blast loading was defined as pressure profiles. Knowing the position and the weight, in TNT equivalent, it is possible to estimate the pressure profile acting on a specific surface, using the different scenarios detailed above.

### 4.2 Numerical results

For a blast analysis it is necessary to define a damage criterion that can be applied to categorize the damage on the masonry panels. The criteria defined by UFC-3-340-02 was applied, meaning that the support rotations will be checked in order categorize the damage on the masonry panels.

Scenario A corresponds to an explosion at a square on the South side of the building. It is a place with possible high concentration of people due to the presence of outdoor cafes. An explosion with 20 kg TNT at 4 meters from a surface will create a reflected pressure of around
1.5 MPa with duration of 1.4 ms. The adopted methodology for applying the load resulted in eleven different pressure profiles applied to the masonry panels according to the distance from the explosion center. The reflection angle was considered constant at 90° for all panels. For the present analysis, the initial instant corresponds to the moment when the blast wave first touches the building. Scenario A was studied with material properties labelled as UFC-3-340-02 and considering the contribution of pavements. This scenario represents a low impact loading in the structure, and in order to easily see the results, only part of the structure (the closer part to the explosion), is presented (Figure 9). The panel on the left, which is closest to the explosion, is the first to be loaded. Then the blast wave reaches the panel on the right. At this time, the first panel is already unloaded and it is still moving due to the structure inertial force. Although the structure has small displacements, the loading is enough to reach the nonlinear behavior of the masonry. There is a concentration of plastic strains on the right side of the panel on the left. It is possible to have in that area some cracking, although it should be negligible. Still, this level of loading is very low for this structure. The closest panel to the explosion has a maximum displacement of around 2.75 mm, keeping a 1.5 mm permanent displacement after the loading. The analysis of the support rotations shows that these are still far away from the failure criteria.

![Figure 9: Scenario A: time history of displacements (left); damaged area for t = 40 ms (right).](image)

Scenario B corresponds to an explosion at the East façade of the building, at 5 meters from the center of the façade. Thus is a close-range large blast and it will generate very high strain rates in the masonry. Both situations regarding the pavements (neglecting and considering its contribution) were considered and the results were compared. Only the first 30 ms of analysis are presented here, as it enough to reach the collapse of the structure considering the damage criteria defined. Figure 10 shows that the global response of the structure changes if we neglect or consider the contribution of pavements. In the first case, the East facade panel behaves as one large masonry panel being supported at ground level and on its side edges. In the second case, considering the contribution of the pavements, the East façade behaves with intermediate supports along its height, similar to three “independent panels”. Due to the dimensions of these panels (very long) it is almost as if they were only supported at the bottom and at the top. The load resulting from this explosion is quite high and the structure response is quite fast. In the first 30 ms the masonry reaches a velocity of around 10 m/s resulting in around 300 mm of maximum displacement, see Figure 11. Analyzing the support rotations, it is clear that, in both situations, the masonry panel rotates beyond the non-reusable state defined in UFC-3-340-02. At this point, it was considered that this part of the structure would have collapsed.

Scenario B’ corresponds to an explosion at the East façade of the building, at 25 meters from the center of the façade. This simulates the possibility of closing to traffic the road right
in front of this façade and the application of bollards preventing vehicles to get closer to the building. In this case the support rotations are kept under the reusable limit established by UFC-3-340-02, for both possibilities of the floors.

Figure 10: Damaged area for scenario B (t = 30 ms): without floors (left); with floors (right).

Figure 11: Time history of displacements for scenario B: without floors (left); with floors (right).

5 CONCLUSIONS

Understanding the strain rate effect on masonry materials is important for proper modelling and design of masonry structures under high velocity impacts or blast loads. A large experimental campaign was performed on different loading regimes and different materials. Almost 250 impact tests were performed during this research and more than 60 handmade clay brick were cut in order to prepare the specimens. Masonry specimens and its components, clay brick and mortar, were tested under quasi-static regime – strain rate of $10^{-5}$ s$^{-1}$ – and dynamic regime with strain rates ranging from 2 s$^{-1}$ up to 200 s$^{-1}$. It was found that the mechanical properties of these materials increase with the increase in strain rate, having DIFs ranging from 2 to 6 for a strain rate of 200 s$^{-1}$. Equations that represent the empirical relations obtained were proposed and they can be used to estimate the response of this kind of materials under different strain rates.

Rossio train station in Portugal was modelled using explicit non-linear dynamics and the results were presented for different explosion scenarios. It was shown that a small package explosion would have a small impact on the structure while a large package explosion would lead to the collapse of the structure. Increasing the standoff distance, as a measure for mitigating the impact of the explosion, was analyzed and proven to be an effective measure.
REFERENCES


