DAMAGE ACCUMULATION IN UNREINFORCED MASONRY DUE TO SEISMIC LOADING

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ABSTRACT

Masonry has low cohesive strength and is susceptible to the brittle failure, when subjected to induced tensile stresses. It fails due to lateral tensile stresses even when masonry prisms are tested in uni-axial static compression and the stress-strain curve remains almost linear up to failure. Although masonry normally resists compressive forces, under lateral in-plane cyclic loads some parts of the volume of masonry walls are subjected to tensile stresses, which are sufficient to cause localized brittle failure. However, localized brittle failure does not always lead to a collapse of the masonry wall. The wall after cracking often retains some capacity to deform under the applied load (ductility).

This phenomenon shows that masonry is intrinsically neither brittle nor a ductile material. The phenomenological characterization of failure for materials such as masonry is conditional and depends on: (a) the state of stress, (b) the strain rate or loading rate, (c) the temperature, and (d) the amount of accumulated micro-structural damage. Some of these issues for masonry have been addressed; others have not been adequately addressed to this day. This paper reports results of the experimental study on accumulation of the microstructural damage in masonry under seismic loading.

1. INTRODUCTION
Masonry buildings degrade and fail when subjected to extreme environmental loads such as earthquakes, and yet still can last for two millennia if they are built well. Post earthquake reconnaissance surveys have shown that masonry shear walls can degrade, crack, rock, slide, and collapse when these walls have been subjected to some forms of dynamic loading. The degradation of the shear walls reduces the stiffness of the masonry elements and affects the ability of the structure to withstand additional cyclic loading. This research has investigated the degradation of the masonry shear walls up to the point of cracking. The elements of rocking, sliding, and collapse are not part of the study. Dynamic loading arises from a variety of sources. The interest in this research lies in the seismic range of amplitudes and frequencies. This set of frequencies is typically within the range of 0 to 10 Hz. Baker (1912) noted that the tensile strength of masonry was strain rate dependent when he reported on tests on mortar. Subsequent experimental research supported the known degradation of masonry shear walls that occurs in an earthquake or windstorm, and showed that the measured stiffness of masonry had a frequency effect (Tercelj, Sheppard, and Turnsek, 1969; Paulson and Abrams, 1990; Klopp, 1996). The first observation from their previous research was of a difference in the measured stiffness resulting from the frequency of loading. These experimental observations have been attributed to a distinct static stiffness and a distinct dynamic stiffness (Freund, 1990). The second observation was of the subsequent degradation of the material with time of loading and increasing strain in a dynamic experiment (Paulson and Abrams, 1990; Krajcinovic, 1996). This paper presents the findings of the research on the progressive degradation of masonry shear walls. The progressive degradation has been investigated for the time dependent in plane loading of the shear walls. The amplitudes
and frequencies have been varied systematically to investigate the properties of the degrading material as the amplitude increases and the frequencies followed a preset pattern.

A test rig was constructed that could apply a two part stress field to a single Wythe masonry panel. The stress field comprised non-proportional bi-axial compression and harmonic shear. A Finite Element analysis showed that a time dependent harmonic shear forcing function used for the experimental loading, could comprise a systematically changing set of single amplitude – frequency tests to match an increasing seismic loading on a shear wall. Ten masonry panels of standard size pressed clay bricks from Sydney, Australia and a one cement: one lime: six sand mortar were tested to failure in the rig. A mathematical fragment has been developed to encapsulate the changes in the stiffness properties of masonry from the static to the dynamic loading and onto the failure of the material at high strains. The fragment has the primitive of the invariant intrinsic elastic constants from the theory of damage mechanics, the derivation of an effective stiffness function, $k_2$, that has strain as the independent variable, and the introduction of the measured stiffness function, $\dot{\varepsilon}_2$, that has been defined as the measured stress field divided by the measured strain field. The measured stiffness function has strain, strain rate and frequency of loading as independent variables. The strain, strain rate and frequency are related by the wave amplitudes.

The measured stiffness function was related to the effective stiffness function with the inequality $k_2 \geq \dot{\varepsilon}_2$, which was supported by the results for the masonry panels. The effective stiffness was recoverable from the measured stiffness for a frequency of the forcing function less than 0.4 Hz. The measured stiffness function was shown to explain the observations of the dynamic stiffness because of the mass and damping terms in Newton’s Second Equation. The damage parameter, $D$, was encapsulated from the effective stiffness data and tri-linear curve recommended for engineering design purposes.

The analytic part presents a discussion of the progressive degradation of the masonry panels based on the data measured from the in plane testing of the panels and the analysis of the results. The critical factors influencing the degradation of the masonry stiffness are identified and discussed.
2. AIMS AND METHODS

Masonry is a general brittle material that is poorly defined using plastic concepts. Ceramic materials, which include pressed masonry shear walls, have an observationally documented distinct static stiffness and distinct dynamic stiffness. The literature survey has shown that a number of alternative definitions and equations have been postulated and experimentally encapsulated to account for the two observed material strengths. The two material strengths have been observed to be dependent on the frequency and the rate of loading. Irrespective of the observations, the properties of the materials such as masonry must be understood in terms of the fundamental principles of elasticity, dynamic motion and the field of damage mechanics. A low strain rate loading test on masonry that was monotonically increasing in force to failure shows a linear relationship between the stress tensor and the strain tensor (Nichols and Totoev, 1997a). This research on the degradation of masonry shear walls was based on the assumptions of Hooke’s law, using an orthotropic model. The orthotropic model, whilst being a continuum, provided a simple method to account for the differing Young’s moduli that were established as normal and parallel to the bed joint. The model retained the shear modulus as a linear equation. The level of orthotropicity was mild and the results can be expressed as isotropic for design purposes. Newton’s second law established the defining differential equation of motion. The analysis focused on the change in the stiffness of the material as it was loading under varying harmonic conditions. Krajcinovic (1996) presented the theory of damage mechanics for a general brittle material. The damage parameter was introduced to provide an internal state
variable that represented the accumulation of damage in the sample. The change in the effective stiffness against strain has traditionally been used as the determinant of the damage model. The experimental study presented in this thesis undertook to establish an estimate of the damage parameter for a masonry shear wall that was subjected to harmonic loads. The broad research aims are:

• To explain and confirm the various experimental observations of the effective and dynamic (or measured) stiffness.
• To investigate the mathematics of the differential equation of motion of the system to satisfactorily explain the various observed stiffness operators.
• To provide a set of numerical values for the damage parameterization that is related to the total strain in a specimen for the loading pattern.

3. THE CONCEPT OF EFFECTIVE AND MEASURED STIFFNESS

Damage mechanics is used to study the process of the degradation of masonry. This study of the degrading stiffness properties of masonry panels subjected to harmonic loads was based on a primitive assumption that an elastic stiffness constant forms an upper bound to an equivalent effective stiffness and that the effective stiffness essentially monotonically decreases with loading (Krajcinovic, 1996). The fundamental definition of the stiffness, $k$, from the physical sciences has only the displacement term as a component. The effective stiffness, $k(t)$, in damage mechanics implicitly carries a time–strain component. The differential equation of motion based on the concept of an effective stiffness that varies with time is:

$$\frac{d^2}{dt^2} F(t)x(t)k + \frac{d}{dt} F(t)x(t)h + dx(t)m = 0$$

The analysis completed by Benedetti and Pezzoli (1996) succinctly demonstrates the alternative definitions that can be provided for the effective stiffness. The differential equation can be transformed using a change of axes to eliminate the constant force term, by assuming that the
forcing function is strictly harmonic with a constant amplitude, and by making the assumption of zero damping. As a practical starting point, this reduces equation (1) to:

\[ \sin(\omega t)(2\pi At \times mw t)k \hat{U} = - (2) \]

A further simplification reduces equation (2) to the form:

\[ ()(2) = (3) \]

Where \( \cdot_2(\tau) \) is defined as the measured stiffness that has the displacement, velocity, and acceleration as linearly independent components, and has the frequency term present in the forcing function. The following inequality derived from equation (2) holds for the seismic range of frequencies:

\[ ()(2) = (4) \]

Equation (4) demonstrates that two operators are determinable from the defining differential equation. The two operators are nominally termed stiffness, however this is only a practical name for the \( \cdot_2(\tau) \) term and is provided purely to retain compliance with the existing engineering literature. Thus, the effective stiffness is represented by \( k_2(\tau) \) and the measured stiffness is represented by \( \cdot_2(\tau) \).

The derivation of the effective stiffness and the measured stiffness had a number of theoretical constraints on the damping and the amplitude. The use of non-zero damping does not change the form of equation (4) because of the limits to and the value for the damping terms are constrained by the principles of entropy and the phase lag. The increasing change in frequency will affect the size of the mass and damping terms proportionally greater than the effective stiffness terms. The practical result is that equation (4) is valid for all harmonic loading cases. The last theoretical aspect is that a Fourier decomposition of a cyclic wave pattern will derive harmonic wave patterns, thus equation (4) will hold for each Fourier wave and thus for the summation of
the waves. There are limitations, but the restrictions to a seismic range of frequencies provide practical constraints that remain within the reasonable Fourier definition range.
The impact of the inequality shown in equation (4) is to introduce a piecewise continuous form to the measured stiffness which will not have a continuous first derivative.
This matter is demonstrated with an abrupt step in the frequency at low loading levels. This is an essential feature of the measured stiffness, which is evident at a change in frequency. Hence this step, introduces as a minimum, a discontinuity in the first derivative. This step occurs without a break in the manifold, only requiring a change in the frequency of the testing. This result implies that the first derivative of the measured stiffness may not exist between all data results points.
The definition of the effective stiffness is predicated on the mathematical concept of a manifold. The first derivative of the function is assumed to exist, as the function within reasonable limits is smooth. The breaking of the manifold, which is evident with the cracking of the masonry panel, negates the boundary conditions required for the solution of the differential equation to be applicable. This breaking of the manifold provides the practical limit to the measurement of the effective stiffness. It also demonstrates that the measured stiffness is not a true effective stiffness. The problem of crack sliding under these broken manifold conditions has been outlined in Krajcinovic (1996). This aspect was not investigated in this research.
The measured stiffness has been shown to approach the effective stiffness as the frequency of the loading function approaches zero. This theoretical limit was estimated assuming zero damping and a 1% systematic error as being acceptable between the measured stiffness and the effective
stiffness. The theoretical limit for the cut off for a quasistatic frequency was established as 0.8 Hz. A pragmatic frequency cut off limit based on damping evident in the test data is 0.3 to 0.4 Hz.

4. MEASURED SHEAR STIFFNESS RESULTS

The measured stiffness results for single frequency testing on Panel AP: 3 are shown in Figure 1.

5. EFFECTIVE SHEAR STIFFNESS RESULTS

Results were obtained for eight panels. The effective shear stiffness was established from the measured stiffness data points using points with a frequency of forcing function that was less than 0.4 Hz. A typical set of results and the error in the results for panel AP: 7 is shown in Table 1.

Table 1. Typical Set of Effective Shear Stiffness Results

<table>
<thead>
<tr>
<th>Strain (µå)</th>
<th>Effective Shear Stiffness (GPa)</th>
<th>Standard Deviation in the Effective Stiffness</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>8.466</td>
<td>0.261</td>
</tr>
<tr>
<td>60</td>
<td>7.172</td>
<td>0.269</td>
</tr>
<tr>
<td>75</td>
<td>6.694</td>
<td>0.056</td>
</tr>
<tr>
<td>100</td>
<td>6.237</td>
<td>0.088</td>
</tr>
<tr>
<td>150</td>
<td>4.929</td>
<td>0.250</td>
</tr>
<tr>
<td>200</td>
<td>4.064</td>
<td>0.325</td>
</tr>
<tr>
<td>400</td>
<td>Insufficient number of tests to compute</td>
<td></td>
</tr>
</tbody>
</table>

The results show the degrading value of the effective stiffness with increasing strain levels.

6. DAMAGE PARAMETERIZATION

The results of the damage parameterization can be used for engineering design purposes to quantify the change in the effective stiffness in masonry subjected to applied strains. Krajcinovic (1996) establishes the definition for the damage parameter. The parameter, D, is related to the effective...
shear stiffness, $E$, by equation (5).

\[ 10.1 \times 10^{-6} = D \]

The critical damage parameter, $D_c$, is defined at the break of the manifold structure. This point represents the cracking of the masonry panel into two separate pieces. The damage parameterization results for the eight panels are shown in Figure 2.

Figure 2. Damage Parameterization Results for the Clay Masonry Panels

The trend in the data is evident in Figure 2. A linear regression analysis provided a straight line fit to the data. The regression equation has an intercept of zero and a slope of 0.0018. The regression coefficient is 0.5787. This error level is acceptable given the scatter in the data points. There is sufficient information in the results for the damage parameterization to provide a review of the alternative models that are available to encapsulate the data. The three models are all linear, with the first a standard regression,
the second bi-linear, and the third tri-linear. The estimated critical points for the three models for the damage parameterization are presented in Table 2.

Table 2. Model Values for the Damage Parameter

<table>
<thead>
<tr>
<th>Strain Level (µå)</th>
<th>Linear Model</th>
<th>Bi-linear Model</th>
<th>Tri-linear Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>25</td>
<td>0.045</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>200</td>
<td>0.36</td>
<td>0.33</td>
<td>0.5</td>
</tr>
<tr>
<td>400</td>
<td>0.72</td>
<td>0.7</td>
<td>0.7</td>
</tr>
</tbody>
</table>

The essential differences in the models relate to the zero point and the value of the damage parameter at a strain level of 200 µå. The linear model offers computational speed, whereas the bi-linear model underestimates the typical range of the damage parameter at 200 µå. The 200 µå level appears to be the critical level at which cracking is initiated in the manifold. The complete breaking of the manifold occurs at 400 µå. The theory of a single manifold breaks down at 400 µå. The tri-linear and linear models are shown in Figure 3. The additional damage parameter term, \( D_{ci} \), has been introduced to identify the onset of cracking in the manifold.

The results demonstrate the loss of stiffness in the masonry when it is subjected to increasing cyclic loading generating a slowly increasing peak tensile principal stress. Three potential design curves have been established from the results. The linear curve is computationally faster to
implement, but not as close to the mean results curve which
is better represented by the tri-linear form. The tri-
linear results are recommended values.

7. OUTCOMES
The major experimental objective was to observe the change in
properties of shear walls subjected to specific loading and
boundary conditions. The measured stress field and strain
field were determined and recorded for a series of masonry
panels. The stress field was determined from the forcing
function at the ITM. The strain field was determined across a
uniformly strained section of the masonry panel. The
loading pattern was systematically varied to investigate
different aspects of the measured stiffness. The purpose was to
provide sufficient data for the statistical analysis for the
effective stiffness and to demonstrate that the hypotheses
were supported by the experimental results. As for all
Newtonian physics, the observation data supports the use of
Hooke’s law within the field of damage mechanics, however the
normal limitations of the Newtonian physics of real materials
apply to these results. Three hypotheses were proposed that
relate the measured stiffness to the effective stiffness. The
experimental data, the observed failure mechanism, and the
analysis of the results confirm that the:
I. Three hypotheses are reasonable for pressed masonry.
II. Measured stiffness corresponds to the dynamic strength concepts.
III. Effective stiffness can be established from the measured stiffness under
some limiting conditions.
IV. Equation (4) holds for the seismic range of frequencies.
V. Measured stiffness has frequency dependence.
VI. Effective stiffness is a monotonically decreasing function dependent on the
strain level and not the frequency of loading.

8. CONCLUSIONS
The principal conclusions are that:
I. The pressed clay masonry behaves as a general brittle material, with a damage
parameter that has a critical value of approximately 0.5 at a first principal
tensile strain of 200µå for the start of the cracking and a value of 0.7 at 400µå
for the failure of the manifold.
II. The static elastic modulus of pressed clay masonry can be used to establish the
starting point for the application of the damage parameter results established in
this research.
III. The minimum characteristic strength for masonry applicable with these results
is 0.2 MPa when tested in accordance with the procedures in AS3700. The site
experience shows that a bond strength less than this level provides an extremely
weak masonry wall that thus fails to conform to reasonable site practice.
IV. An effective limit between quasi-static loading and dynamic loading rates can
be established at a frequency of 0.3 to 0.4 Hz. This applies to amplitude of
stroke no greater than 5 millimeters.
V. For pressed masonry, a homogenized isotropic model provides a reasonable
method for applying these results.
VI. Static and dynamic forces with the same peak amplitude are not equivalent
loads and the principal of equivalent energy levels needs to be applied.
VII. The results provide a source of data to develop fragility curves for masonry
buildings and for considering the impact of frequency matching of buildings
and seismic traces. This issue of frequency matching is a problem in larger
seismic events such as the Nahanni event where the broad band of energy will
still supply resonant frequencies as the building degrades and the natural
frequency response changes.
VIII. The use of a damage parameter method in place of the linear elastic
assumptions used in current design standards is a reasonable design step. The
analysis of simple structures usually does not warrant this level of complexity,
but the introduction of the method into finite element codes for complex
masonry structures is recommended.
9. REFERENCES
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