[Title of Document]: DEGRADATION OF PRESSED CLAY MASONRY SHEAR WALLS SUBJECT TO HARMONIC LOADING

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DEGRADATION OF PRESSED CLAY MASONRY SHEAR WALLS
SUBJECTED TO HARMONIC LOADING

John M. Nichols ¹ and Yuri Z. Totoev ²

ABSTRACT

Un-reinforced pressed clay masonry shear walls are studied in the context of damage mechanics. Un-reinforced masonry walls are commonly used in intraplate tectonic regions to provide shear stiffness to buildings. This experimental study investigated the changing properties of the effective stiffness of series of masonry walls. A systematically increasing harmonic in-plane strain field was applied to the walls using a uni-axial ram concurrent with the application of a non-proportional bi-axial constant compression stress field. The change in the effective shear stiffness was established from the measured stress and strain fields. The effective shear stiffness decreases to approximately one half of the shear modulus at the point of failure of the single masonry manifold. This result matches a critical damage parameter of one half, which is usually observed in studies on brittle materials. The strain data shows that the change in the effective stiffness is a linear function of the peak amplitude of the strain and independent of the frequency. This data provide a method for establishing the damage levels in masonry that has been subjected to earthquakes and for estimating effective stiffness values for design.

1. INTRODUCTION

Repeated field observations after earthquakes have shown that masonry shear walls degrade and may cause the collapse of structures when subjected to a dynamic loading (Page, 1992). These observations lead to a need to understand the properties of the

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effective stiffness of the masonry as it degrades. Classical continuum mechanics assumes that the Young’s modulus for a material such as masonry is constant to the point of failure. The theory of damage mechanics accepts this hypothesis, but modifies the result by assuming that the damage parameter, D, can be defined and measured for an isotropic material. The scalar representation of the damage parameter is:

\[ D = \frac{E - \bar{E}}{E} \]

where E is Young’s modulus and \( \bar{E} \) is the effective stiffness at some strain level that is non zero. Kachanov, et al.\(^3\) suggested that ‘the second order tensor representation of damage satisfies all rational criteria of accuracy for the approximation of the effective properties.’ The second order tensor is a matrix representation.

Heyman (1995) presented the definitive text on historic and load bearing masonry. Epperson and Abrams (1989) make the point that the older a building is, the greater the probability that it will encounter a major environmental event and that most of these events are a lateral loading. This type of loading is a harsh test of a gravitational non-ductile structure, such as historic masonry of any age. Unreinforced masonry is a widely used construction material for housing, flats, and commercial premises across large areas of the world. This method of construction is common in those parts of the world identified as intraplate zones. The designation of intraplate covers the interior of the tectonic plates. This regional definition is now generally considered as being 2 degrees from the plate boundary (Wysession et al. 1995).

Page (1973) identifies the structural action of the shear wall as the principal element in moving from historic to structural masonry. A shear wall is dependent on a high moment of inertia about one axis, high-compressive strength brickwork, and some tensile capacity, preferably augmented with a compressive stress generated by dead load from higher storeys. This type of element has created the problem of slenderness about the other principal axis, which requires careful design to avoid out-of-plane failure even if reserve strength exists in plane. These structures rely on shear walls or connected elements on all three orthonormal planes to tie the shell together. This method meets the
requirements of sound building practice coupled with the assumption of some tensile capacity (AS3700) in the masonry walls, which is used to resist transient dynamic loads.

Lourenço and Rots (1997) have identified the need to establish micro-mechanical models and obtain experimental data for masonry that are accurate beyond the structural pre-peak regime. Part of this work is then to obtain a mathematical model for masonry and to undertake the necessary numerical modelling. This model should allow for the damage accumulation in the material and the change in effective stiffness properties with time.

Experimental research, which supports the known degradation of the masonry under cyclic loading such as occurs in an earthquake or windstorm, has shown that the measured stiffness of masonry has a frequency effect (Tercelj, Sheppard, and Turnsek, 1969; Paulson and Abrams, 1990; Klopp, 1996). This research has been confirmed in a number of shear wall and shaking table experiments undertaken in the last three decades. The first observation, from this previous research, is of the initial difference in the measured stiffness, resulting from the frequency of loading. The second coupled experimental observation is of the subsequent degradation of the material with time of loading in a dynamic experiment (Paulson and Abrams, 1990; Krajcinovic, 1996). These experimental observations have been attributed to a distinct static stiffness and a distinct dynamic stiffness. Often this experimental work is based on a particular time series such as the Irpinia earthquake (Benedetti and Pezzoli, 1996). This use of a pseudo-random cyclic event can obscure the mathematical features of interest, although experimentally this problem can be ameliorated using simple harmonic loading functions.

The experiments, completed as part of this research, investigate the degradation of shear-wall elements that are subjected to a varying loading pattern. The in plane, loading
pattern involves static, non-proportional, compression stress, and a harmonic, shear stress. The research work uses a set of frequencies for the harmonic, shear stress that spans from an equivalent static through the seismic range of frequencies. The effective stiffness properties are determined using shear strain developed under controlled displacement.

LITERATURE REVIEW

A review of the mathematics of stiffness and the existing experimental data established a relationship between the various stiffness definitions. These definitions, which were adopted for this thesis, are the elastic modulus, the effective stiffness, and the measured stiffness. Each elastic modulus is an invariant intrinsic property. An effective stiffness is the degraded stiffness that has a basis that only includes the displacement term. A measured stiffness has a component basis that includes the acceleration, damping, and displacement terms.

The stress field and the strain field were used to determine the measured stiffness. A Fast Fourier transform analysis of the measured stiffness results recovered the changes in the loading pattern as causing the observed differences in these results. Two conclusions are reached from this analysis. The first conclusion is that a measured stiffness represents a sound approximation to the effective stiffness, provided the harmonic frequency of the loading term was less than 0.3 to 0.4 Hertz. The second observation is that the elastic modulus as an upper bound is recoverable from the time-series data. The stress measurements and the measured stiffness are constrained by inertial effects at higher frequency, which is directly attributable to the basis for the measurements.
The changes in the effective stiffness with time and frequency of loading were then investigated for each shear wall element.

The mathematical concept that underpins the numerical analysis is that of a Fourier series. In this sense, the definition of numerical analysis is the traditional pure mathematics term. We are seeking to understand the mathematical concepts that explain the experimental observations, not discussing the very limited field of computer simulation and modelling that is often considered the field of numerical analysis. The Fourier series uses a set of harmonic functions to provide an equivalent loading to a cyclic pattern. The discrete Fourier transform, which is a powerful analytical tool, permits the transformation of a time-domain wave into a frequency-domain representation (……).

The differential equation of motion for a harmonic forcing function has a basis with three components. These terms are acceleration, velocity, and displacement. This basis, coupled with a harmonic forcing function, provides the interesting feature of one being able to define several relationships between the forcing function and the displacement. These relationships are generically termed stiffness, and their meaning depends on the underlying theoretical or experimental assumptions. Accepting the concept of the two distinct forms of defined stiffness, a separate definition has been provided for each. The dynamic stiffness is in reality a mathematical operator. The use of the term stiffness to describe this operator is so this dissertation uses the accepted engineering terms without introducing secondary concepts that obscure the results (Nichols, 2000)
Each definition is dependent on the varying components of the basis. Two simple stiffness definitions and an upper bound primitive for the elastic constants provide the set of mathematical tools for solving the problem of the decay of masonry under high strains. These definitions are consistent with the principles of damage mechanics, the physics of springs, and the published results of previous experimental work on masonry.

These tools are not new; however, this paper provides a formal relationship between the various stiffness definitions. The primary definitions are for a measured stiffness and an effective stiffness. The measured stiffness is deemed the experimentally determined stiffness that is obtained by dividing the stress field results by the strain field results. The basis for this stiffness is the set of terms including the acceleration, velocity, and displacement. This basis directly implies frequency dependence, because of the direct frequency component in the velocity and acceleration and the indirect frequency component in the harmonic term controlling the displacement. The effective stiffness is deemed the mathematical functional representation of the stiffness defined explicitly by the displacement term of the differential equation of motion for a harmonic function. This functional representation has no frequency dependence (Kaplan and Lewis, 1971) in a strict static test. The standard relationship where the effective stiffness and the measured stiffness are identical functions is the generalized Hooke’s law (Lekhnitskii, 1963), when applied using a static forcing function. This functional relationship is achieved with the types of experimental method used by Page (1979). The types of experiments conducted on shear walls or oscillators at seismic frequencies yield measured stiffness data sets that have been shown to be frequency dependent (Bolotin, 1964, 13; Tercelj, Sheppard, and Turnsek, 1969; Baker, 1912).
Richter (1958) provides details of the estimated amplitudes that are the expected surface movements in a moderately strong earthquake. This data is presented in Table 1.

**Table 1 Harmonic Oscillation Frequencies (Hertz)**

<table>
<thead>
<tr>
<th>Acceleration ( a ) (g)</th>
<th>Amplitude of Surface Movement (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.0001</td>
</tr>
<tr>
<td>1</td>
<td>500</td>
</tr>
<tr>
<td>0.1</td>
<td>160</td>
</tr>
<tr>
<td>0.01</td>
<td>50</td>
</tr>
</tbody>
</table>

He stated that 'in this table the shaded numerals indicate the combinations of amplitude and acceleration, the corresponding frequencies, most common in the analysis of moderately strong ground motion.'

Macchi’s paper outlines the principles of a diamond shaped rig that is capable of measuring the constitutive properties of a material. The paper discusses the alternative test rigs including shear and racking tests. He explains why the diamond shaped shear test provides the only method able to arrive at an estimate of the constitutive properties.

Magnes and Calvi provided a thorough review of the seismic analysis of masonry walls in existing dwellings. Their work on the damage mechanic and the effective stiffness is interesting and provides confirmation of the starting points for this research work.

Kachanov, *et al.*\(^4\) suggested that ‘the second order tensor representation of damage satisfies all rational criteria of accuracy for the approximation of the effective properties.’ The second order tensor is a matrix representation. The theory of damage mechanics is predicated on the assumption that the intrinsic elastic constants are invariant properties, measured using tensile rather than compression tests. The effective stiffness is

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then noted to change as the material fails or degrades. This evolution is quantified using the damage parameter D. The scalar representation of the damage parameter is:

\[
D = \frac{E - \bar{E}}{E}
\]

This equation (1) is a simplistic representation of the reality of any material; nevertheless, it demonstrates the normalization procedure implicit in the concept of a damage tensor. Krajcinovic presents Kachanov’s concept for an integrity tensor. It can be viewed as mathematically simpler, but conceptually not as satisfying as the current accepted definition for the damage parameter. The damage parameter \( D_p \) is zero at the commencement of the experiment and its value is given by the change in the effective stiffness \( \bar{E} \) relative to the Young’s modulus \( E \). This representation is simply a method for comparing two materials with different Young’s modulus. The theoretical maximum value for \( D \) is one; however, a critical value of 0.5 is generally reached at the point of fracture of the manifold or the sample.\(^5\)

The two failure mechanisms have variously been termed static and a dynamic mechanism. The common observation is that masonry is initially stiffer when subjected to a dynamic loading when compared to an equivalent static loading. There are two essential features to these observations from the experimental work. Theses observations are:

**Observation 1:** The specimen is initially stiffer.\(^6\) This observation implies that there is a degradation of the stiffness with time.

\(^5\) The point of view as to a manifold implies a mathematical model of the failure and the view of a sample implies the engineering view of a failure of a sample.

\(^6\) Op. cit., Page, A.W., (1979), Abstract and Chapter 1. As Page noted the effective stiffness is dependent upon the loading history.
Observation 2: Given a dynamic loading that is considered equivalent to a static loading, as in the experiments of Paulson and Abrams. As these two loading patterns are applied to the specimens, they result in an experimentally measured higher initial stiffness for the dynamic loading in comparison to the static loading. This observation implies that equivalent force loadings may not generate equivalent energy and momentum solutions, which are related to the differential equation that governs the motion and strains of the specimens.

The hypothesis is that an alternative method of absorbing energy is creating the observations that have been termed, for the want of a better name, the dynamic stiffness. This hypothesis then leads directly to the acceptance of the view that there is not necessarily a change in the true effective stiffness even if there is a change in the quantity termed the dynamic stiffness. Of course, this does not preclude the view that the effective stiffness is changing, but its rate of change does not have to be at the same rate of change as the dynamic measurements. The experimental work has to be structured to identify these mechanisms and to quantify the separate results so that a satisfactory and mathematically sound explanation is provided for the experimental observations. This conclusion now permits the development of a theory to explain the observations within the constraints of the mathematics of the differential equation of motion and the measurement of the true effective stiffness.

The various research studies provide the basis for establishing a number of definitions of the various physical entities that are discussed and measured in this dissertation. The measured stiffness \( \lambda^2(t) \) is deemed an experimental stiffness that is obtained by dividing

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the measured stress field results by the measured strain field results. The effective stiffness $k^2(t)$ is deemed the mathematical functional representation of the stiffness defined explicitly by the displacement term of the differential equation of motion for a harmonic function. This functional representation has no frequency dependence (Kaplan and Lewis, 1971, 554). The function measures the degradation of the stiffness with level of damage using equation Error! Reference source not found.. The relationship where the effective stiffness and the measured stiffness are identical functions is the generalized Hooke’s Law (Lekhnitskii 1963, 8). The concept of the invariant elastic stiffness constants was explained in § Error! Reference source not found. Error! Reference source not found.. These definitions are established in. The relationship in Error! Reference source not found. that was postulated between the effective stiffness $k^2(t)$ and the measured stiffness $\lambda^2(t)$ was:

$$k^2(t) > \lambda^2(t)$$

This study is to establish an estimate of the damage parameter for a masonry shear wall subjected to harmonic loads. The research aims are:

I. To explain and confirm the various experimental observations of the effective and measured stiffness.

II. To investigate the mathematics of the differential equation of motion of the system to explain satisfactorily the various observed stiffness operators.

III. 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.

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8 Refer pg. Error! Bookmark not defined.
The bricks selected for the detailed experiments are an Austral pressed brick of Sydney, Australia origin. Their commercial name is *Chocolate Mottled Brown*. A photograph of the brick units is shown in Figure 1. This photograph shows brick units in the lower left quadrant. A fully completed panel and a partially completed panel demonstrate the method of construction of the panels. The brick has a shallow frog.
Figure 1 Austral Mottled Chocolate Brown Bricks

Sugo (2000) has extensively tested these bricks and documented a range of their properties. The measured properties for these bricks are presented in Table 2.

Table 2 Brick Details

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Detail</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manufacturer</td>
<td></td>
<td>Austral</td>
</tr>
<tr>
<td>Name</td>
<td></td>
<td>Chocolate Mottled Brown</td>
</tr>
<tr>
<td>Type</td>
<td></td>
<td>Pressed</td>
</tr>
<tr>
<td>Frog</td>
<td></td>
<td>Shallow Prismoidal</td>
</tr>
<tr>
<td>Length Range</td>
<td>mm</td>
<td>225-230</td>
</tr>
<tr>
<td>Breadth Range</td>
<td>mm</td>
<td>108-111</td>
</tr>
<tr>
<td>Depth Range</td>
<td>mm</td>
<td>74-76</td>
</tr>
<tr>
<td>Initial Rate of Absorption</td>
<td>kg/m²/min</td>
<td>5.6⁹</td>
</tr>
<tr>
<td>Weight</td>
<td>kg</td>
<td>3.8</td>
</tr>
</tbody>
</table>

The mortar used in the construction of the shear wall element is a Portland cement: lime:sand mix that is 1:1:6 by volume. No additives were used in the mix.

⁹ This is quite a high value for a research brick.
The panel dimensions for design purposes were set at 1200 millimetres in both of the principal axes. This size of panel uses 70 bricks per panel, allowing for a 10-mm joint thickness. The joints are not tooled, but simply finished flush. This mortar has a mean compressive stress of $2.2 \pm 0.2$ MPa.

Table 3 Day Series of Prism Flexural Strength Results

<table>
<thead>
<tr>
<th>Description</th>
<th>245</th>
<th>246</th>
<th>329</th>
<th>331</th>
<th>77</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean Flexural Strength:</td>
<td>0.85</td>
<td>0.29</td>
<td>0.37</td>
<td>0.62</td>
<td>0.42</td>
<td>MPa</td>
</tr>
<tr>
<td>Standard Deviation:</td>
<td>0.16</td>
<td>0.06</td>
<td>0.11</td>
<td>0.11</td>
<td>0.07</td>
<td>MPa</td>
</tr>
<tr>
<td>Coefficient of Variation:</td>
<td>19</td>
<td>20</td>
<td>29</td>
<td>18</td>
<td>16</td>
<td>%</td>
</tr>
<tr>
<td>Characteristic Strength Factor K:</td>
<td>0.83</td>
<td>0.82</td>
<td>0.75</td>
<td>0.84</td>
<td>0.85</td>
<td></td>
</tr>
<tr>
<td>Characteristic Flexural Strength:</td>
<td>0.51</td>
<td>0.16</td>
<td>0.17</td>
<td>0.44</td>
<td>0.20</td>
<td>MPa</td>
</tr>
</tbody>
</table>

The results highlight the range of values that can be obtained even under strict quality control for masonry. The difference between Series 245 and 246 is explained by pre-wetting the bricks before laying the Series 246. The weaker walls proved extremely difficult to move and test particularly those that tested in the range of 0.16 to 0.2 MPa flexural strength in the prism test results.
The load application system consists of two separate and distinct mechanisms. The first mechanism is a compression frame designed to supply quasi-uniform pressure to the masonry panel. The mechanism supplies in-plane, bi-axial, non-proportional, compression stress to reflect the loading on a shear wall from the dead load of a structure. The second mechanism is a shear yoke that is designed to apply a harmonic shear stress to the edge of the panels. The shear yoke fits between a 50 mm base-plate to anchor the
shear yoke to the strong floor and the 250 kN Instron hydraulic ram. This ram provides the harmonic, shear stress that is applied to the masonry panel. This loading replicates the seismic loading applied to a building, which in turn loads the shear wall element.

Table 4 Features of the Pattern

<table>
<thead>
<tr>
<th>Feature</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>The pattern commences with a prestress level that ensures that the sample and the rig are maintained in a tensile state. The amplitude is measured about the prestress level to provide the necessary translation of axes for the solution.</td>
</tr>
<tr>
<td>2</td>
<td>The amplitude is held constant whilst the frequency is varied systematically. The frequency sweeps through a range from 0.06 Hz to 10 Hz.</td>
</tr>
<tr>
<td>3</td>
<td>The amplitude is increased by a constant amount and the group of frequencies repeated. This is a simplistic representation to outline the features of the pattern; it does not represent every incremental step of the test pattern.</td>
</tr>
</tbody>
</table>

The specific pattern structure used for AP: 6 is presented in Table 5.

Table 5 Panel AP: 6 Specific Pattern Details

<table>
<thead>
<tr>
<th>Amplitude (mm)</th>
<th>Frequency (Hz)</th>
<th>Replicates</th>
<th>Count</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.54</td>
<td>0.06: 0.1: 0.3: 1.0: 5.0: 10.0</td>
<td>15</td>
<td>90</td>
</tr>
<tr>
<td>3.81</td>
<td>0.06: 0.1: 0.3: 1.0: 5.0: 10.0</td>
<td>180</td>
<td></td>
</tr>
<tr>
<td>5.08</td>
<td>0.06: 0.1: 0.3: 1.0: 5.0: 10.0</td>
<td>270</td>
<td></td>
</tr>
<tr>
<td>6.35</td>
<td>0.06: 0.1: 0.3: 1.0: 5.0: 10.0</td>
<td>360</td>
<td></td>
</tr>
<tr>
<td>7.62</td>
<td>0.06: 0.1: 0.3: 1.0: 5.0: 10.0</td>
<td>450</td>
<td></td>
</tr>
<tr>
<td>8.89</td>
<td>0.06: 0.1: 0.3: 1.0: 5.0: 10.0</td>
<td>540</td>
<td></td>
</tr>
<tr>
<td>10.16+</td>
<td>Varies</td>
<td>640</td>
<td></td>
</tr>
</tbody>
</table>

The development of a sound statistical program requires the evolution of the pattern once an effect has been quantified and measured. The evolution of the pattern then used a repeating of the frequencies at constant amplitude for Panel AP: 9. The revised pattern is presented in Table 6.

Table 6 Panel AP: 9 Pattern Developments

<table>
<thead>
<tr>
<th>Amplitude (mm)</th>
<th>Frequency (Hz)</th>
<th>Replicates</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.54</td>
<td>0.06: 0.1: 0.3: 1.0: 5.0: 10.0</td>
<td>15</td>
<td>90</td>
</tr>
<tr>
<td>3.81</td>
<td>0.06: 0.1: 0.3: 1.0: 5.0: 10.0</td>
<td>180</td>
<td></td>
</tr>
<tr>
<td>5.08</td>
<td>0.06: 0.1: 0.3: 1.0: 5.0: 10.0</td>
<td>270</td>
<td></td>
</tr>
<tr>
<td>6.35</td>
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<td>360</td>
<td></td>
</tr>
<tr>
<td>7.62</td>
<td>0.06: 0.1: 0.3: 1.0: 5.0: 10.0</td>
<td>450</td>
<td></td>
</tr>
<tr>
<td>8.89</td>
<td>0.06: 0.1: 0.3: 1.0: 5.0: 10.0</td>
<td>540</td>
<td></td>
</tr>
<tr>
<td>10.16+</td>
<td>Varies</td>
<td>640</td>
<td></td>
</tr>
</tbody>
</table>

10 The amplitudes are nominal values. The Instron is calibrated in 1/10 of an inch and the dial gauge has a systematic error of +10 percent.
The purpose of this change is to investigate the effect of frequency changes at constant amplitude where the frequencies are broken into two blocks and repeated. This test used five replicates per amplitude frequency point.

The development of the loading pattern looked to establish a rational basis for the selection of the forcing function. The finite element and Fast Fourier transform analysis, which is detailed in Error! Reference source not found., demonstrates that a harmonic function provides an appropriate function for the experimental work. The essential issues with this study are the effects on the effective stiffness of the frequency of the forcing function and the strain rate, which is quantified with the amplitude of the forcing function. These two variables are changed in a systematic manner to establish graphical results for the change in the measured stiffness with time. In this case, it is pseudo-time based on a thirty-second standard block.

**Table 7 Applied Static Pressures to Panels**

<table>
<thead>
<tr>
<th>Shear Wall Element</th>
<th>Vertical Pressure MPa</th>
<th>Horizontal Pressure MPa</th>
<th>Ratio of the Pressures</th>
</tr>
</thead>
<tbody>
<tr>
<td>AP: 3</td>
<td>2.8</td>
<td>1.5</td>
<td>1.9</td>
</tr>
<tr>
<td>AP: 4</td>
<td>2</td>
<td>1.1</td>
<td>1.8</td>
</tr>
<tr>
<td>AP: 5</td>
<td>1.8</td>
<td>0.9</td>
<td>2</td>
</tr>
<tr>
<td>AP: 6</td>
<td>1.8</td>
<td>1.2</td>
<td>1.5</td>
</tr>
<tr>
<td>AP: 7</td>
<td>1.8 (1.9)(^{1})</td>
<td>1.2</td>
<td>1.5</td>
</tr>
<tr>
<td>AP: 8</td>
<td>1.9</td>
<td>1.3</td>
<td>1.5</td>
</tr>
<tr>
<td>AP: 9</td>
<td>1.9</td>
<td>1.3</td>
<td>1.5</td>
</tr>
<tr>
<td>AP: 10</td>
<td>2.7</td>
<td>1.2</td>
<td>2.25</td>
</tr>
</tbody>
</table>

\(^{1}\) The E series of tests.
A few experiments on panel AP: 3 were completed at a pre-stress level of 175 kN. The value of 150 kN was adopted, after testing on panel AP: 3, as it is mid range in the nominal force limits of the Instron. The peak force of 250 kN when resolved and translated to a shear is equivalent to about 1.3 MPa. This limits the maximum compressive stress, so that the first principal stress is driven slowly into tension during the incremental stepping of the harmonic loading. The signals that were available from the Instron were the displacement at the Instron and the force applied at the load cell location. It should be noted that the reaction frame is not equivalent to a fixed point and the force and displacement at the Instron are the forcing function values, but these do not relate directly to the force associated with the effective stiffness values of Hooke’s law.

The structure of this chapter provides a summary of the results, a typical failure mechanism, and the results for the measured shear stiffness for the eight panels. The establishment of the effective stiffness for each panel is then related to the particular loading pattern and the measured shear stiffness results. The features of the Damage Parameter D are then determined from the measurements of the effective stiffness and are related to the change in strain in the panels.
The failure pattern, which was observed in the masonry panels, was the pattern expected from a simple Mohr’s circle analysis. The failure plane always initiated at a point and progressed across the panel element.

A typical failure pattern is shown in Error! Reference source not found.. This panel cracked starting at the point A and progressing over about 10 tests to the point circled x. At this point, the cracking was arrested and the testing continued for about twenty tests before the crack extended further.

A similar pattern of cracking failure was observed in all tests. The crack, in this panel, was initiated in the mortar joint near point A. There was no systematic evidence that the boltholes initiated the cracking. However, the drilling on a mortar joint may contribute to the crack initiation.
Panel AP: 10 was tested at an age of 218 days, it has a prism flexural strength of $0.37 \pm 0.11$ MPa. The mean pre-stress force was 150 kN and the number of tests to failure was approximately 800. The test set-up and the detailed results are presented in Appendix Error! Reference source not found.. The frequency pattern was the same as for Panel AP: 9 up until test number 640. A set of tests at frequencies of one and ten Hertz was

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12 Refer pg. Error! Bookmark not defined..
used after test number 640 until the panel failed. The results for the peak strain at each test are shown in Figure 2.

**Error! Not a valid link.**

**Figure 2 Panel AP: 10 Strain for each test**

The interesting feature is the increase in the strain level at low amplitudes for all frequencies that is evident up until test 200-250. The graph of the measured shear stiffness is shown in Figure 3.

**Error! Not a valid link.**

**Figure 3 Panel AP: 10 Measured Shear Stiffness**

The duplicate sets of frequency sweeps at one-amplitude are shown in Figure 3. The bounding curve for the effective stiffness is evident for the range of the tests. The peak effective stiffness was 7.7 GPa.

The results shown in **Error! Reference source not found.** represent the full set of 640 tests. At an initial viewing the interpretation of the information contained in the graph is obscured because of the systematic variation in the frequency at each amplitude point. The measured shear stiffness for a single amplitude point represented between tests numbered 271 to 361 is shown in Figure 4. Each point is the mean of fifteen replicates.

**Error! Not a valid link.**

**Figure 4 Panel AP: 6 Tests at a Single Amplitude**

The critical element to this plot is the observation that the mean effective shear stiffness from test 270 and test number 361 can be determined from **Error! Reference source not found.**
These tests represent the first test at a frequency of 0.06 Hertz, which is a quasi-static loading rate. The results have an effective shear stiffness $k^2(t)$ of about 8.4 at test 271 and of about 7.5 at test 361. The conclusion is reached that in this series of tests from 271 to 361 that the effective stiffness is degrading. There is however no indication of a break in the manifold. The results plotted in Figure 4 have a relationship between the measured shear stiffness $\lambda^2(t)$ and the frequency $f$ that is shown in

\[ \lambda^2(t) = 8.4 - 1.5f + 0.086f^2 \]

The resultant fitted equation has a regression co-efficient of $R^2 = 0.988$. The frequency term clearly dominates the shape of the measured shear stiffness from test point 270 to 360. The parabola minimum occurs at 9 Hertz. This is a reasonable estimate given the sparsity of the points to define the parabola. This pattern is repeated in each of the 90 block test points for constant amplitude set. The results of the plot shown in Figure 4 can be compared to the results shown in Error! Reference source not found. for Panel AP: 3. The typical results shown in panel AP: 3 were tested at a constant frequency of one Hertz. These results for panel AP: 3 for comparable test points match the observation in the loss of the effective stiffness $k^2(t)$ of about 8.4 at test 270 and of about 7.5 at test 361 for panel AP: 6. This result is entirely consistent with the results for Panel AP: 4 and the principle of a smooth change in the effective stiffness demonstrated in Panel AP: 5. The results for the measured shear stiffness that are shown in Figure 4 are clearly dominated by the frequency of the loading pattern. The reasonable comments that can be

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\[13\] Refer pg. Error! Bookmark not defined.
made from this graph and the associated equation (3) are presented in Error! Reference source not found.

). Equation Error! Reference source not found. can be substituted into equation Error! Reference source not found. to yield equation (4).

\[(4) \quad k^2(t)x(t) - mw^2 x(t) = A \sin \omega t\]

The equation (4) can be simplified to equation (5).

\[(5) \quad (k^2(t) - mw^2) x(t) = A \sin \omega t\]

and with further simplification\(^{14}\) using equations Error! Reference source not found. and (7) to

\[(6) \quad F(t) = \lambda^2(t)x(t)\]
\[(7) \quad \lambda^2(t) = k^2(t) - m\omega^2\]

It is a simple self-evident step then to the inequality that

\[(8) \quad k^2(t) \geq \lambda^2(t)\]

1.1.1 Comments
This observational data demonstrates on these panels the difference in the response of this masonry to a dynamic load, when compared to the static load. These results confirm that the measured stiffness has frequency dependence. The confirming element in this analysis is the standard statistical method using the Fast Fourier transforms, which is

\(^{14}\) One can make the conceptual leap that the forcing function can be other than a harmonic function
illustrated with the results from Panel AP: 6. There is no other satisfactory or physically sound explanation for this result.

The theoretical relationship between the effective stiffness and the measured stiffness was presented in Error! Reference source not found..\textsuperscript{15} A standard statistical analysis using Fast Fourier transforms has demonstrated that the change in the frequency present in the loading pattern is the sole contributor to the variation between the effective stiffness and the measured stiffness. These detailed results are presented in Appendix Error! Reference source not found..\textsuperscript{16} The relationship is confirmed as:

\begin{equation}
(9) \quad k^2(t) \geq \lambda^2(t)
\end{equation}

The effective stiffness $k^2(t)$ has been shown to have a theoretical error of less than one percent at 0.8 Hertz for zero damping when compared to the measured stiffness $\lambda^2(t)$ and a practical static limit for non-zero damping from these eight panels test results of 0.4 Hertz. The limit has been identified with the symbol $\Psi$.

The damage parameter $D$ has been estimated using this frequency limit of $\Psi = 0.4$ for the eight panels. A typical set of results is presented for Panel AP: 7 in Table 9.

<table>
<thead>
<tr>
<th>Strain (micro-strain)</th>
<th>Effective Shear Stiffness</th>
<th>Standard Deviation in Effective Shear Stiffness</th>
<th>Damage Parameter D</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>8.466</td>
<td>0.261</td>
<td>0.00</td>
</tr>
<tr>
<td>60</td>
<td>7.172</td>
<td>0.269</td>
<td>0.15</td>
</tr>
<tr>
<td>75</td>
<td>6.694</td>
<td>0.056</td>
<td>0.21</td>
</tr>
<tr>
<td>100</td>
<td>6.237</td>
<td>0.088</td>
<td>0.26</td>
</tr>
<tr>
<td>150</td>
<td>4.929</td>
<td>0.250</td>
<td>0.42</td>
</tr>
</tbody>
</table>

\textsuperscript{15} Refer pg. Error! Bookmark not defined.
\textsuperscript{16} Refer pg. Error! Bookmark not defined.
The results for the mean values for the effective stiffness for the tests with a frequency less than $\Psi = 0.4$ is presented with the standard deviation. The Damage Parameter is then estimated from these results using equation Error! Reference source not found.. A summary of the calculated results for the damage parameter for the eight panels is presented in Figure 5.

Figure 5 Summary of the Damage Parameter Results

The results are shown for each panel in Figure 6.

Figure 6 Damage Parameter Results for each Panel

The encapsulation of the damage parameter from this information is presented in Error! Reference source not found.. The two interesting sets of results are for Panel AP: 5 and Panel AP: 8.

Panel AP: 5 failed in a static test, rather than a dynamic tests as for the other panels. This panel, which was constructed on day 331, had a high value for the shear stiffness. This type of variation is common in masonry, but is not considered typical for design purposes. The results are interesting but not considered to be statistically significant for these test purposes.

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17 Insufficient tests are available to compute a statistically significant value.
18 Refer pg. Error! Bookmark not defined..
Panel AP: 8 failed in a dynamic test with an amplitude of 6.3 mm and a frequency of 0.06 Hertz. The stiffness declined from seven to five GPa at the point of failure. The strain in this panel showed two significant jumps firstly to 200 micro-strains, again at a low frequency and then the final jump to 350 micro-strains. This panel was tested with a time of 90 seconds. These results show a panel degrading at a slower rate than the remaining panels.

The remaining results show a clear trend in the data at zero strain, 200 micro-strains, and 400 micro-strains. The 200 micro-strain mark appears to point to the initiation of the development of the cracking in the panel. The 400 micro-strain mark represents failure of the panel with the creation of two manifolds. The theory of elasticity of a manifold fails at this point.

The results show that even at reasonably low values of strain the masonry is degrading. The results for the measured stiffness were analysed using a linear regression and discrete fast Fourier transform technique. The results show that the discontinuities in the stiffness visible in the measured stiffness results are related in a one-one correspondence to the applied loading pattern. The results confirm the stiffness theory presented in Error! Reference source not found.. The effective stiffness can be recovered from the measured stiffness using a quasi-static frequency limit of 0.3-0.4 Hertz.

The results indicate that a minimum value of 0.2 MPa as a characteristic strength was achievable with the bricks. There is no evidence to support a change in this value as the minimum adopted flexural strength value for AS 3700 Masonry Structures. The results do point to a problem with masonry that is poorly constructed, which in the case of this
experimental work is quite clearly identified as having a characteristic flexural strength less than 0.2 MPa.

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 a non-zero damping will not change the form of equation Error! Reference source not found. 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 Error! Reference source not found. 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 Error! Reference source not found. will hold for each Fourier wave and thus for the summation of the waves. There are limitations, but the limit to seismic frequencies provides practical constraints that remain within the reasonable Fourier definition range.

The impact of the inequality shown in equation Error! Reference source not found. 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 for all 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 that are 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, 374-379). This aspect was not investigated in this research.

1.1.2 Summary
The measured stiffness has been shown in the limit, as the frequency of the loading approaches zero, to approach the effective stiffness. This theoretical limit was estimated assuming zero damping and a one per cent systematic error as being acceptable between the measured stiffness and the effective stiffness. The theoretical limit for the cut-off for a quasi-static frequency was established as 0.8 Hertz. A pragmatic frequency cut-off limit based on damping evident in the test data is 0.3 to 0.4 Hertz. This result was seen in the measured stiffness data shown in Figure 4.

This next subchapter presents details of the estimation of the effective stiffness and the damage parameterization based on these results. Once this data is established, the second stage involving the establishment of the engineering curve that is used in seismic design can proceed. The transition for a physics data set to an engineering data requires the application of engineering judgement. This judgement is applied in the step of determining change in the effective stiffness with strain level.
Table 10: Typical Model Values for the Damage Parameter

<table>
<thead>
<tr>
<th>Strain Level (microns)</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 micro-strains. The linear model offers computational speed, whereas the bi-linear model underestimates the typical range of the damage parameter at 200 micro-strains. The 200 micro-strain level appears to be the critical level at which cracking is initiated in the manifold. The complete breaking of the manifold occurs at 400 micro-strains. The theory of a single manifold breaks down at 400 micro-strains. The tri-linear and linear models are shown in Figure 7.

Figure 7: Damage Parameterization Models for Pressed Masonry Shear Walls

The additional term $D_{ci}$ has been introduced to identify the onset of cracking in the manifold.

Table 11: Summaries of Panel Test Purposes and Methods

<table>
<thead>
<tr>
<th>Laboratory Designation</th>
<th>Thesis Number</th>
<th>Test Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>210</td>
<td>AP: 1</td>
<td>This panel was used to confirm the operation of the LVDT units and the test rig in compression. The panel used an extruded brick.</td>
</tr>
<tr>
<td>226</td>
<td>AP: 2</td>
<td>This panel was used to confirm the operation of the shear yoke and to gain further data on the use of the LVDT’s and the data acquisition software. The panel used an extruded brick.</td>
</tr>
<tr>
<td>246A</td>
<td>AP: 3</td>
<td>This panel was tested at a constant frequency of 1 Hertz and increasing amplitude in a classic block pattern.</td>
</tr>
<tr>
<td>245A</td>
<td>AP: 4</td>
<td>This panel was tested frequencies of 1 and 5 Hertz and increasing amplitude.</td>
</tr>
</tbody>
</table>
The purposes for the early tests up to Test AP: 6 were to:

I. Identify and determine the physical properties of the measured stiffness.

II. To relate this measured stiffness to the static and dynamic stiffness discussed in the literature (Paulson and Abrams, 1990).

III. Investigate the systematic variation of the frequency and amplitude on the measured modulus. This investigation uses the concepts of a FFT coupled with linear regression to fit a combined linear equation and Fourier series to the data.

IV. Estimate the elastic stiffness constants at the start of the test of each masonry panel.

V. Confirm the definition for the effective stiffness, from the start of testing to the breaking of the manifold. The broken manifold destroys the assumptions required for measuring the change in the effective stiffness.

VI. Quantify the rate of degradation of the masonry shear walls as a function of the time of loading.

The purpose of the final tests from AP: 7 to AP: 10 were to:
VII. Further, investigate the properties of the rate of degradation of the masonry shear walls as a function of the time of loading and frequency, which confirms the results established in panel AP: 6.

VIII. Confirm the assumptions that underpinned the development of the definitions in the classical mechanics of stiffness, which are presented in Error! Reference source not found..

IX. Further related to a single frequency and an investigation of the time dependent changes in damping in the final stage of Panel AP: 10.

The principal objectives of the study were presented in Error! Reference source not found. Error! Reference source not found.. This thesis has presented the key findings of the research in this chapter. These key findings relate to the determination of the progressive degradation of masonry shear walls that are subjected to harmonic loads. The central facet to the findings is presented in Error! Reference source not found.. This subchapter provides a brief review of the principal objectives and relates these objectives to the research outcomes. The original research outcomes, which are relevant to the research objectives, are divided into two categories:

I. Experimental Outcomes

II. Analytical Outcomes

1.1.3 Experimental Outcomes
The first objective was to investigate the dynamic properties of shear walls constructed from a single wythe of masonry. The dissertation has addressed the properties of pressed
masonry constructed in a 1.2 metre square panel. The dynamic properties were investigated within the seismic range of frequencies.

The second objective was to determine the appropriate basis for the development of a loading pattern to be used in the experimental work. The basis for the development of the loading pattern was established from a finite element analysis of a two storey and a seven storey masonry structures analysed using a range of earthquake traces. The pattern was based on the estimated loading at the shear wall element and not for the free field acceleration.

The third objective was to design and construct a test rig that will support the experimental work. This is the fundamental basis for the experimental procedures. The detailed design and procedures for the test rig and the program are presented in the appendices.

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 Instron. The strain field was determined in a uniformly strained section of the masonry panel. The loading pattern was systematically varied to investigate different aspects of the measured stiffness. The purpose is to provide sufficient data for the statistical analysis for the effective stiffness and to demonstrate that the hypotheses are 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 in Error!
Reference source not found. 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 corresponded to the dynamic strength concepts presented in Error! Reference source not found. Error! Reference source not found.

III. Effective stiffness could be established from the measured stiffness under some limiting conditions.

IV. Equation $k^2(t) > \lambda^2(t)$ holds for the seismic range of frequencies.

V. Measured stiffness was shown to have frequency dependence.

VI. Effective stiffness $k^2(t)$ is a monotonically decreasing function that is dependent on the strain level and not the frequency of loading.

The results of the analysis of the measured stiffness are presented in Chapter 4 and Chapter 5. The analytical outcomes that follow from this experimental work are presented in the next section.

1.1.4 Analytical outcomes
The first stage used the fast Fourier transform method to investigate the measured stiffness of the shear walls and identify the conditions of loading in terms of amplitude and frequency, which yield the experimentally observed static stiffness and the dynamic
stiffness. The mathematical constraint was identified in \textbf{Error! Reference source not found.} between the measured stiffness defined as $\lambda^2(t)$ and the effective stiffness defined as $k^2(t)$, which was $k^2(t) > \lambda^2(t)$. The use of Fast Fourier transforms showed that the measured stiffness was controlled by the frequency element of the loading pattern. The effective stiffness could be recovered from the measured stiffness for some test conditions. The critical test condition to recover the effective stiffness was that the frequency of loading was to be less than 0.3 to 0.4 Hertz. This frequency $\Psi$ defines the effective quasi-static limit of loading.

The next stage was to determine the time domain degradation of the effective stiffness from the measured stiffness and to encapsulate the damage parameter and determine the critical damage level.

This dissertation presents the experimental and analytical work; the results and conclusions for this study of the progressive degradation of masonry shear walls that are subjected to harmonic shear loading. Masonry shear walls form an essential element to many building types. Masonry, as a structured continuum, has a high compressive strength and an inferior tensile strength. It is classified as a general brittle material. Shear walls that are subjected to tensile stresses that exceed the elastic capacity of the masonry can degrade, crack, rock, and slide. This research investigated the properties of the masonry up to the point of cracking. This tensile stress regime in buildings is typically generated by environmental loads such as seismic events. The purpose of the research was to investigate the properties of the masonry shear walls as they degrade in a known stress field. The stress field subjected the shear walls to a non-proportional, uniform, biaxial, compressive stress field and a harmonic, shear stress field. This conclusions
chapter presents the background and introduction to the study, outlines a summary of the results, analysis and the applications of the research findings, and presents a section on the future directions for research into the dynamic properties of masonry shear walls.

Experimental observation has shown a consistent difference in the static and dynamic strength of ceramics including masonry. Baker (1912) probably reported the first observation of the increased strength for the dynamic loading mechanism. This observation related to the dynamic loading of mortar samples in tension. The difference between the static and dynamic response characteristics of masonry shear walls was clearly shown by Paulson and Abrams (1990). Freund (1990) provides the mechanical explanation for the two responses based on the difference in the rate of stress wave loading. A literature review presented in Error! Reference source not found. outlines the existing knowledge on the static and dynamic response from experimental studies of shear walls and buildings. Macchi (1982) documents the four methods used to test masonry shear walls. The four test procedures are shown on Error! Reference source not found.. The fourth procedure of using a masonry panel subjected to uniform biaxial compression and harmonic shear was selected as appropriate for this research study.

The literature review established the constraining information and previous studies in the areas of an historical perspective, masonry practice, seismicity, masonry buildings, damage mechanics, and fracture mechanics.

In Error! Reference source not found., the degradation of masonry was considered in terms of the previous experimental work, which included observational data on analytical aspects of the dynamic properties of masonry and other materials. This information,

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coupled with the data from the literature review, provided the basis for the problem
definition and the solution method. The problem definition was to investigate the
dynamic properties of masonry shear walls and to explain the results in terms of the
theories of mechanics and damage mechanics. The solution method established the basis
for the experimental and analytical research work.

In Error! Reference source not found., the experimental study was presented in terms
of the selection and construction of the masonry panels, the design of the experimental
equipment and methodology. A pressed clay masonry brick of Sydney, Australia origin
was used with a [1C: 1L: 6S] mortar to construct the 1.2 metre test panels. The test rig is
shown assembled in Error! Reference source not found.. The test rig design,
hydraulic and electronic systems are detailed in the appendices. Central to the
development of the experimental methodology and the subsequent analysis of the results
was the review of the mechanics of stiffness presented in Error! Reference source not
found.. It was postulated that there are two distinct mathematical entities representing
the static and dynamic stiffness. The first entity is the effective stiffness that is deemed
the mathematical functional representation of the stiffness defined explicitly by the
displacement term of the differential equation of motion for a harmonic function. This
functional representation has no frequency dependence (Kaplan and Lewis, 1971, 554;
Krajcinovic, 1996). The second entity is the measured stiffness that is defined to be the
experimentally determined stiffness that is obtained by dividing the stress field results by
the strain field results for each recorded time-step (Paulson and Abrams, 1990). The
design of the loading pattern and the experimental methodology was designed to
systematically explore the statistical and physical properties of the static and dynamic stiffness.

In Error! Reference source not found., the results for the testing of the masonry panels were presented in terms of the measured stiffness data from the two critical panels AP: 3 and AP: 6. The first panel AP: 3 clearly demonstrates the evolution of the measured stiffness with time of loading and increasing strain rate. This panel was effectively tested at a constant frequency of one Hertz. This result was shown on Error! Reference source not found.. The second panel AP: 6 clearly demonstrates the fundamental relationship between the effective stiffness $k^2(t)$ and the measured stiffness $\lambda^2(t)$, which is $k^2(t) > \lambda^2(t)$. The first step in this demonstration was to confirm the frequency basis for the measured stiffness. This frequency dependence was clearly shown in Figure 4. The parabolic equation between the frequency $f$ (Hertz) and the measured stiffness $\lambda^2(t)$ was established as:

$$\lambda^2(t) = 8.4 - 1.5f + 0.086f^2$$

The result applies to the particular loading pattern for this test set. The effective stiffness $k^2(t)$ has a continuous first derivative up until the point of cracking of the manifold. This relationship had been demonstrated for other general brittle materials (Vecchio and Collins, 1986; Krajcinovic, 1996). The effect of the relationship $k^2(t) > \lambda^2(t)$ can now be clearly seen in the results for panel AP: 6 shown in Error! Reference source not found. and in the limit as the frequency $f$ approaches zero. The three hypotheses,
which are presented in Error! Reference source not found. for the relationship between the effective stiffness and the measured stiffness, are confirmed for this general brittle material.

In Error! Reference source not found., the damage parameterization was established for the pressed clay masonry panels. A review was provided for the research outcomes for the experimental and analytical work. The analysis and specific application of the results was presented and then extended to examples of applications for standards and codes. This work was based on the FEMA 273 Manual (1997). The design curve for the damage parameter $D$ established for pressed clay masonry is shown in Figure 7 Damage Parameterization Models for Pressed Masonry Shear Walls.25

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 200 micro-stains, for the start of the cracking and of 0.7 at 400 micro-strains for the failure of the manifold.

II. The standard static properties of masonry being the elastic modulus and the bond wrench tests 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 where these results are applicable is 0.2 MPa; when tested in accordance with the procedures in AS 3700. The site experience shows that a bond strength less than this level

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provides an extremely weak masonry wall, that thus fails to conform to reasonable site practice.

IV. The relationship between the effective stiffness and the measured stiffness $k^2(t) > \lambda^2(t)$ and the relationship between the frequency of loading and the measured stiffness shown in equation (10)\textsuperscript{26} demonstrates the effect of the damping and acceleration terms on the apparent strength or stiffness of a general brittle material as it is loaded at higher strain rates. This confirms the observations first made by Baker (1912) with tests on mortar samples.

V. An effective limit between quasi-static loading and dynamic loading rates can be established at a frequency $\psi$ of 0.3 to 0.4 Hertz. This applies to amplitude of stroke no greater than 5 millimetres.

VI. For pressed masonry, a homogenized isotropic model provides a reasonable basis for applying these results.

VII. Static and dynamic forces with the same peak amplitude are not equivalent loads and the principal of equivalent energy levels needs to be applied.

VIII. The results provide a source of data for assisting in the development of fragility curves for masonry buildings and to consider 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.

\textsuperscript{26} Refer pg. 38.
IX. The use of a damage parameter method in place of the linear elastic assumptions used in codes such as the FEMA 273 manual (1997) 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.

X. Future research should investigate the damping characteristics of the panels from the data. The types of procedures developed for NLFIT by Kuczera (1994) would be a suitable method. The properties of extruded and repressed bricks can be investigated with the test rig.

Masonry that is detailed for seismic loads and constructed to a reasonable level of quality control can withstand loads generated by seismic events (Melchers and Page, 1992; Tenacolungu and Abrams 1992). This research work, which extends the static research of the last few decades (Page, 1979; Ganz, and Thurlimann, 1982), has concluded that the degrading properties of pressed clay masonry are consistent with the model for a general brittle material (Krajcinovic, 1996). The effective stiffness has been shown to halve as the material was strained using harmonic shear stresses in the normal seismic range (Richter, 1958). The strain criteria to account for the increasing damage level was found to be the first principal stress as it passed into a tensile range. The equivalence of a static and dynamic force requires consideration of the energy and momentum equations, rather than the assumption that peak amplitude of force provides equivalent measured damage. This finding provides an alternative formulation for the difference between the static and dynamic strengths to that provided by Freund (1990), who explained the phenomena in terms of the stress wave-loading rate.
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Page, A.W., (1992), The design, detailing and construction of masonry - the lessons from the Newcastle earthquake, *Australian Civil Engineering Transactions*, 34, 4, 343- 53