SEISMIC STRENGTHENING OF ADOBE-MUDBRICK HOUSES

By
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EXECUTIVE SUMMARY

This thesis presents the research and development of a low-cost, low-tech reinforcement system to improve the earthquake resistance of adobe mudbrick houses. The outcome of this research project is a reinforcement system which can be readily implemented by rural homeowners in developing countries using locally available resources (materials, tools and skills), without the need for ongoing external support. The proposed reinforcement system incorporates bamboo poles placed vertically against the walls, and connected with through-wall string ties, and strands of wire running horizontally. A continuous timber ring beam is placed on top of the walls. The system can be used for new-build constructions, as well as for the retrofit-strengthening of existing dwellings. The system has the potential to substantially and sustainably reduce the vulnerability of traditional adobe houses around the world.

This thesis describes the multi-disciplinary approach undertaken for this project, which includes field research in El Salvador, review of literature, extensive experimental testing, Experimental Modal Testing and Analysis (EMTA) and the development of dissemination and implementation initiatives. A number of further research needs are also identified.

Field research in El Salvador

In early 2001, the small Central America nation of El Salvador was rocked by two major earthquakes, registering $M_w$ 7.7 and $M_w$ 6.6. The earthquakes claimed almost 1,200 lives and affected over 1.6 million people. More than 110,000 adobe houses were destroyed.

The field research component of this thesis has involved a variety of post-earthquake relief, research and reconstruction activities in El Salvador since 2001. The following aspects are presented in this thesis:

- Case study of adobe in El Salvador, including a discussion of the history and use of adobe housing, as well as some of the common features and deficiencies in traditional adobe houses.
• Evaluation of the features and effects of the 2001 El Salvador earthquakes, with a particular focus on the impacts to adobe housing.

• A review of reconstruction activities and improved adobe initiatives (promotion, training and construction projects) one year after the earthquakes.

• The design and construction of an improved adobe child-care centre in a small rural community in El Salvador.

• A review of the general state of housing reconstruction and the housing deficit in El Salvador in 2005.

• An assessment of the challenges and opportunities for the widespread implementation and acceptance of safer adobe construction and retrofit-strengthening techniques.

Literature review
Substantial seismic adobe research has been undertaken in Peru, Mexico, the U.S.A. and Colombia since the 1970s. To date, experimental testing has tended to focus on qualitative results (observations). Research findings have been included in a variety of adobe guidelines and manuals. These research and dissemination activities have made a significant contribution to the current state-of-knowledge. Despite these efforts, however, there has been a lack of large-scale application and community-level acceptance of these practices. The main reason for this lack of broadscale uptake is that most of the proposed systems are too complex and/or too complicated to be widely used without sustained external intervention.

Experimental testing
The experimental testing component of this research project is divided into four main phases:

• Static testing of adobe prisms to determine characteristic material properties (compressive, shear and tensile bond strengths). It was found that significant improvements in the shear and flexural bond strength of adobe masonry can be practically achieved by: (i) wetting the surface of each brick prior to laying:
(ii) using a thin mortar joint; and/or (iii) applying a modest compressive load during curing.

- Shake table testing of eleven 1:2 scale u-shaped adobe wall units, representing traditional and improved adobe structures. Each specimen was subjected to a series of simulations, using a modified input time history from the El Salvador earthquake of January 13, 2001. For each specimen a unique ‘time scaling factor’ was determined, based on the relationship between the natural frequency of the specimen and the dominant frequency range of the input excitation. This factor was used to time scale the input spectra to ensure dynamic similitude (between specimens) and induce damaging near-resonance conditions. Qualitative and quantitative data from each test was collected and analysed, as discussed below.

- Shake table testing of a 1:2 scale model adobe house, which was retrofit-strengthened with external vertical bamboo, external horizontal wire, and a timber ring beam. Detailed analysis of results were undertaken, as discussed below.

- Detailed analysis of results from the shake table testing of the u-panel units and the model house. This included a review and comparative analysis of the qualitative results (observations, photographs, video footage) and quantitative results (displacement-time records, relative deformation, and vertical and horizontal flexure). Common crack patterns (vertical corner cracking, vertical midspan cracking, and horizontal and diagonal cracking) were observed. These damages were due to combinations of overturning, vertical flexure and horizontal flexure. The most successful improvement systems were seen to reduce movement in the wall units and enhance the overall strength of the structure. Such systems effectively delayed the onset of initial cracking, and reduced the severity of cracking during repeated high intensity simulations. Most importantly, collapse of reinforced structures was prevented in all tests. Results from the preparation and testing were used to develop a ‘specimen rating matrix’ which presents the seismic capacity, cost and complexity of each reinforcement system. The matrix highlights the important of considering the technical and practical aspects of any proposed reinforcement system. These factors could be incorporated in a detailed multi-criteria evaluation matrix,
which would be a useful tool in the planning and realisation of any construction and implementation project.

**Experimental Modal Testing and Analysis (EMTA)**

EMTA was undertaken in conjunction with the experimental testing of the u-shaped wall units and the model house. EMTA was used to determine the dynamic characteristics (natural frequencies, damping ratios and mode shapes) of specimens both prior to and during shake table testing. Results highlighted the discontinuities introduced by internal vertical reinforcement (increased damping, decreased stiffness) and the changes in dynamic properties during the strengthening process prior to shake table testing. The influence of penetrations (windows and doors) was clearly evident. EMTA during the shake table testing showed the progressive loss of stiffness, and the general increase in modal damping, of the specimens as the level of damage increased. The mode shapes matched the flexure graphs produced during the experimental testing. EMTA was demonstrated to be a useful tool to reflect the physical response and changes in dynamic characteristics of adobe structures. Results provide greater insight into the structural behaviour than observations alone, and may be a practical tool for damage detection and condition monitoring of adobe structures.

**Dissemination and implementation**

The framework of two initiatives for the dissemination and implementation of research findings are presented in this thesis. These initiatives are designed to transfer the outcomes of seismic adobe research and application activities to communities around the world where people continue to live in vulnerable adobe houses. These initiatives included:

- The World Adobe Forum: a website dedicated to the sharing of information about safer adobe construction.

- A sample implementation program designed to increase the desire, capacity and confidence of local homeowners and builders to retrofit and/or construct safer adobe houses.
Further research

A number of further research needs have been identified and presented in this thesis. These research needs relate to both technical and practical aspects of improved adobe, and include:

- Post-earthquake reconnaissance.
- Further experimental testing of adobe bricks, prisms, wall units and model houses (static, quasi-static and dynamic testing).
- Development of a reliable numerical model through nonlinear heterogeneous Finite Element (FE) modelling.
- Extensive parametric studies (using the validated FE model) to assess a broad range of design and construction variables and improvement systems, without the need for resource-intensive physical testing.
- Implementation and application activities, including an evaluation of the effectiveness of promotion and training programs, and the development of a comprehensive multi-criteria evaluation tool.

This thesis highlights the importance of a multi-disciplinary approach which considers both the social and technical aspects of disaster mitigation activities. This approach is necessary to ensure the development of solutions which are socially-appropriate (low-cost and low-tech) and technically-sound (seismically safe). Such solutions have the opportunity to significantly reduce the vulnerability of adobe houses around the world.
CERTIFICATE OF AUTHORSHIP / ORIGINALITY

I certify that the work in this thesis has not previously been submitted for a degree nor has it been submitted as part of requirements for a degree except as fully acknowledged within the text.

I also certify that the thesis has been written by me. Any help that I have received in my research work and the preparation of the thesis itself has been acknowledged. In addition, I certify that all information sources and literature used are indicated in the thesis.

Dominic Dowling

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NOMENCLATURE

AS     Australian Standard
ASIA   Asociación Salvadoreña de Ingenieros y Arquitectos (Salvadoran Association of Engineers and Architects) [El Salvador]
ASTM   American Society for Testing and Materials [U.S.A.]
b     Breadth / width
BSSC   Building Seismic Safety Council [U.S.A.]
CAFOD  Catholic Agency for Overseas Development (NGO) [United Kingdom]
CIA    Central Intelligence Agency [U.S.A.]
COEN   Comité de Emergencia Nacional (National Emergency Committee) [GOES]
COSMOS Consortium of Organizations of Strong-Motion Observation Systems
CoV    Coefficient of Variation
d     Depth
DIGESTYC Dirección General de Estadísticas y Censos (General Office of Statistics and Censuses) [GOES]
DIN    Deutsches Institut für Normung [Germany]
E     East
ECLA/CEPAL Economic Commission for Latin America and the Caribbean / Comisión Económica para América Latina y El Caribe [UN]
EERI   Earthquake Engineering Research Institute [U.S.A.]
EGPA   Estimated peak ground acceleration
EMTA   Experimental Modal Testing and Analysis
f     Frequency
F     Force
f_c   Compressive strength
FE     Finite Element
FEMA   Federal Emergency Management Agency [U.S.A.]
FRF    Frequency Response Function
FUNDASAL Fundación Salvadoreña de Desarrollo y Vivienda Minima (Salvadoran Development and Minimum Housing Foundation) [NGO, El Salvador]
g     Acceleration due to gravity (9.81ms⁻²)
GDP    Gross Domestic Product
GOES   Gobierno de El Salvador / Government of El Salvador
GSAP   Getty Seismic Adobe Project [U.S.A.]
Hz     Hertz
IAEE   International Association for Earthquake Engineering
ISO    International Standards Organisation
<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Full Form</th>
</tr>
</thead>
<tbody>
<tr>
<td>IZIIS</td>
<td>Institute of Earthquake Engineering and Engineering Seismology, Skopje, Macedonia.</td>
</tr>
<tr>
<td>kg</td>
<td>Kilograms</td>
</tr>
<tr>
<td>kPa</td>
<td>Kilopascals</td>
</tr>
<tr>
<td>L</td>
<td>Length</td>
</tr>
<tr>
<td>LSM</td>
<td>Low Strength Masonry</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear Variable Differential Transformer (displacement transducers)</td>
</tr>
<tr>
<td>m</td>
<td>Metres</td>
</tr>
<tr>
<td>MAC</td>
<td>Modal Assurance Criterion</td>
</tr>
<tr>
<td>MDOF</td>
<td>Multi-degree-of-freedom</td>
</tr>
<tr>
<td>MIF</td>
<td>Mode Indicator Function</td>
</tr>
<tr>
<td>mm</td>
<td>Millimetres</td>
</tr>
<tr>
<td>MPa</td>
<td>Megapascals</td>
</tr>
<tr>
<td>MSJC</td>
<td>Masonry Standards Joint Committee [U.S.A.]</td>
</tr>
<tr>
<td>n</td>
<td>Number of specimens</td>
</tr>
<tr>
<td>N</td>
<td>Newtons</td>
</tr>
<tr>
<td>N</td>
<td>North</td>
</tr>
<tr>
<td>NC</td>
<td>Not Captured</td>
</tr>
<tr>
<td>NGO</td>
<td>Non-Governmental Organisation</td>
</tr>
<tr>
<td>NISEE</td>
<td>National Information Service for Earthquake Engineering</td>
</tr>
<tr>
<td>NR</td>
<td>Not Reported</td>
</tr>
<tr>
<td>°C</td>
<td>Degrees Celsius</td>
</tr>
<tr>
<td>P</td>
<td>Maximum load</td>
</tr>
<tr>
<td>p.a.</td>
<td>Per annum</td>
</tr>
<tr>
<td>PUCP</td>
<td>Pontificia Universidad Católica del Perú (Catholic University of Peru)</td>
</tr>
<tr>
<td>RESESCO</td>
<td>Reglamento Para la Seguridad Estructural de las Construcciones (Regulation for the Structural Security/Safety of Constructions) [El Salvador]</td>
</tr>
<tr>
<td>RMS</td>
<td>Root Mean Square</td>
</tr>
<tr>
<td>s</td>
<td>Seconds</td>
</tr>
<tr>
<td>S</td>
<td>South</td>
</tr>
<tr>
<td>S.A.</td>
<td>Surface Area</td>
</tr>
<tr>
<td>SD</td>
<td>Standard Deviation</td>
</tr>
<tr>
<td>SDOF</td>
<td>Single-degree-of-freedom</td>
</tr>
<tr>
<td>ST</td>
<td>Shake table</td>
</tr>
<tr>
<td>t</td>
<td>Time</td>
</tr>
<tr>
<td>TRS</td>
<td>Test Response Spectrum</td>
</tr>
<tr>
<td>UCA</td>
<td>Universidad Centroamericana ‘José Simeón Cañas’ [El Salvador]</td>
</tr>
<tr>
<td>UES</td>
<td>Universidad de El Salvador (University of El Salvador)</td>
</tr>
</tbody>
</table>
UN       United Nations
UNAM     Universidad Nacional Autónoma de México
UNDP     United Nations Development Program
USGS     United States Geological Survey [U.S.A.]
UTS      University of Technology, Sydney, Australia
VMVDU    Vice-Ministerio de Viviendas y Desarrollo Urbano (Vice-Ministry of Housing and Urban Development) [GOES]
W        West
w.r.t.   with respect to
τ        Shear strength
ζ        Damping ratio
To the people of El Salvador, whose indefatigable spirit and courage endures and inspires, despite a history of war, oppression, corruption, and natural disasters.
1 INTRODUCTION

1.1 Background

Devastating earthquakes in Asia, the Middle East, Africa and Latin America have served as recent reminders of the vulnerability of non-engineered, low-cost dwellings to seismic forces (Figure 1-1 and Figure 1-2). The loss of life and livelihood is often drastic, with millions of people in the poorest communities most severely affected. Adobe mudbrick housing is particularly vulnerable because of its inherently brittle nature, widespread use, generally poor construction quality and the limited awareness of concepts of aseismic design and construction. Despite this limitation, however, there is little doubt that adobe will continue to be the main construction material for the majority of the rural poor who simply cannot afford any alternative.

Figure 1-1 Earthquake damage to adobe houses.
Over the last three decades there has been considerable research focused on reducing the vulnerability of adobe houses. These efforts have included a variety of static and dynamic tests of adobe structures which have yielded some technically-proven means of improving the seismic capacity of such dwellings. Despite these efforts, however, there has been a distinct lack of large-scale application and community-level acceptance of these practices. The main reason for this lack of broad-scale uptake is that most of the proposed systems are too complex and/or too costly to be widely used without sustained external intervention. Other factors include: deficiencies in the promotion and support of improved adobe; negative stigma associated with adobe (seen as representative of poverty and lack of progress); and international attention and aid generally focused on disaster response, rather than disaster preparedness. A combination of social and technical solutions is required to reduce the vulnerability of adobe structures, and local and international collaboration is necessary to realise this goal.

![Total destruction of adobe house, El Salvador, 2001 (López, UES/EERI).](image-url)
1.2 Adobe

Definition
Adobe is defined as: ‘a sun-dried brick used for building’ (Collins, 1982). Adobe is also widely known as ‘mudbricks’.

The term ‘improved adobe’ is used in this thesis to refer to any systems and techniques which are intended to improve the seismic performance of adobe structures.

Distribution
Adobe is one of the oldest and most widely used construction materials in the world. It is estimated that one-third of the world’s population live in a home made from unfired earth, of which adobe is a major form (Houben and Guillaud, 1994). Earth building is more common in developing countries, where it is reportedly used by approximately 50% of the population (Houben and Guillaud, 1994), predominantly in rural areas. Adobe is widely used in Latin America, Africa, the Middle East and Asia (Figure 1-4a), with buildings ranging from simple, single-storey dwellings to large and elaborate monuments, churches and cultural sites.

Advantages
The advantages of adobe as a building material include:

- Low cost (soil is usually free, labour is usually community and family based);
- Easy to use (no specialist skills are required to make and lay adobe blocks);
- Durable (long life, seen in many very old structures);
- Energy efficient (low energy required to make and lay bricks);
- Ecologically sustainable (natural materials, recyclable, low embodied energy); and
- High thermal capacity (natural regulation of temperature).

The first two advantages are responsible for the predominant use of adobe in developing countries. The latter factors are responsible for the re-emergence of adobe as an attractive housing option in developed countries, such as the U.S.A., New Zealand and Australia (Figure 1-3).
Disadvantages

The disadvantages of adobe include:

- Labour intensive and slower construction process (due to fabrication of bricks, and daily build-height limitations);
- Susceptible to water damage if inadequately protected;
- Negative stigma in many developing countries (often associated with poverty or lack of progress); and
- Vulnerable to damage by earthquakes (inherently low-strength, brittle material used mostly in unreinforced buildings).

This final feature has been tragically observed in earthquake-prone areas around the world. Figure 1-4 shows global maps indicating the distribution of earthen architecture/construction, and regions of moderate, high and very high seismic hazard. The maps show an alarming matching of earthen building use and earthquakes.

Figure 1-3 Modern adobe home (Earthways, Wollombi, Australia).
Figure 1-4 Global maps: (a) earthen architecture; (b) seismic hazard (www.terracruda.com).
1.3 **Adobe performance in earthquakes**

The vulnerability of traditional adobe structures to earthquakes has been known for centuries. Table 1-1 shows a list of major earthquakes in regions where adobe housing commonly exists. What is most disturbing is that, despite the tremendous advances in earthquake engineering and risk mitigation measures in recent years, the loss in developing countries continues to be high. A large proportion of the fatalities and injuries in earthquakes is due to the collapse of buildings, with low-strength unreinforced masonry (e.g. adobe, stone, brick) being particularly vulnerable.

<table>
<thead>
<tr>
<th>Year</th>
<th>Location</th>
<th>Deaths</th>
<th>Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>2005</td>
<td>Pakistan (Kashmir)</td>
<td>80,361</td>
<td>7.6</td>
</tr>
<tr>
<td>2003</td>
<td>Iran (Bam)</td>
<td>31,000</td>
<td>6.6</td>
</tr>
<tr>
<td>2001</td>
<td>India (Gujarat)</td>
<td>20,023</td>
<td>7.7</td>
</tr>
<tr>
<td>1999</td>
<td>Turkey</td>
<td>17,118</td>
<td>7.6</td>
</tr>
<tr>
<td>1990</td>
<td>Western Iran</td>
<td>40,000 – 50,000</td>
<td>7.7</td>
</tr>
<tr>
<td>1976</td>
<td>Guatemala</td>
<td>23,000</td>
<td>7.5</td>
</tr>
<tr>
<td>1976</td>
<td>China (Tangshan)</td>
<td>255,000 (official) 650,000 (unofficial)</td>
<td>7.5</td>
</tr>
<tr>
<td>1970</td>
<td>Peru</td>
<td>66,000</td>
<td>7.9</td>
</tr>
<tr>
<td>1948</td>
<td>USSR (Turkmenistan, Ashgabat)</td>
<td>110,000</td>
<td>7.3</td>
</tr>
<tr>
<td>1935</td>
<td>Pakistan (Quetta)</td>
<td>30,000 – 60,000</td>
<td>7.5</td>
</tr>
<tr>
<td>1932</td>
<td>China (Gansu)</td>
<td>70,000</td>
<td>7.6</td>
</tr>
<tr>
<td>1927</td>
<td>China (Tsinghai)</td>
<td>200,000</td>
<td>7.9</td>
</tr>
<tr>
<td>1920</td>
<td>China (Gansu)</td>
<td>200,000</td>
<td>7.8</td>
</tr>
<tr>
<td>1727</td>
<td>Iran (Tabriz)</td>
<td>77,000</td>
<td>-</td>
</tr>
<tr>
<td>1556</td>
<td>China (Shansi)</td>
<td>830,000</td>
<td>~8</td>
</tr>
<tr>
<td>1138</td>
<td>Syria (Aleppo)</td>
<td>230,000</td>
<td>-</td>
</tr>
<tr>
<td>856</td>
<td>Iran (Damghan)</td>
<td>200,000</td>
<td>-</td>
</tr>
</tbody>
</table>

*Source: USGS (2006)*
Less severe earthquakes around the world have also caused devastating losses, especially to the housing sector in developing countries. These include recent earthquakes in Latin America (Argentina, Bolivia, Brazil, Chile, Colombia, Costa Rica, Ecuador, El Salvador, Guatemala, Honduras, Mexico, Nicaragua, Peru), Asia (Afghanistan, China, India, Indonesia, Kashmir, Mongolia, Pakistan, Philippines, Russia, Taiwan, Tajikistan, Turkey), the Middle East (Iran, Israel, Jordan) and Africa (Algeria, Morocco, Mozambique, Tanzania). They serve as clear reminders that urgent, appropriate and broad-reaching action is required.

Comprehensive earthquake damage statistics from the El Salvador earthquakes of early 2001 are presented in Chapter 2. The earthquakes had a particularly devastating impact on adobe housing (more than 110,000 adobe houses destroyed). The experiences of the author in El Salvador during and after the earthquakes have served as a reference point and motivation for the realisation of this project.

Recent advances in performance-based earthquake engineering have led to the development of detailed performance criteria and design guidelines [e.g. Vision 2000 (SEAOC, 1995); NEHRP Guidelines (BSSC, 1997)]. Although these criteria provide necessary and detailed categorisation of damage and serviceability, the fundamental performance criteria discussed by Liu (1985, cited by Tolles and Krawinkler, 1990) are considered to be more appropriate in the case of low-cost housing in developing countries. These performance criteria are:

1) No damage during minor earthquakes.
2) Tolerable damage during moderate earthquakes.
3) Heavy damage but no collapse during extraordinarily severe earthquakes.

These performance criteria and objectives have been adopted in this research project. The main priority of any improved adobe reinforcement system is to save lives. Primarily this means giving residents ample time to escape from their homes in the event of an extreme earthquake.
1.4 Objectives and scope

The main objective of this research project was:

To develop a low-cost, low-tech reinforcement system to improve the earthquake resistance of new and existing adobe mudbrick houses for application in developing countries without the need for ongoing external intervention.

In order to achieve this objective the following criteria were considered:

- **Seismic capacity**: minimisation of cracking during low-intensity shaking, and delay / prevention of severe damage and collapse during extreme earthquake events.

- **Simplicity of construction**: selected reinforcement systems should be simple to implement, such that given a moderate amount of training, the technique can be adopted at the local level with confidence.

- **Cost of construction and choice of materials**: selected reinforcement systems should use materials which are locally available and affordable to the homeowners.

Other research objectives included:

- Increase understanding of dynamic behaviour of adobe structures.

- Assess the seismic capacity, complexity and cost of a variety of reinforcement systems, both new and established.

- Determine appropriate conditions for the dynamic testing of adobe structures.

- Establish a database of experimental data for later use in finite element (FE) modelling of adobe structures.

- Understand some of the challenges and opportunities for the widespread application of simple techniques to improve the seismic resistance of adobe dwellings.

- Identify future research needs (technical and social).

- Obtain and share information about safer adobe construction practices (research and application).
These objectives were met through a comprehensive, multi-disciplinary approach, which included:

- Field research involving a variety of post-earthquake response, reconstruction and research activities in El Salvador since 2001. Activities included: earthquake reconnaissance; a 'lessons learned over time’ investigation focused on adobe housing reconstruction; and the design and construction of an improved adobe child-care centre in a small rural community.

- Review of the current literature, covering previous adobe research (including static and dynamic testing, and Experimental Modal Testing and Analysis, EMTA), and existing guidelines and manuals for safer adobe design and construction.

- Testing of adobe prisms to determine key structural properties (compressive, shear and tensile bond strength) and assess the effectiveness of simple construction techniques to improve these properties.

- Development of a robust means of shake table testing adobe structures, to ensure dynamic similitude and induce damaging near-resonance conditions.

- Construction, shake table testing and comparative analysis of 1:2 scale adobe structures (u-panels and model house) to assess the behaviour of unreinforced and reinforced structures, and evaluate the seismic capacity, cost and complexity of different improvement systems.

- Detailed Experimental Modal Testing and Analysis (EMTA) of adobe structures prior to and during shake table testing to understand the key dynamic characteristics, and changes thereof, during testing.

- Development of the concept and framework of the World Adobe Forum, a website dedicated to sharing information about safer adobe construction.

- Development of a sample implementation strategy, which outlines the steps involved in a broad-scale promotion and training program.

- Identification of further research needs, both technical and social.
The research undertaken as part of this thesis is one stage in a long and iterative process involving:

- Implementation projects, which include promotion, awareness-building, training (practical and simple theory), and periodic refresher training.
- Ongoing review of implementation progress, successes, lessons learned and identification of future research needs (technical and social).
- Ongoing research (technical and social) and modifications to techniques.

The following research areas were identified, but considered beyond the scope of this thesis:

- Testing of advanced and modern materials as reinforcement (e.g. plastic mesh and tubing, synthetic mesh fabrics, steel reinforcement, carbon fibres, reinforced concrete elements, fibre-reinforced polymers, epoxy grouts, etc). The research described in this thesis centred on the use of locally-available and affordable materials.
- Detailed analysis of the most appropriate soil composition (sand-silt-clay proportions), moisture content and curing conditions for adobe bricks. Substantial research has been done in this area by others, however, at the grass-roots level people continue to use the material which is available. The bricks used in this research were fabricated to be 'consistently average' bricks, that is, not too strong, not too weak, and with general consistency.
- Finite Element Modelling (FEM) of adobe structures. Due to the complexity of modelling the highly non-linear (post-cracked) state, finite element modelling was not undertaken. The vast array of experimental test data recorded during this research is suitable for use in future modelling.
- Static testing of walls (in-plane and out-of-plane). This project focused on the complex dynamic behaviour and interactions of adobe structures rather than the load-deformation characteristics and relationships of wall panels. Without a finite element modelling component in the research, this information was not necessary.
- Micro-level mechanics, including stress concentrations, stress analysis, crack propagation and fracture mechanics. This thesis focuses on the global, general behaviour of adobe structures.
1.5 Thesis outline

Chapter 1 is the introductory chapter which provides the background, context and rationale for the research, including a discussion of recent major earthquakes and their impact on adobe structures. It also defines the objectives and scope of this research project and outlines the contents of this thesis.

Chapter 2 is a case study on El Salvador, based largely on the author’s experiences there during and since the devastating earthquakes of early 2001. These experiences have provided exposure to the realities, opportunities, challenges and needs related to appropriate housing solutions in vulnerable communities. This exposure has been fundamental to understanding the core issues related to adobe housing, and to the development and realisation of the research objectives discussed above. The chapter is divided into seven main components:

- El Salvador: profile, context, and traditional use of adobe.
- The 2001 El Salvador earthquakes: general features and damage statistics.
- Damage patterns and failure mechanisms of adobe houses subjected to earthquakes.
- ‘Lessons learned over time’ investigation: review of reconstruction activities one year after the earthquakes, with particular focus on improved adobe initiatives (promotion, training and construction projects) and some of the strengths and limitations of these initiatives (undertaken in 2002).
- Improved adobe child-care centre: project design and construction, lessons learned and outcomes (undertaken in 2002).
- El Salvador in 2005: reconnection with people, projects and progress four years after the earthquakes.
- Discussion of some of the challenges facing the widespread application of improved adobe design and construction initiatives.

Chapter 3 is a literature review, covering two main aspects:

- Research related to evaluating and improving the seismic performance of adobe structures, with most attention given to shake table testing of adobe models.
- Review of existing improved adobe design and construction manuals and guidelines.
Chapter 4 describes the fabrication of the adobe bricks used in this research project, and presents a series of compressive, shear and tensile bond strength tests of adobe prisms. These tests were undertaken to determine key structural properties and assess the effectiveness of simple construction techniques to improve these properties (e.g. using wet or dry bricks during construction, varying mortar thicknesses, and application of different compressive loads during curing).

Chapter 5 presents the characteristics of the UTS shake table and the process of selection, verification and modification of the input time history (taken from the El Salvador earthquake of January 13, 2001). The chapter describes the important process of accurately time scaling the input spectra for each specimen to ensure dynamic similitude and induce damaging near-resonance conditions. This feature has not been reported for other adobe testing.

Chapter 6 details the core component of this research: the shake table testing of 1:2 scale u-shaped adobe wall units. The chapter describes the preparation, instrumentation, testing and observations for eleven wall units, which represented traditional and improved adobe structures. The reinforcement systems tested included: corner pilasters/buttresses, internal horizontal chicken wire mesh, external chicken wire mesh wrapping, internal vertical bamboo/timber poles, external horizontal bamboo poles, external vertical bamboo poles, external horizontal wire reinforcement, and a timber ring beam. These improvements were tested individually and/or in combination with other interventions. The chapter focuses on the system developed at UTS, which incorporates external vertical bamboo, external horizontal wire, string, and a timber ring beam. A number of variations of this system were tested, including a retrofitted model and an ‘optimised’ model. The chapter provides the specifications of each model, as well as the preparation, testing, results and lessons learned from each test and specimen. Results are presented in the form of observations, photographs and displacement-time history graphs which show the response of the wall relative to the shake table motion. The chapter focuses on the observed behaviour and recorded responses of each individual specimen (i.e. an objective presentation of ‘what happened’). Accompanying video footage of the tests is included in Appendix A.
Chapter 7 provides a detailed, comparative analysis and discussion of results from the u-shaped adobe wall unit tests (i.e. the ‘why’ and ‘what does it mean’). The relative behaviour of different specimens and reinforcement systems is considered, utilising displacement data (absolute, relative, peak and root mean square displacements) and flexure snapshots which show the response of each wall at critical stages. Drawing on this information and the observations in Chapter 6, as well as commonly reported damage patterns (as presented in Chapter 2) the chapter includes a discussion of the factors which cause damage, and the contributions of different reinforcing systems in ameliorating these damage factors. The chapter concludes with a matrix which evaluates the seismic capacity, cost and complexity of each reinforcement system.

Chapter 8 contains details of the Experimental Modal Testing and Analysis (EMTA) undertaken for the u-shaped adobe wall units. EMTA provides information on the key dynamic properties (natural frequencies, damping ratios and mode shapes) of each structure and was undertaken in two stages: (i) EMTA using an impact hammer to excite each specimen prior to shake table testing; and (ii) EMTA using the shake table to excite each specimen. Chapter 8 describes the test procedure and instrumentation for each stage, and discusses the dynamic characteristics both before and during shake table testing. Detailed EMTA theory and extensive results are included in Appendix D.

Chapter 9 describes the construction, preparation, shake table testing, observations and analysis of model house 4A (1:2 scale), which was retrofitted with external vertical bamboo, external horizontal wire and a timber ring beam. The chapter includes the observed behaviour and recorded responses for each simulation (the ‘what’ happened), followed by some comparative analysis and discussion of general crack patterns and failure modes (the ‘why’ and ‘what does it mean’). Accompanying video footage of the construction, strengthening and testing of model house 4A is included in Appendix A.

Chapter 10 contains details of the Experimental Modal Testing and Analysis (EMTA) of model house 4A. EMTA was undertaken at each key stage of the retrofitting process (using the impact hammer) and then for the series of shake table tests. The chapter describes the test procedure and instrumentation, and discusses the dynamic characteristics during the retrofit-strengthening stages (impact hammer tests) and during shake table testing.
Chapter 11 presents the concept and framework for the World Adobe Forum, a website dedicated to sharing information about safer adobe construction. The chapter describes the background, objectives, stakeholders, means and scope of the website, which is currently under development. Sample templates are presented for key subject areas: experimental testing/analysis; field research; and application/implementation.

Chapter 12 presents the framework for a potential implementation program. The main objective of the proposed implementation strategy is to enhance the desire, capacity and confidence of local homeowners and builders to retrofit and/or construct safer adobe houses. This involves a combination of promotion, training and construction activities. The chapter includes a sample two-page brochure outlining the retrofit-strengthening process.

Chapter 13 identifies and discusses further research needs, including: post-earthquake reconnaissance; experimental testing (static and dynamic); finite element modelling (FEM) and experimental modal testing and analysis (EMTA); and implementation and application.

Chapter 14 contains a summary of the overall research process, results and conclusions.

The final sections of the thesis contain a list of references, and appendices containing supplemental information. The appendices include:

- Video footage of UTS research, including construction, preparation and testing of specimens (Appendix A).
- Media exposure and promotions related to this project, including national television and radio segments and an article in New Scientist magazine (Appendix B).
- Ground motion recording station characteristics from the El Salvador earthquake of January 13, 2001 (Appendix C).
- Theory and additional results from the EMTA (Appendix D).
2 CASE STUDY: ADOBE IN EL SALVADOR

2.1 Introduction

In early 2001 two major earthquakes rocked the small Central American country of El Salvador. The two quakes (registering Mw 7.7 and Mw 6.6) devastated the social and physical infrastructure of the small nation. The earthquakes claimed almost 1,200 lives and affected over 1.6 million people (UNDP, 2001). The housing sector was particularly hard-hit, with over 166,000 houses destroyed and 110,000 houses damaged (DIGESTYC, 2001). In some regions, up to 85% of the houses were destroyed. The vulnerability of traditional adobe houses was clearly demonstrated, with 113,000 adobe houses destroyed and 43,000 adobe houses damaged (DIGESTYC, 2001). Overall, 44% of the pre-earthquake adobe housing stock was affected (damaged + destroyed), and adobe houses accounted for 57% of the total affected houses (DIGESTYC, 1999 & 2001).

This chapter is a case study on El Salvador, based largely on the author’s experiences there during and since the devastating earthquakes (Dowling, 2004a,b&c). The chapter is divided into seven main components:

- El Salvador: profile, context, and traditional use of adobe.
- The 2001 El Salvador earthquakes: general features, damage statistics and impacts to adobe housing.
- Damage patterns and failure mechanisms of adobe houses subjected to earthquakes.
- ‘Lessons learned over time’ investigation: review of reconstruction activities one year after the earthquakes, with particular focus on improved adobe initiatives (promotion, training and construction projects) and some of the strengths and limitations of these initiatives (undertaken in 2002).
- Improved adobe child-care centre: project design and construction, lessons learned and outcomes (undertaken in 2002).
- El Salvador in 2005: reconnection with people, projects and progress four years after the earthquakes.
- Discussion of some of the challenges facing the widespread application of improved adobe design and construction initiatives.
Exposure to these aspects has been fundamental to understanding the core issues related to adobe housing, and to the development and realisation of the objectives of this research project.

2.2 El Salvador

2.2.1 Profile

El Salvador is the smallest and most densely populated country in Central America. It covers an area of 21,040 km² (CIA, 2002) and has an estimated population of 6,756,800 (UNDP, 2005a). El Salvador is divided into 14 departments and is bordered by the Pacific Ocean to the south, Guatemala to the west and Honduras to the north and east (Figure 2-1). El Salvador has a long history of seismic activity, and is affected by subduction and upper-crustal earthquakes. Over the last century, El Salvador has experienced an average of one major, destructive earthquake per decade (Bonmer et al., 2002).

Figure 2-1 Map of El Salvador (United Nations).
El Salvador’s recent history is replete with disasters, both natural and human in origin. Natural disasters have included major earthquakes in 1965, 1982, 1986 and 2001, Hurricane Mitch in 1998, Hurricane Stan in 2005, volcanic eruptions and various periods of drought and flooding. The human-generated disasters have included social, environmental and economic injustices (poverty, landlessness, deforestation, unemployment and a huge disparity in wealth distribution). These features have come to a head in violent uprisings by the marginalised, and aggressive repressions by the military and ruling class. A series of conflicts over the last 100 years increased strong social imbalances and created an environment of fear and uncertainty across the country. In 1992, the 12-year civil war ended with the signing of peace accords, and the country began the process of rebuilding social, institutional and physical infrastructure and systems.

El Salvador occupies position 104 in the list of 173 countries assessed by the UNDP Human Development Index, which corresponds to countries classed as having medium human development (UNDP, 2005b). The UNDP (2005b) reports that for the years 1990-2003, 31.1% of the population lived on less than US$1 a day, 58.8% lived on less than US$2 a day, and 48.3% of the population lived below the national poverty line. Despite general improvements during the period, the disparity between urban and rural poverty increased, with the average rural income only 40% of the average urban income in 1999 (UNDP, 2001). Furthermore, the UNDP (2001) reported that the inequality levels in El Salvador remain amongst the highest in the world, with the richest 20% of the population receiving 56.2% of overall income in 1999, while the poorest 50% received only 16.4%.

2.2.2 Adobe in El Salvador

History

Adobe has been used in El Salvador since pre-conquest times (early 16th Century), although the indigenous population mainly used light materials, such as straw, palm and reeds. In colonial times adobe houses became more prominent and included stone foundations, thick walls (up to 1.5 meters thick), stabilising pilasters, and a timber roof structure supporting clay roof tiles (Moreira and Rosales, 1998). Over time, however,
adobe revealed itself to be very vulnerable to earthquake damage due to its large mass, low strength, and brittleness. This, combined with the introduction of 'modern' construction materials (such as cement, corrugated iron, brick and stone masonry, followed by pre-cast concrete, steel, aluminium, plywood, asbestos-cement, and concrete blocks in recent times), generated changes in the usage patterns and attitudes to traditional materials. Despite these changes, adobe is still extensively used in rural communities and urban areas outside San Salvador.

Distribution

In 1999 it was estimated that approximately 1.6 million people (26% of the population) lived in adobe homes, with 70% of these located in rural areas, 26% in urban areas outside San Salvador and less than 4% in the San Salvador Metropolitan Area (DIGESTYC, 1999). Figure 2-2 shows the housing distribution in El Salvador, and Table 2-1 describes the common housing types in El Salvador. The housing distribution in 2004 is presented in Section 2.7 below, and provides an interesting insight into the impacts of the 2001 earthquakes, and the process of reconstruction.

**Total country (urban + rural): 1,383,145 houses**

<table>
<thead>
<tr>
<th>Material</th>
<th>Number</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adobe</td>
<td>359,969</td>
<td>26%</td>
</tr>
<tr>
<td>Timber</td>
<td>36,857</td>
<td>3%</td>
</tr>
<tr>
<td>Lamina</td>
<td>28,856</td>
<td>2%</td>
</tr>
<tr>
<td>Concrete / Misto</td>
<td>863,940</td>
<td>63%</td>
</tr>
<tr>
<td>Other</td>
<td>20,265</td>
<td>1%</td>
</tr>
</tbody>
</table>

Figure 2-2 Housing distribution in El Salvador: housing material, number of houses and proportion of total housing stock (DIGESTYC, 1999).
Table 2-1 Common housing materials used in El Salvador.

<table>
<thead>
<tr>
<th>Category</th>
<th>Material</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>'Modern' materials</td>
<td>Concrete</td>
<td>Any concrete building, including concrete block, concrete panel walls, reinforced concrete, etc.</td>
</tr>
<tr>
<td></td>
<td>Mixto</td>
<td>System of confined masonry, consisting of lightly reinforced concrete beams and columns with infill brick masonry.</td>
</tr>
<tr>
<td>Traditional / low-cost materials</td>
<td>Adobe</td>
<td>Sun-dried mud brick.</td>
</tr>
<tr>
<td></td>
<td>Bahareque</td>
<td>Matrix of vertical and horizontal timber or cane elements with mud or stone in-fill and/or mud plaster. Also known as 'wattle and daub' or 'quincha'.</td>
</tr>
<tr>
<td></td>
<td>Timber</td>
<td>Any type of timber construction.</td>
</tr>
<tr>
<td></td>
<td>Lamina</td>
<td>Metal lamina sheeting, typically corrugated, which is supported by a timber frame.</td>
</tr>
<tr>
<td></td>
<td>Other</td>
<td>e.g. plastic sheeting, palm fronds, cardboard, discarded materials.</td>
</tr>
</tbody>
</table>

Table 2-2 shows some of the factors influencing the type of construction in urban and rural areas.

Table 2-2 Factors influencing urban and rural construction.

<table>
<thead>
<tr>
<th>Feature</th>
<th>Urban</th>
<th>Rural</th>
</tr>
</thead>
<tbody>
<tr>
<td>Income</td>
<td>Greater access to income sources&lt;br&gt;Medium level of poverty</td>
<td>Minimal access to income sources&lt;br&gt;High level of poverty</td>
</tr>
<tr>
<td>Materials</td>
<td>Greater access to 'modern' materials&lt;br&gt;(bricks, cement, steel, etc.)</td>
<td>Traditional materials are widely available (soil, bamboo, rocks, low-grade timber, etc.)</td>
</tr>
<tr>
<td>Builders</td>
<td>Greater access to skilled builders with experience in 'modern' construction</td>
<td>Usually rely on family or community-based artisans with or without experience in construction</td>
</tr>
<tr>
<td>Land space</td>
<td>Restricted</td>
<td>Unrestricted</td>
</tr>
<tr>
<td>Enforcement of codes</td>
<td>Little control</td>
<td>Little or no control</td>
</tr>
</tbody>
</table>
2.2.3 Traditional adobe construction in El Salvador

Traditional adobe construction is usually undertaken in an ad-hoc, unregulated manner. The systems and construction techniques vary considerably according to local practices, local materials and local skills. The following generalised aspects relate to the traditional form of adobe construction in El Salvador. Some of the more serious limitations of the current practices are presented, although it should be noted that these shortcomings are not representative of all adobe houses. Some houses feature many deficiencies whereas others are well built and maintained.

**Land ownership** is a key factor influencing the current housing situation in El Salvador. Wisner (2001) suggests that the roots of disaster vulnerability lie in the imbalance of land ownership and the consequent violent struggles that have become a feature of El Salvador’s history. The process of land redistribution, a key part of the 1992 Peace Accords, has been bureaucratic and slow and as such, a large proportion of families do not possess official land titles. This has created uncertainty, with a general reluctance to commit effort and funds to the adequate construction of a house, when the legal title to the land is in question. This feature has been particularly significant in post-earthquake reconstruction, with donor agencies requiring beneficiaries to demonstrate legal land ownership in order to receive support. Families without this legal title have been left largely unsupported.

**Site selection** is linked with land ownership and rarely takes into account local hazards. Most families simply utilise land that is available, and consequently, houses are often built in high-risk areas, which may be subject to flooding, volcanic activity, and slope or soil instability.

**Labour** is usually family or community based and is normally directed by the family head or someone with experience in construction. Formal training is rarely available and techniques (both good and bad) are passed on informally.

Adobe blocks are generally made with the locally available soil, and may contain unsuitable proportions of sand, silt and clay, as well as undesirable organic material, large particles and foreign matter. The mud is often inadequately mixed and placed into
moulds that are poorly constructed and the blocks are commonly cured in direct sunlight or exposed to rain. These factors often result in severe cracking, erosion or deformation of the blocks.

Construction: crude foundations consist of stones held by a weak cement or mud mortar without reinforcement. Foundations are often shallow and are not raised above the natural ground level, such that the first course of blocks is susceptible to water ingress from the ground and excess rainwater. A damp-proof course or layer is generally not used. Often, the floor consists of bare earth, which has been compacted and smoothed over time. In other cases, a thin (10 - 30mm) concrete screed is applied or tiles laid. In newer houses walls are generally thin (200 – 300mm) and mortar joints tend to be thick (greater than 30mm), and in some cases are thicker than the blocks themselves (Figure 2-3). Common features also include un-level horizontal courses, insufficient horizontal block overlap and vertical joints running several courses high. Foreign material, such as ceramic, metal, timber and glass, is frequently embedded in the walls, which are often out of plumb. Ring beams, pilasters and reinforcement, either vertical or horizontal, are hardly ever used. The walls may be unprotected or rendered with cement, lime or mud mortar. The majority of houses are one-storey high.

![Figure 2-3 Typical adobe house, La Paz, El Salvador.](image)

The roof structure is normally made from heavy rough-hewn timbers that support a heavy clay tile covering (Figure 2-3). The structure generally rests directly upon the
walls, with little or no attachment. The tiles are not normally attached to the frame, nor to each other.

Other houses consist of crude timber columns supporting the roof structure, with in-fill adobe walls (Figure 2-4). The walls are seldom attached to the columns and behave as unrestrained cantilever walls, which are highly susceptible to collapse during seismic events. The advantage of this system is that the roof is less prone to collapse if the columns and roof diaphragm have sufficient strength.

Many buildings have been built in stages or repaired using different materials and techniques. Such construction generally lacks continuity, homogeneity and uniformity. During an earthquake the response of each component is different, which often causes damage to the adjoining components due to pounding.

Maintenance of adobe buildings is generally poorly done. Many of the buildings that failed during the earthquakes of early 2001 were older buildings that were poorly maintained. The attack of natural agents, such as water, wind, animals, insects and plants, is often left unattended and reduces the structural integrity of the building.

Figure 2-4 Timber columns with unrestrained adobe walls, Usulután, El Salvador.
2.3 The 2001 El Salvador earthquakes

2.3.1 General

The earthquakes of January 13 and February 13, 2001 (M\text{w} 7.7 and M\text{w} 6.6, respectively) caused widespread damage across El Salvador (Figure 2-6). Almost every department of El Salvador was affected in some way, with the housing sector being particularly hard-hit (Figure 2-5). The damage from the January 13 earthquake was much greater than that of the February 13 earthquake, but because of the short time period between the two events, many of the damage statistics have been combined. Table 2-3 shows various features of the two earthquakes. The majority of fatalities in the first quake were due to a large landslide which enveloped a section of the city of Santa Tecla and claimed approximately 500 lives (La Prensa Grafica, 2001). The loss of life across El Salvador would have been undoubtedly higher if the earthquakes had occurred during the night when sleeping residents would have been trapped in their homes. In such a case, a death toll in the tens of thousands would have been not unlikely.

Figure 2-5 Severely affected village, El Salvador (López, UES/EERI).
<table>
<thead>
<tr>
<th>Date</th>
<th>Saturday January 13, 2001</th>
<th>Tuesday February 13, 2001</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time</td>
<td>11.33 am (local time)</td>
<td>8.22 am (local time)</td>
</tr>
<tr>
<td>Geographic Co-ordinates</td>
<td>13.049 N, 88.660 W</td>
<td>13.671 N, 88.938 W</td>
</tr>
<tr>
<td></td>
<td>(USGS)</td>
<td>(USGS)</td>
</tr>
<tr>
<td>Magnitude</td>
<td>$M_w$ 7.7 (USGS)</td>
<td>$M_w$ 6.6 (USGS)</td>
</tr>
<tr>
<td>Depth</td>
<td>60 km (USGS)</td>
<td>10 km (USGS)</td>
</tr>
<tr>
<td>Fatalities</td>
<td>844 (COEN, 2001)</td>
<td>315 (COEN, 2001)</td>
</tr>
<tr>
<td>Injuries</td>
<td>4,723 (COEN, 2001))</td>
<td>3,399 (COEN, 2001)</td>
</tr>
<tr>
<td>Affected population</td>
<td>1,616,782 people (UNDP, 2001)</td>
<td>(=25% of the population)</td>
</tr>
<tr>
<td>Houses (Figure 2-10)</td>
<td>166,529 destroyed (DIGESTYC, 2001)</td>
<td>110,065 damaged (DIGESTYC, 2001)</td>
</tr>
<tr>
<td>Roads</td>
<td>2,300 km damaged (La Prensa Grafica, 2002)</td>
<td></td>
</tr>
<tr>
<td>Health facilities (Figure 2-8)</td>
<td>20 hospitals and 75 health centres damaged (Ministry of Health, 2001)</td>
<td>3 hospitals and 22 health centres damaged (Ministry of Health, 2001)</td>
</tr>
<tr>
<td>Churches (Figure 2-9)</td>
<td>344 damaged (COEN, 2001)</td>
<td>73 damaged (COEN, 2001)</td>
</tr>
<tr>
<td>Public Buildings</td>
<td>1,155 damaged (COEN, 2001)</td>
<td>82 damaged (COEN, 2001)</td>
</tr>
<tr>
<td>Economic losses</td>
<td>US$1,255.3 million</td>
<td>US$348.5 million</td>
</tr>
<tr>
<td>Total losses: US$1.6 billion (=12% of 2001 GDP) (UNDP, 2001)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Landslides (Figure 2-6)</td>
<td>574 (COEN, 2001)</td>
<td>71 (COEN, 2001)</td>
</tr>
<tr>
<td>Other</td>
<td>Eight people killed in Guatemala. Felt from Mexico City to Colombia (USGS)</td>
<td>Felt throughout El Salvador and in Guatemala and Honduras (USGS)</td>
</tr>
</tbody>
</table>

Notes:
COEN: Comité de Emergencia Nacional (National Emergency Committee), Government of El Salvador.
UNDP: United Nations Development Program.
DIGESTYC: Dirección General de Estadísticas y Censos (General Office of Statistics and Census), Government of El Salvador.
Figure 2-6  Housing destruction and landslides (López, UES/EERI).

Figure 2-7  Completely destroyed adobe house, El Salvador (López, UES/EERI).
Figure 2-8 Damaged adobe hospital, El Salvador (López, UES/EERI).

Figure 2-9 Damaged adobe church, Apastapeque, El Salvador.
2.3.2 Housing damage

The UNDP (2001) reported that the estimated losses to the housing sector totalled US$333.8 million, which accounts for 21% of the total damage incurred. Figure 2-10 shows the number of houses affected and details the impacts on the common housing types in El Salvador. Damaged houses are considered houses which were damaged but able to be repaired. Destroyed houses are considered houses which were assessed as uninhabitable (Figure 2-5 to Figure 2-7). The term ‘affected houses’ refers to both damaged and destroyed houses.

![Diagram showing the percentage breakdown of housing types and the number of affected houses]

*Data from DIGESTYC, 1999
All other data from DIGESTYC, 2001

Figure 2-10 Flowchart of affected houses (Data from DIGESTYC, 1999 & 2001).
Various estimates as to the extent of damage have been produced (ECLA/CEPAL, COEN, DIGESTYC, VMVDU). For this thesis, data from the Dirección General de Estadísticas y Censos (DIGESTYC, 1999 and 2001), which is part of the Government of El Salvador (GOES) Ministry of the Economy, has been adopted because a comparison can be made between official pre-earthquake housing statistics (1999) and official post-earthquake damage statistics (2001) to provide an overall picture of the extent of damage relative to pre-existing housing stocks. Analysis of these data sets reveals the following features:

- 20% (276,594) of all houses in El Salvador were affected, consisting of 166,529 (12%) uninhabitable (destroyed) and 110,065 (8%) repairable (damaged).
- 37% of the affected houses were in urban areas and 63% were in rural areas.
- Of the affected houses, 57% (157,070) were made of adobe; of the houses destroyed, 68% (113,469) were made of adobe; and of the houses damaged, 40% (43,601) were made of adobe.
- 8.3% (71,444) of all houses constructed of 'modern' materials (concrete and mixto) were affected, consisting of 20,432 (2.4%) destroyed and 51,012 (5.9%) damaged.
- 39.5% (205,150) of all houses constructed of 'low-cost' materials (adobe, bahareque, lamina, and discarded materials) were affected, consisting of 146,097 (28.1%) destroyed and 59,053 (11.4%) damaged.
- 44% (157,070) of all houses constructed of adobe were affected, consisting of 113,469 (32%) destroyed and 43,601 (12%) damaged. The damage percentages for houses of bahareque were similar (44% affected: 34% destroyed and 10% damaged), although prior to the earthquakes only 5% of houses were made of this material.

Based on these statistics, combined with field observations, the following comments can be made:

- Houses constructed of 'low-cost' materials were more vulnerable than those constructed of 'modern' materials.
- Adobe houses were more prone to complete destruction, rather than repairable damage (Figure 2-11). This suggests the high susceptibility of traditional adobe
houses to sudden and catastrophic failure, thus increasing the risk of fatalities, injury, and loss of possessions. This tendency toward destruction also meant that there was a disproportionately smaller number of damaged adobe houses available for assessment of damage patterns and failure modes.

- Contrary to widespread local opinion, houses constructed of ‘modern’ materials (concrete and mixto) were not ‘immune’ to damage, reaffirming that the type of building material is not the only factor that influences seismic performance.
- Damage in rural areas was more extensive, reflecting greater use of traditional/low-cost materials, reduced access to resources (materials, tools, technical support), and consequent lower standards of quality.

It should be noted that these statistics cover the whole country and that some departments, such as Morazán, La Unión, and Chalatenango, were practically unaffected, while others, notably Cuscatlán, La Paz, San Vicente, and Usulután were severely affected. In the municipality of San Agustín, Usulután, it was reported that 93.5% of all houses were affected in some way, with 83% of all houses destroyed (DIGESTYC, 2001).

Figure 2-11 Completely destroyed adobe house (López, UES/EERI).
2.4  **Damage patterns and failure mechanisms**

This section details the common damage patterns observed in traditional adobe houses affected by earthquakes, such as seen in El Salvador. A brief description of each damage pattern is given, followed by a discussion of the failure mechanisms that cause the damage and a description of the exacerbation features that increase the structural response and stresses that generate failure (Dowling, 2004b). This information is based on the observations of the author in earthquake-affected El Salvador, in combination with some theoretical analysis and a review of literature (e.g. IAEE, 1986; Flores et al, 2001; Tolles et al, 1996 & 2002).

2.4.1  **Vertical cracking at corners**

**Failure mechanism:** large relative displacement between orthogonal walls. Shear walls (subject to in-plane forces) have a low response (displacement). Transverse walls (subject to out-of-plane forces) have little resistance to bending and overturning action, resulting in a larger response. The large relative displacement between shear and transverse walls induces stresses at the connection of the walls (highest stresses at the top). This type of failure is very common because relative response is largest at the wall-wall interface. Cracking occurs when the material strength is exceeded in either shear (Figure 2-12a) or tearing (Figure 2-12b). Oblique seismic forces will induce a combination of both shear and tearing stresses (Figure 2-13). Vertical corner cracking may lead to the post-failure overturning of the wall panel, as detailed below.

**Exacerbation features:**

1) Large roof mass, which is transferred to the walls. The larger mass generates larger inertial forces, which in turn cause larger response (displacement).

2) Poor block arrangement (block-mortar interfaces are planes of weakness whose seismic resistance is lowered by inadequate overlap, thick mortar joints, and the presence of vertical joints).

3) Thin walls, which have a lower relative area of resistance.

4) Long walls, which attract greater out-of-plane response about the vertical axis due to bending, which induces a splitting-crushing cycle at the corners, thus reducing the shear area.
Figure 2-12 Vertical corner cracking due to (a) shear failure; and (b) tearing failure.

Figure 2-13 Typical damage patterns: vertical cracking, corner dislocation, spalling of render (El Salvador).

2.4.2 Vertical cracking and overturning of upper part of wall panel

Failure mechanism: out-of-plane seismic forces inducing bending about the vertical axis (dominantly) and the horizontal axis. Bending about the vertical axis causes a splitting-crushing cycle generating vertical cracks in the upper part of the wall (Figure 2-13). These vertical cracks reduce the resistance to bending about the horizontal axis in the damaged panel, which may result in overturning (Figure 2-14a and Figure 2-15).
Exacerbation features:

1) Poor roof anchoring, where roof beams and trusses often rest directly on the wall, creating zones of high stress concentration (exacerbated by large roof mass). Both localised shear (dominant) and localised bending stresses are generated in these zones.

2) Poor block arrangement (inadequate overlap, thick mortar joints, vertical joints).

3) Long, thin, and slender walls, which are more susceptible to bending about the vertical and horizontal axes.

Figure 2-14 (a) Overturning of upper part of wall; (b) Vertical cracking and overturning of wall panel.

Figure 2-15 Typical damage patterns: inclined cracking, overturning of wall panels, vertical corner cracking, displacement of roof tiles, El Salvador (López, UES/EERI).
2.4.3 Overturning of wall panel

Failure mechanism: out-of-plane seismic force acting on a wall panel with lack of edge restraint on three sides (wall-wall corner connection, wall-roof connection). The lack of fixity of the wall-wall connections may be due to vertical cracking at the corners (Figure 2-12) or the presence of timber columns at the corners that are inadequately attached to the walls (Figure 2-4). In these cases, the wall-foundation interface behaves as a pin connection, which has little resistance to overturning when an out-of-plane force is applied (Figure 2-14b). This type of failure is particularly common for long walls without intermediate lateral restraint, such as boundary walls and garden walls. For buildings, the wall panel will generally overturn outward, due to the outward force exerted by the roof in the absence of an adequate roof diaphragm. This type of failure frequently results in the total collapse of the building, as commonly observed in El Salvador in 2001.

Exacerbation features:

1) Same features as for vertical cracking.
2) Poor conditions at the base of the wall, due to moisture damage and/or poor mortar-block bonding, which have little rotational restraint.

2.4.4 Inclined cracking in walls

Failure mechanisms:

1) Out-of-plane deformation due to bending, causing ‘bulging’, which generates ‘X’ pattern cracking (Figure 2-16a). Adequate restraint on three or four sides is required for this condition.
2) Large in-plane shear forces, which generate tensile stresses at ~45 degrees, thus causing single-direction inclined cracking, or ‘X’ pattern cracking due to cyclic loading (Figure 2-16b).
Exacerbation features:

1) Poor block arrangement.
2) Thin walls, which have a smaller shear area and are more susceptible to bending about the vertical axis.
3) Slender walls (large height-thickness ratio), which are more susceptible to bending about the horizontal axis.
4) Long walls, which attract greater out-of-plane response due to bending.
5) Openings (doors and windows), which induce high stress concentrations, as well as reducing the effective area of the walls. The wall panel between openings acts as a slender column and as such is subject to greater shear stresses, as well as greater compressive stresses due to the weight of the wall above. The influence of openings is further exacerbated when they are located close to other zones of high stress concentration, such as corners and other openings (Figure 2-15 and Figure 2-16b).

![Diagram of Seismic Force and Cracking](image)

(a)  
(b)

Figure 2-16 Inclined cracking in wall due to (a) 'bulging'; and (b) in-plane shear.
2.4.5 Dislocation of corner

Failure mechanism:
Initial failure is due to vertical corner cracking induced by shear or tearing stresses, as described above. The lack of fixity at the corners allows greater out-of-plane displacement of the wall panels, which generates a pounding impact with the orthogonal wall. The top of the wall has a larger response, which causes a greater pounding impact, thus inducing greater stresses that lead to failure (Figure 2-13 and Figure 2-17). An oblique seismic force will cause both orthogonal walls to respond, and the pounding impact will be greater (Figure 2-18a).

Exacerbation features:
1) Same features as for vertical corner cracking.
2) High stress concentrations due to poor roof anchoring and lack of uniform distribution of roof loads.

Figure 2-17 Sequence leading to corner dislocation.
2.4.6 Horizontal cracking in upper section of wall panel

**Failure mechanism:** out-of-plane or in-plane shear failure, which generally occurs when there is a ring beam or the roof is securely attached to the top of the wall. A seismic force creates a relative difference in lateral movement between the wall and roof, which induces high shear stresses leading to cracking when the shear resistance is exceeded (Figure 2-18b).

**Exacerbation features:**

1. Lack of adequate fixity of ring beam to wall.
2. Poorly anchored roof structure, which creates zones of high stress concentration.
3. Poor mortar-block bonding, creating a horizontal plane of weakness. The resistance to shear due to friction is greatly reduced at the top of the wall because there is less applied weight force.

![Image of horizontal cracking](image)

Figure 2-18 (a) Corner dislocation (López, UES/EERI); (b) Horizontal cracking in upper section of wall panel.

2.4.7 Displacement of roof structure

**Failure mechanism:** relative displacement between different components of the roof structure and the walls. The worst-case scenario of this type of failure is the total collapse of the roof structure.
Exacerbation features:

1) Inadequate three-dimensional roof diaphragm, with a lack of triangulation and diagonal bracing.
2) Poor connection to walls and between different roof components. In many cases, the only connection between roof components is afforded by nails, which provide limited resistance to shear.
3) Heavy roof mass, which attracts greater seismic force due to inertia.

2.4.8 Falling roof tiles

Failure mechanisms:

1) No attachment between tiles and roof structure. The roof tiles are simply held by friction, which is easily overcome during seismic events, resulting in the roof tiles sliding off the roof (Figure 2-15).
2) The failure of purlins, which support the tiles, resulting in tiles falling inside the building.

Exacerbation features:

1) Deteriorated purlins, which easily fail when an additional force is applied.

2.4.9 Links to improved design and construction

It should be stressed that in the above assessments the seismic force is generally considered as acting in a single plane. This should not alter an appreciation that during a seismic event any given wall in a particular structure will act as both a transverse wall and a shear wall, being subjected to components of both in-plane and out-of-plane forces (in alternately opposite directions). A wall will influence and be influenced by other structural members, and will experience bending, tensile, compressive and shear stresses. The resulting loads and stress conditions are a complex blend of these factors, and combinations of the above damage patterns are common. Despite this limitation, assessment of simplified scenarios allows a greater understanding of the performance and response of structural elements, which supports the development of concepts of improved seismic design and construction.
2.5 'Lessons Learned Over Time' investigation (2002)

2.5.1 Introduction

Much of the information presented in this section was obtained during field trips by the author to El Salvador in 2002. The field trips formed part of a 'Lessons Learned Over Time' investigation, supported by the Earthquake Engineering Research Institute (EERI), which focused on adobe housing reconstruction after the 2001 El Salvador earthquakes. The resulting report was published by EERI as a dual-language publication (English and Spanish) as part of their Learning From Earthquakes Series (Dowling, 2004a). In addition to an assessment of the context and effects of the earthquakes (as detailed in the previous sections) the investigation considered a number of factors with respect to earthquake reconstruction, and adobe housing in particular. These factors included stakeholder attitudes and involvement, funding, training programs, publications, construction projects, and opportunities and challenges for improved adobe initiatives. A summary of these aspects is presented in this section, although it should be noted that they reflect the situation in El Salvador in 2002, and some aspects may have changed since (see Section 2.7 below for a discussion of the situation in 2005).

2.5.2 Earthquake reconstruction

In the aftermath of the 2001 earthquakes, significant international assistance was offered to El Salvador to support medium and long-term development activities. Medium-term projects focused on the rehabilitation and reconstruction of damaged infrastructure, such as buildings (health and education facilities, houses, churches, public buildings) and lifelines (roads, water and telecommunications networks). Long-term initiatives have included infrastructure development, as well as aspects of disaster mitigation and preparedness, improving institutional capacity, and fostering sustainable development.

Many observers have suggested that the reconstruction process has been slow. At the time of the investigation (2002) many people lacked basic facilities and were still living in temporary shelters. Some severely affected communities had not received any aid assistance since the provision of emergency shelters by the Government of El Salvador (GOES) and relief agencies shortly after the earthquakes. This slow response was
mainly due to the immensity of the housing shortage and the lack of resources to assist those in need. Other reasons include isolation, corruption, and poor municipal and community organisation and co-operation. Many of the same issues were evident four years after the earthquakes, as reported in Section 2.7 below.

Since the earthquakes, many families have utilised both a ‘day house’ and a ‘night house’. The ‘day house’ typically consists of a damaged adobe or bahareque house that has been partially repaired and is used for daytime activities, such as cooking, relaxing, and meeting (Figure 2-19a). The ‘night house’ is generally made of lamina sheeting that has been constructed as temporary accommodation by the GOES, aid agencies, or the residents themselves (Figure 2-19b). These shelters have been dubbed hornos (ovens) by the local folk, due to the oppressive build-up of heat during the day, and as such, they are only used for sleeping at night. The families are distrustful of the ‘day houses’, due to the risk of failure in an earthquake, with a greater risk of injury if the occupants are sleeping within.

(a) ‘Day house’  
(b) ‘Night house’

Figure 2-19 (a) ‘Day house’ (partially repaired adobe house);  
(b) ‘Night house’ (temporary shelter).

The provision of considerable foreign aid to support earthquake recovery activity has led to unprecedented, rapid development in El Salvador. The influx of this assistance has had mixed blessings. On one hand, the aid has brought rapid relief to large numbers of affected people through the provision of basic needs (food, water, shelter and clothing). Additionally, many of the most affected people now have improved living conditions, thus reducing their vulnerability to future disasters. On the other hand, many relief activities fail to consider the long-term implications of fast-paced, short-
term, externally-driven initiatives. Some of the impacts include loss of traditional and local identity, increased reliance on external aid, and reduced capacity for effective, internal response to adversity. These impacts are considered to be inconsistent with the principles of sustainable development. This thesis presents a viable option to address these problems.

2.5.3 Improved adobe initiatives

Since the earthquakes of 2001, there has been a renewed and concentrated focus on construction projects, training programs and publications aimed at promoting the principles of improved seismic resistance of adobe buildings.

A number of initiatives have been undertaken to support and encourage the use of improved adobe in housing construction. Three approaches have emerged:

- **Promotion** to enhance the overall awareness, acceptance and understanding of improved adobe. These activities take place at community and institutional levels to inform key stakeholders (local homeowners, and governmental and non-governmental entities) about the application and value of improved adobe systems.

- **Training programs and skills building projects** to increase the capacity of local artisans in safer adobe construction.

- **Construction projects**, with improved adobe considered a viable alternative to concrete block in agency-supported housing construction projects. In these cases, the main focus is on the provision of adequate housing, but a subsidiary benefit is the informal learning by those who assist in the construction of their homes, under the direction of trained technicians.

Encouragingly, some overseas aid has been provided to support these endeavours (although not on the scale of projects using ‘modern’ materials, such as concrete block). Despite such efforts, the sheer magnitude and wide dispersion of need means that many people have been left unsupported, and are either living in temporary shelters or rebuilding in the traditional manner because they are unaware of simple techniques that can be adopted to improve the seismic resistance of their homes.
The majority of improved adobe projects have been undertaken by the NGO sector. The main improvement systems being advocated and used include:

- Reinforced concrete foundation and plinth (Figure 2-20a).
- Internal vertical reinforcement (bamboo or cane, Figure 2-20b).
- Internal horizontal reinforcement (bamboo, cane or barbed-wire, Figure 2-20b).
- Reinforced soil-cement or concrete ring beam (reinforced with steel or bamboo, Figure 2-21a).
- Corner pilasters/buttresses (Figure 2-20b, Figure 2-21b).
- Light-weight roof (lamina sheeting or micro-cement tiles, Figure 2-21b).

Many of these techniques were adopted in the design and construction of the child-care centre described in Section 2.6 below.

![Figure 2-20](image)

(a) Concrete foundation and plinth; (b) Bamboo reinforcement.
Each project and organisation possessed a number of strengths and limitations, some of which are generalised and presented below.

**Strengths**

Some of the strengths of different projects and organisations include:

- In-depth promotion and training program prior to commencement of construction.
- Construction on beneficiary-owned land in existing communities, with access to services and facilities and little disruption to community structure.
- Model house built first to demonstrate the technique and train builders and beneficiaries (Figure 2-21b and Figure 2-22).
- Beneficiary involvement in aspects of project design, construction and management.
- Strong community interest and participation.
- Working in a community with long-term involvement with the organisation.
- Local organisation, with background and experience in community adobe construction.
- Focus on utilising locally available soil for block fabrication.
- Good consistency of materials.
- Excellent supervision and quality control.
- Strong focus on training and capacity building.
- Good environmental considerations in construction.
- Provision for house extension.
- Improved rain protection offered by four pitch roof.

Limitations

Some of the limitations of different improved adobe housing projects include:

- Problems with community organisation and willingness to participate.
- Slow construction timeframe.
- Significant logistical problems, including transporting water into the community.
- No horizontal reinforcement in the foundation.
- Potential creation of cracks or weaknesses by using a machete to hack voids for reinforcement in blocks.
- Additional complexity of a variety of spacings between vertical reinforcement.
- Vertical bamboo bars protruding from ring beam, and thus exposed to insect attack.
- Only a single room house.
- Minimal resistance to roof uplift provided by the common wall-ring beam attachment systems.
- Open gables mean houses are not secure from intruders (Figure 2-23).
- Exposed wall at rear of house due to single pitch roof.
- Additional complexity and cost of four pitch roof.
- Additional expense and vulnerability to insect attack for houses with an independent roof structure supported by timber columns placed in the ground (Figure 2-23, see discussion in Section 2.7 below).
- Incompatibility and additional cost of cement-based render placed on wall (see discussion in Section 2.7 below).

These factors and other limitations are discussed in greater detail in Section 2.6 below.

The post-earthquake improved adobe activities in El Salvador represent one of the first major attempts for large-scale implementation of such a system in Latin America. As expected, the process has faced various challenges, many of which have been due to the
The pioneering nature of the projects. Challenges have also related to the attitudes and awareness of the general community, the Government of El Salvador (GOES), donors, non-governmental organisations (NGOs) and the media towards adobe specifically and development in general. A significant challenge has been posed by resource limitations, including adequate funding, trained and experienced personnel, and time. The solution to these challenges is a combination of promotion, training, and social and technical initiatives and a strategic approach (discussed further in Chapter 12). Further research and development is an essential component in each of these activities, as described in Chapter 13.

Figure 2-22 Improved adobe house (FUNDASAL project).

Figure 2-23 Improved adobe house (Atlas project).
2.6 Child-care centre construction (2002)

The Expedition El Salvador project involved the construction of a child-care centre using principles of improved adobe design and construction (Figure 2-24). The project was located in the small rural community of El Condadillo, in the municipality of Estanzuelas, situated in the northern part of Usulutan, El Salvador. The design and construction was co-ordinated by the author of this thesis.

The project was a collaborative venture involving individuals, groups and institutions in El Salvador, the United Kingdom and Australia. The main driving force for the project came from a group of enthusiastic engineering students and staff from Imperial College, London, who desired to develop and participate in a practical project to support the reconstruction efforts in post-earthquake El Salvador.

The project had the following key objectives:

- Provision of an important community facility (child-care centre).
- A hands-on community training program in improved adobe construction (Figure 2-25).
- Promotion of improved adobe as a viable and safe construction system.
- Introduction to international and community development for engineering students from Imperial College, London.
- Opportunity to evaluate the effectiveness and practicality of a variety of improved adobe construction techniques.

Figure 2-24 Child-care centre under construction (2002).
The child-care centre consisted of one room (9.3m x 3.6m) plus a covered verandah area (12m x 3.8m). The total project cycle was approximately 11 months (January to December 2002) and the construction component was approximately four months (August to December 2002). The direct construction costs were ~US$5,000, which covered materials, tools, transport and a master builder (Don Adán Rosales). Other in-kind donations included labour, supervision and additional materials, tools and transport. Labour was provided by local community members and nine student volunteers from Imperial College, London.

![Figure 2-25 Community training by Don Adán.](image)

The design of the building combined a variety of existing improved adobe construction systems with some new initiatives (Figure 2-26 and Table 2-4). The design satisfied relevant seismic design criteria outlined in the adobe supplement of the El Salvador building code (RESESCO, 1997) and the International Association of Earthquake Engineering Guidelines for Earthquake-Resistant Non-Engineered Construction (IAEE, 1986). (These guidelines are discussed in greater detail in Chapter 3: Literature review.) The design underwent several changes during the course of the project, due to site limitations, community input, new ideas and lessons learned. The design and construction specifications for the building are presented in Table 2-4.
Figure 2.26 Preliminary design of child-care centre showing plinth, pilasters, door openings and vertical reinforcement.

[Note: internal wall was not included in actual construction, due to community input]

Table 2.4 Child-care centre design and construction specifications.

<table>
<thead>
<tr>
<th>Location</th>
<th>El Condadillo, Estanzuelas, Usulután, El Salvador.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Collaboration</td>
<td>CAFOD UK (in-country construction costs), British Embassy El Salvador, Shell, GESAL, Imperial College London, University of Technology Sydney, Municipality of Estanzuelas + others.</td>
</tr>
<tr>
<td>Costs</td>
<td>Direct construction costs US$5,000 (tools, materials, transport, master builder) + in-kind donations.</td>
</tr>
<tr>
<td>Supervision + labour</td>
<td>Engineer + builder, working with local community members + student volunteers from Imperial College, London.</td>
</tr>
<tr>
<td>Dimensions</td>
<td>Internal area: 9.3m x 3.6m (33.5m²), 1 room, plus covered patio 12m x 3.8m. Total covered area: ~105m²</td>
</tr>
<tr>
<td>Foundations</td>
<td>400mm wide, 250-750mm deep, rocks + cement mortar, reinforced with three 13mm steel bars, with 6mm stirrups every 250mm.</td>
</tr>
<tr>
<td>Plinth</td>
<td>300mm wide, 300mm high, rocks + cement mortar (Figure 2-30b). No reinforcement. Top surface roughened.</td>
</tr>
<tr>
<td>Damp-proof course</td>
<td>Plastic sheeting between plinth + adobe wall.</td>
</tr>
<tr>
<td>Adobe soil</td>
<td>Local + 'imported' soil, approximately 40% sand, 40% tierra blanca¹, 20% clay. Mixed manually. Used for bricks + mortar.</td>
</tr>
</tbody>
</table>

¹ A pyroclastic ash deposit (see Rolo, et al, 2004, for geologic + engineering characterisation).
<table>
<thead>
<tr>
<th>Table 2-4 cont’d</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Blocks</strong></td>
</tr>
<tr>
<td><strong>Mortar joints</strong></td>
</tr>
<tr>
<td><strong>Vertical reinforcement</strong></td>
</tr>
<tr>
<td><strong>Horizontal reinforcement</strong></td>
</tr>
<tr>
<td><strong>Walls</strong></td>
</tr>
<tr>
<td><strong>Pilasters</strong></td>
</tr>
<tr>
<td><strong>Openings</strong></td>
</tr>
<tr>
<td><strong>Lintels</strong></td>
</tr>
<tr>
<td><strong>Ring beam</strong></td>
</tr>
<tr>
<td><strong>Ring beam – wall attachment</strong></td>
</tr>
<tr>
<td><strong>Roof - ring beam attachment</strong></td>
</tr>
<tr>
<td><strong>Roof structure</strong></td>
</tr>
<tr>
<td><strong>Roof cover</strong></td>
</tr>
<tr>
<td><strong>Gables</strong></td>
</tr>
<tr>
<td><strong>Wall finish</strong></td>
</tr>
<tr>
<td><strong>Floor</strong></td>
</tr>
<tr>
<td><strong>Other infrastructure</strong></td>
</tr>
</tbody>
</table>

\(^2\) *Gynernium sagittatum* (Hays, 2001)
Figure 2-27 Site prior to commencement of work.

Figure 2-28 Site clearing and excavation.

Figure 2-29 Block fabrication and drying.

[Note: drainage ditches and plastic covers]
There were a number of limitations which reduced the efficiency and effectiveness of the project, and necessitated changes to the original design and construction plans, which in turn impacted on the cost, complexity and time-frame of the project. These limitations included:

- Site constraints: small, sloped site (10m x 15m, with 2.4m height difference) with limited access and a large dead tree trunk (Figure 2-27 and Figure 2-28).
- Logistics: lack of water and suitable soil on-site.
- Weather: project undertaken during wet season which impacted on site excavation, brick fabrication and wall construction.
- Lack of widespread and consistent community interest and participation, with only a small proportion of the community actively involved.
- Tight timeframe: which increased project pressure, and reduced opportunity for more detailed training.

Notwithstanding these limitations, there was a positive community response to the design and construction techniques presented. In general, the local community were impressed with the solidity of the building and quality of the construction and many workers commented that they would use some of the ideas in future adobe construction. However, it was also acknowledged that the building, as constructed, was too expensive, too complicated and too labour-intensive to be widely adopted. There were a number of factors which contributed to the high cost and complexity of this particular building, which may be reduced in the construction of a family dwelling, including:

- The function of the building (child-care centre) necessitated the use of higher factors of safety in the design.
- Site constraints, as detailed above.
- Uncertainty relating to the characteristics of various materials used, so principles of over-design were utilised.
- The project was designed to demonstrate a best-practice example of improved adobe construction, with participants encouraged to use the systems which matched their personal resource and skill levels.

Some construction photographs are shown in Figure 2-30 to Figure 2-34.
Figure 2-30 Construction of (a) plinth; and (b) adobe walls.

Figure 2-31 Adobe walls under construction, including plinth, pilasters, vertical and horizontal reinforcement, plastic rain covers.
Figure 2-32 Adobe walls under construction.
[Note: slight figure alignment offset due to photograph splicing]

Figure 2-33 Ring beam channel blocks and reinforcement.
[Note: steel pins and wire straps for roof attachment]

Figure 2-34 (a) Roof frame under construction; (b) Roof cover and verandah area.
The project yielded valuable lessons, which relate to both the technical and social aspects of this project (in particular), and improved adobe training and construction projects (in general). These lessons include:

- **Background and preparation**: a thorough and detailed assessment of the project, community and proposed site should be undertaken before detailed planning and design begins. The strengths, weakness, opportunities and threats should be considered and managed.

- **Stakeholder engagement, participation and management**: key stakeholders should be identified and involved in all aspects of the planning and execution of the project. Community needs, interests and availability should be thoroughly considered and incorporated in the design and management of the project. An active and effective ‘social promoter’ should be engaged to facilitate this process. Adequate time and resources should be allocated to effectively manage the involvement and contribution of project participants and collaborators, particularly when many different individuals and organisations are involved.

- **Timely training**: additional time and resources should be allowed for the introduction of new techniques, including a discussion of the rationale (theory), combined with practical application. An accompanying design, construction and maintenance manual should be included in any training program.

- **Weather**: the timing and execution of the project should consider climatic factors, such as wet/dry season, excessive heat, etc.

As mentioned above, the main problem with the project and techniques introduced was the cost and complexity. The following ‘improvement systems’ were particularly problematic:

- **Internal vertical reinforcement** poses significant challenges which make the construction process more labour-intensive and time-consuming. Specially prepared full- and half-size bricks with voids must be used, and carefully fitted around the vertical reinforcement. It is suspected that the inclusion of voids may actually weaken the bricks. If the vertical poles are misaligned (during the pouring of the foundations and/or plinth, or due to the natural and unavoidable curvature of the bamboo/cane) the location of the voids in the bricks are inaccurate, meaning that bricks need to be realigned and/or cut to fit. There are
also concerns about the durability of natural materials, such as cane or bamboo, which cannot be assessed or replaced once they are encased in the wall. If the materials have deteriorated (due to moisture or insect attack or natural deterioration) the resulting void will actually weaken the structure, rather than improve its seismic capacity. The use of internal vertical bamboo or cane reinforcement should be the focus of further research.

- Pilasters (also known as buttresses) are designed to increase the stability of the structure and were constructed externally at each corner of the building and both internally and externally at third points along the long wall (Figure 2-31a and Figure 2-32). The inclusion of pilasters increased the complexity and resource requirements of the project. The preparation of the formwork for the plinth layer became a complex task, requiring multiple right-angled corners and various adjustments (Figure 2-30a). Additional bricks were required and the configuration of the bricks within the pilasters was slightly complicated, which created some initial confusion among the local workers. The inclusion of the pilasters necessitated the expansion of the roof cover in order to maintain the desired minimum overhang of 600mm (Figure 2-34b). This resulted in an increase of 13% in the plan area of the roof, with various elements of the roof structure designed to support cantilever actions of more than one metre. These increases in costs and complexity should be considered in conjunction with the aseismic contribution offered by the pilasters and should be the subject of further research.

Despite these limitations, the majority of the project objectives were met: a safe and secure child-care centre was constructed (Figure 2-35), the local community engaged in a hands-on introduction to improved adobe construction (Figure 2-36), the Imperial College students gained invaluable first-hand exposure to international and community development, and a number of practical lessons were learned about improved adobe design and construction, as described above and in subsequent chapters.

On a return visit in 2005, an inspection of the child-care centre revealed the building to be in good condition (Figure 2-37), despite some minor maintenance issues requiring attention (drainage, latrine, repairs in community-built kitchen annex).
Figure 2-35 Child-care centre in use (S. Oates, Shell International).

Figure 2-36 Working with the local community.

Figure 2-37 Child-care centre (2005).
2.7 El Salvador in 2005

2.7.1 Introduction
During a brief visit to El Salvador in early 2005, the author took the opportunity to reconnect with people, projects and progress. In some respects, the situation was more desperate and confronting than the period directly after the earthquakes. In the aftermath of the earthquakes there was significant reconstruction activity, with the Government of El Salvador (GOES) and a bevy of local and international agencies providing support across the country. There was hope and optimism at the community level that this support would cover all affected families. By 2005 the designated earthquake reconstruction funds were almost completely exhausted, and yet many people continued to live as they did just after the earthquakes -- housed in appalling conditions (shelters made of cardboard, plastic, palm fronds and lamina tin, plus deteriorated 'temporary shelters', and damaged adobe and bahareque houses). The key difference in 2005 was that much of the hope and optimism had disappeared, replaced with resignation and despair. The reality of the situation is that most families that occupied these 'houses' in 2005 will almost certainly continue to live in such precarious conditions for the foreseeable future (Figure 2-38 and Figure 2-39). The key housing statistics (Table 2-5) present an alarming picture across the nation, which highlights the need for a broad-scale, sustainable approach to address the severe housing deficit in El Salvador.

Figure 2-38 Lamina tin house, El Salvador (2005).
2.7.2 Housing statistics

The proportions of housing materials (Table 2-5) provides a sobering picture of the vulnerability of houses in El Salvador. Despite the overall increase in the number of houses made of ‘modern’ materials (concrete and mixto), low-cost materials still account for a major portion of housing. This means that at least 30% of the population (representing almost 500,000 families) live in houses which are highly vulnerable to natural disasters. The large increase in the number of lamina houses shows that the ‘temporary’ shelters provided by the GOES and relief agencies in the immediate aftermath of the earthquakes have become more permanent houses. The decreases reported for adobe and bahareque housing are largely due to the damage wrought by the earthquakes (discussed in Section 2.3 above).

Table 2-5 Key housing statistics for 2000 and 2004 (UNDP, 2005a).

<table>
<thead>
<tr>
<th></th>
<th>2000</th>
<th>% of TOTAL</th>
<th>2004</th>
<th>% of TOTAL</th>
<th>change</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL</td>
<td>1,403,279</td>
<td>100%</td>
<td>1,593,528</td>
<td>100%</td>
<td>-</td>
</tr>
<tr>
<td>Concrete / Mixto</td>
<td>891,082</td>
<td>63.5%</td>
<td>1,128,218</td>
<td>70.8%</td>
<td>+7.3%</td>
</tr>
<tr>
<td>Adobe</td>
<td>355,030</td>
<td>25.3%</td>
<td>246,997</td>
<td>15.5%</td>
<td>-9.8%</td>
</tr>
<tr>
<td>Bahareque</td>
<td>72,971</td>
<td>5.2%</td>
<td>49,399</td>
<td>3.1%</td>
<td>-2.1%</td>
</tr>
<tr>
<td>Timber</td>
<td>36,485</td>
<td>2.6%</td>
<td>33,464</td>
<td>2.1%</td>
<td>-0.5%</td>
</tr>
<tr>
<td>Lamina</td>
<td>26,662</td>
<td>1.9%</td>
<td>119,515</td>
<td>7.5%</td>
<td>+5.6%</td>
</tr>
<tr>
<td>Other</td>
<td>21,049</td>
<td>1.5%</td>
<td>15,935</td>
<td>1.0%</td>
<td>-0.5%</td>
</tr>
<tr>
<td>Modern</td>
<td>891,082</td>
<td>63.5%</td>
<td>1,128,218</td>
<td>70.8%</td>
<td>+7.3%</td>
</tr>
<tr>
<td>Low-cost</td>
<td>512,197</td>
<td>36.5%</td>
<td>465,310</td>
<td>29.2%</td>
<td>-7.3%</td>
</tr>
<tr>
<td>Houses constructed</td>
<td>10,513</td>
<td>0.7%</td>
<td>27,947</td>
<td>1.8%</td>
<td>1.1%</td>
</tr>
<tr>
<td>Qualitative deficit</td>
<td>489,010</td>
<td>34.8%</td>
<td>512,230</td>
<td>32.1%</td>
<td>-2.7%</td>
</tr>
<tr>
<td>Quantitative deficit</td>
<td>36,511</td>
<td>2.6%</td>
<td>32,590</td>
<td>2.1%</td>
<td>-0.5%</td>
</tr>
<tr>
<td>TOTAL deficit</td>
<td>525,521</td>
<td>37.4%</td>
<td>544,820</td>
<td>34.2%</td>
<td>-3.3%</td>
</tr>
</tbody>
</table>

Notes:
- See Table 2-1 for descriptions of housing materials.
- See Section 2.7.3 for definitions of qualitative and quantitative deficits.
2.7.3 Housing deficit

Even prior to the devastating earthquakes of early 2001 there was a severe housing deficit in El Salvador, with almost 40% of the population living in sub-standard housing conditions (UNDP, 2005a). According to the GOES Vice-Ministry of Housing and Urban Development (VMVDU, 2001), there are six basic housing requirements: 1) safe/secure walls, 2) safe/secure roof, 3) hygienic floor, 4) adequate sanitation facilities, 5) access to potable water, and 6) access to electricity. The total housing deficit consists of the quantitative deficit (homes which are deficient in all six of the basic housing requirements) and the qualitative deficit (homes which are deficient in up to five of the six basic housing requirements). This distinction, however, does not give a true indication of the actual housing needs. An unsafe and dilapidated house may be considered in the qualitative deficit category simply because it has access to a latrine, electricity and/or potable water.

In 1999, the Vice-Ministry of Housing and Urban Development noted that 78.1% of the housing deficit was represented by families living below the poverty line and that this level of income was insufficient to finance construction or to obtain access to finance schemes (VMVDU, 1999). This renders these families reliant on external support, or self-directed construction using the available resources. Despite the unprecedented levels of construction in El Salvador in the wake of the earthquakes, the construction activity has been insufficient to cover all of the housing needs. Another solution is urgently required.

Figure 2-39 Precarious housing, El Salvador (2005).
2.7.4 Improved adobe project re-visited

Despite the limited duration of the visit in 2005, there was an opportunity to re-visit one of the improved adobe projects profiled as part of the EERI 'Lessons Learned Over Time' investigation from 2002 (Dowling, 2004a). The project was co-ordinated by Atlas Logistique (a French NGO) and involved the construction of 37 safer adobe houses in a small community, 19 de Junio, San Vicente. Three years later most of the 'new' adobe houses in the community were in a very poor state, with deterioration of timber elements and wall render, and some significant cracking in the walls. The residents were disappointed and upset at the state of their homes, but with Atlas Logistique no longer present in El Salvador, there was little they felt able to do. The main deficiencies were:

- Significant cracking above lintels (most probably due to settlement, Figure 2-40);
- Termite attack of timber elements, which included window and door frames (Figure 2-42b), and most critically, timber columns supporting the roof (there was no connection between the wall and roof, Figure 2-41); and
- Spalling of plaster (due to the use of a cement-based render/plaster, which is incompatible with the adobe bricks, Figure 2-42).

![Figure 2-40](image) (a) Before (2002) (b) After (2005)

*Figure 2-40* (a) Lintel and wall, with ‘keys’ for render; (b) Cracking above lintel, and deteriorated timber door frame.
These deficiencies highlight some of the problems associated with the design and construction approach taken by Atlas Logistique. Considering that a light-weight roof was used, there appears to be little sense in having a separate timber structure to support the roof. This is especially problematic if the timber columns are not adequately protected from insect attack. In addition to the incompatibility of cement-based render and adobe walls, the cement is an unnecessary expense. An attractive finish could have been easily achieved using mud, sand and lime. Above all else, the deficiencies observed highlight the ineffectiveness of the project as a capacity-building program; none of the beneficiaries demonstrated the desire or capacity to undertake even simple maintenance on their homes. The project represents an unsustainable approach to aid.
2.8 Challenges for improved adobe

Based on these experiences and observations in El Salvador, coupled with a review of relevant literature, a number of challenges associated with the widespread implementation of improved adobe have been identified (Dowling, 2004a). Some of these challenges relate specifically to improved adobe, others are particular to the social, cultural and institutional features of a given region (in this case El Salvador), and other challenges can be considered common at an international level. Some of these key challenges include:

- Lack of confidence in adobe as an adequate construction material in areas of high seismicity. This attitude has been promulgated by the generally poor performance of traditional adobe buildings in earthquakes.
- Lack of widespread promotion and support of improved adobe. This can be attributed to a number of factors, including limited institutional resources, traditional attitudes and a large and dispersed target population.
- Lack of experience and skill in aseismic design and construction.
- Lack of individual and family resources, which restrict skills development and the use of better materials and techniques.
- Resistance to change. New initiatives are often viewed with suspicion, particularly if tangible and practical examples are not available.
- Competition from other materials and construction forms which are perceived to be modern and superior. Adobe is often perceived as indicative of poverty, whereas ‘modern’ materials are symbolic of progress and prosperity.
- Increasing levels of aid dependency. The prevalence of agency-supported housing projects whereby beneficiaries are given a new house often increases the expectation of outright aid assistance. This means that programs which focus on skills building and the promotion of local, community-based construction may face low levels of interest and support.

- In many regions there is a ‘short memory’ relating to the impacts of natural disasters. In El Salvador some observers have commented that despite a long history of natural disasters, many Salvadoran people and institutions have failed to take appropriate actions to reduce vulnerability and prepare for future events (Wisner, 2001; UNDP, 2001; Bommer et al, 2002; local observers, 2002).
• Lack of commitment to long-term programs of risk mitigation and disaster preparedness. This is commonly due to varying levels of institutional consistency and competence (UNDP, 2001).

• Uncertainty relating to some aspects of current improved adobe construction, coupled with the poor outcomes of some projects (e.g. the Atlas Logistique project described above). These uncertainties and poor outcomes may reduce the credibility of improved adobe from the perspective of potential donors, institutions and beneficiaries.

• Lack of awareness of the importance of maintenance. Adobe is a naturally deteriorating material which requires periodic maintenance to preserve structural integrity. In each earthquake the most affected adobe houses are generally older and poorly maintained buildings which have significantly reduced seismic capacity.

In each of these cases a single solution does not exist, but rather a combination of promotion, training, social and technical initiatives is required. Various opportunities and recommendations to offset these challenges are discussed in this thesis.
2.9 Summary

This chapter presented a case study on El Salvador covering the following components:

- El Salvador profile, which introduced the general context of El Salvador, including a discussion of the history and use of adobe housing, as well as some of the common features and deficiencies in traditional adobe houses.
- The 2001 El Salvador earthquakes, which detailed some of the general features and damage statistics of the earthquakes, with a particular focus on the impacts on adobe housing.
- Damage patterns and failure mechanisms, which described the common damages to adobe houses subjected to earthquakes, including a discussion on the failure mechanisms and exacerbation features.
- ‘Lessons learned over time’ investigation, which included a review of reconstruction activities one year after the earthquakes. A number of improved adobe initiatives (promotion, training and construction projects) were presented, including a discussion of some of the strengths and limitations of these initiatives.
- Improved-adobe child-care centre, which described the project design and construction, lessons learned and outcomes of a small project co-ordinated by the author.
- El Salvador in 2005, which described the general state of housing reconstruction and the vast housing deficit four years after the earthquakes. A visit to an improved adobe construction project revealed some significant deficiencies in the practices used.
- Challenges for improved adobe, which presented some of the key obstacles facing the widespread application of improved adobe.

Research undertaken in El Salvador has been a key component of this overall research project. The experiences have helped to develop an understanding of the core issues related to adobe housing, from both technical and practical perspectives. The field research has highlighted the importance of developing an appropriate solution, which is within the resource means (cost, skills, materials, tools) of the intended users. Such a solution has the opportunity to significantly reduce the vulnerability of adobe houses to earthquakes in El Salvador, and many other parts of the world.
Chapter 3. Literature review

3 LITERATURE REVIEW

3.1 Introduction

This literature review focuses on two main aspects:

- Research related to evaluating and improving the seismic performance of adobe structures, with most attention given to shake table testing of adobe models.
- Review of existing improved adobe design and construction manuals and guidelines.

In the last 30 years there have been a number of research projects focused on evaluating and improving the earthquake performance of adobe structures. The most notable research has been undertaken in Peru, Mexico, the U.S.A. and Colombia. This research has included a variety of material property tests, static tests of adobe wall panels and the shake table testing of adobe wall units and model houses. This chapter focuses mainly on the shake table testing, and presents the testing approaches, specimen specifications and key outcomes of seismic adobe research at each location. The results of this research have been used to develop a number of design and construction manuals and guidelines, some of which are reviewed later in this chapter.

This literature review covers both formal and informal literature published in English and Spanish. There is evidence to suggest that meaningful adobe research has also been undertaken in other parts of the world (e.g. France, India, Iran and Japan), however, such information is not widely available, at least in English or Spanish.

Despite the significant advances in adobe research in recent years, the amount of research focused on non-engineered constructions has been relatively small compared with the extensive research on so-called modern materials (e.g. steel, concrete, composites) and advanced systems (e.g. passive and active control). Due to the relatively limited amount of adobe-focused research, literature relating to conventional fired-brick masonry was also considered as part of this thesis. Valuable information and ideas with respect to the experimental testing and analysis of masonry were obtained from a variety of sources (e.g. Bruneau, 1994; Tomazevic et al., 1996; Doherty et al., 2002; Lam et al., 2003; Alcocer et al., 2004; Reneckis et al., 2004; Griffith et al., 2004;
Vaculik et al, 2004; Lawrence et al, 2005). A detailed review of these sources is beyond the scope of this thesis.

In addition, various resources were reviewed to support other components of the research presented throughout this thesis. These include:

- Material property testing of masonry prisms (Chapter 4).
- Experimental Modal Testing and Analysis, EMTA (Chapters 8 and 10, and Appendix D).
- Implementation (Chapter 12).
- Finite Element Modelling (FEM) (Chapter 13, Further research)
3.2 PUCP, Peru

The Pontificia Universidad Católica del Perú (PUCP) has been a key player in adobe research since the 1970s. Research has included material property testing (soils, bricks and prisms), static testing of wall panels and model houses using a reaction frame and a tilting table, and dynamic testing of wall panels and model houses using a shake table. The results of these tests have been incorporated into a number of the improved adobe guidelines and manuals, discussed in Section 3.7 below. The key stages and research findings are presented in this section, with greatest emphasis on the shake table testing.

3.2.1 Material property tests

Research has included a number of tests on soils, individual bricks and multi-brick assemblages (Vargas and Ottazzi, 1981; Vargas et al, 1984). Outcomes have included:

- Enhanced understanding of the properties of soil and the ideal conditions for improved adobe bricks and mortar. Analysis has included appropriate proportions of sand, silt and clay, the use of additives, and the ideal moisture content of adobe mud.
- A number of simple field tests to assess the suitability of soil for adobe bricks and mud mortars.
- Data on the properties of bricks and prisms, including compressive, tensile and shear strength, considering the effects of different mortar (mud, mud + straw, sand + cement), slenderness ratios, and age of prisms at testing.

3.2.2 Static tests of wall panels

Various static tests of adobe wall panels have been undertaken, including:

- Out-of-plane flexural tests of wall panels (Figure 3-1a) with different reinforcement, including timber ring beams and horizontal, vertical and diagonal cane (Vargas and Ottazzi, 1981).
- In-plane shear tests of wall panels, considering different pre-compressive loads, slenderness ratios and reinforcement (e.g. horizontal and vertical cane and timber) (Vargas and Ottazzi, 1981).
• In-plane shear tests of I-shaped wall panels (cyclic loading parallel to the longitudinal wall, which included a window penetration). Different reinforcement systems have been tested, including internal vertical and horizontal cane, internal horizontal plastic mesh, external wire mesh strips, external ‘geogrid’ polymer mesh (Figure 3-1b) and internal vertical steel reinforcing bars at the corners (Blondet et al., 2005).

Figure 3-1 Static tests of adobe wall panels: a) out-of-plane flexure; b) in-plane shear (PUCP).

Figure 3-2 Tilting table tests of model adobe houses: (a) unreinforced; (b) reinforced (PUCP).
3.2.3 Tilting table tests of model adobe houses

In the 1970s eight model adobe houses were subjected to a constant acceleration (due to gravity) when tilted on a tilting table (Figure 3-2). Damage patterns were consistent with real adobe houses subjected to severe earthquakes, and a number of reinforcement systems (using timber and wire) improved the capacity of the models to resist such loading (Vargas and Ottazzi, 1981). The results have been an important forerunner to the shake table testing undertaken at PUCP, detailed below.

3.2.4 Dynamic tests

General

In 1979, PUCP acquired an earthquake simulation table and commenced an extensive program of dynamic testing of wall panels and model houses. The PUCP shake table is 4 x 4m and operates in one horizontal direction (uni-axial). The time history from the May 1970 Huaraz earthquake in Peru has been used for most simulations. Intensity-scaling of the ground motion has been undertaken to assess the behaviour of the structures under different dynamic load conditions. For the highest intensity simulations a peak shake table displacement ($D_o$) of 130mm, and peak shake table acceleration ($A_o$) of 1.2g, have been reported (Blondet et al, 2006a). These values correspond to a shake table frequency in the region of only 1.5Hz, which is at the lower end of ground motion frequencies. There does not appear to be a consistent pattern to the sequence of simulations undertaken during different testing programs at PUCP. The following simulation sequences have been reported:

- $D_o = 10, 15, 40, 60, 80, 100, 120$ and $140$mm (Ottazzi et al, 1989).
- $D_o = 30, 60, 80, 100, 120$ and $140$mm (Matos, 1997; Zegarra et al, 1999).
- $D_o = 30, 80$ and $130$mm (Blondet et al, 2006a).
- $D_o = 15, 30, 60, 80, 100, 120$ and $120$mm (Blondet et al, 2006a).

This variability of test sequences and parameters makes it difficult to make clear comparisons between specimens. Time-scaling of the input motion has not been undertaken, and there is no information in the literature detailing the natural frequency of the specimens, the dominant frequency of the shake table motion, or the relationship
between the two. This suggests that dynamic similitude has not been considered (the importance of which is discussed later in this thesis).

The focus of testing at PUCP has been the delay and/or prevention of collapse of adobe structures. Emphasis has been on the qualitative performance (observations) rather than the quantitative response (displacement, acceleration, dynamic amplification, etc). There is no reported data for the low intensity simulations when much valuable information about the pre-cracked, linear elastic behaviour of the structure can be determined.

**U-shaped wall panels**

A number of shake table tests of u-shaped adobe wall panels have been undertaken at PUCP (Zegarra *et al.*, 1999; Quiun *et al.*, 2005). The specimens have consisted of one long wall (3.30m in length) and two short side ‘wing’ walls (0.92m in length). Walls were 1.80m high and 0.28m thick. The panels were tested with the long wall perpendicular to the direction of motion (Figure 3-3). No restraint was applied to the ends of the ‘wing’ walls (as described in Chapter 6).

U-shaped wall panels with the following systems have been tested at PUCP:

- **Wall 1**: Traditional, unreinforced.
- **Wall 2**: Timber boards placed in a horizontal channel near the top of the wall, attached with staples passing through the wall and covered with a sand-cement render.
- **Wall 3**: Rope placed in a horizontal channel near the top of the wall, connected with wire passing through the wall and covered with a sand-cement render.
- **Wall 4**: Chicken wire mesh placed in vertical and horizontal strips at the corners and along the top part of the wall, attached with nails and covered with a sand-cement render.
- **Wall 5**: Welded mesh placed in vertical and horizontal strips at one corner, and chicken wire mesh placed in vertical and horizontal strips at the other corner, attached with nails and covered with a sand-cement render.
- **Wall 6**: Welded mesh placed in a horizontal strip at the top of one corner, attached with plywood brackets. The other corner was reinforced with wire
in a mesh-like configuration, attached with nails. All mesh was covered with a sand-cement render.

Zegarra et al (1999) and Quiñon et al (2005) noted that each reinforcement intervention provided some improvement to the walls, with collapse delayed. The timber boards and rope (Walls 2 and 3) provided protection at the location of the reinforcement but contributed only a little to the overall performance of the structure. The wall with welded mesh vertically and horizontally (Wall 5) performed the best, closely followed by Wall 4 (chicken wire mesh vertically and horizontally). The modes of failure of the walls, however, were not all consistent with the common failure patterns exhibited by adobe houses subjected to earthquakes (as shown in Chapter 2). In a number of cases reported for the PUCP testing there was extensive damage to the shear walls, with diagonal and horizontal cracking consistent with rotation (overturning) of the long wall (Figure 3-3a). It is apparent that to more accurately simulate the boundary conditions of a real house it is necessary to add some sort of restraint to the ‘wing’ wall to prevent overturning. Such a restraint would concentrate the seismic forces on the corners and out-of-plane long wall, as discussed in Chapter 6.

The other feature of note is the abundant use of cement, which has been subsequently shown to increase the risk of brittle and sudden collapse (Blondet et al, 2005). Furthermore, the use of cement adds considerable cost to the retrofit-strengthening of an adobe structure.

![Figure 3-3 U-shaped wall panels (PUCP).](image-url)
Model houses

A number of shake table tests of adobe houses have been undertaken at PUCP. Figure 3-4 shows the typical plan and elevation details of the model houses. The houses can most accurately be described as 3:4 scale. Some houses were tested with a flat roof, others with a double-pitched roof, and others with a slightly inclined roof (Figure 3-4 and Figure 3-5). It should be noted that none of the shear walls contained a door opening, the vulnerability of which is demonstrated in Chapters 9 and 10 of this thesis.

![Figure 3-4 Plan and elevation of adobe houses tested at PUCP (Blondet et al, 2006b)](image)

[Red arrows indicate direction of motion]

![Figure 3-5 Model adobe houses on shake table at PUCP.](image)

Table 3-1 shows the specifications and reported results for the model adobe house testing undertaken at PUCP.
<table>
<thead>
<tr>
<th>Specifications</th>
<th>Reported results</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traditional, unreinforced.</td>
<td>Exhibited typical failure patterns: vertical corner cracking, separation of walls leading to collapse.</td>
<td>Ottazzi et al, 1988</td>
</tr>
<tr>
<td>Unreinforced with improved quality adobe bricks and construction.</td>
<td>“Increased strength and rigidity of wall prior to cracking caused by a severe earthquake”, then common failure patterns and collapse.</td>
<td>Ottazzi et al, 1988</td>
</tr>
<tr>
<td>Internal vertical and horizontal cane reinforcement (Figure 3-6b). Seven houses were tested with different quality adobe bricks and reinforcement configuration, with/without a ring beam (eucalypt branches) and with/without windows.</td>
<td>“Horizontal and vertical cane reinforcement, combined with a solid ring beam impeded separation of walls at the corners, maintaining the integrity of the structure after cracking... their presence is indispensable to prevent collapse even during a severe seismic event.”* Vertical reinforcement alone is not sufficient to prevent collapse; horizontal reinforcement is also required.</td>
<td>Ottazzi et al, 1988</td>
</tr>
<tr>
<td>Confined adobe, with reinforced concrete columns at each corner and reinforced concrete ring beam.</td>
<td>Minor to moderate cracking in shear and out-of-plane adobe walls. No cracking in concrete elements</td>
<td>Matos, 1997</td>
</tr>
<tr>
<td>Welded mesh and chicken wire mesh placed in vertical and horizontal strips at the corners and along the top part of the walls, attached with nails and covered with a sand-cement render.</td>
<td>“Enhanced the seismic security of the houses... increased the strength of the walls... controlled their displacement and postponed collapse.”* Vargas et al (2005) also noted the incompatibility between the cement-based render and the adobe masonry, which failed to work together.</td>
<td>Zegarra et al, 1999</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vargas et al, 2005</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Quiun et al, 2005</td>
</tr>
<tr>
<td>Specifications</td>
<td>Reported results</td>
<td>References</td>
</tr>
<tr>
<td>-------------------------------------------------------------------------------</td>
<td>-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
<td>------------------------------</td>
</tr>
<tr>
<td>Welded mesh placed in vertical and horizontal strips at the corners and along the top part of the walls, attached with nails and through-wall ties, and covered with a sand-cement render. Continuous reinforced concrete ring beam on top of walls.</td>
<td>&quot;Moderate damages during a severe earthquake simulation... on the verge of collapse after a 'catastrophic' earthquake, due to shear separation of the concrete ring beam at the corners.&quot; Quiun et al (2005) also noted that performance could be improved by including connectors and shear pins at the base (connecting the mesh and foundation) and the top (connecting the mesh and ring beam).</td>
<td>Quiun et al, 2005</td>
</tr>
<tr>
<td>External polymer mesh reinforcement (industrial 'geogrid' mesh or 'soft plastic construction fencing mesh') – full/partial wrapping, attached with polypropylene ties through the wall (Figure 3-5, Figure 3-6b, Figure 3-7a and Figure 3-8b). One half of each house covered with mud render.</td>
<td>Collapse prevented in all cases. The polymer mesh was &quot;compatible with the adobe walls, working together even for high intensity simulations.&quot; The mesh &quot;starts working after the walls have cracked.&quot; The mud plaster &quot;increases the shear strength and stiffness of the walls... and protects the polymer mesh from ultraviolet radiation.&quot; &quot;It is not necessary to completely cover the walls with the polymer mesh... placing the mesh in critical locations should be sufficient to prevent collapse.&quot; The soft plastic mesh was &quot;deformed and broken in several places, indicating that the amount provided was barely adequate.&quot;</td>
<td>Blondet et al, 2005 Blondet et al, 2006b</td>
</tr>
<tr>
<td>External vertical cane and external horizontal jute twine, attached with jute twine ties through the wall, and partially covered with mud render (Figure 3-7b).</td>
<td>Elastic behaviour up to moderate intensity simulations, then vertical corner cracking. Spalling of render. Large displacement of transverse walls. Partial and total collapse prevented. Walls with mud render were less damaged (due to increased stiffness and strength).</td>
<td>Torrealva &amp; Acero, 2005</td>
</tr>
</tbody>
</table>

Notes: * translated from the original Spanish document by the author of this thesis.
Figure 3-6 Preparation of model adobe houses at PUCP: (a) internal cane reinforcement; (b) external polymer mesh reinforcement.

Figure 3-7 Preparation of model adobe houses at PUCP: (a) external plastic construction mesh; (b) external cane and jute twine reinforcement.

Figure 3-8 Damaged model adobe houses at PUCP: (a) unreinforced; (b) reinforced with polymer mesh.
The research at PUCP has made a significant contribution to the understanding of the behaviour of adobe structures when subjected to earthquake forces. Furthermore, the research has yielded a number of effective reinforcement systems to delay and/or prevent serious damage and collapse of adobe structures, even during high intensity ground motion. A number of these solutions have been included in a variety of improved adobe guidelines and manuals (as described in Section 3.7 below). Some of these systems have been incorporated in construction projects (e.g. in El Salvador in the aftermath of the 2001 earthquakes, as described in Chapter 2) and some pilot retrofit-strengthening projects (Vargas et al, 2005; Blondet et al, 2005). Large-scale implementation of the solutions, however, has not occurred. It would seem that this is due mainly to the cost and complexity of the proposed reinforcement systems, which put them beyond the resource and skill levels of the rural poor in developing countries. The exception to this is the system of external vertical cane and horizontal jute twine reinforcement (Figure 3-7b), which is a similar system to the one developed as part of this thesis. [Testing of this system at PUCP was done several months after the first externally reinforced specimen (3E) was tested at UTS.] The other promising option is the use of the soft plastic construction mesh (Figure 3-7a), which is reported to be cheap, although its availability in rural areas is unknown, and its susceptibility to deterioration when exposed to ultraviolet radiation is an important consideration.

The main limitations of the testing undertaken at PUCP include:

- Lack of dynamic similitude between different specimens. This is evident in the variety of testing sequences (described above) and the lack of time scaling of the input shake table motion (the importance of which is described in Chapters 5 and 6 of this thesis).
- The lack of quantitative data obtained and reported. The focus of testing at PUCP has been on qualitative results and performance, however quantitative data is useful to understand the pre- and post-cracked behaviour of adobe structures. Furthermore, this information is important for the development and validation of finite element models (discussed in Chapter 13).
3.3 **UNAM, Mexico**

Seismic adobe research at the *Universidad Nacional Autónoma de México* (UNAM) began in the 1970s with the aim of “establishing bases for the evaluation of the safety of existing adobe buildings in the country and for the assessment of the effectiveness of different strengthening procedures” (Meli, 1984). The research has included:

- A survey of adobe construction around the country, including a review of behaviour of adobe buildings in recent earthquakes.
- Material property tests of adobe bricks and prisms from different regions in Mexico (discussed in Chapter 4).
- A series of static tests of adobe walls with different reinforcement configurations. In-plane horizontal cyclic loading was applied to unreinforced and mesh-reinforced adobe walls. Results confirmed the improvement in lateral deformation capacity obtained with welded mesh and chicken wire mesh reinforcement (Flores *et al.*, 2001).
- Shake table testing of seven model adobe houses (1:2.5 scale) with different reinforcement systems (Figure 3-9 and discussed below).
- Analytical studies of adobe structures using finite element methods (linear elastic), validated with experimental test results (Bazán *et al.*, 1980). This is discussed in Chapter 13: Further research.

![Diagram and photograph of model houses](image)

**Figure 3-9** Dimensions and layout of model houses, UNAM (Flores *et al.*, 2001).

Dynamic testing at UNAM was undertaken on a 2.4m x 4.5m uni-axial shake table. The dimensions and layout of the model houses are shown in Figure 3-9, and the
specifications of each model are presented in Table 3-3. A number of simulations were undertaken using input ground motions from three earthquakes: El Centro, U.S.A. (1940), Managua, Nicaragua (1972) and Oaxaca, Mexico (1973). Modifications to the input time histories were undertaken to account for the dimensional scaling of the specimens, and the maximum displacement capacity of the shake table ($\pm 2.54 \text{cm}$). The records were time-scaled by a factor of 1/2.5 (matching the geometrical scaling of the specimens). Table 3-2 shows the peak accelerations and displacements for the original and modified earthquake records. Due to shake table limitations only 90\% of the intensity (displacement-based) of the El Centro and Managua earthquakes was reproducible, whereas up to 9 times the intensity of the Oaxaca earthquake was possible, due to its small displacement. For the first three models all three records were used, at varying intensities. It was noted that the Oaxaca record had a greater capacity to induce damage in the models, so for the remaining models that record only was used.

Table 3-2 Peak accelerations and displacements for the original and modified earthquake records, UNAM (Hernández et al, 1981).

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Acceleration (g)</th>
<th>Displacement (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Original</td>
<td>Modified</td>
</tr>
<tr>
<td>El Centro</td>
<td>0.35</td>
<td>0.85</td>
</tr>
<tr>
<td>Managua</td>
<td>0.37</td>
<td>0.85</td>
</tr>
<tr>
<td>Oaxaca</td>
<td>0.20</td>
<td>0.50</td>
</tr>
</tbody>
</table>

Initially, each model was tested with a timber roof frame and tile roof covering. The frame and tiles were then removed and masses were added simulating a light roof ($50 \text{kg/m}^2 = 0.49 \text{kPa}$) and a heavy roof ($500 \text{kg/m}^2 = 4.9 \text{kPa}$). Six accelerometers were used to record the dynamic response at different locations. After each simulation a free vibration test was performed to measure the period and damping of each wall. These results were used to determine the level of damage suffered by the models, with a loss of stiffness and increase in damping indicative of damage. These results were used to develop a relationship between the intensity of the simulation and the damage experienced. ‘First damage’ was identified by a clear increase in natural period (decrease in natural frequency), and ‘major damage’ corresponded to a 50\% increase in
natural period (Meli, 1984). These damage designations were then compared with results from the unreinforced model houses (models #1 and #4) to obtain a relative intensity at first and major damage (Table 3-3). Published results have tended to focus on this relative performance and little qualitative observational information has been presented.

Table 3-3 Model adobe house testing at UNAM: specifications and results.

<table>
<thead>
<tr>
<th>Model$^1$</th>
<th>Specifications$^2$</th>
<th>Natural frequency$^3$ (Hz)</th>
<th>Relative intensity$^4$</th>
<th>First damage</th>
<th>Major damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1</td>
<td>Traditional, unreinforced house.</td>
<td>12.6</td>
<td></td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>#2</td>
<td>Reinforced concrete ring (collar) beam (Figure 3-10a).</td>
<td>14.7</td>
<td></td>
<td>2.0</td>
<td>2.7</td>
</tr>
<tr>
<td>#3</td>
<td>Repair and retrofit of damaged model #1, with welded mesh covered with concrete render; + small reinforced concrete ring beam.</td>
<td>12.5</td>
<td>&gt;2.0</td>
<td>&gt;3.0</td>
<td></td>
</tr>
<tr>
<td>#4</td>
<td>Traditional, unreinforced house.</td>
<td>13.9</td>
<td></td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>#5</td>
<td>Repair and retrofit of damaged model #4, with horizontal ties (steel bars) at the top of the wall (Figure 3-10b).</td>
<td>9.1</td>
<td></td>
<td>1.7</td>
<td>2.1</td>
</tr>
<tr>
<td>#6$^5$</td>
<td>Interior walls.</td>
<td>N.R.$^*$</td>
<td>1.5</td>
<td></td>
<td>1.4</td>
</tr>
<tr>
<td>#7$^5$</td>
<td>Horizontal and vertical ties.</td>
<td>N.R.$^*$</td>
<td>2.0</td>
<td></td>
<td>2.7</td>
</tr>
</tbody>
</table>

Notes:
1) Direction of motion perpendicular to long walls (Figure 3-9).
3) Natural frequency prior to shake table testing (Meli, 1984; Flores et al, 2001).
4) “Intensities of an accelerogram recorded at Oaxaca (1979) needed to produce a given level of damage, divided by those corresponding to the unreinforced adobe models” (Meli, 1984).
5) For models #6 and #7 little information has been reported. N.R.$^*$ = not reported.
Figure 3-10 Reinforcement details: (a) Model #2; (b) Model #5 (Flores et al, 2001).

Published reports emphasised that test results “cannot be directly applied to making quantitative statements about the seismic strength of actual houses” (Meli, 1984), due to the limitations of scale model testing in a single horizontal direction. The reports do, however, suggest that qualitative statements can be made about the relative effectiveness of the different strengthening methods tested. They confirmed that each system tested provided a significant improvement in seismic capacity, but that “only those that included both horizontal and vertical reinforcement resulted in a radical improvement of the behaviour” (Meli, 1984).

The main limitations of the tests conducted at UNAM include:

- All of the improvement systems tested use modern materials (concrete and steel) and are thus relatively expensive, and most probably beyond the resource means of the rural poor.
- There is a lack of detailed qualitative and quantitative data reported for the testing.
3.4 **Stanford University, U.S.A.**

Adobe research at Stanford University, California commenced in the 1980s, as the focus of a PhD dissertation titled ‘Seismic studies on small-scale models of adobe houses’ (Tolles and Krawinkler, 1990). The main component of the research was the shake table testing of six model adobe houses (1:6 scale) to “study the dynamic behaviour of low strength masonry (LSM) buildings and to evaluate the relative benefits of simple structural improvement techniques designed to prevent collapse during strong ground motion” (Tolles and Krawinkler, 1990). Figure 3-11 shows the configuration and specifications of the model houses. Table 3-4 shows the specifications and reported results for the model adobe houses tested at Stanford University.

Testing was undertaken on a uni-axial shake table, using the N21E component of the July 1952 Taft earthquake (California). The record chosen possessed a significant high frequency content, which was suitable for the dynamic testing of stiff adobe structures. Each model was subjected to at least two low intensity simulations to determine the pre-cracked elastic response. Amplitude was then increased by approximately 30% for each subsequent test. The mid-span top of each wall was instrumented with accelerometers and LVDT displacement sensors. Results were presented for the pre-cracked, elastic state and the post-cracked post-elastic state of each model house. Significant qualitative and quantitative information was reported for each simulation.

Experimental Modal Analysis (EMA) was undertaken prior to testing to determine the mode shapes and natural frequencies of each specimen. This is discussed in greater detail in Chapters 8 and 10. The results of the EMA and static tests of the model houses prior to testing were used to develop and validate a Finite Element (FE) model of the houses. The FE analysis was limited to static and modal analyses of the model buildings. Tolles and Krawinkler (1990) identified the potential to use the FE models to “study the elastic response of these buildings to earthquake motions.”

The Stanford University researchers also undertook a number of static tests of adobe prisms, which are discussed in greater detail in Chapter 4.
Figure 3-11 Model adobe houses tested at Stanford University
Table 3-4  Model adobe house testing at Stanford University: specifications and results (Tolles and Krawinkler, 1988 & 1990).

<table>
<thead>
<tr>
<th>Model</th>
<th>Specifications</th>
<th>Reported results</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1</td>
<td>Traditional, unreinforced house, with lead weights to simulate a heavy earthen roof.</td>
<td>Initial cracking during Test #4 (0.23g). Collapse of E, S + W walls during Test #7 (0.42g).</td>
</tr>
<tr>
<td>#2</td>
<td>Traditional, unreinforced house, with no roof load.</td>
<td>Collapse of W wall, severe cracking in E + S walls during Test #7 (0.42g). No observed damage prior to Test #7.</td>
</tr>
<tr>
<td>#3</td>
<td>Same as model house #1, but with roof beams securely anchored to the tops of the walls.</td>
<td>Substantial damage during Test #4 (0.23g). Collapse of S wall, severe cracking in E + W walls during Test #7 (0.42g).</td>
</tr>
<tr>
<td>#4</td>
<td>Same as model house #1, but with a timber ring beam added to the tops of the walls.</td>
<td>Initial cracking during Test #5 (0.26g). Total/partial collapse of E, S + W walls during Test #9 (0.54g).</td>
</tr>
<tr>
<td>#5</td>
<td>A previously damaged model house, with large cracks filled with adobe mortar, and anchored roof beams and ties added.</td>
<td>Initial cracking during Test #5 (0.26g). Partial collapse of E + W walls during Test #9 (0.54g).</td>
</tr>
<tr>
<td>#6</td>
<td>Same as model house #3, but tested in a different orientation (parallel to the long axis).</td>
<td>Initial cracking during Test #5 (0.26g). Verge of collapse of E + S walls during Test #7 (0.42g).</td>
</tr>
</tbody>
</table>
In addition to the research findings relating to the material property testing and EMA (discussed in Chapters 4, 8 and 10), the major outcomes of the research, as reported by Tolles and Krawinkler (1990), include:

- **Linear-elastic modelling techniques can NOT predict the collapse load of a building.**
- **The dynamic, post-elastic behaviour of LSM buildings must be understood in order to design proper details that help prevent collapse.**
- **Construction details that provide structural continuity can greatly improve the post-elastic behaviour of LSM buildings.**
- **It is most important to use structural collapse as the criterion for evaluating a suggested improvement or repair technique.**
- **One of the most important aspects of an improvement technique is preventing overturning of the walls.**
- **Bond beams are the best single improvement technique tested in this program.**
- **Securely anchored roof beams improve the out-of-plane behaviour of the walls.**
- **The behaviour of in-plane walls can be improved by tying the top of the wall together with a continuous plate anchored to the top of the wall.**
- **Pre-existing damage may have little effect on the collapse level of a building if the building is properly detailed.**

The main limitations of the research include:

- The use of small models (1:6 scale) which neglect the effects of gravity, and make it difficult to accurately represent the important mortar-brick interface.
- The testing program investigated only a small number of reinforcement systems, mostly related to the connection between the walls and the roof.

Notwithstanding these limitations, the research at Stanford University has made a major contribution to the area of seismic studies of adobe structures. In particular, the research has added a significant amount of detailed quantitative information relating to both the pre-cracked elastic, and post-cracked post-elastic behaviour of model adobe
houses. The researchers noted a clear distinction between the elastic and post-elastic responses of the buildings, and posed two key questions (Tolles and Krawinkler, 1990):

- "Which aspects of the design affect the \textit{elastic} behaviour?... Elastic design techniques can be used to minimise cracking during small to moderate earthquakes but cannot predict collapse."

- "Which aspects of the design affect the \textit{post-elastic} behaviour?" These aspects dictate the collapse resistance capacity of the structure, which is the most important design consideration.

Adobe research at Stanford University continued as part of the Getty Seismic Adobe Project (GSAP) as described below.
3.5 Getty Seismic Adobe Project, U.S.A.

Some of the most important seismic adobe research to date has been undertaken as part of the Getty Seismic Adobe Project (GSAP), sponsored by the Getty Conservation Institute. The focus of the GSAP research has been the preservation and protection of historic adobe structures, including mission buildings, churches and ranches. The objective of the GSAP research was "to develop seismic retrofitting strategies for historic adobe buildings that are both structurally effective and have a minimal and primarily reversible impact on historic fabric" (Tolles, 2005). Although the strengthening interventions have been developed for historic buildings, there are a number of key outcomes which are relevant to the seismic improvement of domestic housing in developing countries. These include an understanding of damage patterns and modes of failure, the influence of a roof diaphragm and the effectiveness of relatively simple and cost-effective measures (e.g. external vertical and horizontal strapping).

The GSAP research has been published in three key components:

- Survey of damage to historic adobe buildings after the January 1994 Northridge earthquake (Tolles et al, 1996). The report documented damage patterns, failure modes, and the influence of pre-existing conditions (e.g. quality of adobe, erosion, moisture).

- Seismic stabilisation of historic adobe structures (Tolles et al, 2000). The report presents the results of shake table testing of small scale (1:5) and large scale (1:2) model adobe houses. These are discussed in greater detail below.

- Planning and engineering guidelines for the seismic retrofitting of historic adobe structures (Tolles et al, 2002). The report includes a summary of the previous two reports, plus an overview of engineering design and outline of the design process (global design, crack prediction and retrofit measures).

The shake table testing of model adobe houses was undertaken in two phases: i) 1:5 scale model houses at Stanford University; and ii) 1:2 scale model houses at the Institute of Earthquake Engineering and Engineering Seismology (IZIIS) in Skopje, Macedonia. Table 3-6 shows the specifications and reported results for the model adobe houses tested as part of the GSAP research.
The 1:5 scale model houses (models 1-9) were tested on the Stanford University uni-axial shake table (Figure 3-12), using the N21E component of the July 1952 Taft earthquake (as described in Section 3.4 above). The shake table displacement was increased by 20-30% for each test in the sequence (Table 3-5). Detailed qualitative results (observations) were presented for each model house. Aside from the displacement and acceleration of the shake table, no quantitative measurements were taken.

The 1:2 scale model houses (models 10-11) were tested on a uni-axial shake table at the Institute of Earthquake Engineering and Engineering Seismology (IZIIS) in Skopje, Republic of Macedonia. Instrumentation was used to record the acceleration and displacement of the structure, as well as the stresses in the reinforcement straps. Comprehensive qualitative and quantitative information was reported for each model. Of particular note was the in-depth discussion of the behaviour of the models at different stages (elastic response, damage progression, and collapse), as well as the detailed diagrams showing the existing and new cracks during each simulation.

<table>
<thead>
<tr>
<th>Test level</th>
<th>Max EPGA* (g)</th>
<th>Max displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Full-scale</td>
<td>1:5 scale</td>
</tr>
<tr>
<td>I</td>
<td>0.12</td>
<td>25.4</td>
</tr>
<tr>
<td>II</td>
<td>0.18</td>
<td>50.8</td>
</tr>
<tr>
<td>III</td>
<td>0.23</td>
<td>76.2</td>
</tr>
<tr>
<td>IV</td>
<td>0.28</td>
<td>101.6</td>
</tr>
<tr>
<td>V</td>
<td>0.32</td>
<td>127.0</td>
</tr>
<tr>
<td>VI</td>
<td>0.40</td>
<td>158.8</td>
</tr>
<tr>
<td>VII</td>
<td>0.44</td>
<td>190.5</td>
</tr>
<tr>
<td>VIII</td>
<td>0.48</td>
<td>254.0</td>
</tr>
<tr>
<td>IX</td>
<td>0.54</td>
<td>317.5</td>
</tr>
<tr>
<td>X</td>
<td>0.58</td>
<td>381.0</td>
</tr>
</tbody>
</table>

Note: EPGA = Estimated peak ground acceleration (Tolles and Krawinkler, 1990)

Figure 3-12 Model house #7 prior to testing (Tolles et al, 2000).
<table>
<thead>
<tr>
<th>Model</th>
<th>Scale (h/t ratio)</th>
<th>Walls</th>
<th>Principle retrofit measure</th>
<th>Results</th>
<th>Collapse level</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1</td>
<td>1:5 (7.5)</td>
<td>N+E</td>
<td>Upper horizontal strap</td>
<td>Out-of-plane collapse that may have been prevented by more closely spaced crossties (Figure 3-13a)</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S+W</td>
<td>Upper and lower horizontal straps</td>
<td>Close to collapse during final test</td>
<td>No collapse</td>
</tr>
<tr>
<td>#2</td>
<td>1:5 (7.5)</td>
<td>N+E</td>
<td>Bond beam and centre-core rods</td>
<td>Stable behaviour in all tests</td>
<td>No collapse</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S+W</td>
<td>Bond beam, lower internal horizontal straps, and vertical straps</td>
<td>Stable behaviour in all tests</td>
<td>No collapse</td>
</tr>
<tr>
<td>#3</td>
<td>1:5 (7.5)</td>
<td>All</td>
<td>Bond beam, lower internal horizontal straps, and vertical centre-core rods</td>
<td>Stable behaviour in all tests</td>
<td>No collapse</td>
</tr>
<tr>
<td>#4</td>
<td>1:5 (5)</td>
<td>N+E</td>
<td>Upper horizontal strap</td>
<td>Near-collapse of east wall; substantial permanent displacements throughout the model (Test level X)</td>
<td>No collapse</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S+W</td>
<td>Upper and lower horizontal straps</td>
<td>Substantial permanent displacements throughout the model (Test level X)</td>
<td>No collapse</td>
</tr>
<tr>
<td>#5</td>
<td>1:5 (11)</td>
<td>All</td>
<td>Unreinforced (traditional)</td>
<td>Collapse of 75% of the walls; complete collapse of out-of-plane walls (Test level VIII)</td>
<td>VII</td>
</tr>
<tr>
<td>Model</td>
<td>Scale (h/t ratio)</td>
<td>Walls</td>
<td>Principle retrofit measure</td>
<td>Results</td>
<td>Collapse level</td>
</tr>
<tr>
<td>-------</td>
<td>-----------------</td>
<td>-------</td>
<td>---------------------------</td>
<td>---------</td>
<td>----------------</td>
</tr>
<tr>
<td>#6</td>
<td>1:5 (11)</td>
<td>N+E</td>
<td>Bond beam, lower horizontal straps, and vertical straps</td>
<td>Out-of-plane walls near collapse; centre pier dislodged (Figure 3-13b)</td>
<td>VII</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S+W</td>
<td>Bond beam, lower horizontal straps, and local ties at piers between the door and windows</td>
<td>Collapse of out-of-plane west wall during test level VIII; collapse of most of south wall during test level IX</td>
<td>VIII &amp; IX</td>
</tr>
<tr>
<td>#7</td>
<td>1:5 (5)</td>
<td>N+E</td>
<td>Partial wood diaphragms + upper strap at attic-floor level, lower straps, and vertical straps</td>
<td>Stable behaviour in all tests</td>
<td>No collapse</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S+W</td>
<td>Partial wood diaphragms + upper strap at attic-floor level and lower straps; no vertical straps</td>
<td>Partial collapse of south (in-plane) wall during test level X (Figure 3-14a)</td>
<td>X</td>
</tr>
<tr>
<td>#8</td>
<td>1:5 (7.5)</td>
<td>N+E</td>
<td>Partial wood diaphragms + upper strap at attic-floor level, lower straps, and vertical straps</td>
<td>Stable behaviour in all tests (Figure 3-14b)</td>
<td>No collapse</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S+W</td>
<td>Partial wood diaphragms + upper strap at attic-floor level, lower straps, and vertical centre-core rods; no lower strap on west wall</td>
<td>Stable behaviour in all tests</td>
<td>No collapse</td>
</tr>
<tr>
<td>#9</td>
<td>1:5 (7.5)</td>
<td>All</td>
<td>Unreinforced (traditional)</td>
<td>Complete collapse of gable-end walls</td>
<td>VI</td>
</tr>
</tbody>
</table>
Table 3-6 cont’d

<table>
<thead>
<tr>
<th>Model</th>
<th>Scale (h/t ratio)</th>
<th>Walls</th>
<th>Principle retrofit measure</th>
<th>Results</th>
<th>Collapse level</th>
</tr>
</thead>
<tbody>
<tr>
<td>#10</td>
<td>1:2</td>
<td>All</td>
<td>Unreinforced (traditional) [Matching model #8]</td>
<td>Mid-height, out-of-plane collapse of east wall</td>
<td>VIII</td>
</tr>
<tr>
<td>(Z1)</td>
<td>(7.5)</td>
<td></td>
<td></td>
<td>(Figure 3-15)</td>
<td></td>
</tr>
<tr>
<td>#11</td>
<td>1:2</td>
<td>N+E</td>
<td>Partial wood diaphragms + upper strap at attic-floor level, lower straps, and vertical straps [Matching model #9]</td>
<td>Stable behaviour in all tests</td>
<td>No collapse</td>
</tr>
<tr>
<td>(Z2)</td>
<td>(7.5)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>S+W</td>
<td>Partial wood diaphragms + upper strap at attic-floor level, lower straps, and vertical centre-core rods; no lower strap on west wall [Matching model #9]</td>
<td>Stable behaviour in all tests</td>
<td>No collapse</td>
</tr>
</tbody>
</table>

Notes:
1) Model houses 1-6 did not include a roof system (Figure 3-13a&b). Model houses 7-11 were Tapanco-style buildings, with gable-end walls, attic-floor and roof system (Figure 3-14a&b).
2) h/t ratio = height / thickness ratio (= slenderness ratio).
3) E (east) and W (west) walls were tested out-of-plane; N (north) and S (south) walls were tested in-plane.
4) Retrofit measures:
   - Vertical and horizontal straps for the 1:5 scale models were made of 3mm wide, flexible, woven nylon strap (as typically used for a bootlace).
   - Bond beams for the 1:5 scale models were made of timber (Douglas fir, 38mm wide and 10mm thick) anchored with 89mm screws.
   - Crossties for the 1:5 scale models consisted of 1.6mm diameter nylon chord, inserted through the wall.
   - Centre cores for the 1:5 scale models consisted of 3mm diameter steel drill rods (models 2 and 3), and 4.8mm diameter steel rods anchored in an epoxy resin (model 8). For the 1:2 scale houses (models 10 and 11) the centre core rods were 14mm diameter ribbed steel rods in an epoxy grout.
5) Test level at collapse, as shown in Table 3-5.
Figure 3-13 (a) Model house #1 (east wall) after test level X;
(b) Model house #6 (east wall) after test level X (Tolles et al., 2000).

Figure 3-14 (a) Model house #7 (south wall) after test level X (2);
(b) Model house #8 (east wall) after test level IX (Tolles et al., 2000).

Figure 3-15 Mid-height, out-of-plane collapse, Model #10, east wall (Tolles et al., 2000).
The key outcomes of the GSAP research which are of particular relevance to this thesis include (Tolles et al., 2000):

- **The retrofit systems tested (horizontal and vertical straps, ties, vertical centre-core rods, and improvements in the anchoring of the roof to the walls)... proved to be successful in reducing the tendency of the model buildings to collapse.**

- **The retrofit method using vertical straps was most effective for reducing the risk of out-of-plane wall collapse. Vertical straps had little or no effect on the initiation and early development of crack damage. When displacements or offsets became significant, however, the strapping system controlled the relative displacement of cracked sections of walls, which, if left uncontrolled, led to instability. When coupled with tied anchorage to the roof and/or floor system, the out-of-plane overturning or mid-height collapse of walls can be prevented.**

- **In-plane damage was much less affected by vertical straps. This is largely because in-plane offsets are smaller in magnitude and more likely to persist after the dynamic motions are completed. Straps can prevent large displacements but not small crack offsets. Straps are also useful in preventing piers from becoming unstable.**

- **Vertical centre-core rods installed in the adobe walls were found to be particularly effective in delaying and limiting the damage to both in-plane and out-of-plane walls. The initiation of cracks was delayed because of the centre-core rods. Some cracks in the in-plane walls that started at the corners of the door and window openings propagated to a centre-core rod and then were arrested for one or two more tests. The cracks never became severe.**

- **In the out-of-plane walls, the centre-core rods acted as reinforcing elements... In addition, the rods acted as dowel pins that minimised the relative motion of adobe blocks.**

- **The performance of small- and large-scale model buildings was very similar in many ways. The general development of cracks, the types of cracks, and the failure modes were similar. The effects of the retrofit measures on building behaviour were also similar. For the most part, the behaviour of the small-scale models was an acceptable predictor of large-scale model performance.**
The principal physical difference between the small- and large-scale models was in gravity loads. As a result, specific differences were seen in both the out-of-plane and in-plane wall performances. Global performances were very similar. The most outstanding difference was the occurrence of diagonal cracks in the in-plane walls. Diagonal cracks allow displacements that are cumulative and slippage that is exacerbated by vertical loads. As a result, diagonal cracks were more of a problem in the larger-scale model and would be expected to be at least as serious a problem in full-scale buildings, in which diagonal cracks near the ends of piers and walls are of particular concern.

Of particular interest was the effectiveness of the external strapping reinforcement, which was reported to be the "most effective for reducing the risk of out-of-plane wall collapse... [although they] had little or no effect on the initiation and early development of crack damage." (Tolles et al, 2000). It can be observed that if the flexible strapping were substituted with a stiffer material, then this early crack development may possibly be delayed. This aspect has been the subject of in-depth testing using bamboo reinforcement as part of this PhD research (Chapters 6, 7 and 9).

The main limitation of the GSAP research was the lack of quantitative data obtained for all specimens. Such data from these extensive tests would be a very useful resource for the development and validation of finite element models (described in Chapter 13), as well as for detailed comparative analysis of the behaviour of each specimen.
3.6 Universidad de los Andes, Colombia

Seismic adobe research has been undertaken in recent years at the Universidad de los Andes in Colombia (Yamin et al, 2003, 2004 & 2005). The research has included:

- A survey of adobe construction around Colombia.
- Material property tests of adobe bricks and prisms (discussed in Chapter 4).
- A series of static tests of adobe walls with different reinforcement configurations. These included in-plane horizontal cyclic loading to assess the horizontal rigidity, deformation and capacity; and out-of-plane inclination tests to evaluate the flexural capacity and failure modes of the walls. These tests were also used to determine material properties for use in finite element modelling.
- Shake table testing of three model adobe houses (1:5 scale) with different reinforcement systems (Figure 3-16, Table 3-8 and discussed below). Four model houses representing the local tapia pisada (rammed earth) style of construction were also tested.
- Some finite element (FE) modelling of both walls and model houses, using solid elements to model the walls, and shell and frame elements to model the reinforcement. FE modelling of adobe structures is discussed further in Chapter 13.

Dynamic testing at Universidad de los Andes was undertaken on a small uni-axial shake table. Each model house was 60cm long, 60cm wide and 90cm high. Each model included a timber roof frame with tiles. The specifications of each model house are presented in Table 3-8, and shown in Figure 3-16. A number of simulations were undertaken using input ground motions from the 1995 Tauramena, Colombia earthquake. Modifications were made to the time domain (generally scaled by a factor of 1/5, corresponding to the dimensional scaling of the models), as well as to the displacement domain. Table 3-7 shows the sequence of testing, and time scaling, displacement scaling, peak displacement and peak acceleration of the shake table for each model house. A consistent sequence of testing was not used for each specimen, which raises questions about the comparability of test results. Qualitative results (observations) only were presented (Table 3-8). Figure 3-17 shows each model house after testing.
Table 3-7 Shake table test sequences for scale model adobe houses #1, #2 and #3, *Universidad de los Andes* (Yamin et al, 2003).

<table>
<thead>
<tr>
<th>Test</th>
<th>Model #1</th>
<th></th>
<th>Model #2</th>
<th></th>
<th>Model #3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Time</td>
<td>Displacement</td>
<td>D&lt;sub&gt;max&lt;/sub&gt;* (mm)</td>
<td>A&lt;sub&gt;max&lt;/sub&gt;** (g)</td>
<td>Time</td>
</tr>
<tr>
<td>1</td>
<td>20%</td>
<td>1%</td>
<td>0.37</td>
<td>0.05</td>
<td>20%</td>
</tr>
<tr>
<td>2</td>
<td>20%</td>
<td>5%</td>
<td>1.85</td>
<td>0.25</td>
<td>20%</td>
</tr>
<tr>
<td>3</td>
<td>20%</td>
<td>10%</td>
<td>3.7</td>
<td>0.5</td>
<td>20%</td>
</tr>
<tr>
<td>4</td>
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<td>15%</td>
<td>5.55</td>
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</tr>
<tr>
<td>5</td>
<td>20%</td>
<td>20%</td>
<td>7.4</td>
<td>1.0</td>
<td>20%</td>
</tr>
<tr>
<td>6</td>
<td>20%</td>
<td>25%</td>
<td>9.25</td>
<td>1.25</td>
<td>20%</td>
</tr>
<tr>
<td>7</td>
<td>20%</td>
<td>35%</td>
<td>12.95</td>
<td>1.75</td>
<td>20%</td>
</tr>
<tr>
<td>8</td>
<td>20%</td>
<td>40%</td>
<td>14.8</td>
<td>2</td>
<td>20%</td>
</tr>
<tr>
<td>9</td>
<td>20%</td>
<td>40%</td>
<td>14.8</td>
<td>2</td>
<td>20%</td>
</tr>
<tr>
<td>10</td>
<td>20%</td>
<td>40%</td>
<td>14.8</td>
<td>2</td>
<td>20%</td>
</tr>
<tr>
<td>11</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>40%</td>
</tr>
</tbody>
</table>

*Notes*

* D<sub>max</sub> = Maximum displacement (shake table)

** A<sub>max</sub> = Maximum acceleration (shake table)
Table 3-8 Model adobe house testing at *Universidad de los Andes*: specifications and results (Yamin *et al.*, 2003, 2004 & 2005).

<table>
<thead>
<tr>
<th>Model</th>
<th>Specifications</th>
<th>Natural frequency (Hz)</th>
<th>Initial cracking</th>
<th>Collapse level</th>
<th>Reported failure mechanisms</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1</td>
<td>Traditional, unreinforced house. (Figure 3-16a)</td>
<td>16.7</td>
<td>Test 3</td>
<td>Partial: Test 8, Total: Test 10</td>
<td>Brittle, catastrophic failure. Total collapse. (Figure 3-17a)</td>
</tr>
<tr>
<td>#2</td>
<td>Reinforced with external vertical and horizontal timber elements on both faces of the wall, connected with bolts passing through the wall. (Figure 3-16b)</td>
<td>20</td>
<td>Test 3</td>
<td>Roof: Test 9, Gables: Test 10</td>
<td>Failure of roof and gable ends / top part of walls. Progressive failure. Total collapse prevented. (Figure 3-17b)</td>
</tr>
<tr>
<td>#3</td>
<td>Welded mesh in vertical and horizontal strips at corners and tops of walls, connected with wire passing through the wall, and covered with a sand-lime render. (Figure 3-16c)</td>
<td>20</td>
<td>Test 3</td>
<td>Total: Test 10</td>
<td>Failure at intersection of walls, failure of mesh and overturning of walls. Catastrophic failure delayed. (Figure 3-17c)</td>
</tr>
</tbody>
</table>
The results showed that a significant improvement in seismic capacity was achieved in both reinforced models. The system of timber framing exhibited enhanced ductility and energy dissipation characteristics, and did not fail catastrophically. Yamin et al (2004) reported that the reinforcement systems provide some “structural continuity and generate certain levels of confinement that reduce the probability of anticipated failure and delay the occurrence of collapse, reducing the probability of casualties.”

The main limitations of the research at Universidad de los Andes include:

- The use of small models (1:6 scale) which neglect the effects of gravity, and make it difficult to accurately represent the important mortar-brick interface.
- No quantitative data showing the relative movement of the walls has been presented for the testing.
3.7 Adobe guidelines and manuals

A large number of manuals and guidelines have been produced in efforts to disseminate safer adobe construction practices (Papanikolaou and Taucer, 2004, identified 18 different manuals used in Latin America alone, although there are more). This section focuses on guidelines and manuals for use in developing countries, and discusses three examples in significant depth:

- RESESCO, Reglamento para la Seguridad Estructural de las Construcciones (1997) Folleto complementario adobe, Asociación Salvadoreña de Ingenieros y Arquitectos, Ministerio de Obras Publicas, El Salvador, 36 p. [In Spanish]
- Equipo Maíz (2001) La casa de adobe sismorresistente, Asociación Equipo Maíz, El Salvador, 91p. [In Spanish]

These publications cover three general categories: international guidelines for general use around the world (IAEE guidelines); government-endorsed guidelines or code supplements (RESESCO adobe supplement); and grass-roots, community construction manuals (Equipo Maíz adobe manual).

For each guideline or manual the following aspects are presented:

- General description, context and scope.
- Design specifications relating to adobe construction.
- Strengths of the publication.
- Limitations of the publication.

Some additional general features of these and other manuals are presented in Section 3.7.4. These include soil selection, wall dimensions, the cost, durability and availability of materials, and the complexity of construction.
3.7.1 International Association for Earthquake Engineering (IAEE)

In 1986 the International Association for Earthquake Engineering (IAEE) published *Guidelines for earthquake resistant non-engineered construction*. The guidelines were developed by an international committee in response to the observation that “most of the loss of life in past earthquakes has occurred due to the collapse of buildings, constructed in traditional materials like stone, brick, adobe and wood, which were not particularly engineered to be earthquake resistant. In view of the continued use of such buildings in most countries of the world, it is essential to introduce earthquake resistant features in their construction.”

The IAEE guidelines include the following chapters:

1) The problem, objective and scope.
2) Structural performance during earthquakes, including earthquake effects, ground shaking effect on structures, effect of site conditions on building damage, other factors affecting damage, failure mechanism of structures, and earthquake damage categories.
3) General concepts of earthquake resistant design, including categories of buildings, general planning and design aspects, structural framing, requirements of structural safety, concepts of ductility, deformability and damageability, concept of isolation, and foundations.
4) Buildings in fired-brick and other masonry units.
5) Stone buildings.
6) Wooden buildings.
7) Earthen buildings.
8) Non-engineered reinforced concrete buildings.
9) Repair, restoration and strengthening of buildings.

Chapters 1, 2, 3, 7 and 9 of the IAEE manual are of particular relevance to the issue of seismic safety of adobe dwellings. The earthquake damage categories presented in Chapter 2 of the IAEE guidelines have been adopted in this thesis to describe the damage to each specimen during the shake table testing (Chapters 6 and 9). The general specifications related to improved adobe design and construction are summarised in Table 3-9.
Table 3-9 Specifications from IAEE guidelines (1986).

<table>
<thead>
<tr>
<th>Specification</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>General layout</td>
<td>Single-storey with attic (in high seismicity zones); rectangular, with dividing walls.</td>
</tr>
<tr>
<td>Foundations</td>
<td>Width: 1-1.5 x width of wall (depending on soil stability); Depth: 0.4m (minimum); stone or fired brick in cement or lime mortar.</td>
</tr>
<tr>
<td>Plinth(^1)</td>
<td>Height: 0.3m (minimum); stone or fired brick in cement or lime mortar.</td>
</tr>
<tr>
<td>Damp-proof course</td>
<td>Water-proof mud, plastic sheeting.</td>
</tr>
<tr>
<td>Adobe Soil</td>
<td>Subjected to the 'dry strength test' (Figure 3-18a).</td>
</tr>
<tr>
<td>Blocks</td>
<td>Various. &quot;...the dimensions of the blocks, nor the way they are placed, have a serious effect on their strength.&quot; Strength tested by a simple bending test (Figure 3-18b)</td>
</tr>
<tr>
<td>Stabilisation</td>
<td>No details given.</td>
</tr>
<tr>
<td>Mortar joints</td>
<td>Same mud material as adobe blocks; straw or sand added to control cracking; quality verified by 'fissure control test'. No dimensions given.</td>
</tr>
<tr>
<td>Walls</td>
<td>The length of an unsupported wall should not be greater than 10 times the wall thickness (t), or greater than (64t^2/h), where (h) is the height of the wall; the height of a wall should be less than 8 times its thickness.</td>
</tr>
<tr>
<td>Pilasters</td>
<td>Corners and intermediate; depth: wall thickness.</td>
</tr>
<tr>
<td>Openings</td>
<td>Width: 1.2m (maximum). Distance between openings and lateral supports: 1.2m (minimum). Total area: less than 0.3 x area of wall.</td>
</tr>
<tr>
<td>Lintels</td>
<td>Embedded into wall 0.5m (minimum) either side of opening.</td>
</tr>
<tr>
<td>Vertical reinforcement</td>
<td>Internal. Cane or bamboo (~20mm diameter), every 400mm (minimum); embedded in foundation; connected to ring beam.</td>
</tr>
<tr>
<td>Horizontal reinforcement</td>
<td>Internal. Split or crushed cane or bamboo, every four courses.</td>
</tr>
<tr>
<td>Ring beam</td>
<td>Timber; continuous.</td>
</tr>
<tr>
<td>Roof - ring beam – wall attachment</td>
<td>No details given.</td>
</tr>
<tr>
<td>Roof + gables</td>
<td>No details given.</td>
</tr>
<tr>
<td>Wall cover</td>
<td>Options: soil, sand, cow dung, cactus 'glue', bitumen.</td>
</tr>
<tr>
<td>Floor</td>
<td>No details given.</td>
</tr>
<tr>
<td>Construction costs</td>
<td>No details given.</td>
</tr>
<tr>
<td>Retrofit-strengthening</td>
<td>Described below.</td>
</tr>
</tbody>
</table>

\(^1\) The plinth is a concrete 'top foundation' which elevates the adobe wall above the ground, thus reducing potential moisture damage.
Figure 3-18 Adobe soil tests: (a) ‘Dry strength test’;  
(b) ‘Adobe strength test’ (IAEE, 1986).

The IAEE guidelines suggest that *strengthening procedures should aim at one or more of the following:*

- *Increasing the lateral strength in one or both directions...*
- *Giving unity to the structure by providing a proper connection between its resisting elements...*
- *Eliminating features that are sources of weakness or that produce concentrations of stresses...*
- *Avoiding the possibility of brittle modes of failure by proper reinforcement and connection of resisting members...*

Some of the retrofit-strengthening techniques presented in the IAEE guidelines include:

- *Inserting new walls, including internal shear walls and cross walls, and external buttresses to avoid torsion due to asymmetry, or long unsupported walls. The problem of connecting the old and new walls is acknowledged, and steel keys are recommended to address this problem.*
- *Strengthening existing walls, including grouting, addition of vertical reinforced concrete coverings on the two sides of the wall (e.g. steel mesh covered with cement-based render), and prestressing the wall (e.g. longitudinal steel bars).*

Some of the strengths of the IAEE guidelines include:

- The guidelines are based on sound engineering principles, and have been developed by international experts.
• The guidelines are presented in a very systematic manner, covering key features, including identification of the problem, structural performance during earthquakes, general concepts of earthquake resistant design, followed by specific issues and recommendations for buildings made of different materials (fired-brick, stone, timber, earth, reinforced concrete).

• The guidelines incorporate techniques for repair and retrofit-strengthening of existing dwellings.

• The guidelines include a mix of technical discussion and clear illustrations, appealing to a broad audience.

Some of the limitations of the IAEE guidelines include:

• There is limited technical depth and detail in some sections, including general roofing detail, and the connections between wall, ring beam and roof.

• Many of the suggestions are still relatively expensive and/or complicated. There is no discussion of these aspects in the guidelines.

• The guideline would benefit from updating to incorporate some of the latest research findings, as described earlier, and throughout this thesis. The guidelines were reprinted in 2004 by NICEE, National Information Centre of Earthquake Engineering, Indian Institute of Technology, Kanpur, India. This suggests that there are no immediate plans to update the guidelines.

Many of the principles in the IAEE guidelines have been adopted by a variety of other manuals and guidelines, including those described below.
3.7.2 RESESCO adobe supplement, El Salvador

In 1994 the El Salvador building code, *Reglamento Para la Seguridad Estructural de las Construcciones* (RESESCO), was published. In 1997 an adobe supplement was produced which provides a set of recommendations for adobe design and construction. The recommendations in the supplement are not legally enforceable, although this is reasonable considering that adobe construction is undertaken in an unregulated manner, particularly in rural areas, as discussed in Chapter 2.

The supplement covers the following subjects:

- General design criteria.
- Material for adobe fabrication.
- Construction.
- Visual aids.

The general specifications are summarised in Table 3-10 (translated from the Spanish). Some sample recommendations are shown in Figure 3-19.

Some of the strengths and limitations of the guidelines are presented after Table 3-10.

Figure 3-19 RESESCO recommendations: (a) foundations and plinth; (b) window openings.
<table>
<thead>
<tr>
<th>Table 3-10 Specifications from REESCO adobe supplement (1997).</th>
</tr>
</thead>
<tbody>
<tr>
<td>General layout                              Single-storey, rectangular, with dividing walls.</td>
</tr>
<tr>
<td>Foundations                                  Width: 1.5 x width of wall (minimum: 0.4m); Depth: 0.4m (minimum); rough rocks and 6:1 sand-cement mortar (Figure 3-19a).</td>
</tr>
<tr>
<td>Plinth                                       0.2m high; rough rocks and 6:1 sand-cement mortar (Figure 3-19a).</td>
</tr>
<tr>
<td>Damp-proof course                           None.</td>
</tr>
<tr>
<td>Blocks                                       Options: Full size: 28x28x8cm; 23x23x10.5cm; 40x40x10cm. Half size: 28x13x8cm; 23x10.5x10.5cm; 40x20x10cm. Voids for reinforcement included in mould. Grooved surfaces. Simple bending test for quality control.</td>
</tr>
<tr>
<td>Stabilisation                               Options: Portland cement: 8% by volume; Lime: 3-8% by volume; natural fibres: 20-30% by volume; asphalt: no proportions given.</td>
</tr>
<tr>
<td>Mortar joints                                10-25mm.</td>
</tr>
<tr>
<td>Walls                                        Height: 2.0m (minimum), 3.0m (maximum). Width: 0.3m (minimum), greater than 1/8 height. Length: less than 10 x width of wall.</td>
</tr>
<tr>
<td>Pilasters                                    Recommended; no dimensions or locations given.</td>
</tr>
<tr>
<td>Openings (Figure 3-19b)                     Width: 1.2m (maximum). Distance between openings and lateral supports: 1.2m (minimum). Total area: less than 0.3 x area of wall.</td>
</tr>
<tr>
<td>Lintels                                      Extended into wall 0.6m either side of opening.</td>
</tr>
<tr>
<td>Vertical reinforcement                       Internal. Cane, eucalypt or bamboo (25-30mm diameter), every 64cm; embedded in foundation; connected to ring beam.</td>
</tr>
<tr>
<td>Horizontal reinforcement                    Internal. Split cane, eucalypt or bamboo, every four courses; tied to vertical reinforcement.</td>
</tr>
<tr>
<td>Ring beam                                   Timber or reinforced concrete; continuous.</td>
</tr>
<tr>
<td>Ring beam – wall attachment                 Timber dowels; galvanised wire loops running down three courses.</td>
</tr>
<tr>
<td>Roof - ring beam attachment                 No details given.</td>
</tr>
<tr>
<td>Roof                                        Timber frame, light cover. No details given.</td>
</tr>
<tr>
<td>Gables                                       Timber; “not adobe”.</td>
</tr>
<tr>
<td>Wall cover                                   Options: soil; 1:10 cement:soil; 1:5 to 1:10 lime:soil.</td>
</tr>
<tr>
<td>Floor                                       Concrete. No details given.</td>
</tr>
<tr>
<td>Construction costs                          No details given.</td>
</tr>
<tr>
<td>Retrofit-strengthening                      No details given.</td>
</tr>
</tbody>
</table>
Some of the strengths of the RESESCO adobe supplement are:

- Provides many practical and effective suggestions for improved adobe construction. The preparation of the supplement was a collaboration between experienced and respected individuals representing local universities and public and private institutions. Most of these suggestions are consistent with accepted international guidelines (e.g. the IAEE guidelines described above).
- The supplement is an acknowledgment by the Government of El Salvador (GOES) and the Salvadoran Association of Engineers and Architects (ASIA) that adobe is and will continue to be an important and popular construction material in El Salvador.

Some of the limitations of the RESESCO adobe supplement are:

- The scope of the supplement is stated as “establishing the minimum requirements for the design and construction of free-standing dwellings of one storey, constructed of adobe”\(^2\) (RESESCO, 1997). The supplement, however, is presented more as a general construction manual than a detailed building code. A reader may presume that the contents represent the only acceptable form of improved adobe construction, when in fact a variety of suitable alternatives exist. A clear definition of the purpose of the document would assist the audience to understand the scope and limitations of the supplement.
- The document is somewhat disjointed, which makes it difficult to follow. Some aspects are unnecessarily repeated, others are inconsistent, and some important parts are omitted or deficient in detail. Most noticeably, there is a lack of detail relating to the important connections between wall, ring beam and roof structure. The document also lacks emphasis on overall aseismic building configuration, focusing instead on individual components of the building.
- There is no discussion of appropriate retrofit-strengthening or repair measures for existing dwellings.

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\(^2\) Translated from the original Spanish document by the author of this thesis.
3.7.3 **Equipo Maíz adobe manual, El Salvador**

In early April 2001, Asociación Equipo Maíz released a Spanish-language adobe construction manual entitled *La Casa de Adobe Sismorresistente* (the seismically-resistant adobe house). This manual was a collaborative compilation, involving the efforts and support of technicians, engineers and architects from local and international organisations. The book is a user-friendly, Spanish-language guide to building an adobe house with improved seismic resistance (Figure 3-20).

The manual consists of three chapters:

- **'Chapter One: About adobe, earthquakes and other concerns'** presents the notion that the impacts of natural disasters may be reduced by appropriate action, as well as promoting the reconstruction process as an opportunity to reconstruct human dignity, social justice and solidarity. The chapter discusses the history and future of adobe in El Salvador and other parts of the world, referring to the advantages and disadvantages of adobe, the 'conspiracy' against adobe, examples of adobe use and development, and a representation of poorly constructed and well constructed adobe houses. The chapter closes with a simple analysis of wall panels subject to in-plane and out-of-plane seismic forces.

- **'Chapter Two: Manufacture of good adobe blocks'** outlines the construction of moulds, the assessment of soil, the preparation of materials, the mixing of adobe mud, the fabrication of test blocks, the calculation of the required number of blocks, the production of the blocks, and the drying of blocks.

- **'Chapter Three: Construction of seismically resistant adobe houses'** provides a description of the key aspects of the construction, including the materials and tools required, site selection, setting out the site, foundations, reinforcement, plinth, walls, mortar, lintels, ring beam, roof, and wall finish.

Table 3-11 shows the technical design specifications presented in the Equipo Maíz adobe manual.

Some of the strengths and limitations of the manual are discussed below.
Figure 3-20 Title page of Equipo Maíz adobe manual (2001).
<table>
<thead>
<tr>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>General layout</td>
</tr>
<tr>
<td>Foundations</td>
</tr>
<tr>
<td>Plinth</td>
</tr>
<tr>
<td>Damp-proof course</td>
</tr>
<tr>
<td>Adobe soil</td>
</tr>
<tr>
<td>Blocks</td>
</tr>
<tr>
<td>Mortar joints</td>
</tr>
<tr>
<td>Walls</td>
</tr>
<tr>
<td>Pilasters</td>
</tr>
<tr>
<td>Lintels</td>
</tr>
<tr>
<td>Vertical reinforcement</td>
</tr>
<tr>
<td>Horizontal reinforcement</td>
</tr>
<tr>
<td>Ring beam</td>
</tr>
<tr>
<td>Ring beam – wall attachment</td>
</tr>
<tr>
<td>Roof - ring beam attachment</td>
</tr>
<tr>
<td>Roof structure</td>
</tr>
<tr>
<td>Gables</td>
</tr>
<tr>
<td>Wall cover</td>
</tr>
<tr>
<td>Floor</td>
</tr>
<tr>
<td>Construction costs</td>
</tr>
<tr>
<td>Retrofit-strengthening</td>
</tr>
</tbody>
</table>

---

3 A pyroclastic ash deposit (see Rolo et al., 2004, for geologic + engineering characterisation).
Some of the strengths of the Equipo Maíz adobe manual are:

- The manual is a very timely publication, being prepared and produced in response to the devastation of the earthquakes of 2001. The speed of this process is highly commendable and indicates the editors’ recognition of the importance of rapid dissemination of this relevant information.

- The book is a very visual manual, containing excellent drawings which are both engaging and informative (Figure 3-20 and Figure 3-21). The illustrations have a distinct ‘Salvadoran flavour’ which reflect the cultural, political and social features of El Salvador and allow the local population to identify with the issues presented. The detailed illustrations combined with clear and concise explanations are considerate of those with limited literacy skills.

- The manual contains many sound suggestions relating to improved adobe construction, and includes the input and support of many esteemed professionals.

- The manual is very affordable, with a retail price of US$2.86 (in 2002). This indicates the commitment of the publishers to provide a low-cost construction manual which is accessible to those with limited resources.

- Equipo Maíz is an established and well respected organisation. The profile of Equipo Maíz is a valuable tool in the promotion and acceptance of the book and the notions contained within.

- The manual would be a useful accompaniment to a detailed training program, with participants encouraged to make additional notes in the manual to reflect the learning undertaken in the training program. The book would then serve as an ongoing reference manual to support construction activities undertaken by the participant.

- The manual does not hesitate to present the social and political aspects of the reconstruction process and of adobe construction, making the publication both a construction manual and a political statement. This feature has also drawn some criticism, as discussed below.
Some of the limitations of the Equipo Maíz adobe manual are:

- Limited technical depth and detail, particularly in the latter sections. Most notably, only one page is dedicated to the roof, and no details are given about the roof structure and the important ring beam-roof connection.

- The manual presents the mix proportion of "one bag of cement per twelve wheelbarrows of soil" (less than 3% by mass) as being suitable for making stabilised blocks (Figure 3-21). Unfortunately, this suggestion is somewhat misleading and does not present the complexities of cement stabilisation. Various studies have concluded that the capacity of cement to improve the structural integrity of adobe is directly related to the clay content of the soil used. Norton (1986) states: "A soil with a 10% clay content (an effective minimum for the soil to be cohesive) may require only 5% cement, but this will increase to nearer 10%, and more, when the clay content is 30%." Testing has revealed that an insufficient cement content can actually lower the strength of an adobe block, relative to the strength of an unstabilised block of the same soil (Woodward, 1996; Dowling, 2001). It is feared that the mix proportion suggested in the Equipo Maíz manual may produce inferior blocks. It is recommended that several trial blocks of different mix proportions are manufactured and tested prior to the fabrication of large quantities of stabilised blocks. Furthermore, cement stabilised blocks should be wet cured for at least the first seven days after fabrication. There is no mention of wet curing of cement stabilised blocks in the Equipo Maíz manual.

- The use of stabilised blocks for the lintels and ring beam is widely accepted, however using stabilised blocks in certain parts of the corners of the building is questionable, both on structural and economic grounds. This is especially the case if the mix proportion used creates weaker blocks, as described above. Other concerns relate to the reduced bonding capacity between stabilised blocks and mud mortar (McHenry, 1984) and the lack of homogeneity created by using mixed construction materials.

- Some observers have suggested that the Equipo Maíz adobe manual has lost some of its value as a construction manual because of the strong political and social commentary. The book contains comments and illustrations which level
criticism at the upper class, the Government, banks, the construction industry, and property developers.

- The complexity of improved adobe construction is not clearly presented. This may give the misleading impression that the construction of an improved adobe house is a straightforward task that can be undertaken without any specialised skills or experience.
- There is no discussion of appropriate retrofit-strengthening or repair measures for existing dwellings.

![Images of adobe construction steps](image)

(a) Mix proportions for stabilised bricks
(b) Mixing mud + making bricks
(c) Layout of walls, pilasters + reinforcement
(d) Finished product

Figure 3-21 Sample illustrations from Equipo Maíz adobe manual (2001).
3.7.4 Other manuals

A number of other manuals were reviewed as part of this research. These cover the same general features as described above, with differing approaches and emphases. These include:

- GTZ COPASA (2002) Terremoto? ¿Mi casa si resiste! GTZ COPASA-PUCP-SENCICO, Arequipa, Peru, 50p. [In Spanish]

Some general aspects of the manuals and guidelines reviewed warrant further discussion. These include soil selection, wall dimensions, and the cost, availability and complexity of materials and improvement initiatives.

Soil selection

Various manuals recommend that specific proportions of sand, silt, and clay will improve the structural integrity of the adobe blocks, although there is no universal agreement as to the most appropriate mix. Table 3-12 is a compilation of soil mix proportions recommended by various sources. Furthermore, the manuals propose a number of ‘simple’ tests to determine the suitability of the soil (e.g. Figure 3-18a above). In reality, however, many of these tests are not simple or reliable enough to
assess the suitability of a given soil. Despite these recommendations, it should be noted that the average rural home builder will generally not have access to a wide variety soil types or to accurate means of testing the soil. The most effective manner of assessing the suitability of the soil is to make several trial bricks (using combinations of the available soils) and cure these under different conditions (direct sun, shade, with additives, etc.). It is suggested that at the end of the curing period blocks should “be able to be handled without crumbling or being easily damaged; and not have developed any crack longer than 75mm and wider than 3mm or deeper, irrespective of length or width, than 10mm” (Middleton, 1987).

**Table 3-12 Recommended soil proportions for adobe bricks.**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>55 – 75%</td>
<td>40%</td>
<td>55 – 70%</td>
<td>50 – 70%</td>
</tr>
<tr>
<td>Silt</td>
<td>10 – 28%</td>
<td>40%</td>
<td>15 – 25%</td>
<td>5 – 20%</td>
</tr>
<tr>
<td>Clay</td>
<td>15 – 18%</td>
<td>20%</td>
<td>10 – 20%</td>
<td>15 – 30%</td>
</tr>
</tbody>
</table>

**Wall dimensions**

The recommendations for appropriate wall dimensions given in many manuals are assumed to originate from the IAEE guidelines (discussed above). Although not implicitly stated in the manuals, it is assumed that these recommendations refer to traditional, unreinforced adobe walls which do not incorporate improved seismic resistance systems (e.g. reinforcement, ring beam, pilasters, roof diaphragm, etc.). The provision of improvement systems is expected to alter these suggested dimensions, although further research is required to ascertain the degree of improvement offered by such changes.

**Cost, availability and complexity**

None of the reviewed manuals discuss the issues of cost, durability and availability of materials, nor the complexity of incorporating improvement initiatives. Most manuals present the proposed systems as being simple and cheap, although in reality this may not be the case. The experiences of the author in El Salvador and in the construction of adobe structures as part of this research project suggest that the cost and complexity of construction is one of the major considerations in any improved systems.
3.8 **Summary**

This chapter presented two key components of the current literature related to seismic safety of adobe structures:

- Research / technical, which described the experimental testing undertaken to date.
- Dissemination / practical, which presented a number of manuals and guidelines currently in use.

These research and dissemination activities have made a significant contribution to the body of knowledge relating to improved adobe design and construction. The experiences of the author of this thesis, in both field research and implementation in El Salvador, and hands-on construction and experimental testing at UTS, have revealed a number of gaps in the current body of knowledge. These include:

- Most of the solutions that have been researched and presented in literature utilise at least some modern and costly materials (e.g. welded mesh, concrete, steel). These materials are often beyond the means of the rural poor, and thus external intervention is required for widespread implementation. Furthermore, many of the systems are too complicated to be incorporated without ongoing external support. There is generally a lack of discussion of the cost and complexity of the proposed initiatives.
- Much of the experimental testing has produced qualitative data only. This data is useful to understand the general behaviour and relative performance of different adobe structures, but does not reveal some of the intricate response patterns detected by instrumentation.
- The main emphasis of improved adobe manuals and guidelines has been the construction of new houses, with little attention given to the retrofit-strengthening of existing dwellings.
- Many of the manuals and guidelines report systems which are based on theoretical calculations and testing undertaken in the 1970s, '80s and '90s. More recent research has highlighted the need to update these guidelines.

The research undertaken as part of this PhD thesis endeavours to address some of these shortcomings.
4 BRICK FABRICATION AND MATERIAL PROPERTY TESTS

4.1 Introduction

This chapter describes the process of brick fabrication undertaken for this research, as well as a number of material property tests of adobe prisms.

All the adobe bricks used in this research project were hand-made, in a traditional manner (puddled mudbricks). The objective of brick fabrication was to make 'consistently average' bricks, such that they were representative of traditional mudbricks, yet as consistent in properties as possible across all the bricks.

A number of tests were undertaken on adobe masonry prisms. These included compressive testing (using adobe triplet prisms), shear testing (using adobe triplet prisms) and flexural bond testing (using adobe couplet prisms). The objectives of these prism tests were:

- To obtain some representative material characteristics (for reporting, and for future finite element modelling, as described in Chapter 13);
- To understand some of the modes of failure and factors which contribute to mortar-brick bond strength;
- To assess the effect of some simple adobe brick laying techniques which are considered to influence the development of mortar-brick bond strength. These techniques include using wet or dry bricks during construction, varying the mortar thickness and applying a compressive load during specimen curing.
4.2   **Brick fabrication**

For this project, in excess of 8,000 adobe bricks were hand-made, using traditional methods of adobe-mudbrick fabrication, as described below. The bricks were half-size (1:2 scale) of dimensions: 150mm x 150mm x 50mm. Each brick weighed approximately 2kg. The main objective of the brick fabrication process was to make ‘consistently average’ adobe bricks, i.e. to achieve consistency or uniformity of bricks and to produce bricks that could be considered to possess average or indicative characteristics as exhibited by commonly used adobe bricks. In order to achieve these objectives, all bricks were fabricated using the same raw materials in the same location (Mittagong, NSW) with the same process (all undertaken and co-ordinated by the author). Video footage of the brick fabrication process is included in Appendix A.

**Fabrication process**

The fabrication process is described below:

1) A number of trial blocks were made, using combinations of raw soil plus different proportions of straw and sand additives. The selected mix was 3:2 soil:sand by volume (detailed soil composition is presented in Table 4-1). In order to ensure greatest consistency, it was decided to avoid the use of straw, which is difficult to accurately measure and mix.

2) Dry components were manually mixed. Water was then added and thoroughly mixed, again manually (using shovels, rakes, hoes and stomping, Figure 4-2a). The mud was then left to ‘sit’ for a number of hours, or overnight, where possible. This ‘sitting’ of the mud acts to break down the clay particles, which can be then more easily dispersed (as reported in IAEE, 1986; Woodward, 1996; Equipo Maiz, 2001). Two large mixing trays/pits were constructed (one of sheet metal and one of concrete) to allow proper ‘sitting’ of the mud, and avoid contamination of the soil.

3) The mud was then thoroughly mixed again, with additional water added as required. The mud was then transported to the brick making area (Figure 4-1). The brick making area consisted of a large concrete slab, which was overlaid with recycled ceiling panels (to elevate the bricks from the slab) and covered with used carpet. The carpet surface provided a clean, uniform surface, which also allowed some flow of air and moisture under and around the bricks.
4) The mud was placed into steel moulds, with four bricks per mould (Figure 4-2b). Care was taken to fill the corners and reduce voids. The top of the brick was smoothed off by hand and the mould removed directly. The moulds were cleaned in water after each brick set. On days of extreme heat, the bricks were covered immediately with recycled corrugated iron sheeting (Figure 4-1). This was particularly important during the initial days of brick curing, to reduce the incidence of brick cracking due to shrinkage induced by rapid drying. The corrugated iron also served to protect the bricks from rain.

5) After several days, the bricks were uncovered and dried in direct sunlight (Figure 4-3 and Figure 4-4a). Over several days the bricks were rotated until all surfaces were thoroughly dried.

6) The bricks were then loosely stacked for further drying (Figure 4-4b), prior to being stacked and wrapped on pallets. The bricks were carefully transported to UTS by truck.

7) After testing, a large amount of brick rubble was returned to the site and pulverised (using a whacker plate upright rammer) and made into new bricks. Other brick rubble was retained at UTS for use as mortar in subsequent specimens.

Figure 4-1 Brick fabrication area.
[Note: bricks in different stages of drying, corrugated iron sheeting covering fresh bricks]
Figure 4-2  (a) Mixing of mud for trial bricks; (b) Fabrication of adobe bricks.

Figure 4-3  Drying of adobe bricks.

Figure 4-4  (a) Brick fabrication area; (b) Stacked adobe bricks.
Quality control

All bricks were manually inspected at each stage of the brick and specimen fabrication process. Bulletin 5, the long-standing Australian guideline for earthwall construction (Middleton, 1987), states:

At the end of this drying period [28 days] the block should:

i) be able to be handled without crumbling or being easily damaged, and

ii) not have developed any crack longer than 75mm and wider than 3mm or deeper, irrespective of length or width, than 10mm.

For this project, considering the desire for the highest practical consistency and the 1:2 scale bricks, any bricks with cracks longer than 10mm, or wider than 1mm were rejected. During the fabrication of the specimens, each brick was dunked in water prior to laying (described in greater detail below). This process had the added advantage of detecting cracks and voids in the brick (evidenced by bubbles) which were then rejected and recycled.

Brick characteristics

Table 4-1 shows the brick and soil characteristics for bricks used in this research.

<table>
<thead>
<tr>
<th>Table 4-1 Brick and soil characteristics.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil composition (bricks + mortar)</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Atterberg Limits (soil)</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Brick curing conditions</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Nominal brick dimensions (LxWxH)</td>
</tr>
<tr>
<td>Brick density</td>
</tr>
</tbody>
</table>

Notes:
* Standard soil analysis revealed a very low proportion of clay. Despite this, the strength of the bricks was adequate (see Section 4.3 below). A cohesive silt is assumed to contribute to the binding of the soil particles.
4.3 Compressive strength of adobe prisms

4.3.1 Specifications

Ten adobe mudbrick triplet prisms were fabricated and tested in compression. The specifications of the adobe compression prisms are shown in Table 4-2.

<table>
<thead>
<tr>
<th>General specimen configuration</th>
<th>Triplet prisms: three bricks + two mortar joints (horizontal stack)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal specimen dimensions (LxWxH)</td>
<td>150mm x 150mm x (150mm + 2 x mortar thickness)</td>
</tr>
<tr>
<td>Nominal mortar thickness</td>
<td>12 – 13mm</td>
</tr>
<tr>
<td>Average specimen height (H)</td>
<td>174mm (SD = 1.7mm; CoV = 1.0%)</td>
</tr>
<tr>
<td>Average H/W ratio</td>
<td>1.157 (SD = 0.012; CoV = 1.0%)</td>
</tr>
<tr>
<td>Average specimen density</td>
<td>1,776kg/m³ (SD 14.2 kg/m³; CoV = 0.8%)</td>
</tr>
<tr>
<td>Average moisture content of mortar</td>
<td>20–23 % (during construction); ~2 % (during testing)</td>
</tr>
<tr>
<td>Curing load</td>
<td>2 bricks (1:2 scale) (~1.7kPa)</td>
</tr>
<tr>
<td>Specimen curing conditions</td>
<td>Standard laboratory conditions (UTS). Covered with plastic for first 7 days.</td>
</tr>
<tr>
<td>Specimen age (at testing)</td>
<td>~ 160 days</td>
</tr>
<tr>
<td>Specimen quantity</td>
<td>10 prisms fabricated</td>
</tr>
<tr>
<td>Loading rate</td>
<td>In compliance with ASTM C 1314 Section 9.3 (described below)</td>
</tr>
</tbody>
</table>

Notes:
SD = Standard Deviation
CoV = Coefficient of Variation (= standard deviation / average)

The prisms were constructed at the same time as the model house 4A (Chapter 9), using the same bricks and mortar. The construction sequence for the adobe compression prisms is shown in Figure 4-5. After curing, the prisms were wetted and rubbed down to create a smooth and consistent finish. The prisms were then left to dry again. The upper and lower bearing surfaces of each prism were filed smooth and level prior to testing. Figure 4-6 shows a typical prism prior to testing.
All ten prisms were built to this level, then the second mortar joint and third (top) brick were added to each specimen, following the same procedure.

Figure 4-5 Construction sequence for adobe compression prisms.
Figure 4-6 Typical adobe prism prior to testing (specimen 1).

4.3.2 Test method

Compressive testing was undertaken in compliance with ASTM C 1314-02a: Standard Test Method for Compressive Strength of Masonry Prisms (ASTM 2002a). The test machine used was a Shimadzu Universal Testing Machine (Type REH, Serial No. 61531) with a spherically-seated upper platen. The machine was designated Grade A, according to AS 2193: Calibration and classification of force-measuring systems (Standards Australia, 2005). Of particular note was the application of load, described in Section 9.3 of ASTM C 1314:

\[
\text{Loading – Apply the load to the prism to one-half of the expected total load at any convenient rate. Apply the remaining load at a uniform rate in not less than 1 nor more than 2 min.}
\]

A uniform crosshead travel of 1.24mm/min was applied, ensuring that the above requirements were met (Figure 4-7).

The compressive strength was calculated using Equation 4-1:

\[
f_c = \frac{P}{A}
\]

Where:

- \( f_c \) = compressive strength (kPa)
- \( P \) = maximum load (kN)
- \( A \) = surface area (m²)
Figure 4-7 Typical load-time graph for compressive tests (specimen 1).

4.3.3 Results

Results summary

Key results from the compression tests are shown in Table 4-3.

Table 4-3 Results from compression tests of adobe prisms.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>H</th>
<th>H/W ratio</th>
<th>Max Load P (kN)</th>
<th>Prism Strength $f_c$ (kPa)</th>
<th>$E_m$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.173</td>
<td>1.15</td>
<td>34.98</td>
<td>1,554.5</td>
<td>147.7</td>
</tr>
<tr>
<td>2</td>
<td>0.171</td>
<td>1.14</td>
<td>34.84</td>
<td>1,548.3</td>
<td>197.1</td>
</tr>
<tr>
<td>3</td>
<td>0.174</td>
<td>1.16</td>
<td>32.77</td>
<td>1,456.3</td>
<td>167.3</td>
</tr>
<tr>
<td>4</td>
<td>0.174</td>
<td>1.16</td>
<td>35.29</td>
<td>1,568.4</td>
<td>181.5</td>
</tr>
<tr>
<td>6</td>
<td>0.176</td>
<td>1.17</td>
<td>33.19</td>
<td>1,475.1</td>
<td>149.0</td>
</tr>
<tr>
<td>7</td>
<td>0.176</td>
<td>1.17</td>
<td>31.29</td>
<td>1,390.6</td>
<td>138.7</td>
</tr>
<tr>
<td>8</td>
<td>0.172</td>
<td>1.15</td>
<td>32.45</td>
<td>1,442.1</td>
<td>112.3</td>
</tr>
<tr>
<td>9</td>
<td>0.172</td>
<td>1.15</td>
<td>34.49</td>
<td>1,532.8</td>
<td>108.2</td>
</tr>
<tr>
<td>10</td>
<td>0.174</td>
<td>1.16</td>
<td>34.28</td>
<td>1,523.7</td>
<td>158.2</td>
</tr>
<tr>
<td>Average</td>
<td>0.174</td>
<td>1.16</td>
<td>33.73</td>
<td>1,499.1</td>
<td>151.1</td>
</tr>
<tr>
<td>SD</td>
<td>0.002</td>
<td>0.012</td>
<td>1.47</td>
<td>60.7</td>
<td>29.3</td>
</tr>
<tr>
<td>CoV (%)</td>
<td>1.0</td>
<td>1.0</td>
<td>4.0</td>
<td>4.0</td>
<td>19.4</td>
</tr>
</tbody>
</table>

Notes:
For all specimens: $W=0.15m$; $L=0.15m$; Hence cross-sectional area, $A=0.0225m^2$. Specimen 5 was omitted due to a technical error.
Aspect ratio correction factor

Unlike conventional fired-brick and concrete block masonry, there are no reliable correction factors to account for the height/width aspect ratio of prisms for adobe-mudbrick masonry. Even for conventional masonry there appears to be no firm agreement with respect to correction factors, as seen in the different factors prescribed by the Australian Standard AS 3700 (2001) and American Society for Testing and Materials standard ASTM C 1314 (2002a). If the AS 3700 procedure is adopted, an aspect ratio correction factor of 0.712 is applied, which gives an average unconfined compressive strength of 1,067 kPa. If the correction factor prescribed in ASTM C 1314 is used (0.673, from linear interpolation) the average unconfined compressive strength is 1,008 kPa. Due to the lack of reliable aspect ratio correction factors for adobe-mudbrick masonry, the raw data is presented for this research, however it should be considered to be slightly stronger than would be expected for prisms with a larger height/width ratio. (Given these circumstances, it seems reasonable to suggest that for this research, an average compressive strength of 1,000 – 1,100 kPa could be assumed.)

Coefficient of Variation (CoV)

Interestingly, the Co-efficient of Variation (CoV) was very low for the compressive strength of the prisms. A value of 4% is lower than is commonly reported for conventional fired-brick masonry and soil-cement brick masonry prism testing (e.g. Walker, 2004; Sarangapani et al, 2005; Venkatarama Reddy and Gupta, 2006). This result suggests that for the tested prisms, there was consistency in brick and mortar properties and construction quality. This also highlights that with due care and attention to quality control processes, bricks and prisms with consistent properties can be produced.

Modulus of Elasticity ($E_m$)

In order to determine the Modulus of Elasticity ($E_m$) for the adobe prisms the chord modulus technique was adopted between stress levels of 5% and 33% of the compressive strength (MSJC, 2002; BSSC, 2000), as shown in Figure 4-8.
Figure 4-8 Chord modulus technique for determining Modulus of Elasticity, $E_m$ (Rai, 2005).

Figure 4-9 Typical stress-strain diagram for compressive tests (specimen 1). [Dashed red line indicates chord modulus for determining $E_m$, as described above]
Failure modes

The compression prisms behaved similar to conventional fired-brick and concrete block masonry: vertical tensile cracking (Figure 4-10a) followed by crushing and spalling. Each prism displayed the typical hour-glass shape characteristic of a conical break (Figure 4-10b), with some face shell separation evident. The stress-strain diagrams for each specimen (e.g. Figure 4-9) show the non-linearity of the adobe prisms, and is characteristic of masonry in general.

![Figure 4-10 Typical post-test crack pattern and hour-glass fracture (specimen 3).](image)

Comparison with other sources

Table 4-4 compares the results from this research (shaded grey) with other tests of adobe-mudbrick prisms undertaken around the world. The results reveal the compressive strength ($f_c$) of the prisms used in this research project to be slightly higher than those tested elsewhere, although this may be attributed to the low height/width ratio of the prisms tested in this research project. (Most of the results reported in Table 4-4 do not indicate whether an aspect ratio correction factor has been used, and in some cases, the height/width (H/W) ratio is not provided.) As discussed above, if an aspect ratio correction factor (based on AS 3700 and/or ASTM C 1314) is applied, an average compressive strength of 1,000 – 1,100 kPa could be assumed for this research.
### Table 4-4 Compressive strength of adobe prisms from this and other sources.

<table>
<thead>
<tr>
<th>Source</th>
<th>Scale (bricks)</th>
<th>Density (kg/m³)</th>
<th>n</th>
<th>H/W ratio</th>
<th>fc (kPa)</th>
<th>CoV (%)</th>
<th>Em (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dowling (UTS)</td>
<td>1:2</td>
<td>1,776</td>
<td>9</td>
<td>1.16</td>
<td>1,499</td>
<td>4.0</td>
<td>151.1</td>
</tr>
<tr>
<td>Hernandez et al (1981) and Flores et al (2001)</td>
<td>1:1</td>
<td>1,800</td>
<td>3+</td>
<td>6.0</td>
<td>1,324</td>
<td>-</td>
<td>245</td>
</tr>
<tr>
<td></td>
<td>1:2.5</td>
<td>1,800</td>
<td>3+</td>
<td>6.0</td>
<td>1,314</td>
<td>-</td>
<td>245</td>
</tr>
<tr>
<td>Meli et al (1980)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>981</td>
<td>30</td>
<td>-</td>
</tr>
<tr>
<td>Moreira &amp; Rosales (1998)</td>
<td>-</td>
<td>1,780</td>
<td>5</td>
<td>2.12</td>
<td>1,495*</td>
<td>11.3</td>
<td>-</td>
</tr>
<tr>
<td>Tolles &amp; Krawinkler (1990)</td>
<td>1:5</td>
<td>1,760</td>
<td>15</td>
<td>2.1</td>
<td>958</td>
<td>6.8</td>
<td>1,520**</td>
</tr>
<tr>
<td>Tolles &amp; Krawinkler (1990)</td>
<td>3:4</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2,413</td>
<td>5.1</td>
<td>-</td>
</tr>
<tr>
<td>Tolles &amp; Krawinkler (1990)</td>
<td>1:5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>3,309</td>
<td>4.9</td>
<td>-</td>
</tr>
<tr>
<td>Yamin et al (2003)</td>
<td>1:1</td>
<td>1,800</td>
<td>-</td>
<td>-</td>
<td>1,196</td>
<td>-</td>
<td>115</td>
</tr>
<tr>
<td></td>
<td>1:5</td>
<td>1,870</td>
<td>-</td>
<td>-</td>
<td>1,549</td>
<td>-</td>
<td>128</td>
</tr>
</tbody>
</table>

**Notes:**
- Original $f_c$ results from Moreira & Rosales (1998) incorporated an aspect ratio correction factor of 1.01 (average) which is consistent with ASTM C 1314. The results presented here are the raw (unadjusted) results.
- The $E_m$ value reported by Tolles & Krawinkler (1990) seems unusually high and inconsistent with other reported $E$ values.
- $n$ = number of specimens tested.
- H/W ratio = height / width ratio.
- $f_c$ = compressive strength.
- CoV = Coefficient of Variation.
- $E_m$ = Modulus of Elasticity.
- a dash '-' indicates data not reported.

Tolles and Krawinkler (1990) reported a difference in strength obtained between tests for 3:4 scale and 1:5 scale adobe masonry assemblies. They found that 1:5 scale prisms behaved more monolithically (in flexure) and were approximately 35% stronger in compression than 3:4 scale prisms made of the same material. They attributed this to different curing conditions of the prisms, as well as the size effect. They suggested that the ratio of the mortar-brick contact area with respect to the brick volume plays a significant role in bond strength development, with a larger surface area to volume ratio causing an increase in the relative amount of water absorption, and hence creating a better brick-mortar bond.
A difference in compressive strength due to size effects was also reported by Yamin et al (2003) who presented a 30% higher compressive strength for 1:5 scale adobe prisms, compared with 1:1 scale prisms. On the other hand, results from Hernandez et al (1981) and Flores et al (2001) do not show any meaningful difference in compressive strength between 1:1 and 1:2.5 scale adobe prisms. It is clear that further detailed testing is necessary to determine more accurately the effect of scaling on brick and prism characteristics.
4.4 Shear strength of adobe prisms

4.4.1 Introduction

During an earthquake, significant shear stresses develop in a structure (Figure 4-11). The mortar-brick interface is a significant plane of weakness, due mainly to differential drying conditions. In order to assess the shear strength of adobe masonry and evaluate a variety of simple techniques for strength improvement, a number of adobe ‘triplet’ prisms were tested in double shear under static conditions (using a modification of the DIN / European Standards EN 1052-3:2002 and EN 1052-4:2000). Parameters chosen for testing employed simple and cost-effective construction methods, including the effect of using wet or dry bricks, varying mortar thicknesses and using different surcharges during curing. Eight sets (ten prisms per set) with different parameters were tested. The shear testing of adobe prisms was undertaken as an undergraduate final-year project by Hannah Price at UTS. Some results have been reported in Price (2005) and Dowling et al (2005b).

![Diagram showing in-plane seismic movement and internal shear forces (Diaz, 2003).](image)

Figure 4-11 Shear forces at brick-mortar interface (Diaz, 2003).
4.4.2 Masonry joint shear tests and standards

Various masonry joint shear strength tests have been developed over the years with a focus on conventional masonry such as fired clay bricks and concrete blocks. An evaluation of over 20 different masonry joint shear strength methods by Jukes and Riddington (1997) showed that the Hofmann-Stöckl couplet test produced the most uniform stress distribution in the joint. The major disadvantage of this method, however, is the complexity of the test set up and apparatus required. Jukes and Riddington (1997) noted that the triplet test method with pre-compression (later adopted as the European Standard, and described below) is a valid test method because of the ease of specimen construction, the lack of complex apparatus and the consistency of results.

The current European Standards DIN EN 1052-3 (2002) and DIN EN 1052-4 (2000) show two methods for determining the shear strength of masonry mortar joints. The first method is a modified triplet test with vertical shear load applied to a straight stack specimen under pre-compression (Figure 4-12a). The second test involves a horizontal shear load applied to a running bond-configured specimen under pre-compression (Figure 4-12b). The second test is specified to include a damp-proof course between the mortar joints, but this is not necessary if standard mortar joint shear strength is being considered.

Current Australian Standard (AS), American Society for Testing and Materials (ASTM), and International Organisation for Standardisation (ISO) standards and codes lack a simple and practical test method for determining the horizontal shear capacity of masonry components. Relevant ASTM test methods (C 1531-02 and E 519-00) involve the testing of large wall panels, which is both resource intensive (materials, equipment and time) and impractical for large numbers of specimens.

Preliminary tests at the UTS confirm that the triplet test is suitable for evaluating the shear strength of adobe prisms (see Dowling et al, 2004a).
4.4.3 Numerical studies

Numerical studies of shear test methods for conventional masonry show the variability of shear and normal stresses along the mortar joint (Riddington et al., 1997; Mirabella Roberti et al., 1998). The research by Riddington et al. (1997) shows that shear and normal stress distributions are less variable for low pre-compressive (normal) loads (Figure 4-13). This would seem ideal for adobe units, where compressive strength is low and adobe buildings are generally single-storey structures with relatively low normal loading. On the other hand, however, “when no pre-compression is applied, tensile stress develops across the joint in the central region and failure is likely to be initiated in this region. Tensile bond strength will therefore have an influence on results when there is no pre-compression.” (Riddington et al., 1997). The numerical analysis undertaken by Riddington et al. (1997) considers the effects of pre-compressive (normal) loading of 0, 1 and 2 MPa for conventional masonry.
4.4.4 Test method

The test method adopted for this research can be considered a slight variation of the two DIN European Standards described earlier. The test set up is shown in Figure 4-14. A modified soil mechanics shear box testing apparatus was used in the experiment. The horizontal application of load to the triplet unit is considered to more accurately represent the load distribution in actual wall units, and the slight increase in normal load (compared with vertical testing) is considered to be insignificant. The load was applied with a horizontal hydraulic jack (Carpanelli Type M63/4) at a constant rate of loading (10mm/min) such that failure of the joint occurred between 20 and 60 seconds. The pushing, restraining and pre-compressive load blocks were all adjustable to allow for minor variations in specimen size. Rubber pads were placed between the blocks and the specimen to accommodate inconsistencies in the specimen surface and to ensure more uniform distribution of the loads. The load and displacement data were logged electronically. Specimens were tested to failure.
The shear strength was calculated using Equation 4-2:

\[ \tau = \frac{F}{2 \times S.A} \]  

4-2

Where:

\( \tau \) = shear strength (kPa)

\( F \) = shear load (kN)

\( S.A. \) = shear surface area (m\(^2\))
4.4.5 Specifications

The specimen and test specifications for the shear prisms are shown in Table 4-5.

<table>
<thead>
<tr>
<th>Table 4-5 Shear prisms: specimen and test specifications.</th>
</tr>
</thead>
<tbody>
<tr>
<td>General specimen configuration</td>
</tr>
<tr>
<td>Nominal specimen dimensions (LxWxH)</td>
</tr>
<tr>
<td>Nominal mortar thickness (variable parameter)</td>
</tr>
<tr>
<td>Average moisture content of mortar</td>
</tr>
<tr>
<td>Specimen curing conditions</td>
</tr>
<tr>
<td>Specimen age (at testing)</td>
</tr>
<tr>
<td>Specimen quantity</td>
</tr>
<tr>
<td>Shear loading rate</td>
</tr>
</tbody>
</table>

Notes:
All external surfaces were saw-cut to provide flat and smooth lateral surfaces. The natural roughness of the brick surfaces (top and bottom) in contact with the mortar joints was retained.

4.4.6 Parameters tested

A number of simple parameters were considered. The effects of these changes are discussed in greater detail in Section 4.6 below.

Wet / dry brick surface

There is no clear agreement within published literature as to whether it is necessary to wet the surface of the bricks prior to laying them. This was considered by comparing one set of prisms for which each brick was quickly dunked in water prior to laying (Set 1A) and one set of prisms for which completely dry bricks were used (Set 1B).
Curing load

Curing loads of two and four 1:2 scale bricks were used (Figure 4-15). In real terms this equates to curing loads of one and two full-size bricks, or 1.7 kPa and 3.4 kPa, respectively. The curing load was placed on top of each specimen approximately 20-30 minutes after fabrication. In practice, the curing load is applied in two ways:

- If construction is continuous, the weight of the subsequent brick courses applies a curing load.
- If construction is interrupted (e.g. due to inclement weather, meal and overnight breaks) surplus bricks may be simply dry-stacked on top of the wall to apply the curing load. In this case, it is impractical and resource intensive to apply more than two full-size bricks.

![Figure 4-15 Curing load of two 1:2 scale bricks.](image)

Mortar thickness

The mortar joint serves to provide an even bedding for the bricks and account for any variations in brick dimensions (Woodward, 1996). Various adobe manuals recommend a mortar thickness of 20-30mm (Woodward, 1996; Equipo Maíz, 2001; Blondet et al., 2003), however in practice mortar joint thickness varies between 10 and 50+ mm (Figure 4-16). Thick mortar joints are commonly used in practice because fewer bricks are required, thus saving construction time and energy. The effect of mortar thickness was assessed by fabricating and testing specimens with mortar thicknesses of 12-13mm, 24-26mm and 48-52mm.
Pre-compressive normal loads

Adobe-mudbrick prisms have relatively low compressive strength (see results in Section 4.3 above). It can be seen, therefore, that for the shear testing of adobe triplet units the applied pre-compressive load must be low to avoid damaging the specimen. Furthermore, adobe dwellings are typically limited to one or two storeys in height. In these cases, the normal loads acting on the walls are low (less than 0.1MPa at the base of a double-storey house with heavy tile roof, and even less for single-storey dwellings and houses with non load-bearing walls), so the approach of applying large normal compressive loads (>0.1MPa) cannot be considered indicative of real-life conditions. In the case of design for high seismic zones where single-storey houses with light roofs are advocated (IAEE, 1986; RESESCO, 1997; Equipo Maíz, 2001) the normal loads at the base of a wall are in the region of 0.05MPa, and even less in the upper portion of the wall where failure is predominantly initiated. As a result, six of the eight sets were tested without a pre-compressive normal load, and two sets were tested with nominal pre-compressive normal loads of 100kPa and 200kPa. The results of these two sets provide limited practical value, and are simply presented to provide an indication of the effects of pre-compression, and show the sensitivity of testing to applied normal loads.
4.4.7 Results

Figure 4-17 shows some sample shear prism failures, and Table 4-6 provides a summary of test results.

Figure 4-17 Shear prism failure.
<table>
<thead>
<tr>
<th>Specimen Set</th>
<th>Brick Surface</th>
<th>Mortar Thickness (mm)</th>
<th>Curing Load (1:2 bricks)</th>
<th>Pre-compression Load (kPa)</th>
<th>No. of specimens</th>
<th>Mean Shear Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Co-efficient of Variation, CoV*</th>
<th>Comparative Shear Strength** (w.r.t. Set '1A')</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>Wet</td>
<td>12-13</td>
<td>0</td>
<td>0</td>
<td>9</td>
<td>49.0</td>
<td>14.3</td>
<td>29%</td>
<td>-</td>
</tr>
<tr>
<td>1B</td>
<td>Dry</td>
<td>12-13</td>
<td>0</td>
<td>0</td>
<td>10</td>
<td>13.7</td>
<td>3.9</td>
<td>28%</td>
<td>31%</td>
</tr>
<tr>
<td>1C</td>
<td>Wet</td>
<td>12-13</td>
<td>2</td>
<td>0</td>
<td>10</td>
<td>58.1</td>
<td>11.2</td>
<td>19%</td>
<td>118%</td>
</tr>
<tr>
<td>1D</td>
<td>Wet</td>
<td>12-13</td>
<td>4</td>
<td>0</td>
<td>10</td>
<td>64.1</td>
<td>11.4</td>
<td>18%</td>
<td>131%</td>
</tr>
<tr>
<td>1E</td>
<td>Wet</td>
<td>24-26</td>
<td>0</td>
<td>0</td>
<td>10</td>
<td>35.4</td>
<td>8.1</td>
<td>23%</td>
<td>69%</td>
</tr>
<tr>
<td>1F</td>
<td>Wet</td>
<td>48-52</td>
<td>0</td>
<td>0</td>
<td>10</td>
<td>10.7</td>
<td>5.2</td>
<td>48%</td>
<td>22%</td>
</tr>
<tr>
<td>1G</td>
<td>Wet</td>
<td>24-26</td>
<td>0</td>
<td>100</td>
<td>8</td>
<td>116.9</td>
<td>9.0</td>
<td>8%</td>
<td>239%</td>
</tr>
<tr>
<td>1H</td>
<td>Wet</td>
<td>24-26</td>
<td>0</td>
<td>200</td>
<td>10</td>
<td>171.3</td>
<td>10.0</td>
<td>6%</td>
<td>349%</td>
</tr>
</tbody>
</table>

Notes:
* Co-efficient of Variation (CoV) = standard deviation / mean shear strength
** Comparative Shear Strength = mean shear strength / mean shear strength of Set 1A (datum)
General observations and comments

- Aside from Sets 1G and 1H (tested with pre-compression load), Set 1D possessed the strongest shear bond strength, followed closely by Set 1C. This was due to the presence of a curing load, as well as a thin mortar joint compared with other cases.
- Sets 1B and 1F exhibited significantly weaker shear bond strength, which highlights the weakness of prisms constructed with dry bricks and/or thick mortar joints.

The results are discussed in greater detail in Section 4.6 below, where they are compared with the results from the flexural bond strength (bond wrench) test, described in the next section.

Co-efficient of Variation (CoV)

Adobe-mudbricks are inherently variable in nature, so a coefficient of variation (CoV) below 30% is deemed to be acceptable. (Shear tests of over 40 sets of conventional masonry joints by Riddington and Jukes (1994) produced results with a CoV of between 4.5% and 37.9%, with most between 10% and 25%.) In this research, an acceptable level of variation was attained in all cases, except Set 1F (very thick mortar joints). More reliable (less variable) results were obtained for the specimens with an applied normal load during curing (Sets 1C and 1D), and with a pre-compressive load during testing (Sets 1G and 1H).

Figure 4-18 shows some indicative strength-strain curves for Sets 1A and 1B (wet/dry brick surfaces). The double peaks show the failure of one mortar joint, followed by the failure of the second mortar joint. The curves show the non-linear nature of adobe masonry units.
Figure 4-18 Typical strength-strain curves for Set A (prism 1A₄) and Set B (prism 1B₄) for adobe masonry triplet shear testing.

Failure modes

In almost all cases, failure occurred at the mortar-brick interface. Figure 4-19 shows the location of initial cracking for the shear testing. The percentages indicate the relative frequency of such initial cracking.

Figure 4-19 Shear testing: location and relative proportion of initial cracking.
The following observations and contributing factors are considered to have influenced the distribution of cracking:

- Initial cracking was more common in the top mortar joint. This may be due to:
  - different boundary conditions acting on the top and bottom bricks (the bottom brick was in contact with the floor of the testing machine, whereas the top brick was unrestrained at the top); and
  - a greater curing load being applied to the bottom mortar joint (due to the weight of the top two bricks).
- Initial cracking was more common at the top of the mortar joint for both upper and lower mortar joints. This may be due to the effect of gravity causing the water in the joint to move down, such that the bottom mortar-brick interface dried more slowly and with a higher initial moisture content, which may have aided bond development.
- The surface of the brick may have influenced the bonding. In all cases, the bricks were laid upside-down, which means that the bottom surface of each brick in the prisms was the top surface of the brick during brick fabrication and curing. This surface had greater exposure to the environment during the initial stages of brick curing, which may have affected the surface texture, and thus joint bond development.

All of these factors could be the subject of further detailed testing and analysis.

**Comparison with other sources**

There has been very little published literature with respect to shear strength testing of adobe triplet prisms. Shear testing of adobe has tended to focus on the diagonal compression-tension testing of adobe wallets, as described in ASTM E 519-00 (ASTM, 2000b). These include tests reported by Meli et al (1980), Hernandez et al (1981), Tolles and Krawinkler (1990), Moreira and Rosales (1998), Flores et al (2001), and Yamin et al (2003).
4.5  *Flexural bond strength of adobe prisms*

4.5.1  Introduction

In order to assess the flexural bond strength of adobe masonry, a number of adobe ‘couplet’ prisms were fabricated and tested using the ‘bond wrench’ test, as described in ASTM C 1072 (2000a) and AS 3700 Appendix D (Standards Australia, 2001). Similar to the shear testing described in Section 4.4 above, a number of simple parameters were considered, including the effect of using wet or dry bricks, varying mortar thicknesses and using different surcharges during curing. Five sets (ten prisms per set) with different parameters were fabricated and tested.

4.5.2  Specifications

The specifications of the bond wrench prisms are shown in Table 4-7.

<table>
<thead>
<tr>
<th>Table 4-7  Bond wrench prisms: specimen specifications.</th>
</tr>
</thead>
<tbody>
<tr>
<td>General specimen configuration</td>
</tr>
<tr>
<td>Nominal specimen dimensions (LxWxH)</td>
</tr>
<tr>
<td>Nominal mortar thickness (variable parameter)</td>
</tr>
<tr>
<td>Average moisture content of mortar</td>
</tr>
<tr>
<td>Specimen curing conditions</td>
</tr>
<tr>
<td>Specimen age (at testing)</td>
</tr>
<tr>
<td>Specimen quantity</td>
</tr>
</tbody>
</table>

Notes:
* External surfaces in contact with the clamps were saw-cut to provide flat and smooth lateral surfaces. The natural roughness of the brick surfaces (top and bottom) in contact with the mortar joints was retained.

Figure 4-20 shows the couplet prisms prior to bond wrench testing.
4.5.3 Test method

The flexural bond strength testing was undertaken according to ASTM C 1072 (2000a). The method consisted of clamping the bottom brick of a couplet prism in a secure frame, and then clamping a loading arm to the top brick. A steadily increasing load was applied at the end of the loading arm until failure of the mortar joint. Figure 4-21 is a schematic diagram of the test set up. Figure 4-22 shows the bond wrench test set up, prior to testing and after failure.

![Figure 4-21 Schematic diagram of flexural bond strength (bond wrench) test.](image-url)
4.5.4 Results

The flexural tensile bond strength was calculated according to the equation prescribed for solid masonry units in ASTM C 1072, shown in Equation 4-3:

\[
F_g = \frac{6 (PL + P_L)}{bd^2} - \frac{(P + P_i)}{bd}
\]

Where:
- \( F_g \) = gross area flexural tensile strength (MPa);
- \( P \) = maximum applied load (N);
- \( P_i \) = weight of loading arm = 80.8 N;
- \( L \) = distance from centre of prism to loading point = 720 mm;
- \( L_i \) = distance from centre of prism to centroid of loading arm = 45 mm;
- \( b \) = cross-sectional width of the mortar-bedded area, measured perpendicular to the loading arm of the upper clamping bracket = 150 mm; and
- \( d \) = cross-sectional depth of the mortar-bedded area, measured parallel to the loading arm of the upper clamping bracket = 130 mm.

A summary of results is shown in Table 4-8.
### Table 4-8 Flexural bond strength test results.

<table>
<thead>
<tr>
<th>Specimen Set</th>
<th>Brick Surface</th>
<th>Mortar Thickness (mm)</th>
<th>Curing Load (1:2 bricks)</th>
<th>No. of specimens</th>
<th>Mean Bond Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Co-efficient of Variation, CoV*</th>
<th>Comparative Bond Strength** (w.r.t. Set '2A')</th>
</tr>
</thead>
<tbody>
<tr>
<td>2A</td>
<td>Wet</td>
<td>24-26</td>
<td>0</td>
<td>7</td>
<td>85.9</td>
<td>25.4</td>
<td>29.6%</td>
<td>-</td>
</tr>
<tr>
<td>2B</td>
<td>Dry</td>
<td>24-26</td>
<td>0</td>
<td>4</td>
<td>36.6</td>
<td>1.9</td>
<td>5.2%</td>
<td>43%</td>
</tr>
<tr>
<td>2C</td>
<td>Wet</td>
<td>12-13</td>
<td>0</td>
<td>9</td>
<td>115.3</td>
<td>17.5</td>
<td>15.2%</td>
<td>134%</td>
</tr>
<tr>
<td>2D</td>
<td>Wet</td>
<td>24-26</td>
<td>2</td>
<td>10</td>
<td>108.2</td>
<td>22.0</td>
<td>20.4%</td>
<td>126%</td>
</tr>
<tr>
<td>2E</td>
<td>Wet</td>
<td>48-52</td>
<td>0</td>
<td>5</td>
<td>44.9</td>
<td>15.6</td>
<td>34.8%</td>
<td>52%</td>
</tr>
</tbody>
</table>

**Notes:**
* Co-efficient of Variation (CoV) = standard deviation / mean shear strength
** Comparative Bond Strength = mean bond strength / mean bond strength of Set 2A (datum)
Observations and comments

- Set 2C (thin mortar joints) possessed the strongest flexural bond strength, followed closely by Set 2D (curing load).
- Sets 2B (dry bricks) and 2E (thicker mortar joints) exhibited significantly weaker flexural bond strength.
- For Sets 2B and 2E a lower number of specimens, \( n \), was evaluated. This was predominantly due to failure or damage of the specimens prior to testing and highlights the weakness of prisms constructed with dry bricks and/or thick mortar joints.
- Results from one prism from Set A and one prism from Set E were considered to be abnormal (according to the process outlined in Appendix H of AS 3700) and were rejected.

The results are discussed in greater detail in Section 4.6 below, where they are compared with the results from the shear strength tests, described in the previous section.

Comparison with other sources

There has been very little published literature with respect to flexural bond strength of traditional adobe-mudbrick prisms. The most closely related masonry testing is that using stabilised pressed earth blocks, which generally have a higher strength than adobe mudbricks, due to mechanical compression during fabrication and the inclusion of additives (e.g. cement, lime, asphalt, etc) in both the bricks and mortar (e.g. Rao et al, 1996; Walker, 1999; Sarangapani et al, 2005).
4.6 Shear strength and flexural bond strength: analysis

In this section, the results from the shear strength and flexural bond strength testing are compared. Of main interest is the effect of the parameter changes (wet/dry bricks, mortar thickness and curing load).

Wet v dry brick surface (Sets 1A, 1B, 2A & 2B)

Figure 4-23 shows the influence of wetting bricks during specimen fabrication. Results from both shear and bond wrench testing show a substantial increase in strength when wet bricks are used. Prisms built with dry bricks possessed 31% of the shear strength and 43% of the flexural bond strength of prisms built with wet bricks.

![Graph showing shear and flexural bond strength comparison between dry and wet brick surfaces.](image)

**Figure 4-23** Influence of dry/wet brick surface on shear and flexural bond strength.

Three main reasons are proposed for this significant difference:

i) The wetting process 'cleans' the brick (removing dry dust and loose soil) which promotes improved adhesion between brick and mortar.

ii) Dry bricks rapidly draw moisture from the mortar, resulting in significant shrinkage cracks, which weaken the structure. The wetted bricks draw less
moisture from the mortar, and the drying rate of the mortar is slower, resulting in less shrinkage and fewer cracks.

iii) The faces of the wetted bricks are softened by the water, which create a gradual transition zone between the dry brick and the wet mortar, via a moist interface. It is surmised that this gradual transition results in an improved ‘merging’ of the brick and mortar.

It should be noted that different bricks will respond to contact with water in different ways, depending on the soil composition of the bricks. Some bricks can be left submerged in water for several minutes without significant damage, whereas others may be severely weakened by such submersion. It is recommended that a representative sample of bricks be tested prior to construction of a complete structure.

**Mortar thickness (Sets 1A, 1E, 1F, 2A, 2C & 2E)**

Figure 4-24 shows the influence of mortar thickness on shear and flexural bond strength. Results from Sets 1A, 1E and 1F show a steady decrease in shear strength as the mortar thickness increases. The same trend is evident for bond wrench strength (Sets 2A, 2C and 2E).

![Graph showing the influence of mortar thickness on shear and flexural bond strength.](image)

*Figure 4-24 Influence of mortar thickness on shear and flexural bond strength.*
The influence of mortar thickness on strength may be attributed to:

i) More significant shrinkage cracks in thicker mortar joints due to increased volume of mortar.

ii) Differential mortar drying times, with mortar in the thin joints drying at a more uniform, rapid rate compared with the varied rate of mortar drying in the thick joint.

iii) Greater instability of the prisms during and after fabrication for the thick joints. This would be somewhat reduced in the construction of a real wall, where the surrounding bricks provide additional stability to the structure.

Curing load (Sets 1A, 1C, 1D, 2A & 2D)

Figure 4-25 shows the influence of curing load on shear and flexural bond strength. Results from Sets 1A, 1C, 1D, 2A and 2D reveal a steady increase in strength as the curing load increases. (The flexural bond strength was not tested for prisms with four bricks curing load.)

![Figure 4-25 Influence of curing load on shear and flexural bond strength.](image)
This improvement can be attributed to the additional compression of the joint, which ‘squeezes out’ entrapped air in the mortar and enhances the physical bonds between soil particles of both the mortar and the moistened brick. It should be noted that an excessive curing load, and/or the use of mortar with a high moisture content may result in undesirable ‘settling’ of the wall as the mortar is squeezed out of the mortar joints.
4.7 Summary

This chapter described the process of brick fabrication undertaken in this project, which adopted a number of quality control measures to ensure the production of ‘consistently average’ bricks.

The chapter also presented the procedure and results of a series of compressive, shear and flexural bond strength tests using adobe prisms. The results of this research indicate that a significant improvement in the shear strength and flexural bond strength of adobe mudbrick masonry can be practically achieved by employing one or more of the following simple processes:

- Wetting the surface of each brick prior to laying.
- Using a thin mortar joint (yet still allowing sufficient thickness to adequately bed the brick and account for inconsistencies in brick surface).
- Applying a normal load during curing (which can be applied by dry-stacking a course of unused adobe bricks on top of the wall during periods of inactivity).

Results also show that by adopting these techniques, and with appropriate attention to the general quality of construction, less variability in structural properties can be achieved.

These results are a practical step towards understanding the structural behaviour of adobe masonry and developing practical techniques to improve overall structural performance.

A number of further research tasks have been identified, including assessment of different bond surfaces, material components and curing conditions, as well as extending testing to include diagonal tension (shear), plus in-plane and out-of-plane push-over tests, and numerical studies to predict and model joint and prism behaviour. These aspects are discussed further in Chapter 13: Further research.
5 SHAKE TABLE TESTING: INPUT SPECTRA AND TIME HISTORY

5.1 Introduction

This chapter describes the features of the UTS shake table and the process of selecting and modifying the input time history to ensure consistency of test conditions between each specimen. In order to subject each specimen to similar test conditions (to allow reliable comparisons between the structural response and overall performance of each specimen) the following objectives were set for the shake table testing:

- Ensure dynamic similitude between all u-shaped adobe wall units, such that the frequency ratio, defined as the ratio of dominant input excitation frequencies to structural frequencies (first natural frequency of each specimen), was identical for each specimen.
- Ensure damaging near-resonance conditions, which are achieved when the natural frequency of each specimen (u-panels and model house) is matched with the dominant frequency range of the input spectrum.

In order to achieve these objectives, the input spectra was uniquely time-scaled for each individual specimen. The custom time scaling for each specimen is a unique and robust means of dynamic testing of adobe structures, which has not been reported elsewhere. The process of time scaling the input spectra (adopted from the Mw 7.7 El Salvador earthquake of January 13, 2001 discussed in Chapter 2) is described in this chapter.

In addition to the time-scaling of the input spectra, scaling of the intensity was undertaken. This was necessary in order to subject each specimen to a series of earthquake simulations of increasing magnitude, to gauge the response prior to cracking (elastic behaviour), as well as for severe damaging conditions (post-elastic behaviour).

The chapter concludes with a comparison of the approach taken by other adobe researchers around the world, which confirms the need for greater consistency in dynamic testing approaches, or at the very least, enhanced reporting of the specific details of the selection and modification of the input spectra for dynamic testing.
5.2 UTS shake table

The dynamic simulation was undertaken on the state-of-the-art MTS uni-axial shake table located at the University of Technology, Sydney (UTS). The table is capable of high fidelity seismic reproductions across a range of key parameters. The shake table specifications are shown in Table 5-1. The shake table is shown in Figure 5-1.

<table>
<thead>
<tr>
<th>Table 5-1 UTS shake table specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size of table</td>
</tr>
<tr>
<td>Maximum Payload</td>
</tr>
<tr>
<td>Overturning Moment</td>
</tr>
<tr>
<td>Maximum Displacement</td>
</tr>
<tr>
<td>Maximum Velocity</td>
</tr>
<tr>
<td>Maximum Acceleration</td>
</tr>
<tr>
<td>Testing Frequency</td>
</tr>
</tbody>
</table>

Figure 5-1 UTS shake table.
5.3 Selection and modification of input time history

5.3.1 Proposed design spectrum

For seismic simulations using a shake table, an appropriate input time history signal is required. Selection and modification of a given input time history is commonly based on the design response spectra recommended by relevant seismic standards. Since adobe structures generally exhibit an elastic response followed by a brittle failure mechanism, the Elastic Response Spectra was used in this study. The proposed design spectrum was a modified version of the El Salvador design spectrum (Figure 5-2) reported by López et al (2004). In this research project two modifications were made to the El Salvador design spectrum as follows:

- Elevation of spectral acceleration levels to better envelope recent and more severe earthquakes.
- Extension of the flat portion of the spectrum to a period of 0 seconds to cover a wider range of frequency contents (this is a conservative approach for very stiff structures).

The second modification is particularly important for masonry structures, which possess inherently higher resonant frequencies.

![Proposed Design Spectrum](image)

Figure 5-2 Proposed Design Spectrum (dashed red line) vs Elastic Response Spectra developed from some El Salvador earthquakes by Lopez et al (2004).
5.3.2 Selection and verification of input time history

Once a suitable design spectrum was chosen, the selection of an appropriate input time history was undertaken. The selection process included calculating the Test Response Spectrum (TRS) for a given input time history and plotting it against the proposed design spectrum. Past earthquake records or artificially generated synthetic earthquakes are commonly used as the input time history. In this study, the input time history from the Mw 7.7 January 13, 2001 El Salvador earthquake was used. [This earthquake, in combination with a Mw 6.6 earthquake on February 13, 2001 in the same area, caused the destruction of over 110,000 adobe houses, as described in Chapter 2.] Figure 5-3 shows the calculated TRS of the January 13, 2001 El Salvador earthquake in relation to the proposed design spectrum. For dynamic testing it is important for the TRS to envelope the proposed design spectrum in the frequency range of interest (in this case, the high frequency resonant range, which is typical of adobe wall units).

![Graph showing the proposed design spectrum and the Test Response Spectrum (TRS).](image)

**Figure 5-3** Calculated Test Response Spectrum (TRS) (solid blue line) in relation to the proposed design spectrum (dashed red line).

[The shaded section is shown in Figure 5-4]

Figure 5-4 shows the shaded section from Figure 5-3 and focuses on the frequency range of the TRS in relation to the target maximum acceleration response of 1.5g (obtained from the proposed design spectrum, Figure 5-2). Figure 5-4 reveals the
dominant frequency range of the input spectrum (TRS) to be approximately 6.7 – 15.9 Hz.

![Graph showing Test Response Spectrum (TRS) and Proposed Design Spectrum]

Figure 5.4 Test Response Spectrum (TRS) (solid blue line) in relation to the target maximum spectral acceleration (1.5g).

5.3.3 Time scaling of input time history

In order to subject each specimen to similar test conditions (to allow reliable comparisons between the structural response and overall performance of each specimen) the following objectives were set for the shake table testing (as mentioned in the introduction of this chapter):

- Ensure dynamic similitude between all u-shaped adobe wall units, such that the frequency ratio, defined as the ratio of dominant input excitation frequencies to structural frequencies (first natural frequency of each specimen), was identical for each specimen.
- Ensure damaging near-resonance conditions, which are achieved when the natural frequency of each specimen (u-panels and model house) is matched with the dominant frequency range of the input spectrum.
Clearly it is not possible to alter the fundamental natural frequency of each specimen without affecting other structural characteristics, nor is it possible to build all specimens with the same natural frequencies. Therefore, in order to achieve these objectives, time scaling of the input spectra was undertaken for each individual specimen. This is a unique and robust means of dynamic testing of adobe structures, which has not been reported elsewhere. The process of time-scaling the input spectra is described below.

By means of Experimental Modal Testing and Analysis (EMTA), the Frequency Response Function (FRF) of each specimen was identified (this process is described in detail in Chapter 8). For the eleven u-shaped adobe wall panels the first natural frequency ranged between 24.8 Hz and 34.1 Hz (presented and discussed in Chapters 6 and 8). Taking u-panel 3A as an example, the specimen possessed a first resonant frequency (natural frequency) of around 29.6 Hz (Figure 5-5).

![Figure 5-5 Frequency Response Function (FRF) of specimen 3A from the modal hammer tests.](image)

It is clear that for specimen 3A (and the other u-panel specimens) the first natural frequency \( f_1 \) lay outside the critical dominant frequency range of the input spectrum (TRS) identified above. This means that use of the 'raw' (unscaled with respect to time) input spectrum would not excite the specimen in the damaging near-resonance range (this was later proved in the shake table tests, described in Chapter 6). In order to appropriately assess the performance of each specimen with respect to the proposed design spectrum (1.5g in the bandwidth of interest) a 'target frequency zone' (shaded green circle in Figure 5-6) was identified. The next step was to determine the relationship between the 'target frequency zone' and the first fundamental frequency of
each specimen. This relationship describes the ratio with which the time domain of the input spectrum (shake table motion) was to be scaled to ensure dynamic similitude and induce damaging near-resonance conditions (thus known as the 'time scaling factor'). For specimen 3A ($f_1 = 29.6$ Hz) a time scaling factor between 2.07 ($29.6$ Hz / 14.3 Hz) and 1.86 ($29.6$ Hz / 15.9 Hz) was necessary. In this case, a time scaling factor of 2.0 was applied.

![Graph showing spectral acceleration vs frequency](image)

**Figure 5-6 'Target frequency zone' (shaded green circle) with respect to the Test Response Spectrum (TRS) and Proposed Design Spectrum.**

[The 'target frequency zone' between 14.3 and 15.9 Hz was selected in preference to the other option of 6.7 Hz which would have required a time scaling factor of 4.4 ($29.6$ Hz / 6.7 Hz) for specimen 3A. Such a large scaling factor would have produced a very, very fast earthquake, with very small displacement and very high acceleration.]

The test results for the u-shaped adobe wall unit testing (Chapter 6) clearly demonstrate the importance of time scaling the input motion to induce damaging near-resonance conditions. Even the unreinforced specimen 3A was undamaged during the 200% intensity simulation using the raw, unscaled (with respect to time) input motion. Later,
the specimen was subjected to a series of simulations using the time-scaled input motion, with the specimen exhibiting distinct and classic failure patterns during the 75% intensity shaking. This outcome exposes the deficiency of other approaches which do not consider the importance of time scaling the input spectra. The preparation, results and analysis of the dynamic testing of all the u-shaped wall units are detailed in Chapters 6 and 7.

5.3.4 January 13, 2001 El Salvador earthquake

The first phase of shake table testing of adobe wall units at UTS was undertaken in late 2001 (described in Dowling, 2001; Dowling, 2002). At that stage, the time history from the January 13, 2001 El Salvador earthquake was chosen because it had a particularly devastating effect on adobe houses in the region (detailed in Chapter 2). From the network of seismometers in El Salvador, the data from Hospital Santa Teresa in Zacatecoluca was selected because it was in one of the most severely affected areas and was the closest station to the epicentre of the earthquake (see Appendix C for recording station characteristics). The recorded peak accelerations were: -0.244g (N-S horizontal); -0.294g (E-W horizontal); and -0.275g (vertical). The UTS shake table moves in one horizontal direction only, so the E-W component of the earthquake motion was selected because it represented the most severe conditions. The recorded time history of the E-W component of the January 13, 2001 El Salvador earthquake is shown in Appendix C.

Because the UTS shake table is displacement driven, the acceleration-time history was converted to an appropriate displacement-time history via a ‘blackbox’ process which considered the frequency, acceleration and displacement parameters of both the ground motion and the shake table output capacity. The resultant displacement-time history contained a peak displacement of less than 10mm, which was considered to be too small to cause severe damage to the structure. To overcome this the time history record was scaled by a factor of two (in the displacement domain) to produce a drive file capable of inducing severe damage. As a result, the acceleration of the record increased (up to 0.708g for the 100% intensity simulation). This file was then adopted as the 100% intensity drive file, which was later scaled with respect to time (to ensure dynamic
similitude, as reported in Section 5.3.3 above) and displacement (to produce different intensity simulations, as described in Section 5.3.5 below).

Later, corrected data for other seismograph stations was released by USGS (U.S. Geological Survey), which retained the pattern of acceleration, but adjusted (increased) the magnitude of the acceleration. (At the time of writing, the data from Hospital Santa Teresa, Zacatecoluca was reported as 'uncorrected'). Fortunately, the corrected acceleration reported at a nearby station (Santiago de Maria, Usulutan, Appendix C) was closely matched (in terms of duration, pattern and amplitude) with the modified acceleration produced by the adopted drive file, discussed above. The recorded peak accelerations at Santiago de Maria were: -0.881g (N-S horizontal); -0.716g (E-W horizontal); and -0.440g (vertical).

It should be noted, however, that the main emphasis of the dynamic testing phase in this research project was the assurance of dynamic similitude and damaging near-resonance conditions. This relied on the appropriate determination of the relationship between the dominant frequency of the input excitation and the first natural frequency of each specimen, as described above.

### 5.3.5 Intensity scaling of input time history

In addition to the importance of considering frequency relationships, another significant factor contributing to damage of structures due to seismic loading is wall displacement, which is directly affected by the ground (shake table) displacement. The shake table displacement was scaled to produce ground motion simulations of varying intensities. In order to study the behaviour and performance of the structures at different load levels, a series of simulations were undertaken with varying displacement intensities, ranging from 40% - 200% for the unscaled (with respect to time) input motion and 20% - 125% for the time-scaled input motion. The low intensity simulations were used to study the elastic, dynamic behaviour of each specimen prior to damage, and the higher intensity simulations were used to study the cracking patterns and failure mechanisms of each specimen.
Table 5-2 shows the peak displacement and peak acceleration of the shake table for each simulation. The time history records for acceleration and displacement for simulation S7 (S100%) are shown in Figure 5-7 and Figure 5-8.

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity</th>
<th>Peak Displacement (mm)</th>
<th>Peak Acceleration (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unscaled</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td>U40%</td>
<td>7.326</td>
<td>0.269</td>
<td></td>
</tr>
<tr>
<td>S2</td>
<td>U100%</td>
<td>18.284</td>
<td>0.708</td>
<td></td>
</tr>
<tr>
<td>S3</td>
<td>U150%</td>
<td>27.288</td>
<td>1.148</td>
<td></td>
</tr>
<tr>
<td>S3</td>
<td>U200%</td>
<td>36.201</td>
<td>1.587</td>
<td></td>
</tr>
<tr>
<td>Scaled</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S4</td>
<td>S20%</td>
<td>3.144</td>
<td>0.464</td>
<td></td>
</tr>
<tr>
<td>S5</td>
<td>S50%</td>
<td>7.753</td>
<td>1.123</td>
<td></td>
</tr>
<tr>
<td>S6</td>
<td>S75%</td>
<td>11.721</td>
<td>2.051</td>
<td></td>
</tr>
<tr>
<td>S7</td>
<td>S100%</td>
<td>15.598</td>
<td>2.124</td>
<td></td>
</tr>
<tr>
<td>S8</td>
<td>S125%</td>
<td>19.352</td>
<td>2.540</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
- All data was taken from shake table movement during testing of u-panel 3J, except data for simulation S3 (U200%), which was taken from testing of u-panel 3D.
- Peak displacement and peak acceleration occurred at time = approx. 19.3 seconds.

It should be noted that because each specimen possessed a unique first natural frequency (ranging from 24.1 Hz to 34.1 Hz, as detailed in Chapters 6 and 9) a slightly different time scaling factor was applied to the raw drive file (input spectra). This resulted in a slightly different magnitude of shake table acceleration for each specimen, although the displacement record was approximately consistent for all specimens. The shake table accelerations reported in Table 5-2 were for u-shaped specimen 3J, which possessed a first natural frequency of 33.8 Hz. Specimens with a lower first natural frequency had a lower time scaling factor, which generated a shake table input spectra with a slightly lower magnitude of acceleration. As mentioned above, however, the main considerations were dynamic similitude, near-resonance conditions and displacement consistency, all of which were maintained in each test.
Figure 5-7 Shake table acceleration for simulation S7 (S100%).

[Taken from testing of u-panel 3J]
Figure 5-8 Shake table displacement for simulation S7 (S100%).

[Taken from testing of u-panel 3.]
5.3.6 'Reverse' time scaling and calibration of output (results)

In order to accurately compare the results between specimens it was necessary to 'reverse time scale' and calibrate the output (results). In some cases there were minor inconsistencies in the sampling rate of the data acquisition systems which meant that the 'reverse time scaling factor' did not exactly match the inverse of the 'time scaling factor' described above. The results were calibrated by identifying the precise start and finish point of each test, comparing these with the original earthquake record and applying an appropriate 'reverse time scaling factor' to ensure accurate matching of the records. Extraneous data recorded at the start and end of each test was removed.

In subsequent chapters, where the results of the u-panel and model house tests are presented, the time domain has been 'reverse scaled' so that the results match the original earthquake record, rather than the unique time-scaled record used for each specimen.
5.4 Comparison with other research approaches

The approach of uniquely time scaling the input spectra for each specimen appears to be unique among the limited worldwide dynamic testing of adobe structures. Chapter 3 describes the approach taken by other researchers around the world. Some researchers report time scaling of the input ground motion based on the linear scale (size) of the specimens. Other researchers appear to have used raw unscaled (with respect to time) input motion, arguing that if each specimen is subjected to the same series of simulations, then a comparative analysis is possible. This approach, however, does not adequately account for the different responses of different structures due to the inconsistent ratio of input excitation and specimen natural frequency (i.e. lack of dynamic similitude).

All researchers report intensity scaling, although the basis of scaling varies between displacement, acceleration and/or frequency of input spectra.

In some cases, there is no discussion of the input spectra selection and scaling approach taken for dynamic testing, which leaves little indication of the severity and conditions of testing.

The varied approaches to dynamic testing of adobe structures suggest there is a need for greater consistency in testing methodology, or at the very least, enhanced reporting of the specific details of the selection and modification of the input spectra for dynamic testing.
5.5 Summary

The following topics were presented in this chapter:

- UTS shake table specifications.

- The process of selection and modification of the input time history (based on the \(M_v\) 7.7 El Salvador earthquake of January 13, 2001) to account for recent and severe earthquakes, and consider the inherent stiffness of adobe structures.

- The process of time scaling the input spectra to ensure dynamic similitude between specimens, and induce damaging near-resonance conditions. This feature is a unique approach for dynamic testing of adobe structures, and provides greater confidence in the comparative analysis between specimens, as reported in subsequent chapters.

- The process of intensity scaling (displacement-based) the input time history, which allowed simulations of varying intensities in order to assess the pre- and post-cracking response of each specimen.

- Comparison with the approach to dynamic testing taken by other adobe researchers around the world, which confirms the unique and robust approach taken as part of this research, and confirms the need for better benchmarking and reporting of the characteristics of dynamic testing for adobe structures.

The approach described in this chapter is the platform for the key component of this thesis: the dynamic shake table testing of adobe structures. The preparation, results and analysis of the testing are described in subsequent chapters.
6 U-SHAPED ADOBE WALL UNITS: PREPARATION, TESTING AND OBSERVATIONS

6.1 Introduction

One of the core components of this PhD research project was the fabrication and shake table testing of eleven adobe mudbrick wall units. Scale model (1:2) u-shaped adobe wall panels were subjected to transient dynamic loading using a shake table to evaluate the response to out-of-plane seismic forces. Time-scaled input spectra were used to ensure dynamic similitude and induce damaging near-resonance conditions (as described in Chapter 5). Reinforcement systems tested included: the use of pilasters/buttresses at the corners, internal/external wire mesh reinforcement, internal/external bamboo poles, fencing wire, string and a timber ring beam. The most promising solutions were shown to effectively impede initial cracking, and to delay major structural damage and ultimate collapse.

This chapter describes the preparation, testing and observations from the u-shaped adobe wall unit experimentation. It contains a general description of the u-shaped adobe wall units, including the rationale for adopting this configuration, the dimensions and specifications, as well as the instrumentation and nomenclature used in this project.

This chapter focuses on the observed behaviour and recorded responses of each individual specimen. This chapter is an objective presentation of ‘what happened’. A detailed, comparative analysis and discussion of results (the ‘why’ and ‘what does it mean’) is included in Chapter 7.

For each specimen the construction and reinforcement process is presented. This includes comments on aspects of the ‘constructability’ of each specimen. The testing sequence is then described, and the observed damages for each stage are presented. The observed damages are described in a qualitative sense, with the aid of photographs. Video footage of key simulations for each wall panel is included in Appendix A (on CD/DVD). The video footage presents a clear and exciting picture of the behaviour of each specimen and complements this chapter.
For masonry buildings, damage is considered to be displacement-induced, so the recorded responses (graphs) from the displacement sensors along the top of the 'long' wall are presented for the main simulations for each specimen. Displacement versus time graphs for each specimen are shown for the main damaging simulation, as well as the simulation prior to major damage. This quantitative data clearly indicates the magnification of the response of the wall with respect to the ground (shake table) motion, for both the undamaged and damaged conditions. (Acceleration data was recorded, but is not presented because it has less influence on the structural damage.)

The results and lessons learned from each test sometimes led to modifications of the test procedure or preparation of subsequent specimens. These lessons learned and subsequent modifications are presented for each specimen. For this reason, in this chapter the specimens are presented in the order in which they were tested (chronological) rather than alphanumeric order.

This chapter provides the objective information and framework necessary for the detailed discussion and comparative analysis in the following chapter.

The process and results of Experimental Modal Testing and Analysis (EMTA) for the u-panel tests are presented in Chapter 9.
6.2 General description

6.2.1 Rationale

As described in Chapter 2, the predominant failure modes of traditional adobe-mudbrick houses subjected to earthquake loads are vertical corner cracking at the intersection of orthogonal walls (Figure 6-1), and cracking and/or overturning due to out-of-plane flexure. In order to assess the capacity of different improvement systems to reduce such failure, a series of shake table tests of 1:2 scale u-shaped adobe-mudbrick wall units (Figure 6-2) was undertaken.

Figure 6-1 Typical earthquake damage to adobe buildings (López, UES/EERI).
1:2 scale u-shaped wall units were chosen for the following reasons:

- To focus attention on the vulnerable corners and out-of-plane ‘long’ wall;
- Manageable size and weight permitted the simultaneous fabrication, curing and preparation of multiple specimens, which were then carefully moved onto the shake table;
- A larger number of specimens could be built and tested (in comparison to resource intensive construction and testing of scale model houses, as described in Chapter 9);

### 6.2.2 Specimen design and fabrication

To reduce material and specimen variability, all bricks and specimens were fabricated by the author of this thesis, using consistent raw materials, curing conditions and construction practices, as described in Chapter 4. The u-panel specimens were constructed on custom-made reinforced concrete decks mounted on steel frames, which were firmly connected to the shake table prior to testing. Specimen dimensions and configuration satisfied design criteria recommended in relevant guidelines (e.g. IAEE,
1986; REESCO, 1997). Each specimen consisted of 333 full bricks (150mm x 150mm x 50mm) and 56 half bricks (150mm x 70mm x 50mm) laid in stretcher bond (Figure 6-3) with 12-13mm thick mortar joints. The mortar was made from the same mud material as the bricks. Simple improved practices detailed in Chapter 4 were undertaken, including: dunking each brick in water prior to laying; and application of two courses of bricks, acting as a curing load, on top of the wall after completion of the wall (and during prolonged breaks during construction). Figure 6-4 shows the fabrication of the u-panels. Video footage of the construction of specimens 3J and 3K is included in Appendix A.

The specifications of the timber, wire and wire mesh reinforcement are provided. In the case of the bamboo reinforcement detailed material property testing was not undertaken. The bamboo selection was based on the principle of ‘consistently average’ bamboo, i.e. bamboo that was not cracked or damaged, and with approximate diameter of 16-22mm.

Each dried specimen was ‘remoulded’, which involved slightly wetting the surface and rubbing with a rough cloth to produce a smooth and attractive finish (Woodward, 1996). Each finished specimen weighed approximately 1,050kg. Prior to testing each specimen was ‘cured’ for a minimum of 28 days, under standard laboratory conditions [average temperature: 21.6°C (temperature range: 19.9–23.1°C); average humidity: 46.5% (humidity range: 28.3–65.7%)].

![Figure 6-3 U-panel specimens: plan layout (a) odd courses, (b) even courses.](image)

[*wall width = 0.15m, except for specimen 3H (= 0.10m)]
6.2.3 Wall restraint

For each specimen a downward restraining force was applied to the tops of the 'wing' walls (acting as in-plane shear walls) to simulate the restraint provided by a continuous wall, and to reduce sliding, rocking and overturning of the complete unit (Figure 6-2 and Figure 6-5). This restraint acted to effectively transfer the bulk of the seismic loading to the areas of main interest: the vulnerable out-of-plane long wall, and the corner connections.

The restraining force was applied by tension bars between a timber beam and platens (resting on the walls) and the concrete base (Figure 6-5). A pressure of ~125kPa was applied. This applied restraining force is a significant difference between this experiment and some other dynamic tests on u-shaped adobe wall panels, which do not include any 'wing' wall restraint (e.g. Zegarra et al, 1999). In such cases, the additional stiffness and restraint contributed by the shear walls ('wing' walls) is neglected, which seldom occurs in real structures (as discussed in Chapter 3). This experiment aims to incorporate the contribution of all shear walls in the dynamic response of the system.
For this series of tests a roof load was not applied. This was done for the following reasons:

- For highly seismic areas, a light-weight roof is advocated because it attracts less seismic force. A light-weight roof adds negligible mass to the structure, and was thus not included.
- Non-load bearing walls are common, e.g. post-and-beam houses, perimeter walls, etc.
- The absence of roof load ensures greater consistency in boundary conditions, especially between specimens with and without a ring beam.

Future testing should consider the effects of adding a heavy roof load.

### 6.2.4 Instrumentation

A series of accelerometers and dynamic LVDT (Linear Variable Differential Transformer) displacement transducers were used to record the dynamic response at key locations on each specimen and the shake table (ST) during the series of simulations. (The method of data collection and processing is described in greater detail in Chapters 5 and 8). The locations of the displacement sensors and accelerometers are shown in Figure 6-6 and Figure 6-7. The sensors were attached to small steel plates which were affixed to the wall.
Of main interest was the response of the midspan-top of the 'long' wall in relation to the ground motion (shake table displacement). For each specimen, graphs showing the displacement of the shake table (ST) and the midspan-top of the 'long' wall (L3) are presented. In the following chapter, the horizontal and vertical flexure of the wall are presented, utilising the data recorded from all LVDTs. Double differentiation of the displacement records was undertaken and compared with the accelerometer data to verify accuracy. The accelerometer data was used predominantly in the process of Experimental Modal Testing and Analysis (EMTA), which is presented in Chapter 8.
6.2.5 Nomenclature

In this chapter a shorthand system has been adopted for easy identification of distinct specimens, simulations, instrument locations, intensities and layout.

Specimens, simulations and locations

The shorthand used to describe the specimen, simulation and instrumentation location is shown in Figure 6-8.

![Diagram of nomenclature]

Figure 6-8 Nomenclature for specimens, simulations and locations.

For example:

- ‘3C – S6 – L3’ refers to Specimen 3C, Simulation S6 and Location L3.
- ‘3J – S10 – ST’ refers to Specimen 3J, Simulation S10 and Location ST (shake table)

Intensity and time scaling

The intensity of a simulation refers to its ‘displacement intensity’ (described in Chapter 5) and is preceded by an annotation ‘U’ or ‘S’, signifying ‘unscaled’ or ‘scaled’ (with respect to time). Thus, simulation S3 (U200%) refers to simulation S3, which uses the original time record (unscaled with respect to time) with a displacement intensity of 200% of the original record. And simulation S8 (S125%) refers to simulation S8, with time-scaled intensity of 125%. (Further discussion of time and intensity scaling of input time history is contained in Chapter 5.)
Specimen layout and direction of motion

Figure 6-9 shows the naming used to designate the layout of the specimens, as well as define the direction of motion.

![Diagram of U-panel layout and motion direction](image)

**Figure 6-9** U-panels: specimen layout and direction of motion.

### 6.2.6 Specimen specifications and results summary

Table 6-1 is a summary of the specifications of each u-shaped adobe wall unit (in chronological-testing order). A preliminary outline of the proposed specimen specifications was developed in the early phases of the research, based on field experience (Chapter 2) and review of literature (Chapter 3). This plan was modified as each specimen was built and tested. Modifications were based on lessons learned, results, field experience and input/feedback from observers and other researchers (these modifications are noted, as appropriate).

Table 6-2 is a summary of the testing sequence and results for all u-shaped adobe wall units.

Table 6-3 shows the grading system used to classify damage to the specimens at each stage of testing.
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Horizontal Reinforcement</th>
<th>Vertical Reinforcement</th>
<th>Ring Beam</th>
<th>Notes</th>
<th>Test Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>3A</td>
<td>(None)</td>
<td>(None)</td>
<td>(None)</td>
<td>Traditional, unreinforced</td>
<td>6 April 2004</td>
</tr>
<tr>
<td>3B</td>
<td>(None)</td>
<td>Corner pilasters</td>
<td>(None)</td>
<td></td>
<td>25 June 2004</td>
</tr>
<tr>
<td>3C</td>
<td>Chicken wire mesh (internal)</td>
<td>(None)</td>
<td>(None)</td>
<td></td>
<td>28 July 2004</td>
</tr>
<tr>
<td>3D</td>
<td>Chicken wire mesh (external wrapping)</td>
<td>Chicken wire mesh (external wrapping)</td>
<td>Timber</td>
<td></td>
<td>13 November 2004</td>
</tr>
<tr>
<td>3E</td>
<td>Chicken wire mesh (internal)</td>
<td>Bamboo (external) *</td>
<td>Timber</td>
<td></td>
<td>30 November 2004</td>
</tr>
<tr>
<td>3G</td>
<td>Chicken wire mesh (internal)</td>
<td>Bamboo (internal) *</td>
<td>Timber</td>
<td></td>
<td>9 December 2004</td>
</tr>
<tr>
<td>3I</td>
<td>Chicken wire mesh (internal)</td>
<td>Bamboo (external) *</td>
<td>Timber</td>
<td></td>
<td>19 January 2005</td>
</tr>
<tr>
<td></td>
<td>Bamboo (external)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3H</td>
<td>Chicken wire mesh (internal)</td>
<td>Bamboo (external)</td>
<td>Timber **</td>
<td>Thin wall (100mm)</td>
<td>14 April 2005</td>
</tr>
<tr>
<td>3F</td>
<td>Fencing wire (external)</td>
<td>Bamboo (external)</td>
<td>Timber **</td>
<td>Retrofit</td>
<td>27 April 2005</td>
</tr>
<tr>
<td>3J</td>
<td>Chicken wire mesh (internal)</td>
<td>Bamboo (external)</td>
<td>Timber **</td>
<td>Optimised</td>
<td>6 May 2005</td>
</tr>
<tr>
<td></td>
<td>Fencing wire (external)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3K</td>
<td>Chicken wire mesh (internal)</td>
<td>Timber poles (internal) *</td>
<td>Timber **</td>
<td></td>
<td>11 May 2005</td>
</tr>
</tbody>
</table>

Notes:

* Vertical reinforcement connected to concrete foundation of test frame (described in Section 6.7 below).

** Timber ring beam connected to wall restraint (described in Section 6.10 below).
Table 6-2 Summary of testing sequence and resultant damage grades* for all u-shaped adobe wall units.

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity** (Displacement)</th>
<th>3A</th>
<th>3B</th>
<th>3C</th>
<th>3D</th>
<th>3E</th>
<th>3G</th>
<th>3I</th>
<th>3H</th>
<th>3F</th>
<th>3J</th>
<th>3K</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unscaled</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>40%</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>100%</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>150% / 200%***</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Scaled</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>20%</td>
<td>0</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50%</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>75%</td>
<td>4</td>
<td>-</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>100%</td>
<td>-</td>
<td>4</td>
<td>4</td>
<td>3</td>
<td>1-2</td>
<td>-</td>
<td>0-1</td>
<td>1</td>
<td>1</td>
<td>0-1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>125%</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>3-4</td>
<td>-</td>
<td>1-2</td>
<td>1</td>
<td>2</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>75% (x2)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2-3</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2-3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>100% (x2)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>2-3</td>
<td>3-4</td>
</tr>
</tbody>
</table>

Notes:
* Damage grades: 0 – no damage; 1 – slight damage (fine cracks); 2 – moderate damage (small cracks, spalling); 3 – heavy damage (large + deep cracks); 4 – destruction (gaps in walls, separation of components); 5 – total collapse. [Based on IAEE (1986), see Table 6-3 for full descriptions.]
** Simulation of El Salvador earthquake, 13th January 2001 (Mw 7.7). (See Chapter 5 for details of time and intensity scaling of input spectrum)
*** S3: U200% for specimens 3A, 3B, 3C, 3D, 3E + 3G. U150% for specimens 3I, 3H, 3F + 3J. (See Section 6.8.3 for details)
Table 6-3 Classification of damage to buildings (IAEE, 1986).

<table>
<thead>
<tr>
<th>Damage grade</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 No damage</td>
<td>-</td>
</tr>
<tr>
<td>1 Slight damage</td>
<td>Fine cracks in plaster; fall of small pieces of plaster</td>
</tr>
<tr>
<td>2 Moderate damage</td>
<td>Small cracks in walls; fall of fairly large pieces of plaster, roof tiles slip off; cracks in chimneys; parts of chimney fall down.</td>
</tr>
<tr>
<td>3 Heavy damage</td>
<td>Large and deep cracks in walls; fall of chimneys.</td>
</tr>
<tr>
<td>4 Destruction</td>
<td>Gaps in walls; parts of building may collapse; separate parts of the building lose their cohesion; inner walls collapse.</td>
</tr>
<tr>
<td>5 Total damage</td>
<td>Total collapse of building.</td>
</tr>
</tbody>
</table>

Extensive video footage from each specimen is included in CD/DVD format in Appendix A. The footage covers key simulations, as well as a ‘post-mortem’ review of each specimen after the series of shake table simulations. Further details are included in Appendix A.
6.3 Specimen 3A

6.3.1 Specifications of 3A

<table>
<thead>
<tr>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen</td>
<td>3A</td>
</tr>
<tr>
<td>Horizontal reinforcement</td>
<td>None</td>
</tr>
<tr>
<td>Vertical reinforcement</td>
<td>None</td>
</tr>
<tr>
<td>Ring beam</td>
<td>None</td>
</tr>
<tr>
<td>1\textsuperscript{st} Natural Frequency</td>
<td>29.6 Hz</td>
</tr>
<tr>
<td>Time Scaling Factor</td>
<td>2.0</td>
</tr>
</tbody>
</table>

![Specimen 3A prior to testing](image)

Figure 6-10 Specimen 3A prior to testing.

6.3.2 Preparation of 3A

Specimen 3A represented a traditional, unreinforced adobe structure (Figure 6-10).

Construction was relatively straightforward, with no variations or special considerations. The advantage of this system is that it is quick and simple to construct, hence its widespread use. The disadvantage of this system is that it has very little seismic resistance, as seen in major earthquakes around the world (see Chapters 1 and 2), and in the results of this test.
6.3.3 Testing and results of 3A

Error! Reference source not found. shows the testing sequence and observations for specimen 3A.

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unscaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td></td>
<td>40%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S2</td>
<td></td>
<td>100%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S3</td>
<td></td>
<td>200%</td>
<td>Separation from base. No damage observed.</td>
</tr>
<tr>
<td>Scaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S4</td>
<td></td>
<td>20%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S5</td>
<td></td>
<td>50%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S6</td>
<td></td>
<td>75%</td>
<td>Severely damaged - collapse imminent (Figure 6-11).</td>
</tr>
</tbody>
</table>

Observations

- During simulation S3 (U200%) the specimen separated slightly from the concrete base, but the structure itself suffered no visible damage.

- During the time-scaled simulation S6 (S75% intensity) the specimen cracked in a brittle manner. The cracks propagated rapidly, with slow-motion video replay at 1/25 second barely able to determine the order of cracking. The left corner (Figure 6-11b) appeared to crack first, followed immediately by the diagonal cracking in the ‘long’ wall (Figure 6-11a) and vertical cracking at the midspan (Figure 6-11a) and right corner (Figure 6-11c).

- Figure 6-12 shows the displacements of the shake table (ST) and the midspan-top of the ‘long’ wall (L3) during simulation S5 (S50%). For this simulation (pre-cracking) the ratio of peak displacements (L3/ST) was 113.4%, indicating only a slight increase in the response of the specimen relative to ground motion. On the other hand, for simulation S6 (S75%), when cracking occurred, the ratio of peak displacements (L3/ST) was 442.5%. This feature can be clearly seen in Figure 6-13 which shows the displacements of the shake table (ST) and midspan-top of the
‘long’ wall (L3) during simulation S6 (S75%). The graphs show the response of the wall before, during and after cracking, which occurred around time $t = 22s$.

Figure 6-11 Specimen 3A after simulation S6 (S75%).
Figure 6-12  Specimen 3A: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S5 (S50%).

[Peak displacements: ST: -8.38mm (at time=19.29s) L3: -9.50mm (at time=19.30s)]
Figure 6-13 Specimen 3A: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S6 (S75%).

(Peak displacements: ST: -12.39mm (@ time=19.29s) L3: +54.83mm (@ time=28.82s))
Lessons learned

The testing of specimen 3A confirmed the destructive nature of ground motions containing sufficient energy and possessing dominant frequencies in the region of the natural frequencies of the wall unit. For these near-resonance conditions to occur it was necessary to modify the input time history spectrum, as described in Chapter 5. Specimen 3A displayed classic failure modes, indicative of damage to real houses subjected to real earthquakes. This feature confirms that the selected specimen configuration, boundary conditions and test response spectrum are acceptable for this type of experiment.

The first test highlighted the importance of video recording each simulation from a variety of angles in order to record the failure of the specimen. (Only one camera was used for specimen 3A, as shown in Appendix A. Additional cameras were used for subsequent tests).
6.4 Specimen 3B

6.4.1 Specifications of 3B

<table>
<thead>
<tr>
<th>Specimen:</th>
<th>3B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal reinforcement:</td>
<td>None</td>
</tr>
<tr>
<td>Vertical reinforcement:</td>
<td>Corner pilasters</td>
</tr>
<tr>
<td>Ring beam:</td>
<td>None</td>
</tr>
<tr>
<td>1st Natural Frequency:</td>
<td>34.1 Hz</td>
</tr>
<tr>
<td>Time Scaling Factor:</td>
<td>2.3</td>
</tr>
</tbody>
</table>

Figure 6-14 Specimen 3B prior to testing.

6.4.2 Preparation of 3B

The only alteration in Specimen 3B was the addition of pilasters (also known as buttresses) in the corners (Figure 6-14). The pilasters were fully integrated with the wall via stretcher bond, with pilaster length equal to the width of wall plus one mortar joint (ie. ~162mm representing ~325mm in full-size).

In theory, the pilasters are designed to resist the overturning experienced by the structure and provide additional stability to the vulnerable corners. The main disadvantage of the system is the increase in complexity, labour and costs of the preparation and construction of the structure. In a real building, the inclusion of
pilasters requires additional materials and labour at the foundation, plinth and roof levels, as well as additional bricks to make the pilasters themselves.

### 6.4.3 Testing and results of 3B

Table 6-5 shows the testing sequence and observations for specimen 3B.

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unscaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td></td>
<td>40%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S2</td>
<td></td>
<td>100%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S3</td>
<td></td>
<td>200%</td>
<td>Separation from base. No damage observed.</td>
</tr>
<tr>
<td>Scaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S5</td>
<td></td>
<td>40%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S7</td>
<td></td>
<td>100%</td>
<td>Severely damaged - collapse imminent (Figure 6-15).</td>
</tr>
</tbody>
</table>

An error in the simulation and intensity scaling procedure meant that the specimen was not subjected to the 20% and 75% intensity time-scaled simulations (S4 and S6). Major cracking occurred during the 100% intensity time-scaled simulation (S7) although it is expected that cracking would have occurred during the 75% intensity simulation (S6). In any case, the failure patterns are considered to be accurate, and cracking and failure of the structure can be judged at occurring between 75% and 100% intensity.

**Observations**

- Specimen 3B displayed similar failure patterns to specimen 3A, although the vertical midspan cracking in specimen 3B was more pronounced (Figure 6-15).
- An unexpected result was the orientation of the vertical crack at the left corner (Figure 6-15b). It was anticipated that the pilaster would provide some restraint to the overturning moment which induces tearing failure in the wall, and that cracking would initiate on the inside corner (parallel to the direction of motion), due to the build-up of shear and bending stresses (as seen in the right corner).
Figure 6-16 shows the displacements of the shake table (ST) and the midspan-top of the ‘long’ wall (L3) during simulation S7 (S100%). The loss of elasticity of the wall can be seen to occur around time $t = 22s$, with the LVDT at the midspan-top of the wall (L3) displaying an offset error.

Figure 6-15 Specimen 3B after simulation S7 (S100%).
Figure 6-16 Specimen 3B: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S7 (S100%).

[Peak displacements: ST: -15.38mm (@ time=23.12s) L3: unreliable, due to sensor error]
Lessons learned

The orientation of cracking in the left corner indicates that the corner pilaster in itself may not be providing the stability desired. This outcome, combined with the added complexity and cost of construction raises questions about the value of incorporating corner pilasters in small structures, and should be the subject of further research, including an assessment of optimal pilaster dimensions (depth and height) and comparison with other improvement systems. [The use of intermediate (midspan) pilasters in long walls is still advocated, although, similarly, optimal pilaster dimensions should be determined.]
6.5 Specimen 3C

6.5.1 Specifications of 3C

<table>
<thead>
<tr>
<th>Specimen:</th>
<th>3C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal reinforcement:</td>
<td>Chicken wire mesh (internal)</td>
</tr>
<tr>
<td>Vertical reinforcement:</td>
<td>None</td>
</tr>
<tr>
<td>Ring beam:</td>
<td>None</td>
</tr>
<tr>
<td>1st Natural Frequency:</td>
<td>33.0 Hz</td>
</tr>
<tr>
<td>Time Scaling Factor:</td>
<td>2.2</td>
</tr>
</tbody>
</table>

![Image of specimen preparation](image)

Figure 6-17 Preparation of specimen 3C
[Blue crosses indicate location of horizontal mesh reinforcing]

6.5.2 Preparation of 3C

Chicken wire mesh reinforcement\(^1\) was laid horizontally in the mortar joints (Figure 6-17a). The mesh was included between each course in the upper part of the wall, with the spacing increasing down the wall (shown by the blue crosses in Figure 6-17b) in an attempt to delay the vertical corner cracking which originates at the top of the wall. The wire mesh was slightly pre-tensioned during construction, although this was found to be time consuming and difficult and was discarded in subsequent specimens.

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\(^1\) Whites Wires ‘Vermin Netting’, width: 150mm, mesh size: 13mm, wire diameter: 0.56mm. The same brand and style of mesh were used in subsequent specimens 3E, 3G, 3I, 3H, 3J and 3K.
The advantage of internal horizontal mesh reinforcing is that it is relatively simple to incorporate into the construction of a new building. The mesh is easily laid in the mortar joints and does not cause obstructions to the construction process. Questions remain about the durability of light-gauge mesh (or organic materials such as bamboo, cane or vines which are sometimes used). Because they are embedded in the structure it is very difficult to assess their condition over time, and impossible to repair or replace if deterioration does occur.

### 6.5.3 Testing and results of 3C

Table 6-6 shows the testing sequence and observations for specimen 3C.

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unscaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td></td>
<td>40%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S2</td>
<td></td>
<td>100%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S3</td>
<td></td>
<td>200%</td>
<td>Separation from base. No damage observed.</td>
</tr>
<tr>
<td>Scaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S4</td>
<td></td>
<td>20%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S5</td>
<td></td>
<td>50%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S6</td>
<td></td>
<td>75%</td>
<td>Minor hairline cracking.</td>
</tr>
<tr>
<td>S7</td>
<td></td>
<td>100%</td>
<td>Severely damaged. Collapse prevented, but imminent (Figure 6-18).</td>
</tr>
</tbody>
</table>

**Observations**

- Again, the main failures were vertical cracking at the corners and vertical cracking at the midspan of the ‘long’ wall (Figure 6-18).
- During simulation S7 (S100%) the wire mesh broke in several critical locations in the upper courses (Figure 6-18 and Figure 6-19a). During demolition of the specimen it was observed that the mud mortar bonded very well to the mesh (Figure 6-19b).
Figure 6-18 Specimen 3C after simulation S7 (S100%).
• Figure 6-20 shows the displacements of the shake table (ST) and the midspan-top of the 'long' wall (L3) during simulation S6 (S75%). Prior to cracking, the ratio of peak displacements (L3/ST) was 122.6%, indicating only a slight increase in the response of the specimen relative to ground motion.

• Unfortunately the displacement data for simulation S7 (S100%) was lost due to computer malfunction. As a result, the displacement graphs and ratios are unknown for this simulation. It is expected that the displacement graph patterns would have been similar to those for simulation S7 (S100%) for specimen 3D (Figure 6-24).

Figure 6-19 Specimen 3C: condition of wire mesh after simulation S7 (S100%).
Figure 6-20 Specimen 3C: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S6 (S75%).

[Peak displacements: ST: -11.69mm (@ time=19.30s) L3: -14.33mm (@ time=23.11s)]
Lessons learned

The inclusion of light-gauge horizontal chicken wire mesh reinforcement appears to have improved the ductility of the structure. It is apparent, however, that more substantial means of reinforcement are necessary to significantly reduce the risk of collapse during severe earthquake loading. The internal horizontal wire mesh is relatively simple to include in new constructions, and could be a useful accompaniment to other forms of reinforcement, as considered in later specimens.
6.6 Specimen 3D

6.6.1 Specifications of 3D

<table>
<thead>
<tr>
<th>Specification</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen:</td>
<td>3D</td>
</tr>
<tr>
<td>Horizontal reinforcement:</td>
<td>Chicken wire mesh (external wrapping)</td>
</tr>
<tr>
<td>Vertical reinforcement:</td>
<td>Chicken wire mesh (external wrapping)</td>
</tr>
<tr>
<td>Ring beam:</td>
<td>Timber</td>
</tr>
<tr>
<td>1st Natural Frequency:</td>
<td>32.8 Hz</td>
</tr>
<tr>
<td>Time Scaling Factor:</td>
<td>2.2</td>
</tr>
</tbody>
</table>

(a) ![Image](image1.png)  (b) ![Image](image2.png)

Figure 6-21 Specimen 3D prior to testing.

6.6.2 Preparation of 3D

Specimen 3D was wrapped in light-gauge chicken wire mesh\(^2\). During the construction of the specimen polypropylene strings\(^3\) were laid horizontally across the mortar joints at ~180mm spacings (vertical and horizontal). A light timber\(^4\) ring beam was placed on top of the wall (Figure 6-21b) and attached via steel bolts grouted into slightly oversized holes in the top course of bricks. Prior to testing, the chicken wire mesh was placed

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\(^2\) Whites Wires 'Bird Wire Galvanised Netting', width: 900mm, mesh size: 13mm, wire diameter: 0.56mm.

\(^3\) All purpose Tapex Twine (polypropylene). This same or similar twine was used in all subsequent specimens.

\(^4\) Radiata pine, MGP10, width: 70mm, height: 45mm. The same material was used for all subsequent specimens.
against the wall and attached firmly via the polypropylene string (Figure 6-21a). Where necessary the wire mesh was overlapped to completely envelope the structure. Other researchers advocate the use of nails or staples to attach the chicken wire mesh to the wall, however these have certain limitations (such as little resistance to ‘pull out’ forces, and the potential cracking of the bricks).

It was decided not to apply any render to the structure because evidence in the field suggests that render is often poorly applied and deteriorates very quickly, and as such, cannot be considered to be a permanent part of the structure (see Chapter 2). Recent shake table tests undertaken at the Catholic University of Peru (PUCP) showed that the application of a mud and straw render/plaster significantly enhanced the seismic performance of a model adobe house reinforced with external geo-grid polymer mesh (Torrealva and Acero, 2005; Blondet et al, 2006b). This outcome suggests that if this reinforcing system is to be used most effectively, training should also include appropriate render application and maintenance.

One advantage of the external mesh wrapping, from a constructability perspective, is that it can be used for both new-build constructions and for the retrofit-strengthening of existing structures. Furthermore, if the wire mesh deteriorates or is damaged it is possible to repair or replace the mesh without undertaking a major structural operation. On the negative side, the mesh can be awkward to apply, especially at inside corners, and a lack of adequate overlap could introduce discontinuities in the reinforcement, making it more vulnerable to failure.
### 6.6.3 Testing and results of 3D

Table 6-7 shows the testing sequence and observations for specimen 3D.

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unscaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td>40%</td>
<td>No damage observed.</td>
<td></td>
</tr>
<tr>
<td>S2</td>
<td>100%</td>
<td>No damage observed.</td>
<td></td>
</tr>
<tr>
<td>S3</td>
<td>200%</td>
<td>Separation from base. No damage observed.</td>
<td></td>
</tr>
<tr>
<td>Scaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S4</td>
<td>20%</td>
<td>No damage observed.</td>
<td></td>
</tr>
<tr>
<td>S5</td>
<td>50%</td>
<td>No damage observed.</td>
<td></td>
</tr>
<tr>
<td>S6</td>
<td>75%</td>
<td>Minor hairline cracking.</td>
<td></td>
</tr>
<tr>
<td>S7</td>
<td>100%</td>
<td>Severely damaged. Collapse prevented (Figure 6-22).</td>
<td></td>
</tr>
</tbody>
</table>

**Observations**

- Again, the main failures were vertical cracking at the corners and vertical cracking at the midspan of the ‘long’ wall (Figure 6-22).

- Minor hairline cracking was observed at the corners (vertical) and midspan (vertical) during simulation S6 (S75%).

- During simulation S7 (S100%) the wire mesh stretched significantly. The wire mesh and polypropylene string did not appear to break in any locations.

- Figure 6-23 shows the displacements of the shake table (ST) and the midspan-top of the ‘long’ wall (L3) during simulation S6 (S75%). For this simulation, the ratio of peak displacements (L3/ST) was 118.8%, indicating only a slight increase in the response of the specimen relative to ground motion. On the other hand, for simulation S7 (S100%), when major cracking occurred, the ratio of peak displacements (L3/ST) was 378.8%. This feature can be clearly seen in Figure 6-24 which shows the displacements of the shake table (ST) and midspan-top of the ‘long’ wall (L3) during simulation S7 (S100%). The graphs show the response of
the wall before, during and after major cracking, which occurred around time $t = 16s$.

Figure 6-22 Specimen 3D after simulation S7 (S100%).
Figure 6-23 Specimen 3D: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S6 (S75%).

[Peak displacements: ST: -11.69mm (@ time=19.28s) L3: -13.89mm (@ time=19.31s)]
Figure 6-24  Specimen 3D: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S7 (S100%).

[Peak displacements: ST: +15.48mm ( @ time=22.80s) L3: +58.63mm ( @ time=30.62s)]
Lessons learned

Wrapping the specimen with light-gauge chicken wire mesh reinforcement and adding a timber ring beam significantly reduced the risk of collapse of the structure. The ring beam appears to have delayed the onset of initial cracking, and the mesh served to contain the structure after cracking occurred. These results represent a significant improvement in the safety of adobe structures during severe seismic activity. Heavy-gauge mesh (wire or plastic) has been demonstrated to provide additional seismic improvement (e.g. Torrealva and Acero, 2005; Blondet et al, 2006b; as discussed in Chapter 3) however in many developing countries such heavy-gauge mesh is not widely available and is relatively expensive, making it beyond the reach of the rural poor.
6.7 Specimen 3E

6.7.1 Specifications of 3E

<table>
<thead>
<tr>
<th>Specimen:</th>
<th>3E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal reinforcement:</td>
<td>Chicken wire mesh (internal)</td>
</tr>
<tr>
<td>Vertical reinforcement:</td>
<td>Bamboo (external)</td>
</tr>
<tr>
<td>Ring beam:</td>
<td>Timber</td>
</tr>
<tr>
<td>1st Natural Frequency:</td>
<td>30.8 Hz</td>
</tr>
<tr>
<td>Time Scaling Factor:</td>
<td>2.05</td>
</tr>
</tbody>
</table>

(a) ![Image](image1)

(b) ![Image](image2)

Figure 6-25 Specimen 3E prior to testing.

6.7.2 Preparation of 3E

Specimen 3E was the first specimen of the series to be externally reinforced with vertical bamboo poles. Other improvements included internal horizontal chicken wire mesh and a timber ring beam (Figure 6-25). Prior to construction the mesh was prepared, which included weaving polypropylene strings through the mesh (perpendicular to the wall) (Figure 6-26a). The mesh was then laid in the mortar joints every three courses during construction. 2mm diameter galvanised wire\(^5\) loops to tie

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\(^5\) Whites Wires 2.00mm diameter galvanised tie wire. This same wire was used for a variety of applications in all subsequent specimens.
down the timber ring beam were laid in the mortar joint four courses below the top of the wall, at 150mm spacing. After construction and curing of the wall the vertical bamboo poles were firmly connected to the concrete support base (Figure 6-26b) and attached to the wall (via the polypropylene string). The timber ring beam was connected to the top course of adobe bricks via pins (resisting shear forces) and the 2mm wire loops (resisting uplifting forces), as well as securely attached to the external bamboo by 2mm wire and staples (Figure 6-25b).

![a] Wire mesh and string  ![b] Bamboo connection with base and wall

Figure 6-26 Specimen 3E: construction details

The process of connecting the vertical bamboo with the concrete foundation proved to be slightly awkward. Tin cans were connected to the concrete base via steel plates and dyna-bolts. Screws were attached to the bamboo at the base to provide some shear connection (resisting uplift). The bamboo was inserted into the cans, which were then filled with a sand-cement mix. In the construction of a new house, tin cans or plastic drink bottles could be embedded in the foundation during construction. The bamboo could later be easily inserted into the cans and grouted into place.
6.7.3 Testing and results of 3E

Table 6-8 shows the testing sequence and observations for specimen 3E.

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unscaled</td>
<td>S1</td>
<td>40%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td></td>
<td>S2</td>
<td>100%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td></td>
<td>S3</td>
<td>200%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>Scaled</td>
<td>S4</td>
<td>20%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td></td>
<td>S5</td>
<td>50%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td></td>
<td>S6</td>
<td>75%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td></td>
<td>S7</td>
<td>100%</td>
<td>Minor hairline cracking.</td>
</tr>
<tr>
<td>S8*</td>
<td></td>
<td>125%*</td>
<td>Severely damaged. Collapse prevented (Figure 6-27).</td>
</tr>
</tbody>
</table>

* Simulation S8 (S125%) was incomplete due to shake table shut down just after the period of intense shaking. This is discussed in greater detail in Chapter 7.

Observations

- Again, the main failures were vertical cracking at the corners, however, in this specimen diagonal flexural cracks were also observed in the ‘long’ wall (Figure 6-27).
- Minor hairline cracking was observed at the corners (vertical) and midspan (diagonal) during simulation S7 (S100%).
- During simulation S8 (S125%) the horizontal wire mesh reinforcement broke in the upper parts at the corners (Figure 6-28). None of the polypropylene string appeared to break.
Figure 6-27 Specimen 3E after simulation S8 (S125%).
• The ‘wing’ walls suffered major damage at the point where the timber ring beam terminated (Figure 6-27 and Figure 6-28). This was due to the combination of shear and tearing stresses imparted from the ring beam attachment bolt onto the stiff shear ‘wing’ wall. In a continuous structure, the ring beam would extend along the length of the shear walls, thus distributing these forces more evenly through the structure. This problem was considered and addressed in subsequent tests.

• Figure 6-29 shows the displacements of the shake table (ST) and the midspan-top of the ‘long’ wall (L3) during simulation S7 (S100%). For this simulation, the ratio of peak displacements (L3/ST) was 147.2%, indicating a moderate increase in the response of the specimen relative to ground motion. This response was heightened during simulation S8 (S125%), when major cracking occurred, with the ratio of peak displacements (L3/ST) being 431.5%. This feature can be clearly seen in Figure 6-30 which shows the displacements of the shake table (ST) and midspan-top of the ‘long’ wall (L3) during simulation S8 (S125%). The graphs show the response of the wall before, during and after major cracking, which occurred around time $t = 23s$.

Figure 6-28 Specimen 3E: mesh and wing wall failure during simulation S8 (S125%).
Figure 6-29 Specimen 3E: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S7 (S100%).

[Peak displacements: ST: -15.69mm (@ time=19.30s) L3: -23.10mm (@ time=23.15s)]
Figure 6-30 Specimen 3E: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S7 (S125%).
(Peak displacements: ST: +19.41mm (at time=22.81s) L3: +83.77mm (at time=29.85s))
Lessons learned

The reinforcement system of specimen 3E provided a significant improvement in the seismic capacity, both delaying the onset of initial cracking and preventing collapse of the damaged wall at high intensity simulations. The occurrence of diagonal cracking in the ‘long’ wall indicates a more even distribution of energy in and around the structure.

Some observers have suggested that the home owners may not like the aesthetic of the bamboo poles on the outside of the wall. In such cases, an attractive finish could be easily achieved by covering the wall and reinforcement with an appropriate render (e.g. lime, sand and/or mud, Figure 6-31). The addition of render also has the potential to improve the seismic capacity of the structure, as shown in shake table tests in Peru (Torrealva and Acero, 2005; Blondet et al, 2006b). If the reinforcement was too thick, it could be split, and/or a small vertical channel could be carved into the wall in which to bed the reinforcement. Periodically, the render could be removed at certain locations and the condition of the reinforcement assessed. Deteriorated reinforcement could be easily removed and replaced, and a new render applied. Cracking and spalling of the render is expected during significant seismic events.
6.8 Specimen 3G

6.8.1 Specifications of 3G

<table>
<thead>
<tr>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen:</td>
<td>3G</td>
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<tr>
<td>Horizontal reinforcement:</td>
<td>Chicken wire mesh (internal)</td>
</tr>
<tr>
<td>Vertical reinforcement:</td>
<td>Bamboo (internal)</td>
</tr>
<tr>
<td>Ring beam:</td>
<td>Timber</td>
</tr>
<tr>
<td>1st Natural Frequency:</td>
<td>25.0 Hz</td>
</tr>
<tr>
<td>Time Scaling Factor:</td>
<td>1.67</td>
</tr>
</tbody>
</table>

![Figure 6-32 Specimen 3G prior to testing.](image)

[Orange arrows indicate the location of the internal vertical bamboo]

6.8.2 Preparation of 3G

Specimen 3G was reinforced with internal vertical bamboo reinforcement, internal horizontal chicken wire mesh (every three courses), and a timber ring beam (Figure 6-32). The bamboo was securely attached to the foundation prior to construction, and half bricks and full bricks with notches were configured to encase the bamboo at alternate courses (Figure 6-33). The ring beam was tied down by strands of ~3mm wire which were looped and laid in the mortar joints four courses from the top and extended upwards, entwining the bamboo poles.
This method, or derivations thereof, have been recommended by a variety of sources (e.g. IAEE, 1986; RESESCO, 1997; MTC, 2000; Equipo Maíz, 2001; Pérez, 2001), as described in Chapter 3. This approach was used in the child-care centre construction discussed in Chapter 2.

![Diagram](image)

(a)

(b)

(c)

Figure 6-33 Layout and configuration of bricks, poles and mesh for specimen 3G.

From a practical viewpoint, the use of internal vertical reinforcement is fraught with difficulties. The method is complex and time consuming, and as a result, it is unlikely to be widely accepted in practice, particularly without the support of trained and skilled masons (who don't like the system anyway!). Complications arise in each stage of wall construction, from preparation of the foundation and initial alignment of reinforcement, to adjusting and trimming the bricks to fit the reinforcement (natural products such as bamboo are seldom straight and defect-free), to the placement and adequate connection of the ring beam. In times of wet-weather during construction, the walls are difficult and time-consuming to completely cover and protect. There are also some concerns
about the durability of the natural materials commonly used as internal vertical reinforcement (e.g. bamboo, reeds, timber). There is little doubt that when the internal reinforcement is completely encased it is afforded some protection from attack by insects, air and moisture, however, it is extremely difficult to adequately assess the condition of the reinforcement over time, and it is not possible to change the reinforcement if deterioration has occurred.

These factors have been the major motivation to explore alternative options, which provide significantly improved seismic performance, yet are still simple, affordable and easily repairable if deterioration does occur.

### 6.8.3 Testing and results of 3G

Table 6-9 shows the testing sequence and observations for specimen 3G.

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unscaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td></td>
<td>40%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S2</td>
<td></td>
<td>100%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S3</td>
<td></td>
<td>200%</td>
<td>Significant cracking (Figure 6-34).</td>
</tr>
</tbody>
</table>

* No time-scaled simulations were undertaken, as described below.

**Observations**

- Specimen 3G did not behave as expected. The specimen experienced significant cracking after the 200% intensity unscaled (time) simulation (S3) (Figure 6-34). This situation did not occur in any of the other specimens, and suggests significant weaknesses in the structure.

- The midspan cracking in the ‘long’ wall occurred at the location of the internal vertical bamboo, indicating a significant difference in the response of the stiff bricks and the flexible bamboo. This may also account for the lower global stiffness (first natural frequency) of the structure prior to testing.
Figure 6-34 Specimen 3G after simulation S3 (U200%).
[Orange arrows indicate the location of the internal vertical bamboo]

- Figure 6-35 shows the displacements of the shake table (ST) and the midspan-top of the ‘long’ wall (L3) during simulation S3 (U200%). For this simulation, the ratio of peak displacements (L3/ST) was 122.6%, indicating a moderate increase in the response of the specimen relative to ground motion. These figures show the dynamic amplification of the response of the wall, especially around $t = 20s$. 
Figure 6.35 Specimen 3G: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S3 (U200%).

[Peak displacements: ST: +36.018mm (@ time=19.303s) L3: -44.153mm (@ time=21.954s)]
Lessons learned

Results indicate that the internal vertical bamboo introduced discontinuity to the structure, which created planes of weakness. The results raise questions about the structural improvement which can be achieved using internal vertical reinforcement, even at low intensity shaking. In order to verify this outcome it was decided to build and test an additional specimen (3K) with a slightly modified (and idealised) version of this reinforcement configuration (discussed later).

It should also be noted that the specimen was not tested beyond the 200% unscaled (time) simulation. Feedback from a variety of adobe experts, plus comments from Tolles and Krawinkler (1990), suggest value in testing specimens to complete destruction in order to assess the capacity of the damaged structure to resist collapse. This aspect was noted and it was decided to subject subsequent specimens (3I, 3H, 3F, 3J and 3K) to additional, high intensity simulations.

It is surmised that the 1:2 linear scaling of the specimen may have contributed to the poor performance of the structure. The use of half bricks with notches to accommodate the bamboo resulted in an effective cover (distance from face of wall to bamboo) of ~60 mm (even less when the bamboo poles were misaligned), whereas in full-size (1:1) application, the effective cover would be in the region of ~120-130 mm for a 300 mm thick wall. This scaling factor may have adversely affected the results in this particular case.

Because the specimen experienced significant cracking during unscaled (with respect to time) 200% intensity simulation (S3) it was decided to limit the intensity of subsequent unscaled (time) simulations to 150%. All other specimens have shown these first three simulations (using the original record) to be of minimal value and to have little impact on the overall behaviour of the specimen.
6.9 Specimen 3I

6.9.1 Specifications of 3I

Specimen: 3I
Horizontal reinforcement: Chicken wire mesh (internal)
Bamboo (external)
Vertical reinforcement: Bamboo (external)
Ring beam: Timber
1st Natural Frequency: 31.6 Hz
Time Scaling Factor: 2.17

Figure 6-36 Specimen 3I prior to testing.

6.9.2 Preparation of 3I

Specimen 3I was constructed in a similar manner to specimen 3E, except that on the outside face of the structure, horizontal bamboo reinforcement was used instead of vertical bamboo reinforcement (Figure 6-36). This was done in an attempt to arrest the vertical crack development at the corners. The horizontal bamboo was securely connected at the corners by drilling the bamboo and inserting small wire pins (Figure 6-36b).
The main practical shortcoming of this method is the large cross-over of the horizontal bamboo reinforcement at the corners (this would also be complicated at window and door openings). This cross-over would cause complications if a render were to be applied to the wall, however, if the walls were to be left un-rendered, then the use of external horizontal reinforcement presents no significant problems in terms of constructability.

6.9.3 Testing and results of 3L

Table 6-10 shows the testing sequence and observations for specimen 3L.

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unscaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td></td>
<td>40%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S2</td>
<td></td>
<td>100%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S3</td>
<td></td>
<td>150%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>Scaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S4</td>
<td></td>
<td>20%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S5</td>
<td></td>
<td>50%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S6</td>
<td></td>
<td>75%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S7</td>
<td></td>
<td>100%</td>
<td>Very minor hairline cracking.</td>
</tr>
<tr>
<td>S8*</td>
<td></td>
<td>125%*</td>
<td>Minor cracking (&lt;3mm) (Figure 6-40).</td>
</tr>
<tr>
<td>S9-S10</td>
<td></td>
<td>75%</td>
<td>(Progressive damage).</td>
</tr>
<tr>
<td>S11-S12</td>
<td></td>
<td>100%</td>
<td>(Progressive damage).</td>
</tr>
<tr>
<td>S13-S14</td>
<td></td>
<td>100%</td>
<td>Severely damaged. Collapse prevented (Figure 6-37).</td>
</tr>
</tbody>
</table>

* Simulation S8 (125%) was incomplete due to shake table shut down just after the period of intense shaking. This is discussed in greater detail in Chapter 7.
Observations

- Specimen 3l performed in a similar manner to specimen 3E, although the initiation and severity of cracking appears to have been delayed and reduced in specimen 3l. The specimen maintained excellent structural integrity, even during high intensity simulations (S7: 100% and S8: 125%) as shown in Figure 6-40. The specimen suffered progressive additional damage during two time-scaled simulations at 75% intensity (S9, S10), followed by four time-scaled simulations at 100% intensity (S11, S12, S13, S14). The results after this series of severe earthquakes are shown in Figure 6-37 and reveal the structure to be still safely standing, despite being significantly damaged.

- Again, the main failures were vertical cracking at the corners, plus vertical and diagonal flexural cracks in the ‘long’ wall (Figure 6-37).

- Very minor hairline cracking was observed at the corners (vertical) and midspan (diagonal) during simulation S7 (S100%).

- Again, the location and pattern of damage in the ‘wing’ wall (Figure 6-37c) coincided with the termination of the ring beam, as noted in specimen 3E. This feature is discussed in greater detail later.

- Figure 6-38 shows the displacements of the shake table (ST) and the midspan-top of the ‘long’ wall (L3) during simulation S7 (S100%). For this simulation, the ratio of peak displacements (L3/ST) was 141.5%, indicating a moderate increase in the response of the specimen relative to ground motion. This response was only moderately increased during simulation S8 (S125%), with the ratio of peak displacements (L3/ST) being 175.7%. This result represents the most significant improvement in comparison with the previous six specimens (3A, 3B, 3C, 3D, 3E and 3G). Figure 6-39 shows the displacements of the shake table (ST) and midspan-top of the ‘long’ wall (L3) for specimen 3l during simulation S8 (S125%). The graphs show the moderate response of the wall during such intense shaking. The condition of the wall after simulation S8 (S125%) is shown in Figure 6-40.
Figure 6-37 Specimen 3I after simulation S14 (S100% repeated).
Figure 6-38 Specimen 3L: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S7 (S100%).

[Peak displacements: ST: -15.60mm (at time=23.12s) L3: -22.08mm (at time=23.14s)]
Figure 6-39  Specimen 3I: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S8 (S125%).
[Peak displacements: ST: -19.47mm (@ time=23.13s) L3: -34.21mm (@ time=23.18s)]
Figure 6-40 Specimen 3I after simulation S8 (S125% intensity).
Lessons learned

The results confirm the importance and value of testing the structure beyond initial significant cracking, with intent to cause severe damage and/or collapse. The specimen performed very well under the repeated extreme loading conditions. Results demonstrated the importance of including some form of horizontal reinforcement, which acts to delay vertical crack formation and propagation.

The damage in the right ‘wing’ wall (Figure 6-37c) at the point where the timber ring beam terminated confirms that the discontinuity of the ring beam significantly influences the crack distribution in the wall. In order to more accurately simulate the continuity of a complete ring beam a small wire loop was attached between the ring beam and the ‘wing’ wall restraint in subsequent specimens, as described in Section 6.10.2 below.
6.10 Specimen 3H

6.10.1 Specifications of 3H

<table>
<thead>
<tr>
<th>Specimen:</th>
<th>3H</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal reinforcement:</td>
<td>Chicken wire mesh (internal)</td>
</tr>
<tr>
<td>Vertical reinforcement:</td>
<td>Bamboo (external)</td>
</tr>
<tr>
<td>Ring beam:</td>
<td>Timber</td>
</tr>
<tr>
<td>Other features:</td>
<td>Thin walls (100mm thick)</td>
</tr>
<tr>
<td>1st Natural Frequency:</td>
<td>33.0 Hz</td>
</tr>
<tr>
<td>Time Scaling Factor:</td>
<td>2.2</td>
</tr>
</tbody>
</table>

Figure 6-41 Specimen 3H prior to testing.

6.10.2 Preparation of 3H

Specimen 3H was built with thinner walls (100mm thick, compared with 150mm for all other specimens). This was done in response to the trend towards thinner walls in actual construction, due mainly to the material, time and brick fabrication saving which such thin walls offer. The height-thickness (h/t) ratio of the walls in specimen 3H was 12, compared with a h/t ratio of 8 for all other specimens. There is abundant theoretical and practical evidence in favour of a lower height-thickness ratio, which provides greater stability and higher resistance to overturning moment (Webster, 1995; Tolles et al, 2000). A h/t ratio of 8 is widely accepted and promoted as an appropriate ratio in a variety of adobe building codes (IAEE, 1986; RESESCO, 1997). What is not apparent
with these recommendations is the influence which reinforcing has on the overall structural behaviour, and the potential to increase the h/t ratio if the structure is adequately reinforced.

In response to the serious damage observed in the ‘wing’ walls at the point where the timber ring beam terminated (specimens 3E and 3I, above) a method to more accurately simulate the continuity of a complete ring beam was sought. This was trialled in specimen 3H by attaching a small wire loop between the ring beam and the ‘wing’ wall restraint (Figure 6-42). This loop was designed to provide a semi-rigid connection to allow some movement, as expected with a continuous timber ring beam.

In all other respects specimen 3H was constructed in a similar manner to specimen 3E.

![Figure 6-42 Specimen 3H: ring beam – wall restraint connection.](image-url)
6.10.3 Testing and results of 3H

Table 6-11 shows the testing sequence and observations for specimen 3H.

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unscaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td></td>
<td>40%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S2</td>
<td></td>
<td>100%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S3</td>
<td></td>
<td>150%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>Scaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S4</td>
<td></td>
<td>20%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S5</td>
<td></td>
<td>50%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S6</td>
<td></td>
<td>75%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S7</td>
<td></td>
<td>100%</td>
<td>Minor hairline cracking. Separation from base.</td>
</tr>
<tr>
<td>S8*</td>
<td></td>
<td>125%*</td>
<td>Widespread cracking (&lt;3mm) (Figure 6-44).</td>
</tr>
<tr>
<td>S9-S10</td>
<td></td>
<td>75%</td>
<td>(Progressive damage).</td>
</tr>
<tr>
<td>S11-S12</td>
<td></td>
<td>100%</td>
<td>Severely damaged. Collapse prevented (Figure 6-43).</td>
</tr>
</tbody>
</table>

* Simulation S8 (S125%) was incomplete due to shake table shut down just after the period of intense shaking. This is discussed in greater detail in Chapter 7.

Observations

- Again, the main failures were vertical cracking at the corners, plus vertical and diagonal flexural cracks in the ‘long’ wall (Figure 6-43).
- Minor hairline cracking was observed at the corners (vertical) and midspan (diagonal) during simulation S7 (S100%).
- The wire loop connection between the ring beam and ‘wing’ wall restraint appears to have eliminated the severe local damage at the end of the ring beam, as seen in specimens 3E and 3l.
- The ring beam broke at a knot in the timber (at the midspan of the wall) during simulation S11 (S100% repeated).
Figure 6-43 Specimen 3H after simulation S12 (S100% repeated).

- Figure 6-45 shows the displacements of the shake table (ST) and the midspan-top of the ‘long’ wall (L3) during simulation S7 (S100%). For this simulation, the ratio of peak displacements (L3/ST) was 125.8%, indicating a moderate increase in the response of the specimen relative to ground motion. This response increased during
simulation S8 (S125%), with the ratio of peak displacements (L3/ST) being 231.9%. Figure 6-46 shows the displacements of the shake table (ST) and midspan-top of the ‘long’ wall (L3) during simulation S8 (S125%). The condition of the specimen after simulation S8 (S125%) is shown in Figure 6-44.

Figure 6-44 Specimen 3H after simulation S8 (S125%).
Figure 6.45 Specimen 3H: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S7 (S100%).
[Peak displacements: ST: -15.57mm (@ time=23.13s) L3: -19.59mm (@ time=23.13s)]
Figure 6.46 Specimen 3H: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S8 (S125%).

(Peak displacements: ST: -19.29mm (@ time=23.12s) L3: -44.74mm (@ time=23.17s))
Lessons learned

The results suggest that for reinforced structures the thickness of the wall seems to be less critical for overall seismic performance than for unreinforced structures. Although the structure was significantly damaged during the series of high intensity simulations, the onset of initial cracking was delayed and collapse was prevented. This outcome highlights the importance of considering the combined and conflicting contributions of thicker walls and external vertical reinforcement. Thick walls have greater mass, which attracts greater seismic force. As a consequence, any external reinforcement used should be strong enough to adequately resist and contain this additional force. It would seem that there may be an optimal height-thickness ratio for lightly reinforced structures which is worthy of further investigation.
6.11 Specimen 3F

6.11.1 Specifications of 3F

<table>
<thead>
<tr>
<th>Specimen:</th>
<th>3F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal reinforcement:</td>
<td>Fencing wire (external)</td>
</tr>
<tr>
<td>Vertical reinforcement:</td>
<td>Bamboo (external)</td>
</tr>
<tr>
<td>Ring beam:</td>
<td>Timber</td>
</tr>
<tr>
<td>Other comments:</td>
<td>Retrofit of unreinforced specimen</td>
</tr>
<tr>
<td>1\textsuperscript{st} Natural Frequency:</td>
<td>33.7 Hz</td>
</tr>
<tr>
<td>Time Scaling Factor:</td>
<td>2.2</td>
</tr>
</tbody>
</table>

Figure 6-47 Specimen 3F prior to testing.

6.11.2 Preparation of 3F

Specimen 3F was initially constructed as an unreinforced specimen, in the same manner as specimen 3A. After the wall panel was dry a process of retrofit-strengthening was undertaken. This consisted of drilling holes through the wall at pre-determined locations (aligned with the desired location of bamboo reinforcing), inserting polypropylene string loops into the holes (Figure 6-48a), wetting the holes and injecting mud into the holes (Figure 6-48b&c). A plastic syringe with the end cut open was used to inject the mud. The mud was the same as the material used in the bricks and mortar.
Once the mud plugs were thoroughly dried, bamboo poles were placed vertically against the wall and securely tied. For this specimen no attachment was made between the vertical bamboo poles and the concrete foundation, as can be more readily expected in real-life application for the retrofit-strengthening of an existing dwelling. A timber ring beam was placed on top of the wall and attached via pins and wire, as described in Section 6.7.2 above. 2mm gauge galvanised fencing wire was looped and tied horizontally between the bamboo poles (Figure 6-47). Once all the bamboo and wire was in position the wires were gradually tensioned (using simple fencing pliers) around the structure to create a tight mesh-like matrix (Figure 6-49). The small wire loops were connected between the ring beam and ‘wing’ wall restraint, as trialled in specimen 3H.
6.11.3 Testing and results of 3F

Table 6-12 shows the testing sequence and observations for specimen 3F.

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unscaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td></td>
<td>40%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S2</td>
<td></td>
<td>100%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S3</td>
<td></td>
<td>150%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>Scaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S4</td>
<td></td>
<td>20%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S5</td>
<td></td>
<td>50%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S6</td>
<td></td>
<td>75%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S7</td>
<td></td>
<td>100%</td>
<td>Minor hairline cracking.</td>
</tr>
<tr>
<td>S8*</td>
<td></td>
<td>125%*</td>
<td>Minor cracking (&lt;3mm) (Figure 6-54).</td>
</tr>
<tr>
<td>S9-S10</td>
<td></td>
<td>75%</td>
<td>(Progressive damage).</td>
</tr>
<tr>
<td>S11-S12</td>
<td></td>
<td>100%</td>
<td>Significantly damaged. Collapse prevented (Figure 6-50).</td>
</tr>
</tbody>
</table>

* Simulation S8 (S125%) was incomplete due to shake table shut down just after the period of intense shaking. This is discussed in greater detail in Chapter 7.

Observations

- Again, the main failures were vertical cracking at the corners, plus vertical and diagonal flexural cracks in the ‘long’ wall (Figure 6-50).
- Minor hairline cracking was first observed at the corners (vertical) and midspan (diagonal) during simulation S7 (S100%).
During simulation S9 (S75% repeated) one of the horizontal wires (second from the top) at the right corner snapped at the twist in the wire (Figure 6-50c and Figure 6-51). The wire was not repaired or replaced for subsequent simulations.
Figure 6-51 Specimen 3F: snapped wire during simulation S9 (S75% repeated).

- Figure 6-52 shows the displacements of the shake table (ST) and the midspan-top of the ‘long’ wall (L3) during simulation S7 (S100%). For this simulation, the ratio of peak displacements (L3/ST) was 121.6%, indicating a moderate increase in the response of the specimen relative to ground motion. This response increased slightly during simulation S8 (S125%), with the ratio of peak displacements (L3/ST) being 147.6%. Figure 6-53 shows the displacements of the shake table (ST) and midspan-top of the ‘long’ wall (L3) during simulation S8 (S125%). The condition of the specimen after simulation S8 (S125%) is shown in Figure 6-54.
Figure 6-52 Specimen 3F: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S7 (S100%).

[Peak displacements: ST: -15.63mm (@ time=23.11s) L3: -19.01mm (@ time=23.11s)]
Figure 6-53 Specimen 3F: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S8 (S125%).

[Peak displacements: ST: -19.41 mm (@ time=23.12s) L3: -28.66 mm (@ time=23.16s)]
Figure 6-54 Specimen 3F after simulation S8 (S125%).
Lessons learned

The retrofit-strengthening system tested provided a major improvement to the overall structural capacity of the specimen. It shows that the presence of internal horizontal chicken-wire mesh is not essential to substantially improve the earthquake resistance of a mudbrick structure.

The omission of the connection between the vertical bamboo and the base slab appears to have made little difference to the overall behaviour of the structure. This result is a significant outcome, because omission of the connection saves considerable time and effort in the construction process, especially for the retrofit-strengthening of existing structures.

The concentration of cracking around the midspan of the ‘long’ wall (also evident in all previous specimens) would suggest there is merit in providing additional vertical reinforcement at this location. This was considered and applied in specimen 3J, below.

Only one horizontal wire connector snapped during the testing. This occurred at the twist in the wire, which indicates the need to exercise caution when tightening the wire. The advantage of the external reinforcement is that the wire (and bamboo) can be periodically checked and changed if damaged or deteriorated. This would be especially important after a significant seismic event in the region.

The same retrofitting procedure was used for model house 4A (Chapter 9). Video footage showing this procedure is included in Appendix A.
6.12 Specimen 3J

6.12.1 Specifications of 3J

Specimen: 3J  
Horizontal reinforcement: Chicken wire mesh (internal)  
Fencing wire (external)  
Vertical reinforcement: Bamboo (external)  
Ring beam: Timber  
1st Natural Frequency: 33.0 Hz  
Time Scaling Factor: 2.2

(a)  
Figure 6-55 Specimen 3J prior to testing.

(b)

6.12.2 Preparation of 3J

Specimen 3J was built to incorporate the most effective and practical modifications and key lessons learned from the previous specimens. As such, it is considered to be the 'optimised' or 'ideal' system.

Specimen 3J was constructed with chicken wire mesh running horizontally in the mortar joints every three courses (Figure 6-56a). Wall thickness was 150mm, as in all specimens except 3H (100mm thick walls). Vertical bamboo reinforcement was placed on each face of the wall (connected via polypropylene strings passing through the wall,
Additional bamboo poles were placed at the midspan of the ‘long’ wall. There was no attachment between the vertical bamboo and the base slab. 2mm gauge galvanised fencing wire was tied horizontally between the bamboo poles at upper, middle and lower wall height (additional reinforcing wires were placed at the external corners). A timber ring beam was placed on the top of the wall and connected via steel pins (to the wall) and wire (to the bamboo) (Figure 6-56d). Wire loops were connected between the timber ring beam and the ‘wing’ wall restraint (as described in specimen 3H), to more accurately simulate the effect of a continuous ring beam. Video footage of the construction of specimen 3J is included in Appendix A.

Figure 6-56  Specimen 3J under construction.
### 6.12.3 Testing and results of 3J

Table 6-13 shows the testing sequence and observations for specimen 3J.

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unscaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td></td>
<td>40%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S2</td>
<td></td>
<td>100%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S3</td>
<td></td>
<td>150%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S4</td>
<td></td>
<td>20%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S5</td>
<td></td>
<td>50%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S6</td>
<td></td>
<td>75%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S7</td>
<td></td>
<td>100%</td>
<td>Minor hairline cracking. Separation from base.</td>
</tr>
<tr>
<td>S8*</td>
<td></td>
<td>125%*</td>
<td>Minor cracking (&lt;3mm).</td>
</tr>
<tr>
<td>S9-S10</td>
<td></td>
<td>75%</td>
<td>(Progressive damage).</td>
</tr>
<tr>
<td>S11-S12</td>
<td></td>
<td>100%</td>
<td>Significantly damaged. Collapse prevented (Figure 6-57).</td>
</tr>
</tbody>
</table>

* Simulation S8 (S125%) was incomplete due to shake table shut down just after the period of intense shaking. This is discussed in greater detail in Chapter 7.

**Observations**

- Specimen 3J performed the best out of those tested. Even after the series of intense simulations the structure was still relatively stable, and was able to be safely and securely removed from the table after testing (all others were demolished *in situ*).
- Again, the main failures were vertical cracking at the corners, plus vertical and diagonal flexural cracks in the ‘long’ wall (Figure 6-57).
- Minor hairline cracking was observed at the corners (vertical) and midspan (diagonal) during simulation S7 (S100%) and S8 (S125%) (Figure 6-58).
- None of the external horizontal wires snapped during the simulations.
Figure 6-57 Specimen 3J after simulation S12 (S100% repeated).
• Figure 6-59 shows the displacements of the shake table (ST) and the midspan-top of the ‘long’ wall (L3) during simulation S7 (S100%). For this simulation, the ratio of peak displacements (L3/ST) was 122.8%, indicating a moderate increase in the response of the specimen relative to ground motion. This response increased slightly during simulation S8 (S125%), with the ratio of peak displacements (L3/ST) being 144.3%. Figure 6-60 shows the displacements of the shake table (ST) and midspan-top of the ‘long’ wall (L3) during simulation S8 (S125%). The condition of the specimen after simulation S8 (S125%) is shown in Figure 6-58.

• Because of the stability of the specimen the instrumentation was not removed for the latter simulations, up to S11 (S100% repeated). Even for simulation S11 (S100% repeated), after the series of damaging simulations, the ratio of peak displacements (L3/ST) increased only moderately from simulation S8 (S125%). Figure 6-61 shows the displacements of ST and L3 during simulation S11 (S100% repeated). For this simulation the ratio of peaks (L3/ST) was 175.3%.
Figure 6-58 Specimen 3J after simulation S8 (S125%).
Figure 6-59 Specimen 3J: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S7 (S100%).

[Peak displacements: ST: -15.60mm (@ time=23.13s) L3: -19.15mm (@ time=23.14s)]
Figure 6-60 Specimen 3J: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S8 (S125%).

[Peak displacements: ST: -27.93mm (at time=19.34s) L3: -19.35mm (at time=23.13s)]
Figure 6-61 Specimen 3J: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S11 (S100% repeated).

[Peak displacements: ST: -15.598mm (@ time=23.112s) L3: -27.340mm (@ time=18.494s)]
Lessons learned

The modifications included in specimen 3J provided a major improvement to the seismic capacity of the structure. The specimen performed extremely well. The reinforcement system acted to delay the onset of initial cracking and reduce the severity of cracking, even during the series of high intensity simulations.

The results also confirmed the observation from specimen 3F that the connection between the vertical bamboo and the base slab appears to make little difference to the overall behaviour of the structure.

It is difficult to determine the contribution of the additional vertical bamboo pole at the midspan of the ‘long’ wall on the overall aseismic capacity. Theoretically, the additional pole would provide some extra support at this critical area, although it could be considered more worthwhile to provide additional horizontal wire reinforcement at the midspan.

The construction process and test results confirm specimen 3J as being an ‘optimised’ or ‘ideal’ system. Consequently, a combination of the systems used in specimens 3F and 3J were selected for the testing of the full model adobe house (4A), as described in Chapter 9.

The relative performance of specimen 3J is analysed and discussed further in the following chapter.
6.13 Specimen 3K

6.13.1 Specifications of 3K

<table>
<thead>
<tr>
<th>Specification</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen</td>
<td>3K</td>
</tr>
<tr>
<td>Horizontal reinforcement</td>
<td>Chicken wire mesh (internal)</td>
</tr>
<tr>
<td>Vertical reinforcement</td>
<td>Timber poles (internal)</td>
</tr>
<tr>
<td>Ring beam</td>
<td>Timber</td>
</tr>
<tr>
<td>1st Natural Frequency</td>
<td>27.0 Hz</td>
</tr>
<tr>
<td>Time Scaling Factor</td>
<td>1.8</td>
</tr>
</tbody>
</table>

![Figure 6-62 Specimen 3K prior to testing](image)

Figure 6-62 Specimen 3K prior to testing

[Orange arrows indicate the location of the internal vertical bamboo]

6.13.2 Preparation of 3K

Specimen 3K was constructed using the same general principles as specimen 3G (Section 6.8 above). This included internal vertical pole reinforcement, internal horizontal chicken wire mesh (every three courses), and a timber ring beam. This configuration was retested because it is widely advocated as being the best means of improving the seismic resistance of adobe structures (as described in Chapter 3 and in Section 6.8), and it seemed necessary to subject the specimen to a series of time-scaled shake table tests. Based on the lessons learned during the construction and testing of specimen 3G a number of modifications were incorporated in specimen 3K. These
modifications represent an ‘idealised’ system, which served to test the best conditions under which this system could be used. The changes made from specimen 3G to specimen 3K include:

- The internal vertical bamboo poles were replaced with timber poles, which were of uniform straightness and consistent 25mm diameter (Figure 6-63a&c).
- Greater care and time was taken to align the timber poles ‘perfectly’ at both the base (Figure 6-63a) and the top (Figure 6-62b) to ensure accurate spacing and verticality and avoid the need to adjust bricks and notches during construction.
- A custom-made diamond-tipped drill bit was fabricated to create notches in the bricks (Figure 6-63b). This was done to ensure minimal damage to the bricks during the notching process (for specimen 3G, the bricks were notched using a more aggressive grinding bit which mauled out the notch).
- Wire loops were connected between the timber ring beam and the ‘wing’ wall restraint (as described in specimen 3H), to more accurately simulate the effect of a continuous ring beam.

In practice these modifications would be too costly, complicated and time-consuming to be widely adopted in the field.

Video footage of the construction of specimen 3K is included in Appendix A.

![Figure 6-63 Specimen 3K: preparation of poles and bricks.](image-url)
6.13.3 Testing and results of 3K

Table 6-14 shows the testing sequence and observations for specimen 3K.

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scaled</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S4</td>
<td></td>
<td>20%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S5</td>
<td></td>
<td>50%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S6</td>
<td></td>
<td>75%</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S7</td>
<td></td>
<td>100%</td>
<td>Minor hairline cracking. Separation from base.</td>
</tr>
<tr>
<td>S8</td>
<td></td>
<td>125%</td>
<td>Minor cracking (&lt;3mm).</td>
</tr>
<tr>
<td>S9-S10</td>
<td></td>
<td>75%</td>
<td>(Progressive damage).</td>
</tr>
<tr>
<td>S11-S12</td>
<td></td>
<td>100%</td>
<td>Significantly damaged. Collapse prevented (Figure 6-64).</td>
</tr>
</tbody>
</table>

The unscaled (with respect to time) simulations S1 (U50%), S2 (U100%) and S3 (U150%) were not undertaken to avoid the risk of damaging the structure prematurely, as done in specimen 3G. Unlike for specimens 3I, 3H, 3F and 3J, simulation S8 (S125%) ran for the entire duration of the record without the shake table shutting down. This was most probably due to the lower natural frequency of the specimen which required a different time scaling factor. This meant that the velocity and acceleration of the simulation were slightly lower than for the other specimens.

Observations

- The structure performed well and resisted collapse under extreme loading. Again, the main failures were vertical cracking in the ‘wing’ walls, plus vertical and diagonal flexural cracks in the ‘long’ wall (Figure 6-64).
- As observed in specimen 3G, the vertical cracking was concentrated around the location of the internal vertical poles. This phenomenon may be attributed to a difference in dynamic response between the stiff mudbrick wall and the flexible timber poles which caused a pounding effect in the out-of-plane ‘long’ wall. In the shear ‘wing’ wall, the vertical poles introduced a discontinuity in the wall, which
reduced the effective cross-sectional area, and increased the likelihood of tearing failure (Figure 6-64b).

Figure 6-64 Specimen 3K after simulation S12 (S100% repeated).
(Orange arrows indicate the location of the internal vertical bamboo)
- During the latter intense simulations the left ‘wing’ wall (Figure 6-64b) experienced major movement (clearly seen in the video footage, in Appendix A). The wall experienced significant overturning movement, akin to a hinged connection at the base of the wall.

- Minor hairline cracking was observed at the corners (vertical) and midspan (diagonal) during simulation S7 (S100%) and S8 (S125%) (Figure 6-67).

- Figure 6-65 shows the displacements of the shake table (ST) and the midspan-top of the ‘long’ wall (L3) during simulation S7 (S100%). For this simulation, the ratio of peak displacements (L3/ST) was 130.8\%, indicating a moderate increase in the response of the specimen relative to ground motion. This response increased slightly during simulation S8 (S125\%), with the ratio of peak displacements (L3/ST) being 157.3\%. Figure 6-66 shows the displacements of the shake table (ST) and midspan-top of the ‘long’ wall (L3) during simulation S8 (S125\%). The condition of the specimen after simulation S8 (S125\%) is shown in Figure 6-67.
Figure 6-65 Specimen 3K: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S7 (S100%).

[Peak displacements: ST: -16.54mm (@ time=19.30s) L3: -21.64mm (@ time=19.33s)]
Figure 6-66 Specimen 3K: absolute displacement of shake table (ST) and midspan-top of wall (L3) for simulation S8 (S125%).

[Peak displacements: ST: -20.63mm ( @ time=19.31s) L3: +32.46mm ( @ time=29.62s)]
Figure 6-67 Specimen 3K after simulation S8 (S125%).
Lessons learned

The ‘idealised’ system tested in specimen 3K performed well. The onset of initial cracking was delayed and the severity of cracking was reduced, even during the series of high intensity simulations.

Results confirmed that the internal vertical poles introduced discontinuity to the structure, which created planes of weakness. These discontinuities are cause for great concern especially during severe ground motion.

Although the structure performed well during the simulations, the construction process confirmed that the main problem with this method is the complexity of construction. Significant skill levels and time are required to build this system effectively, both of which are often scarce in poor rural areas.
6.14 Summary

The successful testing of eleven u-shaped adobe wall units revealed the following general outcomes:

- Major improvements in the seismic capacity of adobe mudbrick structures can be achieved using low-cost and low-tech means. Such improvements are viable and effective for both new-build constructions and for the retrofit-strengthening of existing dwellings.

- U-shaped adobe wall panels (with appropriate ‘wing’ wall restraint) exhibit classic failure patterns when subjected to shake table testing using a suitable input time history. Damages were consistent with real structures subjected to real earthquakes.

- Test results confirm the importance of appropriate time scaling of input time history to induce damaging near-resonance conditions in a structure. Time scaling is also necessary to ensure dynamic similitude between specimens, such that accurate comparisons may be made between the performance of different specimens. (A comparative analysis of the u-panel results is presented in greater detail in the following chapter).

- Test results challenge the assumption that corner pilasters/buttresses will adequately restrain the out-of-plane overturning moment induced in a wall, which create vertical corner cracking due to tearing failure in the orthogonal wall.

- Test results indicate that there is some uncertainty relating to the structural performance of internal vertical reinforcement, with significant cracking occurring at a lower intensity than expected. It would appear that the presence of internal vertical reinforcement introduces significant discontinuities in the structure. This aspect, coupled with the complexity of construction, raises questions about the viability of using this form of reinforcement.

- Test results indicate that significant improvement in the earthquake resistance of adobe mudbrick structures can be obtained by using external vertical and/or horizontal bamboo reinforcement, external horizontal wire and/or internal horizontal chicken wire mesh reinforcement and a ring beam. These additions, when securely tied together, create an integrated matrix which restrains movement and enhances the overall strength of the structure. The reinforcement system acted to delay the onset of initial cracking, and reduce the severity of cracking during repeated high intensity shaking. Most importantly, collapse of reinforced structures was prevented.
7 U-SHAPED ADOBE WALL UNITS: COMPARATIVE ANALYSIS

7.1 Introduction

This chapter provides a detailed, comparative analysis and discussion of results from the eleven u-shaped adobe wall unit tests (the ‘why’ and ‘what does it mean’). This analysis builds on the observations presented in Chapter 6 (which described ‘what happened’ to each individual specimen). This chapter endeavours to interpret and explain the factors which cause damage, and considers the capacity of different reinforcing systems to reduce these damages.

In Chapter 6 the response and capacity of each specimen was presented individually. The photographs, displacement-time history graphs and observations indicated both a commonality of damage patterns, and clear differences in performance and seismic response. In this chapter, the relative behaviour of different specimens and reinforcement systems is considered. Specimen 3J is taken as a benchmark because it represents an ‘optimal’ system and performed the best of the eleven specimens tested. Absolute displacement, relative displacement, peak displacements, root mean square (RMS) displacement and flexure snapshots are presented to understand these differences in behaviour. Particular attention is given to the performance of six key specimens (3A, 3C, 3F, 3H, 3J and 3K), because they represent the most common or important systems, and/or they exhibited typical response patterns. These specimens represent unreinforced (3A), lightly reinforced (3C), retrofit (3F), thin wall (3H), externally reinforced (3J) and internally reinforced (3K) systems.

In Chapter 6 it was observed that each specimen exhibited vertical corner cracking at the intersection of the orthogonal walls, plus vertical, horizontal and diagonal cracking in the out-of-plane ‘long’ wall (with variations in the location and extent of damage due to the reinforcing system used). This chapter considers the main factors which contribute to these damage patterns:

- Large relative displacement between the out-of-plane ‘long’ wall (experiencing an overturning moment) and the stiff in-plane shear ‘wing’ wall, leading to
tearing (tensile) stresses in both the mortar-brick interface and the individual brick units at the corner intersection (contributing to vertical corner cracking).

- Horizontal flexure induced in the out-of-plane 'long' wall, causing a splitting-crushing cycle at the mid-span of the 'long' wall and the intersection with the orthogonal shear 'wing' walls (leading to mid-span vertical cracking in the 'long' wall, and contributing to vertical corner cracking and diagonal cracking).

- Vertical flexure and overturning in the out-of-plane 'long' wall (leading to horizontal cracking and contributing to diagonal cracking, in combination with the horizontal flexure).

This chapter presents each of the key factors described above and draws on the extensive test data to assess the contribution of different reinforcing systems in ameliorating these distresses.

The specimen specifications presented in Chapter 6 are reproduced in Table 7-1. For ease and clarity the specimens are presented in alpha-numeric order in this chapter, rather than chronological order. The same nomenclature and instrumentation designations are used as in Chapter 6. In this chapter, results from time-scaled simulations S6 (S75%), S7 (S100%) and S8 (S125%) are presented. The results from specimen 3G have been omitted from this chapter because no time-scaled simulations were undertaken for specimen 3G, as described in Chapter 6.

At the end of this chapter the seismic capacity, plus the complexity and cost of construction of each system is presented. This information is summarised in a matrix, which reflects the importance of the non-technical factors in the assessment of the suitability of different reinforcement systems.
Table 7-1 U-panels: specimen specifications.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Horizontal Reinforcement</th>
<th>Vertical Reinforcement</th>
<th>Ring Beam</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>3A</td>
<td>(None)</td>
<td>(None)</td>
<td>(None)</td>
<td>Traditional, un-reinforced</td>
</tr>
<tr>
<td>3B</td>
<td>(None)</td>
<td>Corner pilasters</td>
<td>(None)</td>
<td></td>
</tr>
<tr>
<td>3C</td>
<td>Chicken wire mesh (internal)</td>
<td>(None)</td>
<td>(None)</td>
<td></td>
</tr>
<tr>
<td>3D</td>
<td>Chicken wire mesh (external wrapping)</td>
<td>Chicken wire mesh (external wrapping)</td>
<td>Timber</td>
<td></td>
</tr>
<tr>
<td>3E</td>
<td>Chicken wire mesh (internal)</td>
<td>Bamboo (external) *</td>
<td>Timber</td>
<td></td>
</tr>
<tr>
<td>3F</td>
<td>Fencing wire (external)</td>
<td>Bamboo (external)</td>
<td>Timber **</td>
<td>Retrofit</td>
</tr>
<tr>
<td>3G</td>
<td>Chicken wire mesh (internal)</td>
<td>Bamboo (internal) *</td>
<td>Timber</td>
<td></td>
</tr>
<tr>
<td>3H</td>
<td>Chicken wire mesh (internal)</td>
<td>Bamboo (external)</td>
<td>Timber **</td>
<td>Thin wall (100mm)</td>
</tr>
<tr>
<td>3I</td>
<td>Chicken wire mesh (internal)</td>
<td>Bamboo (external) *</td>
<td>Timber</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Bamboo (external)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3J</td>
<td>Chicken wire mesh (internal)</td>
<td>Bamboo (external)</td>
<td>Timber **</td>
<td>Optimised</td>
</tr>
<tr>
<td></td>
<td>Fencing wire (external)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3K</td>
<td>Chicken wire mesh (internal)</td>
<td>Timber poles (internal) *</td>
<td>Timber **</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
* Vertical reinforcement connected to concrete foundation of test frame (described in Chapter 6).
** Timber ring beam connected to wall restraint (described in Chapter 6).
7.2 Data selection and results summary

7.2.1 Introduction

In this chapter the following quantitative response data is considered:

- **Absolute displacement** refers to the displacement as measured. It is often presented with the shake table (ST) displacement and is used to show the actual movement of the specimen, as well as the amplification of the response of the wall with respect to the excitation (ST displacement). The displacement-time graphs in Chapter 6 present the absolute displacement of each specimen during key simulations.

- **Relative displacement** refers to the displacement at a given location (e.g. L3) with respect to the shake table (ST) displacement (i.e. L3-ST). This is used to more accurately consider the response of the wall at a given time and place.

- **Peak displacement** refers to the absolute value of the maximum or minimum displacement. The peak displacement may refer to the peak of the absolute displacement, or the peak of the relative displacement. This distinction is noted, as appropriate. The peak displacement is simple to determine, is a tangible entity, and is traditionally used to represent the maximum response of a structure under excitation. The main drawback of the peak displacement is that irregular behaviour and spikes may be misunderstood to represent the overall response of the structure.

- **Root mean square (RMS)** (also known as the quadratic mean) is a statistical measure of the magnitude of a varying quantity. It provides an indication of the magnitude of the displacement over the duration of the simulation, and is more representative of the actual response of a structure for non-stationary excitation (e.g. earthquakes).

The RMS can be calculated for a collection of \( N \) values \( \{x_1, x_2, \ldots, x_N\} \) using equation 7-1:
\[ X_{rms}^2 = \sqrt{\frac{1}{N} \sum_{i=1}^{N} X_i^2} = \sqrt{\frac{X_1^2 + X_2^2 + \ldots + X_N^2}{N}} \]  

7-1

It can also be calculated more simply using equation 7-2:

\[ X_{rms}^2 = \bar{x}^2 + \sigma_x^2 \]  

7-2

Where:

\[ \bar{x} = \text{arithmetic mean}; \text{ and} \]

\[ \sigma_x = \text{standard deviation}. \]

In this research project, equation 7-2 was used to determine the RMS. In this case the mean is fixed (an average over the entire duration of the simulation), although when a specimen cracks the mean may change during the course of the simulation (non-linear, inelastic behaviour). This aspect is observed and discussed in Section 7.3.3 below.

In this chapter the displacement data is presented in the form of summary tables, time history graphs and flexure snapshots, which show the response of each specimen during key simulations.
7.2.2 RMS and peak displacements

Figure 7-1 to Figure 7-3 shows the relationship between the peak of the relative displacement (plotted against the left, vertical axis) and the RMS of the relative displacement (right, vertical axis) for the full simulation sequence S1-S8 for specimens 3A, 3J and 3K. It can be seen that for the lower intensity simulations (prior to significant damage in the structure) there is strong correlation between the peak and RMS. On the other hand, for the latter simulations, when damage to the structure has occurred, there is a poor correlation between the peak and RMS. This would suggest that for undamaged specimens, both the peak and RMS appropriately represent the response of the structure, but for damaged specimens the RMS is considered to be more representative (although the peak displacement is still worthy of consideration).

![Graph showing peak and RMS displacements](image)

**Figure 7-1 Specimen 3A: peak and RMS of relative displacement at midspan-top of wall (L3-ST) for simulations S1 – S6.**
Figure 7-2 Specimen 3J: peak and RMS of relative displacement at midspan-top of wall (L3-ST) for simulations S1 – S8.

Figure 7-3 Specimen 3K: peak and RMS of relative displacement at midspan-top of wall (L3-ST) for simulations S4 – S8.
7.2.3 Absolute displacement

Table 7-2 shows the peak absolute displacement at the midspan-top of the wall (L3) and the ratio of peaks with respect to the peak shake table displacement (ST) for simulations S6, S7 and S8 for all specimens.

Table 7-3 shows the RMS of absolute displacement at the midspan-top of the wall (L3) and the ratio of RMS with respect to the RMS of the shake table displacement (ST) for simulations S6, S7 and S8 for all specimens.

For these tables, a colour-coding system has been developed (in conjunction with the observations in Chapter 6) to signify the relative response of each specimen: (i) negligible response [green]; (ii) moderate response [dark yellow]; (iii) major response [orange]; and (iv) severe response [red].

Both the peak and RMS of the displacement indicate the progression of damage of each specimen, and provide a comparison between the performance of different specimens. Of particular note is the significant improvement provided by the external bamboo reinforcement (specimens 3E, 3F, 3H, 3I and 3J) compared with the unreinforced (3A) and lightly reinforced specimens (3B, 3C and 3D). Specimen 3K exhibited significant movement even during the lower intensity simulation S6 (S75%), and experienced more movement than both specimens 3F and 3J in all simulations. Specimens 3F and 3J exhibited almost identical behaviour. This suggests that the internal chicken wire mesh reinforcement (3J) does not significantly influence the response of the structure at the pre-damaged and lightly damaged states. It also shows that the retrofit-strengthening technique (3F) effectively produces an equivalent seismic capacity as can be achieved for a new-build construction (3J).
Table 7-2 All specimens: peak absolute displacement (mm) at midspan-top of wall (L3) and ratio of peaks with respect to peak shake table (L ST) displacement for simulations S6, S7 and S8.

<table>
<thead>
<tr>
<th>Intensity</th>
<th>3A</th>
<th>3B</th>
<th>3C</th>
<th>3D</th>
<th>3E</th>
<th>3F</th>
<th>3H</th>
<th>3I</th>
<th>3J</th>
<th>3K</th>
</tr>
</thead>
<tbody>
<tr>
<td>w.r.t. Peak (L ST)</td>
<td>443%</td>
<td>-</td>
<td>123%</td>
<td>119%</td>
<td>123%</td>
<td>114%</td>
<td>116%</td>
<td>120%</td>
<td>109%</td>
<td>124%</td>
</tr>
<tr>
<td>S7 100%</td>
<td>Peak (L3)</td>
<td>-</td>
<td>47.077</td>
<td>-</td>
<td>58.627</td>
<td>23.100</td>
<td>19.006</td>
<td>19.591</td>
<td>22.077</td>
<td>19.152</td>
</tr>
<tr>
<td>w.r.t. Peak (L ST)</td>
<td>-</td>
<td>306%</td>
<td>-</td>
<td>379%</td>
<td>147%</td>
<td>122%</td>
<td>126%</td>
<td>142%</td>
<td>123%</td>
<td>131%</td>
</tr>
<tr>
<td>S8 125%</td>
<td>Peak (L3)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>83.774</td>
<td>28.656</td>
<td>44.738</td>
<td>34.211</td>
<td>27.925</td>
</tr>
<tr>
<td>w.r.t. Peak (L ST)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>432%</td>
<td>148%</td>
<td>232%</td>
<td>176%</td>
<td>144%</td>
</tr>
</tbody>
</table>

**Notes:**
* Data from 3C-S7 was lost due to a technical error.

**Key:**
- **Green:** negligible response (up to 120%)
- **Dark yellow:** moderate response (120-149%)
- **Orange:** major response (150-199%)
- **Red:** severe response (above 200%)
<table>
<thead>
<tr>
<th>Intensity</th>
<th>Intensity</th>
<th>3A</th>
<th>3B</th>
<th>3C</th>
<th>3D</th>
<th>3E</th>
<th>3F</th>
<th>3H</th>
<th>3I</th>
<th>3J</th>
<th>3K</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>w.r.t. RMS (L ST)</td>
<td>353%</td>
<td>-</td>
<td>116%</td>
<td>112%</td>
<td>110%</td>
<td>104%</td>
<td>105%</td>
<td>109%</td>
<td>104%</td>
<td>112%</td>
</tr>
<tr>
<td>S7</td>
<td>S100%</td>
<td>-</td>
<td>35.860</td>
<td>-</td>
<td>14.893</td>
<td>5.544</td>
<td>4.723</td>
<td>5.004</td>
<td>5.255</td>
<td>4.833</td>
<td>5.489</td>
</tr>
<tr>
<td></td>
<td>w.r.t. RMS (L ST)</td>
<td>-</td>
<td>850%</td>
<td>-</td>
<td>35%</td>
<td>129%</td>
<td>111%</td>
<td>118%</td>
<td>123%</td>
<td>113%</td>
<td>124%</td>
</tr>
<tr>
<td>S8*</td>
<td>S125%</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>16.074</td>
<td>8.126</td>
<td>10.850</td>
<td>8.279</td>
<td>8.123</td>
<td>9.215</td>
<td></td>
</tr>
<tr>
<td></td>
<td>w.r.t. RMS (L ST)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>250%</td>
<td>127%</td>
<td>171%</td>
<td>152%</td>
<td>127%</td>
<td>140%</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
* For simulation S8, the RMS was calculated for the first 26 seconds of the simulation for all specimens to ensure consistency. (For all specimens, except 3K, the shake table shut down prematurely between 26 and 30 seconds, due to table overloading.)

Key:
- **Green**: negligible response (up to 110%)
- **Dark yellow**: moderate response (110-129%)
- **Orange**: major response (130-170%)
- **Red**: severe response (above 170%)
7.3  Relative displacement

7.3.1  Introduction

This section contains a number of graphs showing the relative displacement at the midspan-top of the ‘long’ wall (L3-ST) for key specimens during simulations S6, S7 and S8 (S75%, S100% and S125%, respectively). Each figure presents the relative displacement of the specimen of interest, plus the relative displacement of specimen 3J. Analysis of these graphs provides a clear insight into the behaviour of each specimen and allows comparisons to be made between specimens.

In this section, the location of focus is the midspan-top of the ‘long’ wall (L3), which in most cases experienced the greatest movement in the structure (except after major cracking in the structure), due to the combination of vertical and horizontal flexure (described in greater detail in Section 7.4 below). All references to displacement, relative displacement and peak displacement in this section (7.3) refer to the movement at the midspan-top of the ‘long’ wall (L3).

Table 7-4 shows the peak relative displacement at the midspan-top of the wall (L3) and the ratio of peaks with respect to the peak relative displacement of specimen 3J (shaded) for simulations S6, S7 and S8 for all specimens.

Table 7-5 shows the RMS of relative displacement at the midspan-top of wall (L3) and the ratio of RMS with respect to the RMS of the relative displacement of specimen 3J (shaded) for simulations S6, S7 and S8 for all specimens.
Table 7-4 All specimens: peak relative displacement* (mm) at midspan-top of wall (L3-ST) and ratio of peaks with respect to benchmark specimen 3J (shaded) for simulations S6, S7 and S8.

<table>
<thead>
<tr>
<th>Intensity</th>
<th>3A</th>
<th>3B</th>
<th>3C</th>
<th>3D</th>
<th>3E</th>
<th>3F</th>
<th>3H</th>
<th>3I</th>
<th>3J</th>
<th>3K</th>
</tr>
</thead>
<tbody>
<tr>
<td>S6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S75%</td>
<td>Peak (L3-ST)</td>
<td>52.335</td>
<td>-</td>
<td>6.398</td>
<td>5.245</td>
<td>4.824</td>
<td>2.157</td>
<td>2.324</td>
<td>4.112</td>
<td>2.258</td>
</tr>
<tr>
<td>w.r.t. Peak 3J (L3-ST)</td>
<td>2,318%</td>
<td>-</td>
<td>283%</td>
<td>232%</td>
<td>214%</td>
<td>96%</td>
<td>103%</td>
<td>182%</td>
<td>100%</td>
<td>242%</td>
</tr>
<tr>
<td>S7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S100%</td>
<td>Peak (L3-ST)</td>
<td>-</td>
<td>62.460</td>
<td>-</td>
<td>55.663</td>
<td>10.947</td>
<td>6.805</td>
<td>8.742</td>
<td>9.064</td>
<td>7.490</td>
</tr>
<tr>
<td>w.r.t. Peak 3J (L3-ST)</td>
<td>-</td>
<td>834%</td>
<td>-</td>
<td>743%</td>
<td>146%</td>
<td>91%</td>
<td>117%</td>
<td>121%</td>
<td>100%</td>
<td>144%</td>
</tr>
<tr>
<td>S8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S125%</td>
<td>Peak (L3-ST)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>87.440</td>
<td>17.013</td>
<td>34.814</td>
<td>20.709</td>
<td>14.508</td>
</tr>
<tr>
<td>w.r.t. Peak 3J (L3-ST)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>603%</td>
<td>117%</td>
<td>240%</td>
<td>143%</td>
<td>100%</td>
<td>151%</td>
</tr>
</tbody>
</table>

Notes:
* Relative displacement is the displacement of the midspan-top of wall (L3) relative to the shake table displacement (ST), i.e. L3-ST.
Table 7-5  All specimens: RMS of relative displacement* (mm) at midspan-top of wall (L3-ST) and ratio of RMS with respect to benchmark specimen 3J (shaded) for simulations S6, S7 and S8.

<table>
<thead>
<tr>
<th>Intensity</th>
<th>3A</th>
<th>3B</th>
<th>3C</th>
<th>3D</th>
<th>3E</th>
<th>3F</th>
<th>3H</th>
<th>3I</th>
<th>3J</th>
<th>3K</th>
</tr>
</thead>
<tbody>
<tr>
<td>S6</td>
<td>11.660</td>
<td>-</td>
<td>1.020</td>
<td>0.862</td>
<td>0.660</td>
<td>0.309</td>
<td>0.395</td>
<td>0.576</td>
<td>0.300</td>
<td>0.807</td>
</tr>
<tr>
<td>w.r.t. RMS 3J (L3-ST)</td>
<td>3,887%</td>
<td>-</td>
<td>340%</td>
<td>287%</td>
<td>220%</td>
<td>103%</td>
<td>132%</td>
<td>192%</td>
<td>100%</td>
<td>269%</td>
</tr>
<tr>
<td>S7</td>
<td>-</td>
<td>35.955</td>
<td>-</td>
<td>14.176</td>
<td>2.318</td>
<td>1.046</td>
<td>1.519</td>
<td>1.795</td>
<td>1.242</td>
<td>1.982</td>
</tr>
<tr>
<td>w.r.t. RMS 3J (L3-ST)</td>
<td>-</td>
<td>2,896%</td>
<td>-</td>
<td>1,142%</td>
<td>187%</td>
<td>84%</td>
<td>122%</td>
<td>145%</td>
<td>100%</td>
<td>160%</td>
</tr>
<tr>
<td>S8**</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>13.906</td>
<td>2.978</td>
<td>6.657</td>
<td>4.083</td>
<td>2.862</td>
</tr>
<tr>
<td>w.r.t. RMS 3J (L3-ST)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>486%</td>
<td>104%</td>
<td>233%</td>
<td>143%</td>
<td>100%</td>
</tr>
</tbody>
</table>

Notes:
* Relative displacement is the displacement of the midspan-top of wall (L3) relative to the shake table displacement (ST), i.e. L3-ST.
** For simulation S8, the RMS was calculated for the first 26 seconds of the simulation for all specimens to ensure consistency. (For all specimens, except 3K, the shake table shut down prematurely between 26 and 30 seconds, due to table overloading.)
7.3.2 Simulation S6 (S75%)  

The relative displacements of the midspan-top of the 'long' wall (L3-ST) for specimens 3A, 3C, 3J and 3K for simulation S6 (S75%) are presented in Figure 7-4 to Figure 7-6. For each specimen the relative displacement is shown for the full simulation (0-60s), a 15-second snapshot (15-30s) and a 5-second close-up shot during intense shaking.

Figure 7-4 shows the major difference in relative displacement between the unreinforced specimen 3A and the reinforced specimen 3J. Major cracking of specimen 3A accounted for the very large ratio of peak relative displacement for specimen 3A with respect to specimen 3J (2,318%). Initial cracking of specimen 3A appears to have occurred around \( t = 19.3 \) seconds, with significant cracking occurring around \( t = 22s \).

Figure 7-5 shows the response of the wall for specimens 3C and 3J. Even though the ratio of peak displacements (L3/ST) for specimen 3C was a moderate 123% (Table 7-2), the ratio of peak relative displacements with respect to 3J (ie. 3A/3J) was a more substantial 283% (Table 7-4). This indicates that the horizontal chicken wire mesh reinforcement (3C) provided a significant improvement to the behaviour of the structure (compared with the unreinforced specimen 3A), but was significantly less effective than the reinforcing system used in specimen 3J. Results indicate that the mesh acted to reduce movement of the wall and, in this simulation, prevented catastrophic cracking. Minor cracking of specimen 3C appears to have occurred around time \( t = 19.3s \).

The response of specimen 3K compared with specimen 3J is shown in Figure 7-6. The graphs show the large relative response of specimen 3K, even for this moderate intensity ground motion. The ratio of peak relative displacement (3K/3J) was only moderately less (242%) than for 3C/3J (283%). These results suggest that specimen 3K experienced a significant loss in stiffness, even prior to simulation S6 (S75%). This aspect is considered in greater detail with respect to the Experimental Modal Testing and Analysis (EMTA), in Chapter 8.

For each of the specimens 3A, 3C and 3K the loss of stiffness (decrease in natural frequency) due to damage in the structure is evident in the graphs, with fewer zero crossings, compared with specimen 3J (most apparent in the close-up snapshots).
Figure 7-4 Specimens 3A and 3J: relative displacement at midspan-top of wall (L3) for simulation S6 (S75%).
Figure 7-5 Specimens 3C and 3J: relative displacement at midspan-top of wall (L3) for simulation S6 (S75%).
Figure 7-6 Specimens 3K and 3J: relative displacement at midspan-top of wall (L3) for simulation S6 (S75%).
7.3.3 Simulation S7 (S100%)

The relative displacements of the midspan-top of the 'long' wall (L3-ST) for specimens 3H, 3J and 3K for simulation S7 (S100%) are presented in Figure 7-7 and Figure 7-8.

Of note in these figures is the offset evident at the end of the simulation for specimen 3J. It would appear that this -1mm offset occurred during time t = 20-30 seconds, and may have been due to minor cracking of the specimen (non-linear, inelastic behaviour), which resulted in the establishment of a new equilibrium position. Alternatively this may have been due to slippage at the base of the whole specimen.

As expected, the amplification of the response at L3 was moderately larger for the thin-wall specimen 3H than for specimen 3J (Figure 7-7). This was most apparent in the period after t = 25 seconds, although this also coincided with the offset experienced by specimen 3J. Specimens 3H and 3J appeared to exhibit the same frequency (stiffness) during the simulation, as seen by the matched pattern (zero crossings) in Figure 7-7c.

For specimen 3K (Figure 7-8) the amplification of the response was much larger than for specimen 3J, even in the initial stages of the simulation when there was relatively little ground motion. This follows from the finding from simulation S6 (above) that specimen 3K was experiencing a progressive loss of stiffness, even from the low intensity simulations (most probably due to discontinuities and cracking around the internal vertical reinforcement). This loss of stiffness (lower frequency) is seen in Figure 7-8c, as well as in the EMTA (Chapter 8). Again, the offset of the displacement sensor in specimen 3J is noted, however in this case, the amplification of the response of specimen 3K is much greater, so the effect of the offset is less apparent.

For specimens 3H and 3K the peak relative displacement at the midspan-top of the wall occurred at the same time (~30.53 seconds), which was much later than the time which the shake table imparted the peak ground motion (~23.13 seconds). This confirms that cracking had occurred in these specimens during the simulation and, as expected, the resultant amplification of the response of the wall was much greater than for the undamaged/slightly damaged specimen 3J. (This feature is also presented with the aid of flexure graphs in Section 7.4.2 below).
Figure 7-7 Specimens 3H and 3J: relative displacement at midspan-top of wall (L3-ST) for simulation S7 (S100%).
Figure 7-8 Specimens 3K and 3J: relative displacement at midspan-top of wall (L3-ST) for simulation S7 (S100%).
7.3.4 Simulation S8 (S125%)

The relative displacements of the midspan-top of the 'long' wall (L3-ST) for specimens 3F, 3H, 3J and 3K for simulation S8 (S125%) are presented in Figure 7-9 to Figure 7-11.

Of particular note in these figures is the abrupt, premature shutdown of the shake table (due to shake table overload) at around time \( t = 25-26 \) seconds (for specimens 3F and 3J) and time \( t = 26-27 \) seconds (for specimen 3H). Despite shake table shutdown, it is believed that the maximum responses were captured. For specimen 3K, the simulation ran for the full duration, as described in Chapter 6.

The responses of specimens 3F and 3J (Figure 7-9, Table 7-2 and Table 7-3) are seen to be almost identical, with specimen 3F exhibiting a very minor increase in response. The RMS of the absolute displacement is closely matched (Table 7-3) indicating very similar behaviour. Figure 7-9c shows the closely matched frequency of response for specimens 3F and 3J, which indicates that the specimens possessed similar stiffness.

Specimen 3H exhibited a significantly larger response than specimen 3J (Figure 7-10), with the peak of the relative displacement of specimen 3H (L3-ST) approximately 240% of the peak of the relative displacement of specimen 3J (L3-ST) (Table 7-4). This matched the RMS of relative displacement which was 233% larger for specimen 3H than for specimen 3J (Table 7-5). The damage to specimen 3H appears to have contributed to a slight loss of stiffness, relative to specimen 3J, as seen by the decreased frequency of zero crossings in Figure 7-10c.

Figure 7-11 shows the response of specimens 3K and 3J. As noted previously (for simulations S6 and S7, above), specimen 3K exhibited significantly greater movement than specimen 3J (also evident in Table 7-2 to Table 7-5). The RMS and peak of the relative displacement of specimen 3K (L3-ST) was 155% and 151%, respectively, of the RMS and peak of the relative displacement of specimen 3J (L3-ST).
Figure 7-9 Specimens 3F and 3J: relative displacement at midspan-top of wall (L3-ST) for simulation S8 (S125%).
Figure 7-10 Specimens 3H and 3J: relative displacement at midspan-top of wall (L3-ST) for simulation S8 (S125%).
Figure 7-11  Specimens 3K and 3J: relative displacement at midspan-top of wall (L3-ST) for simulation S8 (S125%).
7.4  **Horizontal and vertical flexure**

In this section a number of flexure ‘snapshots’ are presented for simulations S6, S7 and S8. These snapshots show the horizontal and vertical response of the wall over a very brief time period (e.g. ~0.1 seconds). The location and layout of the LVDT displacement sensors used is shown in Figure 7-12. For simulations S6 and S7 (S75% and S100%, respectively) the horizontal flexure is presented with smoothed connecting lines between the data points. For the higher intensity simulation S8 (S125%) where cracking generally occurred, the data points are connected with straight lines. For the vertical flexure, straight connecting lines are used in all cases.

For each simulation (S6, S7 and S8) the flexure of each specimen is also presented in terms of the RMS of the relative displacement. This information provides a more accurate representation of the response of each specimen over the entire duration of the simulation.

For vertical flexure the relative displacement of the base of the wall is presented as being zero (i.e. representing no sliding of the wall on the support slab). For the lower intensity simulations (up to and including S6: S75%) this can be seen to be generally accurate. For the higher intensity simulations, however, the vertical flexure graphs clearly indicate movement at the base. In these cases, the movement of the wall at the base has been extrapolated from the displacement data at the top and third-height of the wall and is used to estimate the sliding at the base. (Given adequate stocks of dynamic LVDT displacement sensors it would have been valuable to place a sensor at the base of the wall to record the response / sliding of the wall.)
Figure 7-12 Specimen and LVDT layout for (a) horizontal flexure and (b) vertical flexure.

Note: The dashed lines in the graphs indicate zero displacement, and the crosses indicate the location of the LVDT displacement sensors.
7.4.1 Simulation S6 (S75%)

The flexural responses of specimens 3A, 3C and 3J at certain snapshots during simulation S6 (S75%) are shown in Figure 7-14 to Figure 7-17. The pre-cracked behaviour of specimen 3A is shown in Figure 7-14 which reveals a moderate flexural response. This response is vastly different from the post-cracking snapshot (Figure 7-15) which shows the large flexure of the wall, in particular on the left hand side where the main vertical corner cracking occurred (Figure 7-13a).

For comparative purposes snapshots of specimens 3C and 3J (Figure 7-16 and Figure 7-17, respectively) are given for the same approximate time (~24.9s) as those from the post-cracked specimen 3A (Figure 7-15). These snapshots show the contribution of the light reinforcement (specimen 3C) and significant reinforcement (3J) in reducing the flexure of the wall, thus delaying the onset of initial cracking.

The pattern of the vertical flexure graph for specimen 3A (Figure 7-15b) was clearly influenced by the location of the LVDT displacement sensors. Figure 7-13b shows the equivalent location of the midspan sensors (L3 and L7) on the front of the wall (the LVDTs were actually located on the back of the 'long' wall). It is not surprising, therefore, to observe the pattern of vertical 'flexure' in Figure 7-15b because the midspan-top of the wall (L3) experienced significant post-cracked overturning, whereas at third-height (L7) the wall experienced much less movement.

![Figure 7-13 Specimen 3A after simulation S6 (S75%).](image)

[Dark blue crosses indicate the transposed locations of the LVDT displacement sensors]
Figure 7.14 Specimen 3A during simulation S6 (S75%): (a) horizontal flexure of top of 'long' wall; (b) vertical flexure at midspan of 'long' wall.

Time: $19.350 - 19.474$ s ($\Delta t = 0.124$ s). Midspan-top peak: $-3.801$ mm/+2.952 mm.
Figure 7-15 Specimen 3A during simulation S6 (S75%): (a) horizontal flexure of top of ‘long’ wall; (b) vertical flexure at midspan of ‘long’ wall.

Time: 24.844 – 25.072s (Δt = 0.228s). Midspan-top peak: -3.174mm/+35.608mm.
Figure 7-16 Specimen 3C during simulation S6 (575%): (a) horizontal flexure of top of 'long' wall; (b) vertical flexure at midspan of 'long' wall.

Time: 24.826 – 24.917s (Δt = 0.091s). Midspan-top peak: -2.046mm/+6.398mm.

[Of particular note in the vertical flexure graph is the minor sliding of the base of the wall, which is projected to be around -0.5mm/+1mm (red dashed lines) for the particular snapshot: t = 24.826 – 24.917 seconds.]
(a)

Figure 7-17 Specimen 3J during simulation S6 (S75%): (a) horizontal flexure of top of ‘long’ wall; (b) vertical flexure at midspan of ‘long’ wall.

Time: 24.806 – 24.898s ($\Delta t = 0.092s$). Midspan-top peak: -0.97mm/+1.04mm.

[Of particular note in these graphs is the very small movement of the wall, in both horizontal and vertical aspects. This shows the effective containment provided by the reinforcement ‘matrix’ around the wall. There is negligible sliding of the base of the wall.]
The RMS of the relative displacement for horizontal and vertical flexure for simulation S6 (S75%) is shown in Figure 7-18 to Figure 7-20. In the horizontal flexure graphs the large response of damaged specimens 3A, 3D, 3C and 3K is plainly shown. For specimens 3A and 3D it is clear that the corners suffered major cracking (large RMS at the corners/ends of the wall, Figure 7-19). For specimen 3C, the large RMS at the midspan suggests major cracking occurred there, with relatively little cracking at the corners (moderate RMS at each corner/end of the wall). Specimen 3K experienced relatively large movement at both the midspan and the ends of the wall, indicating either cracking at these locations, or greater flexure at the midspan, and/or rocking of the specimen (described in greater detail in the following section). The specimens with external bamboo reinforcement (3E, 3F, 3H, 3I and 3J) experienced less movement, with specimens 3F and 3J exhibiting superior performance. In terms of base sliding, specimen 3D experienced the least movement, followed by specimens 3H, 3J, 3F, 3K, 3I and 3E (Figure 7-18). Damaged specimen 3C experienced the greatest amount of base sliding.

![Graph showing RMS of relative displacement for vertical flexure for simulation S6 (S75%)](image)

**Figure 7-18** Key specimens: RMS of relative displacement for vertical flexure for simulation S6 (S75%) [RMS: 0-1.1mm in linear scale].

[3A not shown - off the scale of the graph]
Figure 7-19 All specimens: RMS of relative displacement for horizontal flexure for simulation S6 (S75%) [RMS: 0.1-100mm in log scale].
[See Figure 7-20 for clearer detail of RMS: 0-1.1mm]

Figure 7-20 Key specimens: RMS of relative displacement for horizontal flexure for simulation S6 (S75%) [RMS: 0-1.1mm in linear scale].
7.4.2 Simulation S7 (S100%)

For simulation S7 (S100%), the flexural responses of specimens 3F, 3H, 3J and 3K at certain snapshots are presented (Figure 7-22 to Figure 7-25). The snapshots chosen show the peak relative displacement of the midspan-top of the ‘long’ wall (L3-ST) for each specimen.

As noted previously (Section 7.3.3 above) the peak relative displacement at the midspan-top of the wall occurred at the same time (~30.53 seconds) for specimens 3H and 3K. This occurred much later than the time at which the shake table imparted the peak ground motion (~23.13 seconds), and suggests that cracking occurred in these specimens during simulation S7 (S100%).

In the cases of the damaged specimens 3H and 3K the peak relative displacement of the midspan-top of the wall occurred in the positive direction (outwards), indicating that vertical cracking at the corners had probably occurred, and the wall panel was experiencing a combination of overturning and flexure. (Without reference data showing the specific movement of the shear ‘wing’ walls it is very difficult to detect precisely when vertical corner cracking occurred. Additional instrumentation of the shear walls is recommended for future testing). For the undamaged / slightly damaged specimens 3F and 3J the direction (negative) of the peak relative displacement matched the direction of the peak displacement of the shake table.

The movement at the base of the walls was more apparent for simulation S7 (S100%) than for the moderate intensity simulation S6 (S75%). This is shown by the extrapolated wall response in the vertical flexure graphs. It is surmised that this movement was due to a combination of sliding, rocking and bulging (horizontal flexure) at the base of the wall. As expected, for the specimens where there was a firm attachment between the vertical reinforcement and the base slab (specimens 3H and 3K) there was significantly less movement at the base.

The horizontal flexure graphs show both positive and negative movement at the ends of the ‘long’ wall, which suggest that the entire structure (rigid body motion for uncracked specimens) was either sliding or rocking slightly during the simulations. Observations
and video footage during the simulations (Appendix A) suggest that rocking was the more dominant action in the structures (Figure 7-21a). It would appear that although the 'wing' wall restraint acted to reduce this movement, it did not eliminate it entirely. The moderate rocking of the specimens served to dissipate some of the seismic energy, without generating severe diagonal cracking in the shear walls, as observed in the u-panel testing undertaken by Zegarra et al (1999). If a greater reduction of this rocking movement is desired, then a larger 'wing' wall should be considered, with a more substantial restraint provided, as shown in Figure 7-21b.

![Diagram showing rocking of panel](image)

(a) As tested  
(b) Improved sample

Figure 7-21 Schematic elevation of 'wing' wall and restraint.
Figure 7-22 Specimen 3F during simulation S7 (S100%): (a) horizontal flexure of top of ‘long’ wall; (b) vertical flexure at midspan of ‘long’ wall.

Time: 22.422 – 22.491s (Δt = 0.069s). Midspan-top peak: -6.805mm/+3.371mm.

[The sliding of the base of the wall is projected to be around -4mm (red dashed line) for the particular snapshot: t = 22.491 seconds.]
Figure 7-23 Specimen 3H during simulation S7 (S100%): (a) horizontal flexure of top of 'long' wall; (b) vertical flexure at midspan of 'long' wall.

Time: 30.426 - 30.528s (Δt = 0.102s). Midspan-top peak: -4.561mm/+8.742mm.

[Clearly, specimen 3H experienced greater flexure than the previous specimen 3F. The potential movement at the base of the wall is estimated to be equivalent to base sliding of around -1mm/+1.5mm (red dashed lines) for the particular snapshot: $t = 30.426 - 30.528$ seconds.]
Figure 7-24 Specimen 3J during simulation S7 (S100\%): (a) horizontal flexure of top of ‘long’ wall; (b) vertical flexure at midspan of ‘long’ wall.

Time: 23.859 – 23.929s (Δt = 0.070s). Midspan-top peak: -7.490mm/+3.571mm.

[The sliding of the base of the wall is projected to be around minus 3.5mm (red dashed line) for the particular snapshot: t = 23.929 seconds.]
Figure 7-25 Specimen 3K during simulation S7 (S100%): (a) horizontal flexure of top of ‘long’ wall; (b) vertical flexure at midspan of ‘long’ wall.

Time: 30.420 - 30.532s (Δt = 0.112s). Midspan-top peak: -6.487mm/+10.813mm.

[Specimen 3K experienced the greatest amount of flexure of the specimens presented. The minor sliding of the base of the wall is projected to be around -1.2mm/+0.8mm (red dashed lines) for the particular snapshot: t = 30.420 - 30.532 seconds. For this specimen the data points from L2 (@250mm from right) were corrupted and hence disregarded.]
The RMS of the relative displacement for horizontal and vertical flexure for simulation S7 (S100%) is shown in Figure 7-26 to Figure 7-28. The large response of severely damaged specimens 3B and 3D is plainly shown. Specimen 3E also experienced significant movement during the simulation, and the results for specimen 3K demonstrate progressive additional damage. Interestingly, specimen 3K experienced the least amount of base movement (most probably due to the secure attachment between the vertical timber poles and the support slab), followed by specimens 3H, 3I, 3F, 3I, and finally, 3E. These results suggest that for external vertical reinforcement, the movement of the base is more strongly influenced by the degree of damage rather than the connection between the vertical reinforcement and the base slab (for specimens 3E, 3I and 3H the vertical reinforcement was securely attached to the base slab).

![Graph showing RMS of relative displacement for vertical flexure for simulation S7 (S100%)](image)

**Figure 7-26 Key specimens: RMS of relative displacement for vertical flexure for simulation S7 (S100%) [RMS: 0-3mm in linear scale].**

[3B not shown - off the scale of the graph]
Figure 7-27 All specimens: RMS of relative displacement for horizontal flexure for simulation S7 (S100\%) [RMS: 0.1-100mm in log scale].
[See Figure 7-28 for clearer detail of RMS: 0-3mm]

Figure 7-28 Key specimens: RMS of relative displacement for horizontal flexure for simulation S7 (S100\%) [RMS: 0-3mm in linear scale].
7.4.3 Simulation S8 (S125%)

For simulation S8 (S125%), the flexural responses of specimens 3F, 3H, 3J and 3K at certain snapshots are again presented (Figure 7-29 to Figure 7-32). In this case, the snapshots were taken at around t = 25 seconds, which corresponded with the final major movement of the shake table prior to premature shutdown due to shake table overload (for the majority of specimens), as shown in Chapter 6. The graphs show that specimen 3H experienced the greatest relative displacement at the midspan-top of the wall, followed by specimens 3K, 3F and 3J.

The estimated movement of the base of the wall on the slab was significantly greater in these simulations than for the previous simulations S6 and S7. Specimen 3F experienced the largest estimated base movement, followed by specimen 3H, 3J and 3K. Interestingly, specimen 3J experienced relatively little base movement, despite lacking a connection between the vertical reinforcement and the support slab. The movement detected at the midspan-base of the wall may be due to horizontal flexure in the ‘long’ wall and/or rocking-bumping of the wall and/or sliding of the whole structure (despite the intention of the wall restraint to minimise this sliding). Again the response recorded for the vertical flexure was influenced by the location of the LVDT displacement sensors with respect to the cracking in the wall (as described in Section 7.4.1 above).
Figure 7-29 Specimen 3F during simulation S8 (S125%): (a) horizontal flexure of top of ‘long’ wall; (b) vertical flexure at midspan of ‘long’ wall.

Time: 24.942 – 25.126s (Δt = 0.184s). Midspan-top peak: -17.013mm/+14.065mm.

[The significant sliding of the base of the wall is projected to be around -15mm/+6mm (red dashed lines) for the particular snapshot: t = 24.942 – 25.126 seconds.]
Figure 7-30 Specimen 3H during simulation S8 (S125%): (a) horizontal flexure of top of ‘long’ wall; (b) vertical flexure at midspan of ‘long’ wall.

Time: 24.984 – 25.165s ($\Delta t = 0.181$). Midspan-top peak: -20.892mm/+34.814mm.

[The sliding of the base of the wall is projected to be around -7mm/+7mm (red dashed lines) for the particular snapshot: 24.984 – 25.165 seconds.]
Figure 7-31 Specimen 3J during simulation S8 (5125%): (a) horizontal flexure of top of ‘long’ wall; (b) vertical flexure at midspan of ‘long’ wall. Time: 24.934 – 25.108s (Δt = 0.174s). Midspan-top peak: -14.508mm/+12.077mm.

[The sliding of the base of the wall is projected to be around -9mm/+2mm (red dashed lines) for the particular snapshot: t = 24.934 – 25.108 seconds.]
Figure 7-32 Specimen 3K during simulation S8 (S125%): (a) horizontal flexure of top of 'long' wall; (b) vertical flexure at midspan of 'long' wall.

Time: 24.958 - 25.097s ($\Delta t = 0.139s$). Midspan-top peak: $-20.832\text{mm}/+21.095\text{mm}$.

[The sliding of the base of the wall is projected to be around $-5\text{mm}/+2\text{mm}$ (red dashed lines) for the particular snapshot: $t = 24.958 - 25.097$ seconds.

For this specimen the data points from L2 (@250mm from right) were corrupted and hence disregarded.]
The RMS of the relative displacement for horizontal and vertical flexure for simulation S8 (S125%) is shown in Figure 7-33 to Figure 7-35. The large response of significantly damaged specimens 3E and 3H is clearly shown. The pattern of the graph for specimen 3H suggests that major cracking occurred in the midspan of the wall, whereas for specimen 3E vertical corner cracking (at both ends of the wall) appears to be the more dominant mode of failure.

Again, specimen 3K experienced the least amount of base movement, followed by specimens 3J, 3I, 3H and 3F. The overall response of specimen 3F in both horizontal and vertical flexure was very moderate, which may be due, in part, to the beneficial effects (energy dissipation) of base sliding / rocking which reduced relative displacement.

![Figure 7-33 Key specimens: RMS of relative displacement for vertical flexure for simulation S8 (S125%) [RMS: 0-7mm in linear scale].](image-url)
Figure 7-34 All specimens: RMS of relative displacement for horizontal flexure for simulation S8 (S125%) [RMS: 0.1-100mm in log scale].
[See Figure 7-35 for clearer detail of RMS: 0-7mm]

Figure 7-35 Key specimens: RMS of relative displacement for horizontal flexure for simulation S8 (S125%) [RMS: 0-7mm in linear scale].
7.4.4 Simulation S11 (S100% repeated)

For most specimens the instrumentation was removed for the latter stage, high intensity simulations (S9+), because in most cases the structure was already extensively cracked, and there was a risk of damaging the delicate instrumentation. For specimen 3J, however, the damage was less severe, so the instrumentation was used to record the behaviour of the specimen for simulations S9 (S75% repeated), S10 (S75% repeated) and S11 (S100% repeated). The flexure graphs from simulation S11 (S100% repeated) are shown in Figure 7-36.

The results show that even after being subjected to a series of severe simulations, and despite being extensively damaged, the specimen still maintained a high degree of integrity. This is an important feature of this reinforcement system, as it guards against collapse in the event of severe aftershocks following a damage-inducing major shock. This aspect means that even for severely damaged houses, the homeowners may safely re-enter their homes to collect their possessions. In the event that emergency shelter is unavailable in the aftermath of a major shake (as seen in the Pakistan earthquake of October 2005) a moderately damaged house may still be occupied until adequate repairs or reconstruction are undertaken.
Figure 7-36 Specimen 3J during simulation S11 (S100% repeated): (a) horizontal flexure of top of 'long' wall; (b) vertical flexure at midspan of 'long' wall.

Time: 26.884 – 27.116s (Δt = 0.232s). Midspan-top peak: -15.705mm/+15.035mm.

[The movement at the midspan-base was projected to be around -12mm/+7mm for the particular snapshot: t = 26.884 – 27.116 seconds.]
7.5 Analysis of crack patterns and failure mechanisms

7.5.1 Introduction

In this section the most common crack patterns and failure mechanisms are discussed in greater detail, with reference to the observations (Chapter 6) and the displacement and flexure characteristics presented in this chapter. This analysis is intended to explain how and why cracking occurred, and considers the capacity of different improvement systems to reduce such failure.

In Chapter 6 it was observed that each specimen exhibited vertical corner cracking at the intersection of the orthogonal walls, plus vertical, horizontal and diagonal cracking in the out-of-plane ‘long’ wall (with variations in the location and extent of damage due to the reinforcing system used). This chapter considers the main factors which contribute to these damage patterns:

- Large relative displacement between the out-of-plane ‘long’ wall (experiencing an overturning moment) and the stiff in-plane shear ‘wing’ wall, leading to tearing (tensile) stresses in both the mortar-brick interface and the individual brick units at the corner intersection (contributing to vertical corner cracking).

- Horizontal flexure induced in the out-of-plane ‘long’ wall, causing a splitting-crushing cycle at the mid-span of the ‘long’ wall and the intersection with the orthogonal shear ‘wing’ walls (leading to mid-span vertical cracking in the ‘long’ wall, and contributing to vertical corner cracking and diagonal cracking).

- Vertical flexure and overturning in the out-of-plane ‘long’ wall (leading to horizontal cracking and contributing to diagonal cracking, in combination with the horizontal flexure).

These aspects are discussed in greater detail in this section.
7.5.2 Vertical corner cracking

Vertical corner cracking is considered to be caused by two main actions:

(i) Large relative displacement between the out-of-plane 'long' wall (experiencing an overturning moment) and the stiff in-plane shear 'wing' wall, generating tearing (tensile) stresses at the top of the corner intersection (Figure 7.37).

This feature is confirmed by the flexure graphs shown in Section 7.4 above, which indicate significant movement at the ends of the walls for certain specimens. (Without detailed data showing the specific movement of the wing walls, however, it is difficult to determine how much of this movement at the ends of the walls is due to vertical corner cracking, and how much is due to rocking and/or sliding of the whole specimen.) In the cases where vertical corner cracking had occurred (e.g., specimen 3A during simulation S6 (Figure 7.15) and specimen 3C during simulation S6 (Figure 7.16)) the graphs clearly show the positive movement at the ends of the wall which represent the overturning moment of the wall panel.

![Figure 7.37 Schematic side elevation of specimen showing exaggerated overturning moment which generates tensile stresses at the corner intersection.](image)
(ii) Horizontal flexure induced in the 'flexible' out-of-plane 'long' wall, causing a tension-compression (splitting-crushing) cycle at the intersections with the stiff orthogonal shear 'wing' walls (Figure 7-38). (Horizontal flexure also induces splitting-crushing at the mid-span of the 'long' wall, which contributes to mid-span vertical cracking in the 'long' wall, as described below).

This feature is confirmed by the flexure graphs shown in Section 7.4 above, which show the horizontal flexure generated in the wall. It is clear that such flexure causes significant tension-compression stresses at each corner intersection.

Figure 7-38 Schematic plan view of specimen showing exaggerated cyclic wall flexure and tension-compression stresses which contribute to vertical corner cracking.

{Key: T = tension; C = compression}
7.5.3 Midspan vertical cracking

Horizontal flexure in the ‘flexible’ out-of-plane ‘long’ wall causes a tension-compression (splitting-crushing) cycle at the mid-span of the wall (Figure 7-39), which leads to vertical cracking when the capacity of the material is exceeded.

This feature is confirmed by the flexure graphs shown in Section 7.4 above, which show the horizontal flexure generated in the wall. It is clear that such flexure causes significant tension-compression stresses at the midspan of the wall.

![Figure 7-39 Schematic plan view of specimen showing exaggerated cyclic wall flexure and tension-compression stresses which contribute to vertical midspan cracking.](Key: T = tension; C = compression)
7.5.4 Horizontal and diagonal cracking

Vertical flexure and overturning of the out-of-plane ‘long’ wall causes a tension-compression (splitting-crushing) cycle in the wall (Figure 7-40). Without the restraint provided by adequate reinforcement, these stresses lead to horizontal cracking in the wall, usually after the formation of vertical corner cracking when there is little resistance to overturning of the damaged wall panel (e.g. specimens 3A and 3B). In the cases where the walls were reinforced (specimens 3C to 3K) the combination of vertical and horizontal flexure led to the formation of diagonal cracking, with cracking often following a stair step formation (45°) along the weak mortar-brick interface, as seen in Chapter 6. Thus diagonal cracking can be attributed to the generation of maximum tensile stresses on the principal planes, and as such, cracking is at approximately 45° to the horizontal and vertical planes.

[Figure 7-40 Schematic side elevation of specimen showing exaggerated vertical flexure and overturning which lead to horizontal cracking.

[Key T = tension, C = compression]]
7.5.5 Contribution of improvement systems

The main objectives of reinforcing systems for adobe buildings are: (i) prevent collapse, and (ii) reduce damage to the structure. Systems which minimise the movement of the wall also reduce the stresses which lead to cracking. The qualitative and quantitative results from the u-panel testing confirm the value provided by adequate reinforcement, both in terms of minimising movement and reducing damage.

Light-gauge chicken wire mesh running horizontally in the mortar joints (specimen 3C) was observed to slightly reduce the horizontal flexure of the wall, which thus delayed the initial cracking. This solution was seen to be effective for low-moderate intensity ground motion (up to S6: S75%), but for higher intensity excitation (S7: S100%) the mesh was insufficient to restrain movement and prevent severe damage.

The addition of a timber ring beam at the top of the wall (specimens 3D to 3K) had the effect of minimising the horizontal and vertical flexure in the wall. For this to be most effective, however, it is important to have an adequate connection between the ring beam and the wall, as achieved by connecting the external vertical bamboo and ring beam with wire and staples. The continuity of the ring beam is also an important factor in its capacity to reduce movement in the structure. Of the specimens with external vertical bamboo reinforcement (specimens 3E, 3F, 3H, 3I and 3J) the specimens without a connection between the ring beam and the wing wall restraint (specimens 3E and 3I) were seen to exhibit significantly greater movement [flexure and midspan-top (L3) displacement] than those specimens which included a wire loop connection between the ring beam and wing wall restraint (specimens 3F, 3H and 3J). Without the wire connection to simulate a continuous ring beam the tearing stresses at the corners were simply shifted to the end of the ring beam, as seen in Chapter 6.

It would appear that the continuity of the ring beam has a greater influence on the seismic capacity than the presence or lack of firm connection between the vertical reinforcement and the support slab. For specimens 3E, 3I, 3H and 3K there was a firm connection between the vertical reinforcement and the base slab. The movement and damage experienced by these specimens, however, was far greater than for specimens.
3F and 3J where there was no attachment at the base of the wall. This may be due, in part, to the dissipation of seismic energy through rocking/sliding of the structure.

The internal vertical reinforcement (specimens 3G and 3K) seemed to introduce discontinuities, and hence weaknesses, in the structure, which were apparent in the observations (Chapter 6) as well as the detailed displacement data in this chapter. Specimens 3G and 3K had lower first natural frequencies than the other specimens, which indicates that the specimens were less stiff. Furthermore, the EMTA results (Chapter 8) show a progressive loss of stiffness of specimen 3K during shake table testing. Furthermore, specimen 3K exhibited significant flexural movement, causing cracking, even at the lower intensity simulations. In reality, this minor cracking may not pose a significant threat to the stability of the structure, but it may introduce a lack of confidence on the part of the homeowners.

Specimens 3F (retrofit) and 3J (‘optimal’) behaved very similarly. This suggests that the internal horizontal chicken wire mesh has little effect on the pre-damaged response of the wall (when part of an advanced reinforcing system which includes external vertical and horizontal reinforcement). Certainly, the wire mesh acts to hold parts of the wall together after major damage, but prior to this the flexural response of the wall appears to be unaffected by the presence of the mesh. This finding represents a significant outcome and shows that the retrofit-strengthening method trialled in specimen 3F can achieve a similar degree of earthquake resistance as a new-build construction (assuming similar boundary conditions and a similar quality of construction). Importantly, omission of the horizontal chicken wire mesh also represents a significant saving in time and money.

Overall, the most effective means of reducing movement and subsequent damage in a structure is the addition of external vertical bamboo reinforcement, external horizontal wire reinforcement and a continuous timber ring beam. These additions, when securely tied together, create an integrated matrix which restrains movement and enhances the overall seismic capacity of the structure. This conclusion was subjected to further confirmation by testing a model house on the shake table, as presented in Chapter 9.
7.6 Specimen rating matrix

In addition to the seismic capacity of a reinforced adobe structure, the adobe practitioner and the home builder are commonly interested in the complexity and cost of construction. This is particularly relevant in developing countries where skill levels and financial resources are often very limited, and hence play a key role in the type of construction undertaken. In order to understand the relationship between the seismic capacity, complexity and cost, a simple matrix has been developed to provide a rating for each specimen under each key factor (Table 7-6). The values in the matrix are both subjective and relative, based on the test results, combined with the perceptions and experiences of the author during construction of the specimens, and during adobe research and application projects in Latin America (discussed in Chapter 2).

The values in the matrix relate to the u-panel wall units as built and tested under controlled and consistent conditions. In a full structure in the field, these values will change, depending on the local context and resources. The cost, for example, of adobe bricks will be influenced by local soil conditions, the capacity (time, skills, resources) of the homeowner to make their own bricks, the cost of transport (soil and/or bricks if necessary), etc. The same applies for the bamboo/cane, which could be available locally free of charge. In another example, in this research there was little additional cost to include the corner pilasters in specimen 3B. In practice, however, the inclusion of corner pilasters equates to a larger and more costly foundation, as well as significant increases in the cost and complexity of the roof structure and sheeting.

In the matrix the seismic capacity is based on observations and analysis of test results. The complexity parameter encompasses the skills and time required to build and reinforce each structure. The cost parameter incorporates the materials, tools and equipment required, and includes the fabrication and transportation of the bricks. In the matrix higher values represent a more desirable result. Thus, a seismic capacity rating of 5 (out of 5) represents a structure with excellent seismic capacity, whereas a rating of 1 is very poor. Similarly a complexity or cost rating of 2.5 (out of 2.5) indicates a structure which is relatively simple / cheap to build and could be achieved with limited resources, experience and skill. A complexity or cost rating of 0.5 (out of 2.5) indicates higher complexity / cost and would require significant resources / experience and skill.
Chapter 7. U-shaped adobe wall units: comparative analysis

For this exercise a weighting of 50% was placed on the seismic capacity, and 25% for both the complexity and the cost parameters. These assignments are subject to discussion and debate, and would be the worthy subject of field research, especially in conjunction with an implementation/training program, whereby the perceptions of the local community government officials, development workers and risk reduction specialists could be gauged before, during and after a project.

It is interesting to note how this weighting might affect the overall rating and ranking of the structures. For example, if a large organisation plans a housing construction project where there are adequate resources to employ skilled tradespeople and use more costly materials then the weighting assigned to the seismic capacity would be much higher, and the weighting assigned to the complexity and cost would be fairly low (given that the cost and complexity is relatively low compared with other forms of construction). In such a case, specimens 3J, 3F and 3K would be the most appropriate systems to use. On the other hand, if an organisation plans a very simple training workshop where improvement techniques are introduced in a poor rural community, with the homeowners then taking responsibility for the construction and financing of their own homes, then the complexity and cost factors would most likely be weighted more heavily (especially if the threat of earthquakes is low, underestimated or ignored, as is often the case in such areas). This would mean that simple and affordable improvement techniques, such as the horizontal chicken wire mesh trialled in specimen 3C, might be more appropriate, even though its seismic capacity is limited. Specimens 3F, 3J and 3H provide a more balanced overall capacity, although appropriate training and support would be required for effective implementation (discussed further in Chapter 12).

The matrix presented in Table 7-6 highlights the importance of considering the core application considerations of complexity and cost, in conjunction with the seismic capacity in order to develop and assess the most appropriate solutions for a given context. A detailed multi-criteria evaluation tool could be developed to incorporate these factors, as well as other technical, social, cultural and institutional aspects which influence the design and realisation of any construction project and implementation program (as discussed in Chapters 12 and 13).
<table>
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<th>Specimen</th>
<th>Features*</th>
<th>Seismic Capacity /5</th>
<th>Complexity /2.5</th>
<th>Cost /2.5</th>
<th>Overall score /10</th>
<th>Ranking</th>
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<td>6.8</td>
<td>5</td>
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<td>Int. horiz. mesh + int. vert. bamboo + ring beam</td>
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<td>1.6</td>
<td>6.0</td>
<td>=9</td>
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<td>7.1</td>
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<td>7.5</td>
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<td>1.7</td>
<td>1.7</td>
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<td>1.5</td>
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* Abbreviations: int. = internal; ext. = external; horiz. = horizontal; vert. = vertical.

† Estimated, due to error in simulation procedure (as described in Chapter 6).

‡ Estimated, but very difficult to accurately determine, due to unexpected cracking at lower intensity testing (as described in Chapter 6).
7.7 Summary

The analysis of the results for the eleven u-shaped adobe wall units revealed the following general outcomes:

- The main crack patterns in damaged adobe structures (vertical corner cracking, vertical midspan cracking, and horizontal and diagonal cracking) are due to combinations of the stresses generated by overturning, vertical flexure and horizontal flexure. The most effective improvement systems reduce movement in the wall, which minimise these stresses, and thus delay the onset of initial cracking and the loss of strength of the structure.

- The most effective systems incorporated external vertical bamboo reinforcement, external horizontal wire reinforcement and a ring beam. These additions, when securely tied together, create an integrated system which restrains movement and enhances the overall strength of the structure. The reinforcement system acted to delay the onset of initial cracking, and reduce the severity of cracking during repeated high intensity shaking. Most importantly, collapse of reinforced structures was prevented. These improvements were achieved using low-cost and low-tech means.

- The excellent performance of specimens 3F (retrofit) and 3J (‘optimal’) demonstrated the proposed improvement system to be viable and effective for both new-build constructions and for the retrofit-strengthening of existing dwellings.

- A continuous ring beam plays a key role in reducing the movement in the structure. The ring beam must be firmly connected to the wall (e.g. via dowels and external vertical bamboo) to be most effective.

- It would appear that the presence or lack of a firm connection between the vertical reinforcement and the support slab has little bearing on the flexural response and the overall performance of the structure. In fact, specimens with no firm connection performed better, which may be attributed to an improved structural system, and/or the energy dissipated due to rocking / sliding of the structure.

- The use of internal vertical reinforcement appears to introduce discontinuities in the structure, which result in significant flexural movement, causing cracking, even at lower intensity simulations. The change of stiffness during the shake table testing is
discussed in greater detail in the following chapter (Experimental Modal Testing and Analysis).

- A 'specimen rating matrix' was developed to present the relationship between the seismic capacity, complexity and cost of each reinforcement system. The matrix revealed specimens 3F (retrofit) and 3J ('optimised') to be the most appropriate solutions for use in developing countries where skill and resource limitations strongly influence the type of intervention undertaken. These factors could be incorporated in a detailed multi-criteria evaluation matrix, which would be a useful tool in the planning and realisation of any construction and implementation project (as discussed in Chapters 12 and 13).
8 U-SHAPED ADOBE WALL UNITS: EXPERIMENTAL MODAL TESTING AND ANALYSIS (EMTA)

8.1 Introduction

Experimental modal testing and analysis (EMTA) is the process of characterising the dynamic properties of a test structure by exciting the structure artificially and identifying its modes of vibration (Ramsey, 1982). These modes of vibration reveal the frequencies at which the structure can be excited into resonant motion, and the predominant wave-like motions it will assume at the resonant frequencies (Richardson, 1978).

Knowledge of the dynamic properties of a structure, which are defined by specific natural frequencies, modal damping and mode shapes, provides an insight into the dynamic behaviour and failure mechanisms of that structure. The EMTA method is based on the assumption of linear elastic behaviour, i.e. prior to cracking.

In conjunction with extensive shake table testing, EMTA played a vital role in this research project. The main objectives of EMTA include:

- Determination of the first natural frequency of all specimens in order to appropriately scale (with respect to time) the input excitation (ground motion) to ensure dynamic similitude and induce damaging near-resonance conditions (discussed in Chapter 5). This was done by measuring the response of each specimen when subjected to a short, sharp impact excitation (impact hammer test) prior to shake table testing.

- Determination of the dynamic characteristics and behaviour of key specimens prior to and during shake table testing. This was done by further processing the response data from the impact hammer test (during the strengthening process and prior to shake table testing) as well as the response data during shake table testing to determine the first three modes (natural frequencies), modal damping and mode shapes.
• Assessment of the change in dynamic behaviour of key specimens during shake table testing. This was done by comparing the dynamic characteristics during the series of shake table tests.

Within published literature, there has been limited use of EMTA to evaluate the dynamic characteristics of adobe mudbrick structures. Tolles and Krawinkler (1990) used a slightly different impact hammer test technique to determine the dynamic features of 1:5 scale adobe houses prior to shake table testing. They identified the natural frequencies, damping and mode shapes for the key dynamic responses (transverse translation, breathing, torsion and longitudinal translation) for the first seven modes. This provided a useful comparison between the characteristics of different specimens (representing unreinforced and reinforced structures) in the undamaged state, prior to testing. In the reviewed literature there were no reported cases of EMTA being undertaken for adobe mudbrick structures during shake table testing.

Extensive work undertaken at UTS which has focused on the structural dynamics and structural integrity assessment of timber bridges using EMTA (Samali et al., 2002) provides a solid platform for the application of EMTA in this project.

This chapter describes the procedure and key results of EMTA for the u-shaped adobe wall units. Of particular interest are specimens 3H, 3J and 3K, which represent improved structural systems. The theory and extensive results are included in Appendix D.
8.2 Modal testing

The modal testing was undertaken in two key stages:

1) Each u-panel was excited by an impact hammer to identify the dynamic properties of the intact (undamaged) structure.

2) Each u-panel was then subjected to a series of shake table tests (described in Chapter 6). For selected specimens EMTA was undertaken during the shake table tests to determine the dynamic characteristics, and changes thereof, during testing.

The test procedure and instrumentation for each stage are described below.

8.2.1 Test procedure and instrumentation: impact hammer tests

Tolles and Krawinkler (1990) reported that the response of a structure can be measured by either exciting the structure at a single point and measuring the response at a number of locations, or exciting the structure at a number of locations and measuring the response at a single point. They chose the latter because it was easier to perform given their system and configuration. For this research project the first option was chosen because it was more compatible with the manner in which the response was to be measured during the shake table tests (i.e. a single one-off excitation measured at a number of locations on the wall), as well as reducing variabilities due to impact hammer hitting (manner and intensity). In this case, the mesh size was dictated by the number of compatible accelerometers available to record the response of the wall. Despite the reasonably coarse mesh used, the main modes of influence (the first three modes) were clearly detectable.

For this research a large 12 lb Modally Tuned ICP Sledge Hammer was used to excite the specimen. (The impact point was located at the top-midspan of the out-of-plane ‘long’ wall, Figure 8-1.) The vibration response of the structure was measured by piezoelectric-type accelerometers (PCB 356A08 and PCB 337A26), which were attached to the outside face of the out-of-plane wall (Figure 8-1). The signals from the hammer (impact force) and accelerometers (acceleration) were first amplified by signal conditioners and then recorded by a data acquisition system. The acquired frequency
range was set from 0 Hz to 512 Hz with 8,192 data points sampled. Three hammer hits were undertaken for each hammer test, with the most average result processed and analysed. Figure 8-1 and Figure 8-2 show the test set up for the impact hammer testing and associated EMTA.

Figure 8-1 U-panel specimens: test set up for the impact hammer tests.

(a) Impact hammer hit  (b) Accelerometers  (c) Modal analysis

Figure 8-2 Key components of the EMTA.
The precise positioning of the accelerometers can be seen in Figure 8-3. This distribution was chosen to enable the acquisition of the first three modes. The location of the midspan accelerometers (P3, P13 and P23) coincided with the nodes of the second mode. This accounts for the small peaks evident in the FRF graphs, however the use of the FRF stabilisation diagrams allowed the second mode to be confidently determined in most cases. These aspects are discussed in greater detail below.

Figure 8-3 U-panel specimens: location of accelerometers.
8.2.2 Test procedure and instrumentation: shake table testing

The specifications and process of the shake table testing were presented in Chapter 5.

The instrumentation configuration adopted for the impact hammer tests (Figure 8-1 and Figure 8-3) was also used for the shake table testing, however, in lieu of the impact hammer input, the input excitation was measured by an accelerometer located on the support frame mounted on the shake table. The signals of the accelerometers were also amplified by a signal conditioner and recorded by a data acquisition system. The acquired frequency range was set from 0 Hz to 80 Hz with 2048 data points sampled. Figure 8-4 shows the set-up for the shake table tests.

![Diagram of shake table setup]

Figure 8-4 U-panel specimens: test set up for the shake table tests.
8.3 **Experimental Modal Analysis: theory**

Experimental Modal Analysis (EMA) is an established and reliable vibration analysis tool, providing information on the dynamic characteristics of a structure and its excitation (Samali *et al.*, 2002). By processing the excitation-response data obtained through experimental modal testing it is possible to determine the key dynamic features of the structure: the natural frequencies, the modal damping and the mode shapes. The process is depicted in Figure 8-5.

![Diagram of Experimental Modal Analysis](image)

The data sampled during the model testing was analysed using the ‘Modal Analysis Module’ of the software LMS CADA-X. First, the time signals (amplitude versus time) were converted (transformed) into frequency spectra (amplitude versus frequency) using Fourier transform. Then, the Fourier transform signals of the accelerometers (output) were divided by the Fourier transform signal of the hammer impact (input), resulting in the ‘Frequency Response Function’ (FRF). The FRF determines how much acceleration response a structure has per unit of force excitation. This is the most important measurement in the experimental modal analysis, as it provides information about the modal parameters of the structure, namely, the natural frequencies, modal damping and the mode shapes (Avitabile, 2001).

![Figure 8-5 Phases of the modal analysis.](image)
assurance criterion (MAC)] and general descriptions of the modal parameters (natural frequencies, damping and mode shapes) are provided in Appendix D.

For the analysis of the u-panels in this project the first three modes were considered. They refer to the ‘first transverse translation mode’ (Mode 1), the ‘first torsion mode’ (Mode 2) and the ‘second transverse translation mode’ (Mode 3), as defined by Tolles and Krawinkler (1990), and shown in Figure 8-6. [The work by Tolles and Krawinkler (1990) was undertaken on model adobe houses, however the modes shown in Figure 8-6 match those identified during the EMTA of the u-panels in this research project, thus the descriptions and titles proposed by Tolles and Krawinkler have been adopted in this chapter, and in Chapter 10: EMTA of model house 4A.] The ‘first transverse translation mode’ clearly matches the flexural response of the u-panels, as presented and discussed in Chapter 7.

![Figure 8-6 Mode shapes and descriptions for Modes 1, 2 and 3 in different phases (α + β).](image)
8.4 EMTA of undamaged u-panels: impact hammer testing

8.4.1 First natural frequencies of all u-panel specimens

The first natural frequency of all specimens was determined in order to appropriately scale (with respect to time) the input excitation (ground motion) to ensure dynamic similitude and induce damaging near-resonance conditions (see Chapter 5 for more details).

Table 8-1 shows the specifications and first natural frequency of each specimen just prior to testing. (See Chapter 6 for greater details of specimen construction and reinforcement systems.)

The following features can be observed:

- Specimen 3B had the highest first natural frequency (34.1 Hz), indicating the additional stiffness and mass provided by the corner pilasters.

- Specimens 3G and 3K had the lowest first natural frequency (24.8 Hz and 27.0 Hz respectively), indicating that the use of internal vertical reinforcement (bamboo/timber) introduced discontinuities into the structure, resulting in a decreased stiffness and mass. In this case, the vertical reinforcement was located very close to the neutral axis, and therefore had less contribution to the bending rigidity.

- For those specimens with external vertical bamboo reinforcement (u-panels 3E, 3F, 3H and 3J) the bending rigidity (= first natural frequency) was higher because the reinforcement was located at the extremities, which is most effective for the reduction of flexural response of the structure.

- Specimens 3E and 3I had a slightly lower first natural frequency (30.8 Hz and 31.6 Hz, respectively) than the other externally reinforced specimens. For these two specimens, the vertical reinforcement was firmly attached to the base slab, suggesting that this may contribute to a loss in stiffness, perhaps due to an enhanced interaction between the flexible vertical reinforcement and the stiff adobe wall.
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Horizontal Reinforcement</th>
<th>Vertical Reinforcement</th>
<th>Ring Beam</th>
<th>Notes</th>
<th>1&lt;sup&gt;st&lt;/sup&gt; Natural Frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3A</td>
<td>(None)</td>
<td>(None)</td>
<td>(None)</td>
<td>Traditional, unreinforced</td>
<td>29.6</td>
</tr>
<tr>
<td>3B</td>
<td>(None)</td>
<td>Corner pilasters</td>
<td>(None)</td>
<td></td>
<td>34.1</td>
</tr>
<tr>
<td>3C</td>
<td>Chicken wire mesh (internal)</td>
<td>(None)</td>
<td>(None)</td>
<td></td>
<td>33.0</td>
</tr>
<tr>
<td>3D</td>
<td>Chicken wire mesh (external wrapping)</td>
<td>Chicken wire mesh (external wrapping)</td>
<td>Timber</td>
<td></td>
<td>33.8</td>
</tr>
<tr>
<td>3E</td>
<td>Chicken wire mesh (internal)</td>
<td>Bamboo (external) *</td>
<td>Timber</td>
<td></td>
<td>30.8</td>
</tr>
<tr>
<td>3F</td>
<td>Fencing wire (external)</td>
<td>Bamboo (external)</td>
<td>Timber **</td>
<td>Retrofit</td>
<td>33.7</td>
</tr>
<tr>
<td>3G</td>
<td>Chicken wire mesh (internal)</td>
<td>Bamboo (internal) *</td>
<td>Timber</td>
<td></td>
<td>24.8</td>
</tr>
<tr>
<td>3H</td>
<td>Chicken wire mesh (internal)</td>
<td>Bamboo (external)</td>
<td>Timber **</td>
<td>Thin wall (100mm)</td>
<td>33.3</td>
</tr>
<tr>
<td>3I</td>
<td>Chicken wire mesh (internal)</td>
<td>Bamboo (external – inside face) *</td>
<td>Timber</td>
<td></td>
<td>31.6</td>
</tr>
<tr>
<td></td>
<td>Bamboo (external – outside face)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3J</td>
<td>Chicken wire mesh (internal)</td>
<td>Bamboo (external)</td>
<td>Timber **</td>
<td>Optimised</td>
<td>33.8</td>
</tr>
<tr>
<td>3K</td>
<td>Chicken wire mesh (internal)</td>
<td>Timber poles (internal) *</td>
<td>Timber **</td>
<td></td>
<td>27.0</td>
</tr>
</tbody>
</table>

**Notes:**
- Vertical reinforcement connected to concrete foundation of test frame (described in Chapter 6).
- Timber ring beam connected to wall restraint (described in Chapter 6).
8.4.2 Dynamic properties of undamaged u-panels 3F, 3H, 3J & 3K

In order to better understand the dynamic characteristics of selected specimens, detailed EMTA of specimens 3F, 3H, 3J and 3K was undertaken using the MDOF approach (described in Appendix D), with the impact hammer used to excite each u-panel. This process revealed the modal parameters (natural frequencies, damping ratios and mode shapes) for the first three modes of each specimen prior to testing. Table 8-2 shows the natural frequencies and damping ratios for specimens 3F, 3H, 3J and 3K prior to testing. The FRF graph, stabilisation diagram and mode shapes for specimen 3J are shown in Figure 8-7. The FRF graphs, stabilisation diagrams and mode shapes for specimens 3F, 3H and 3K were almost identical, and are included in Appendix D. The only exception to this was the FRF graph for specimen 3K, which is shown in Figure 8-8. Discussion of these results is included below.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Mode 1</th>
<th>Mode 2</th>
<th>Mode 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_1$</td>
<td>$\zeta_1$</td>
<td>$f_2$</td>
</tr>
<tr>
<td></td>
<td>(Hz)</td>
<td>(%)</td>
<td>(Hz)</td>
</tr>
<tr>
<td>3F</td>
<td>33.70</td>
<td>1.01</td>
<td>NC</td>
</tr>
<tr>
<td>3H</td>
<td>33.33</td>
<td>1.19</td>
<td>47.91</td>
</tr>
<tr>
<td>3J</td>
<td>33.82</td>
<td>1.08</td>
<td>42.19</td>
</tr>
<tr>
<td>3K</td>
<td>26.95</td>
<td>0.19</td>
<td>35.48</td>
</tr>
</tbody>
</table>

Notes:
NC – not captured. See discussion in text for explanation.
Figure 8-7 Specimen 3J: dynamic characteristics prior to shake table testing (undamaged).

[The orange arrows in the FRF graphs indicate the detected modes]

Figure 8-8 Specimen 3K: FRF graph prior to shake table testing (undamaged).
The following trends and observations were noted:

- Specimens 3F, 3H and 3J had similar fundamental (first) frequencies (33.7 Hz, 33.3 Hz and 33.8 Hz, respectively). These three specimens were externally reinforced with bamboo.

- Specimen 3K had a notably lower fundamental frequency (26.95 Hz). This lower stiffness can be attributed to the discontinuities in the structure due to the internal vertical reinforcement, as discussed previously.

- For each specimen the first and third modes were clearly visible in the FRF graphs. The second mode, however, was barely perceptible in the FRF graphs because the impact point corresponded with one of the nodes of the second mode. For specimens 3H, 3J and 3K the second mode was determined from the stabilisation diagram, however, for specimen 3F the second mode was undetermined.

- For specimens 3F, 3H and 3J (external vertical reinforcement) the FRF graphs were very similar, with clear, sharp peaks for the first and third modes. In contrast, the FRF graph for specimen 3K (internal vertical reinforcement) revealed more rounded peaks (Figure 8-8). This suggests higher damping in the system (most probably a result of increased internal friction due to the internal vertical reinforcement). Interestingly, the damping calculated by the LMS CADA-X software (Table 8-2) suggested that the damping in specimen 3K was lower than the other specimens, although the FRF diagrams (which more accurately represent the damping) reveal the opposite to be true. This feature highlights the potential inaccuracy of the damping estimations for complex structures, as discussed in Appendix D.
8.4.3 Strengthening stages of u-panel 3J

In order to evaluate the influence of different interventions during the strengthening of specimen 3J, EMTA was undertaken at key strengthening stages, as shown in Table 8-3. Stage 'a' refers to the 'as-built' specimen (with the base frame securely connected to the shake table), which contained only internal horizontal wire mesh reinforcement. Stage 'b' refers to the specimen with the 'wing' wall restraint applied (Figure 8-9a, described in Chapter 6). For stage 'c', the timber ring beam was attached to the wall, via steel bolts grouted into oversized holes in the top course of bricks. Stage 'd' refers to the specimen prior to testing (as presented in Section 8.4.2 above), which included the external vertical bamboo reinforcement, horizontal wire reinforcement, and wire connectors between the bamboo and the ring beam, and the ring beam and the 'wing' wall restraints (Figure 8-9b, described in Chapter 6).

![Figure 8-9 Specimen 3J: strengthening stages 'b' & 'd'.](image)

The frequencies and damping ratios for the first three modes for each strengthening stage of specimen 3J are shown in Table 8-3, and Figure 8-10 and Figure 8-11. The FRF graphs and mode shapes for strengthening stage 'd' (ready for testing) were presented in Figure 8-7, above. The mode shapes for the other strengthening stages were matched with stage 'd', and are included in Appendix D. The FRF graphs for each strengthening stage are shown in Figure 8-12.
Table 8-3 Specimen 3J: natural frequencies (f) and damping ratios (ζ) during strengthening process.

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Mode 1</th>
<th></th>
<th>Mode 2</th>
<th></th>
<th>Mode 3</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>t₁ (Hz)</td>
<td>ζ₁ (%)</td>
<td>t₂ (Hz)</td>
<td>ζ₂ (%)</td>
<td>t₃ (Hz)</td>
<td>ζ₃ (%)</td>
</tr>
<tr>
<td>3J – 'a' (internal horiz. mesh)</td>
<td>29.99</td>
<td>2.74</td>
<td>54.84</td>
<td>2.98</td>
<td>63.41</td>
<td>2.21</td>
</tr>
<tr>
<td>3J – 'b' (mesh + 'wing' wall restraint)</td>
<td>33.60</td>
<td>1.40</td>
<td>42.70</td>
<td>0.59</td>
<td>59.49</td>
<td>1.60</td>
</tr>
<tr>
<td>3J – 'c' (mesh + restraint + ring beam)</td>
<td>33.62</td>
<td>1.27</td>
<td>42.40</td>
<td>0.62</td>
<td>59.67</td>
<td>1.58</td>
</tr>
<tr>
<td>3J – 'd' (mesh + restraint + ring beam + external bamboo + wire)</td>
<td>33.82</td>
<td>1.08</td>
<td>42.19</td>
<td>0.44</td>
<td>59.98</td>
<td>1.40</td>
</tr>
</tbody>
</table>

Figure 8-10 Specimen 3J: natural frequency trend of Modes 1, 2 & 3 during strengthening.
Figure 8-11 Specimen 3J: trend of damping ratio of Modes 1, 2 & 3 during strengthening.

(a) 3J: strengthening stage 'a' (mesh only)  (a) 3J: strengthening stage 'b' (+ restraint).

(a) 3J: strengthening stage ‘c’ (+ ring beam)  (a) 3J: strengthening stage ‘d’ (+ bamboo+wire)

Figure 8-12 Specimen 3J: FRF graphs during strengthening stages.
The following trends and observations were noted:

- In the ‘as-built’ state (stage ‘a’) the system demonstrated high damping. In this case the damping estimations (Table 8-3 and Figure 8-11) confirm the high degree of damping which is characterised by the rounded peaks in the FRF graph (Figure 8-12a).

- The addition of the ‘wing’ wall restraint (stage ‘b’) had a significant impact on the dynamic characteristics of the system. The first natural frequency increased (representing increased stiffness) and the damping decreased. The decrease in damping is clearly evident in the shapes of the peaks in the FRF graph, which are clear and sharp (Figure 8-12b). This pattern is also evident in the FRF graphs for strengthening stages ‘c’ and ‘d’ (Figure 8-12c&d).

- The stiffness and damping of the system were stable for stages ‘b’ and ‘c’, which indicates that the attachment of the ring beam to the wall via steel bolts grouted into the top course of bricks had little impact on the dynamic characteristics.

- There was a slight increase in stiffness at stage ‘d’, which indicates enhanced rigidity when the structure was fully reinforced.

As noted above, the inclusion of the ‘wing’ wall restraint had a clear impact on the dynamic characteristics of the structure: damping decreased and stiffness increased. The damping estimations and FRF graphs for the model house 4A (see Chapter 10) show similar low damping characteristics to the restrained, lightly damped specimen 3J (stages ‘b’, ‘c’ and ‘d’). As discussed in Chapters 2 and 6, the u-shaped adobe wall unit testing undertaken at the Catholic University in Peru (PUCP) did not include any ‘wing’ wall restraint (Zegarra et al. 1999). As noted previously, the inclusion of the ‘wing’ wall restraint alters the structural behaviour and failure patterns, which are seen to be more indicative of full size structures subjected to real earthquakes. Furthermore, the method adopted in Peru is considered to be less severe because there was higher damping in the system. The lightly damped system (as tested in this research project) represented more severe conditions because there was less dissipation of energy throughout the structure. This outcome highlights the importance of providing a ‘wing’ wall restraint when testing u-shaped adobe wall units.
8.5 EMTA of u-panels 3H, 3J & 3K: shake table testing

EMTA was undertaken for u-panels 3H, 3J and 3K during the shake table testing. This was only possible using the multi-degree-of-freedom (MDOF) approach which relies on the FRF stabilisation process to determine the dynamic characteristics (described in Appendix D). In this section some general trends relating to the key dynamic characteristics and indicators (natural frequencies, damping ratios, FRF graphs and MAC) of the first mode are presented and discussed. The first three mode shapes were generally matched with the hammer test results presented in Figure 8-6 and Figure 8-7. A complete set of the modal parameters (natural frequencies, damping ratios and mode shapes) and associated FRF graphs and stabilisation diagrams for Modes 1, 2 and 3 for specimens 3H, 3J and 3K for the shake table tests are included in Appendix D.

Frequency trends

Figure 8-13 shows the frequency trends (first mode) for specimens 3H, 3J and 3K for the shake table testing. The graph clearly shows the loss of stiffness (decrease in first fundamental frequency) of each specimen with increasing intensity of shake table testing.

![Graph showing frequency trends for specimens 3H, 3J, and 3K](image)

Figure 8-13 Specimens 3H, 3J and 3K: frequency trends of Mode 1 for shake table tests.
It is interesting to note that for specimen 3H the unscaled (with respect to time) earthquakes (S1 – S3) did not affect the natural frequency, and hence stiffness of the structure, despite increasing intensity. This phenomenon further highlights the importance of time scaling the input spectra in order to create an earthquake capable of causing damage to specimens (as described in Chapter 5).

For specimen 3H, EMTA detected a loss of stiffness during simulation S6, although visual observations detected no damage during simulation S6 (minor hairline cracking was first noted during simulation S7, see Chapter 6). This suggests that EMTA is a potentially useful tool in the condition monitoring of adobe structures.

For specimen 3J structural stiffness was maintained up to and including simulation S6 (S75%), with cracking of the structure detected by EMTA during the high intensity simulation S7 (S100%). This outcome demonstrates the superior performance of specimen 3J, especially with respect to minimising damage during low to moderate intensity ground motions.

For specimen 3K there was a notable and consistent loss of stiffness from simulation S5 (S50%) onwards. This outcome demonstrates the loss of integrity of the structure, even at the lower intensity simulations, which is probably due to the discontinuities and varied structural response introduced by the internal vertical reinforcement, as discussed previously.

It is also observed that shake table tests determined a higher first natural frequency for all specimens tested. Given the clear and reliable FRFs for the impact hammer tests, compared with the ‘noisy’ shake table results (discussed below), it is believed that the hammer test results are more accurate. It is recommended, therefore, that for future testing an impact hammer test be performed after each simulation if the dynamic properties of each specimen at that state are of interest.
Modal damping trends

Figure 8-14 shows the modal damping trends (first mode) for specimens 3H, 3J and 3K for the shake table testing. Because the FRF graphs are very unclear (shown below), there is even less confidence in the damping results than for the impact hammer tests (discussed above). Despite the questionable reliability of the damping estimations, experimental testing is the only means of determining damping (it cannot be estimated by numerical modelling) so even approximate estimations are valuable. The damping trends shown in Figure 8-14 clearly demonstrate an increase in damping as each structure is progressively damaged. This is to be expected, as cracking increases the capacity of the structure to dissipate energy due to the increased relative movements and internal friction between various components of each specimen after damage.

Figure 8-14 Specimens 3H, 3J and 3K: modal damping trends of Mode 1 for shake table tests.
FRF graphs

Figure 8-15 shows the FRF graphs for shake table simulations S2 – S8 for specimen 3J. The graphs are much ‘messier’ than the FRF graphs for the impact hammer tests (e.g. Figure 8-12). The ‘messy’ graphs reflect the noise generated during the shake table testing, as well as the complexity of the structural interactions and responses due to the complex input excitation (shake table movement). This ‘noise’ is even more apparent during the more severe simulations, when both the input ground motion and the response of the structure were greater. The ‘messy’ graphs also indicate a loss of integrity of the structure, with the previously monolithic, integrated structure cracking and fragmenting to form multiple segments. These multiple segments exhibited multiple responses which generated multiple frequencies which are evident in the ‘messy’ graphs. It is clear that without the use of the FRF stabilisation process it would be impossible to identify the important vibration modes of the structure.

[The orange arrows in the FRF graphs indicate the detected modes (Figure 8-15)]
Figure 8-15 Specimen 3J: FRF graphs for simulations S2 – S8.
MAC trends

Figure 8-16 shows the MAC trends (first mode) for specimens 3H, 3J and 3K for the shake table testing. The MAC relationship provides an indication of the change in mode shapes with respect to a given datum (in this case, the MAC of each specimen at simulation S4: S50%). [A MAC value close to 1.0 (100%) indicates that two modes are well-correlated and a value close to 0 is indicative of uncorrelated modes.] The graph reveals that the mode shapes for each specimen (first mode) are fairly consistent up to and including simulation S6 (S75%), after which the shapes change substantially. The greatest change occurred for specimen 3H, and the least change occurred for specimen 3J.

Figure 8-16  Specimens 3H, 3J and 3K: MAC trends of Mode 1 for shake table tests.
8.6 Summary

Experimental Modal Testing and Analysis (EMTA) was successfully conducted both prior to and during the shake table testing for the u-shaped adobe wall units. The key processes and outcomes include:

- EMTA using an impact hammer to excite each specimen prior to shake table testing to determine the fundamental frequency. This information was then used to appropriately scale (with respect to time) the input excitation to ensure dynamic similitude and induce damaging near-resonance conditions (described in detail in Chapters 5 and 6). The u-panels with internal vertical reinforcement (specimens 3G and 3K) possessed the lowest first natural frequency (lowest stiffness), due to discontinuities introduced by the reinforcement. Specimen 3B had the highest first natural frequency due to the additional stiffness and mass provided by the corner pilasters.

- EMTA using an impact hammer to excite selected specimens (3F, 3H, 3J and 3K) prior to shake table testing. The multi-degree-of-freedom approach used in this research allowed the determination of the key dynamic characteristics (natural frequencies, damping and mode shapes). The results confirmed the validity of the process undertaken, with the detected mode shapes matching those identified in previous adobe research (Tolles and Krawinkler, 1990).

- EMTA using an impact hammer to excite specimen 3J during the different stages of strengthening to evaluate the influence of each strengthening intervention on the dynamic characteristics. Most notably, the addition of the ‘wing’ wall restraint increased the first natural frequency (stiffness) and decreased the modal damping of the structure.

- EMTA using the shake table to excite selected specimens (3H, 3J and 3K) to assess the changes in dynamic characteristics during the series of shake table tests. Results showed the progressive loss of stiffness, and general increase in modal damping, of the specimens as the level of damage increased. This was particularly evident for specimen 3K (internal vertical reinforcement) which experienced a steady loss of stiffness commencing at the low intensity simulation S4 (S20%). In contrast, the externally reinforced specimens 3H and 3J maintained integrity (stiffness) up to simulation S6 (S75%) after which
damage to the structure became apparent. The mode shape of the dominant first mode (‘first transverse translation mode’) clearly matched the flexural response of the walls, as presented in Chapter 7.

Results confirm that EMTA is a useful tool for evaluating the differences and changes in dynamic characteristics of adobe mudbrick structures. Results provide greater insight into the structural behaviour than mere visual observations, and may be a practical tool for damage detection and condition monitoring of adobe structures.

EMTA was also undertaken for model house 4A, and is discussed in Chapter 10.
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9 MODEL HOUSE 4A: PREPARATION, TESTING, OBSERVATIONS AND ANALYSIS

9.1 Introduction

This chapter describes the construction, preparation, testing, observations and analysis for model house 4A. The model house was reinforced with external vertical bamboo (connected with string through-wall crossties), external horizontal wire and a timber ring beam. The results in this chapter include the observed behaviour and recorded responses for each simulation (the 'what' happened), followed by some comparative analysis and discussion of general crack patterns and failure modes (the 'why' and 'what does it mean').

The testing of a 1:2 scale model house was the natural progression from the comprehensive testing of the u-shaped adobe wall units, as described in Chapters 6 and 7. The reinforcing system chosen for the house was based on a combination (with slight modifications) of the systems used in specimens 3F and 3J, which were the most effective of the u-shaped panels. This consisted of retrofitting an unreinforced house with external vertical bamboo, external horizontal wire and a timber ring beam (Figure 9-1). The construction and testing of the model house provided an opportunity to assess the overall 'constructability' and seismic capacity of the proposed improvement system. The inclusion of penetrations (two doors and one window) added another, real, dimension to the construction and retrofitting process, as well as strongly influencing the seismic behaviour. Of key interest were the interactions between the shear walls and the out-of-plane 'long' walls. A main objective was to avoid or minimise the presence and/or severity of vertical corner cracking, which is widely known to be the main cause of catastrophic failure of adobe buildings in earthquakes (see Chapter 2). Other areas of interest included the effect of penetrations (in both shear and out-of-plane walls), the response of the base of the structure (sliding or rocking), and the connection and response of the ring beam.

As with the u-panel testing, a sense of the behaviour of the specimen can be gauged by a combination of visual aids (photographs and video footage) and displacement graphs.
which show the response of the specimen for each simulation. Video footage is included in Appendix A.

A summary of outcomes and lessons learned is included at the end of the chapter.

The results indicate that the proposed system (including external vertical bamboo, external horizontal wire and a timber ring beam) is extremely effective at delaying major structural damage and ultimate collapse.

Experimental Modal Testing and Analysis (EMTA) was undertaken for model house 4A both prior to and during shake table testing, as described in Chapter 10.

Model house 4A was the centrepiece of a segment of ABC TV’s Catalyst program (www.abc.net.au/catalyst), which aired on 18 May 2006. The segment is included as a DVD in Appendix B.

Figure 9-1 Model house 4A prior to testing.
9.2 Design and preparation of model house 4A

9.2.1 Specifications of model house 4A

Specimen: 4A (model house)
Horizontal reinforcement: Fencing wire (external) – top, middle and bottom
Vertical reinforcement: Bamboo (external)
Ring beam: Timber
Other comments: Retrofit of unreinforced specimen
1st Natural Frequency: 24.1 Hz (Time Scaling Factor: 1.6)

9.2.2 Design of model house 4A

Schematic drawings and plan layouts of model house 4A are shown in Figure 9-2 to Figure 9-4. The model house was designed as a 1:2 scale representation of a simple dwelling. Specimen 4A was designed to ‘match’ the u-shaped wall units, with key dimensions maintained where possible (e.g. mortar and brick size and configuration, wall height and thickness, length of unsupported wall, spacing and dimensions of vertical bamboo poles, and details of the timber ring beam). As with the u-shaped wall units (Chapter 6), the dimensions and configuration of specimen 4A satisfy design criteria recommended in relevant guidelines (e.g. IAEE, 1986; RESESCO, 1997).

Model house 4A consisted of 1,020 full bricks (150 x 150 x 50mm) and 152 half bricks (150 x 70 x 50mm) laid in stretcher bond. All mortar joints were 12–13mm thick.

One window and two door openings were included in the structure. The locations of these penetrations were chosen to introduce asymmetry to the structure, and to study the impact of penetrations in different areas. The door openings were 0.52m wide x 1.0m high and the window opening was 0.52m wide x 0.50m high. These penetrations were left unframed and open during testing, to simulate the worst case scenario.

As with the u-shaped panels, no roof load was applied, although a slightly more rigid ring beam diaphragm was included in model house 4A. No wall restraint was applied to the tops of the shear walls. Two shear pins were located at the base of each shear wall, providing a light connection between the house and the concrete base slab.
Figure 9-2 Model house 4A: schematic drawing of unreinforced structure (view from SE).
Figure 9-3 Model house 4A: plan layout (a) odd courses, (b) even courses.
Figure 9-4 Model house 4A: elevation of N wall showing configuration of reinforcement.

[Vertical bamboo (green lines), horizontal wire (dashed orange lines), through-the-wall string connectors (blue crosses) and timber lintel and ring beam (brown lines)]
9.2.3 Construction of model house 4A

The shell structure of model house 4A was constructed to represent a traditional, unreinforced adobe dwelling (Figure 9-5 to Figure 9-7). Video footage of the construction of model house 4A is included in Appendix A. The same procedure as for the construction of the unreinforced u-shaped adobe wall units was adopted (see Chapter 6, especially specimens 3A and 3F). One difference, however, was the inclusion of two small shear pins at the base of each shear wall, which provided a light connection between the house and the concrete base slab (Figure 9-5). These shear pins were included because of concerns that the whole structure might slide off the base if no attachment were provided.

Timber frames, to demarcate the door and window penetrations, were used during construction, with the walls built around them (Figure 9-6d,e&f). These frames were later removed. The lintels were made of timber, with chicken wire mesh nailed to each surface in contact with the wall in an effort to enhance the bond (Figure 9-7a). The lintels were extended the length of two bricks into the wall (~310mm).

Two courses of bricks, acting as a curing load, were laid on top of the wall during prolonged breaks during construction and after completion of the wall (Figure 9-8), as described in Chapters 4 and 6.

Figure 9-5 Model house 4A: base slab and shear pins prior to construction.
Figure 9-6 Model house 4A: under construction.
The finished specimen weighed approximately 3,000kg. Because of its size and mass, the house was built *in-situ* (on the shake table) to reduce the risk of damage during transportation. The model house was 'cured' under standard laboratory conditions and was tested 49 days after completion of the walls. During this period, the retrofit-strengthening and preparation of the specimen was undertaken, as described below.
9.2.4 Retrofit-strengthening of model house 4A

Model house 4A was retrofit-strengthened in a similar manner to specimen 3F, as described in Chapter 6.

The following procedure was undertaken:

- Location of holes marked, aligned with the desired location of the vertical bamboo reinforcing (Figure 9-9a). To reduce time and materials, it was decided to attach the vertical bamboo poles (via string) every five courses for model house 4A (compared with every three courses for u-shaped wall panels 3E, 3F, 3H, 3I and 3J).
- Holes drilled through the wall, using a power or hand-drill (Figure 9-9a).
- Polypropylene string loops inserted through holes. Each hole was wetted inside, and then injected with mud using a modified syringe (Figure 9-9b).
- Once the mud ‘plugs’ were completely dried (Figure 9-10), bamboo poles (16-22mm diameter) were tied securely (vertically) against the wall (Figure 9-11b). As with specimen 3F, no attachment was made between the vertical bamboo poles and the concrete base. This was done to simulate more realistic in-field conditions.
- A continuous timber ring beam (radiata pine softwood) was placed on top of the wall (Figure 9-10 and Figure 9-11). The ring beam included a midspan cross-beam running transversely (N-S), and plywood diagonal braces at each corner (Figure 9-10b). The ring beam was attached using:
  - bamboo dowels (inserted into oversized holes drilled into the top course of bricks and filled with mud);
  - vertical wire loops passing through the wall three or eight courses down the wall, and securely tied down to the ring beam (Figure 9-10); and
  - horizontal wire loops connected between the vertical bamboo poles and stapled to the ring beam (Figure 9-11a).
- Galvanised fencing wire was looped horizontally, and gently tightened between the vertical bamboo poles at the base, mid-height and top of the wall (Figure 9-11b and Figure 9-12). Once all the bamboo and wire was in position the horizontal wires were gradually tensioned (using simple fencing pliers) around the structure to create a tight mesh-like matrix (Figure 9-13).
Figure 9-9 Model house 4A: (a) drilling holes, and (b) injecting mud into holes.

Figure 9-10 Model house 4A: with strings and timber ring beam.

Figure 9-11 Model house 4A: (a) ring beam connection, and (b) fully retrofitted.
Figure 9-12 Model house 4A: details of vertical bamboo and horizontal wire reinforcement.

(a) SW corner (internal)  (b) S wall: midspan SE panel (internal)  (c) SE corner (internal)

Figure 9-13 Model house 4A: fully retrofitted.

Video footage of the retrofit-strengthening process is included in Appendix A.
9.2.5 Instrumentation of model house 4A

Similar to the u-shaped wall units, a series of dynamic LVDT (Linear Variable Differential Transformer) displacement transducers and accelerometers were used to record the displacement and acceleration at key locations on each specimen and the shake table (ST) during the series of simulations. The locations of the accelerometer and displacement sensors are shown in Figure 9-14 and Figure 9-15, respectively. The sensors were attached to small steel plates which were affixed to the wall.

Due to instrumentation limitations, space restrictions and the desire to leave at least one wall and corner relatively free of obstructions (for visual observation and video footage) it was decided to heavily instrument the N out-of-plane 'long' wall, plus the vulnerable E shear wall (with door opening). Displacement sensors were also placed on the top of the SE and SW corners of the S out-of-plane 'long' wall to record the response of the shear walls. Figure 9-16 shows model house 4A instrumented and ready for testing.

Figure 9-14 Model house 4A: location of accelerometers.
Figure 9-15 Model house 4A: location of LVDT displacement transducers.
Figure 9-16  Model house 4A: instrumented and ready for testing.
9.2.6 Test procedure for model house 4A

Intensity and time scaling

As with the u-shaped adobe wall units, the time history for the El Salvador earthquake of January 13, 2001 was used for model house 4A. In this case, however, only time-scaled simulations were undertaken [i.e. no simulations were undertaken using the original ground motion record (unscaled with respect to time)]. The process of time scaling the ground motion, and determination of the natural frequency of the specimens is described in detail in Chapters 5 and 8. By means of EMTA (using an impact hammer to excite the specimen), the first natural frequency of the specimen was found to be 24.1 Hz, so a time scaling factor of 1.6 was applied to induce damaging near-resonance conditions and ensure dynamic similitude (with the u-shaped panels, plus future model house tests). As with the u-panels, scaling of the intensity refers to scaling of the displacement intensity. (It should be noted that the simulation numbers for model house 4A do not match the simulation numbers from the u-shaped wall panels.) The testing sequence for model house 4A is shown in Table 9-1.

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity</th>
<th>Peak Displacement (mm)</th>
<th>Peak Acceleration (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>10%</td>
<td>?</td>
<td>?</td>
</tr>
<tr>
<td></td>
<td>S2</td>
<td>25%</td>
<td>-3.927</td>
<td>+0.39</td>
</tr>
<tr>
<td></td>
<td>S3</td>
<td>50%</td>
<td>-7.855</td>
<td>+0.90</td>
</tr>
<tr>
<td></td>
<td>S4</td>
<td>75%</td>
<td>-11.679</td>
<td>+1.20</td>
</tr>
<tr>
<td></td>
<td>S5</td>
<td>100%</td>
<td>-15.606</td>
<td>+1.66</td>
</tr>
<tr>
<td></td>
<td>S6</td>
<td>125%</td>
<td>-19.120</td>
<td>-2.03</td>
</tr>
<tr>
<td></td>
<td>S7</td>
<td>100%</td>
<td>-15.606</td>
<td>+1.66</td>
</tr>
<tr>
<td></td>
<td>S8</td>
<td>‘Shakedown’</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 9-1 Model house 4A: testing sequence, intensity, and peak displacement and acceleration of the shake table (ST).
As with the u-shaped wall units, the shake table shut down prematurely during the S125% intensity simulation. This occurred just after reaching peak displacement of the table, and is not considered to have significantly influenced the results.

Simulation S8 ‘shakedown’ was undertaken to push the limits of both the model house and the shake table. It involved subjecting the specimen to a sine-sweep, covering a range of frequencies (1-20Hz) and displacements (1-30mm) in an effort to identify the resonant frequencies of the damaged house and shake the house to pieces. After intense shaking for over ten minutes, the shake table eventually shut down (due to overloading).

Displacement and acceleration data was obtained for simulations S1-S6 (up to S125% intensity), after which the sensors were removed to prevent damage to the instrumentation. As with the u-panel testing, the main emphasis was on the displacement (response) of the specimen, so the analysis of results in this chapter focuses on the displacement data. The accelerometer data was predominantly used in the process of EMTA, which is presented in Chapter 10.

Nomenclature of model house 4A

The same shorthand system for identification of distinct simulations, instrumentation locations, intensities and layout has been adopted as for the u-shaped wall units (Chapter 6), and is summarised in Figure 9-17.

![Figure 9-17 Nomenclature for specimens, simulations and locations.](image)

For example:
- ‘4A – S3 – L3’ = model house 4A, simulation S3 (S50%) and LVDT L3.
- ‘4A – S6 – LST’ = model house 4A, simulation S6 (S125%) and LVDT LST (shake table)

The specimen and instrumentation layout are shown in Figure 9-14 and Figure 9-15.
9.2.7 Video footage of model house 4A

Six video cameras were used to record the response of the structure during the simulations. These cameras were positioned to capture the following angles:

1) **East**: view of E shear wall (perpendicular).
2) **South-east**: view of SE corner, showing S and E walls (obtuse).
3) **South**: view of S ‘long’ wall (perpendicular).
4) **South-west**: view of door opening of S wall (obtuse).
5) **West**: view along N ‘long’ wall (parallel).
6) **North-east**: view of NE corner, showing window and door openings in N and E walls respectively (obtuse).

Comprehensive video footage is included in CD/DVD format in Appendix A, and is a recommended accompaniment to this chapter.

![Figure 9-18 Test day: the house and its maker! (Evans, UTS, 2005)](image-url)
9.3 Testing and results of model house 4A

9.3.1 Results summary

Table 9-2 shows the testing sequence and summary of observations for specimen 4A. The damage grades presented in IAEE (1986) and used for the u-panel specimens (Chapter 6) have also been adopted for the model house (Table 9-3, reproduced from Chapter 6).

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity</th>
<th>Damage grade</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scaled</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td></td>
<td>10%</td>
<td>0</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S2</td>
<td></td>
<td>25%</td>
<td>0</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S3</td>
<td></td>
<td>50%</td>
<td>0</td>
<td>No damage observed.</td>
</tr>
<tr>
<td>S4</td>
<td></td>
<td>75%</td>
<td>1</td>
<td>Rocking of E shear wall. Separation at base. Minor cracking from lintel above door (E wall).</td>
</tr>
<tr>
<td>S5</td>
<td></td>
<td>100%</td>
<td>2-3</td>
<td>Major rocking of E shear wall. Major cracking in E shear wall. Minor cracking in all other wall panels.</td>
</tr>
<tr>
<td>S6</td>
<td></td>
<td>125%</td>
<td>3</td>
<td>Severe rocking of E shear wall. Severe cracking in E shear wall. Moderate cracking in all other wall panels.</td>
</tr>
<tr>
<td>S7</td>
<td></td>
<td>100%</td>
<td>3</td>
<td>Progressive additional damage in all wall panels. Severely damaged. Collapse prevented.</td>
</tr>
<tr>
<td>S8</td>
<td>‘Shakedown’</td>
<td></td>
<td>4</td>
<td>Progressive additional damage in all wall panels. Severely damaged. Collapse prevented.</td>
</tr>
</tbody>
</table>

Notes:
* Damage grades: 0 – no damage; 1 – slight damage (fine cracks); 2 – moderate damage (small cracks, spalling); 3 – heavy damage (large + deep cracks); 4 – destruction (gaps in walls, separation of components); 5 – total collapse. [Based on IAEE (1986), see Table 9-3 for full descriptions.]
<table>
<thead>
<tr>
<th>Damage grade</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 No damage</td>
<td>-</td>
</tr>
<tr>
<td>1 Slight damage</td>
<td>Fine cracks in plaster; fall of small pieces of plaster</td>
</tr>
<tr>
<td>2 Moderate damage</td>
<td>Small cracks in walls; fall of fairly large pieces of plaster, roof tiles slip off; cracks in chimneys; parts of chimney fall down.</td>
</tr>
<tr>
<td>3 Heavy damage</td>
<td>Large and deep cracks in walls; fall of chimneys.</td>
</tr>
<tr>
<td>4 Destruction</td>
<td>Gaps in walls; parts of building may collapse; separate parts of the building lose their cohesion; inner walls collapse.</td>
</tr>
<tr>
<td>5 Total damage</td>
<td>Total collapse of building.</td>
</tr>
</tbody>
</table>

The results and observations for simulations S4 (S75%) and beyond are recorded in this section. Results for simulations S1 – S3 are not included as they are associated with ‘no damage’ pre-cracking cases, and hence are of little value. For each simulation the following information is presented:

- Observations and photographs, focused on each wall panel. Damage grades have been assigned for each wall panel;
- Summary of the peak displacement and RMS of displacement for each LVDT displacement sensor;
- Graphs depicting the absolute displacement of key locations on the house;
- Graphs showing the peak horizontal and vertical flexure of the N ‘long’ wall; and
- Discussion of results.
### 9.3.2 Simulation S4 (S75%) 

**Observations**

Table 9-4 describes the damage grades and observations for model house 4A during simulation S4 (S75%).

<table>
<thead>
<tr>
<th>Location</th>
<th>Damage grade*</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>E shear wall</td>
<td>1</td>
<td>- Rocking of wall.</td>
</tr>
<tr>
<td>(w/ door)</td>
<td></td>
<td>- Separation at base (Figure 9-19a).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Separation of lintel from wall above.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Minor cracking from lintel above door (Figure 9-19b).</td>
</tr>
<tr>
<td>N 'long' wall</td>
<td>0</td>
<td>- No observed damage.</td>
</tr>
<tr>
<td>W shear wall</td>
<td>0</td>
<td>- No observed damage.</td>
</tr>
<tr>
<td>S 'long' wall</td>
<td>0</td>
<td>- No observed damage.</td>
</tr>
</tbody>
</table>

**Note:** For description of damage grades see Table 9-3.

---

**Figure 9-19** Model house 4A: damage from simulation S4 (S75%) to E wall.
Structural response

Table 9-5 shows the peak absolute displacement and RMS of absolute displacement of each LVDT sensor, plus the ratio of these with respect to the peak and RMS of the shake table displacement (ST) for simulation S4 (S75%).

**Table 9-5 Model house 4A: simulation S4 (S75%) - peak and RMS of absolute displacement, and ratio of peaks and RMS with respect to shake table displacement (ST).**

<table>
<thead>
<tr>
<th>TOP</th>
<th>S wall</th>
<th>N wall</th>
<th>S wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SE</td>
<td>NE</td>
<td>Mid</td>
</tr>
<tr>
<td>L ST</td>
<td>L12</td>
<td>L1</td>
<td>L2</td>
</tr>
<tr>
<td>w.r.t. Peak (ST)</td>
<td>100%</td>
<td>141%</td>
<td>135%</td>
</tr>
<tr>
<td>RMS</td>
<td>2.908</td>
<td>3.468</td>
<td>3.656</td>
</tr>
<tr>
<td>w.r.t. RMS (ST)</td>
<td>100%</td>
<td>119%</td>
<td>126%</td>
</tr>
<tr>
<td>THIRD-HEIGHT</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>+ BASE</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>L ST</td>
<td>L6</td>
<td>L7</td>
<td>L8</td>
</tr>
<tr>
<td>w.r.t. Peak (ST)</td>
<td>100%</td>
<td>114%</td>
<td>107%</td>
</tr>
<tr>
<td>RMS</td>
<td>2.908</td>
<td>3.328</td>
<td>3.125</td>
</tr>
<tr>
<td>w.r.t. RMS (ST)</td>
<td>100%</td>
<td>114%</td>
<td>107%</td>
</tr>
</tbody>
</table>

The key response features include:

- The greatest movement occurred at each end of the E shear wall (i.e. L1 at the NE corner, and L12 at the SE corner). Sensors L1 and L12 experienced a closely matched response, which indicates that rocking of the E wall occurred, with no major vertical cracks present.

- The least movement was recorded at each end of the W shear wall (i.e. L5 at the NW corner, and L13 at the SW corner). The movement at sensors L5 and L13 closely matched the movement of the shake table, which indicates the extreme stiffness in the W shear wall.
• For each individual sensor (except L1 and L12) there was a strong correlation between the ratio of the peak displacement and the ratio of the RMS of displacement. This is consistent with the findings from the u-panel testing and analysis (Chapter 7) which suggested good matching between peak and RMS for uncracked sections of a wall.

• The response at L11 (midspan-base of the N wall) was well matched with the movement of the shake table, indicating little or no sliding or rocking of the base at that location.

• Movement of the N wall decreased along the wall (E to W) at both the top (L1-L5) and third-height (L6-L10). The response at L1 (top NE corner) and L3 (top midspan of N wall) and the movement of the shake table (LST) are shown in Figure 9-20. (The response at L5 was omitted because it closely matched the movement of the shake table, as described above). The graphs clearly show the amplification of the response, in particular for L1. This is also clearly evident in the horizontal and vertical flexure of the N wall as shown in Figure 9-21 [snapshot showing relative displacement (with respect to shake table displacement) and capturing peak response at L1].

• It is clear that the presence of an opening in a shear wall degrades the stiffness significantly. This is clearly observed in the different responses of the E shear wall (with door opening) and the W shear wall (without opening). This difference in response had a significant effect on the entire structure, with the movement in the long walls being strongly influenced by the large response of the E shear wall.
Figure 9-20 Model house 4A: absolute displacement of L1, L3 and ST (shake table) for simulation S4 (S75%).

[Peak displacements: L1: +15.814mm; L3: +13.930; ST: -11.679mm]
Figure 9-21 Model house 4A during simulation S4 (S75%): (a) horizontal flexure of top of N wall; (b) vertical flexure at midspan of N wall.

Time: 25.109 - 25.243s ($\Delta t = 0.134$s).
9.3.3 **Simulation S5 (S100%)**

**Observations**

Table 9-6 describes the damage grades and observations for model house 4A during simulation S5 (S100%).

<table>
<thead>
<tr>
<th>Location</th>
<th>Damage grade</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>E shear wall (w/ door)</td>
<td>2-3</td>
<td>- Significant rocking of specimen (see video footage).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Separation at base (Figure 9-22a).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Significant cracking vertically + diagonally from ends of lintel above door (Figure 9-22b).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Major cracking around S shear connector at base (Figure 9-22a).</td>
</tr>
<tr>
<td>N 'long' wall NE panel</td>
<td>2</td>
<td>- Horizontal cracking from lintel above window (NE corner).</td>
</tr>
<tr>
<td>(w/ window)</td>
<td></td>
<td>- Minor diagonal cracking downwards from window (Figure 9-22c).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Minor vertical cracking from base upwards towards window.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Separation at base.</td>
</tr>
<tr>
<td>N 'long' wall NW panel</td>
<td>2</td>
<td>- Minor diagonal cracking (running from base at midspan and linking with diagonal cracking in W panel 7 courses from top) (Figure 9-22d).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Separation at base, E of diagonal crack from base.</td>
</tr>
<tr>
<td>W shear wall</td>
<td>1-2</td>
<td>- Minor vertical cracking at NW corner (running down 7 courses then linking with diagonal cracking in NW panel of N 'long' wall).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- No separation at base.</td>
</tr>
<tr>
<td>S 'long' wall SW panel</td>
<td>2</td>
<td>- Minor diagonal cracking from lintel above door (especially on E side) (Figure 9-22e).</td>
</tr>
<tr>
<td>(w/ door)</td>
<td></td>
<td>- Separation at base, E of door opening.</td>
</tr>
<tr>
<td>S 'long' wall SE panel</td>
<td>2</td>
<td>- Minor vertical cracking at midspan (Figure 9-22f).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Separation at base.</td>
</tr>
<tr>
<td>Internal pilasters</td>
<td>1</td>
<td>- Separation at base, but no observed structural damage.</td>
</tr>
</tbody>
</table>

**Note:** *For description of damage grades see Table 9-3.*
Figure 9-22 Model house 4A: damage from simulation S5 (S100%).
Structural response

Table 9-7 shows the peak absolute displacement and RMS of absolute displacement of each LVDT sensor, plus the ratio of these with respect to the peak and RMS of the shake table displacement (ST) for simulation S5 (S100%).

<table>
<thead>
<tr>
<th>TOP</th>
<th>S wall</th>
<th>N wall</th>
<th>S wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SE</td>
<td>NE</td>
<td>Mid</td>
</tr>
<tr>
<td>L ST</td>
<td>L12</td>
<td>L1</td>
<td>L2</td>
</tr>
<tr>
<td>Peak</td>
<td>15.606</td>
<td>43.825</td>
<td>42.538</td>
</tr>
<tr>
<td>w.r.t. Peak (ST)</td>
<td>100%</td>
<td>281%</td>
<td>273%</td>
</tr>
<tr>
<td>w.r.t. RMS (ST)</td>
<td>100%</td>
<td>201%</td>
<td>217%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>THIRD-HEIGHT + BASE</th>
<th>N wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>NE</td>
</tr>
<tr>
<td>L ST</td>
<td>L6</td>
</tr>
<tr>
<td>Peak</td>
<td>15.606</td>
</tr>
<tr>
<td>w.r.t. Peak (ST)</td>
<td>100%</td>
</tr>
<tr>
<td>w.r.t. RMS (ST)</td>
<td>100%</td>
</tr>
</tbody>
</table>

The key response features include:

- Again, the greatest movement occurred at each end of the E shear wall, with sensors L1 (NE corner) and L12 (SE corner) experiencing a closely matched response. For this simulation, the rocking of the wall was significantly greater (confirmed by video footage, Appendix A).

- Again, the least movement was experienced at each end of the W shear wall, with sensors L5 (NW corner) and L13 (SW corner) experiencing movement closely matched with that of the shake table (Figure 9-23). This was especially evident for L13, which displayed almost identical movement to the shake table.
This indicates that despite severe shaking, the stiffness of the shear wall (without penetrations) was maintained. Of note in the displacement graphs is the slight offset evident at L5 (NW corner), which corresponds with the minor vertical cracking in the NW corner (see Figure 9-26e, below, for photograph of advanced damage in this region).

- For all locations where damage was evident (all sensors except L5, L13 and L10) there was a generally poor correlation between the ratio of the peak displacement and the ratio of the RMS of displacement.
- The response at L11 (midspan-base of the N wall) indicated minor sliding and/or rocking at the base, with a movement of approximately 2mm.
- The response at L1 (top NE corner) and L3 (top midspan of N wall) and the movement of the shake table (LST) is shown in Figure 9-24. (Again, the response at L5 was omitted because it closely matched the movement of the shake table, as described above). The graphs clearly show the amplification of the response at L1 and L3, as well as the offset of the wall (approx. +3.8mm) evident at the end of the simulation for L1 (Figure 9-24a).
- Figure 9-25 shows a snapshot of the relative horizontal and vertical flexure of the N wall. The snapshot corresponds with the peak response at L1, and clearly shows the significant movement at the E end of the wall, and the stability at the W end, which is due to the influence of the door opening in the E shear wall, which is significantly less stiff than the opposite W shear wall without penetration.
Figure 9-23 Model house 4A: absolute displacement of L5, L13 and ST (shake table) for simulation S5 (S100%).

(Peak displacements: L5: -15.367mm; L13: -15.761; ST: -15.606mm)
Figure 9-24 Model house 4A: absolute displacement of L1, L3 and ST (shake table) for simulation S5 (S100%).

[Peak displacements: L1: -42.538mm; L3: -35.484; ST: -15.606mm]
Figure 9-25 Model house 4A during simulation S5 (S100%): (a) horizontal flexure of top of N wall; (b) vertical flexure at midspan of N wall.

Time: 26.719 – 26.953s ($\Delta t = 0.234$s).
9.3.4 Simulation S6 (S125%)

Observations

Table 9-8 shows the damage grades and observations for model house 4A during simulation S6 (S125%).

<table>
<thead>
<tr>
<th>Location</th>
<th>Damage grade*</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>E shear wall (w/ door)</td>
<td>3</td>
<td>- Significant rocking of specimen (see video footage).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Further cracking vertically + diagonally from ends of lintel above door</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(Figure 9-26a&amp;b).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Major damage around S shear connector at base (Figure 9-26a).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Significant cracking around N shear connector at base (Figure 9-26b).</td>
</tr>
<tr>
<td>N 'long' wall NE panel</td>
<td>2-3</td>
<td>- Further horizontal cracking from lintel above window (NE corner)</td>
</tr>
<tr>
<td>(w/ window)</td>
<td></td>
<td>(Figure 9-26b).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Further diagonal cracking downwards from window (Figure 9-26c).</td>
</tr>
<tr>
<td>N 'long' wall NW panel</td>
<td>2-3</td>
<td>- Further diagonal cracking (running from base at midspan and linking with</td>
</tr>
<tr>
<td></td>
<td></td>
<td>diagonal cracking in W panel) (Figure 9-26d).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Development of minor diagonal cracking from top NE side and linking with</td>
</tr>
<tr>
<td></td>
<td></td>
<td>diagonal cracking from base at third-height (Figure 9-26d).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Very slight separation at base, W of diagonal crack (first occurrence).</td>
</tr>
<tr>
<td>W shear wall</td>
<td>2</td>
<td>- Further vertical cracking at NW corner (Figure 9-26e).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Very slight separation at base (first occurrence).</td>
</tr>
<tr>
<td>S 'long' wall SW panel</td>
<td>2-3</td>
<td>- Further diagonal cracking from lintel above door.</td>
</tr>
<tr>
<td>(w/ door)</td>
<td></td>
<td>- Very slight separation at base, W of door opening (first occurrence).</td>
</tr>
<tr>
<td>S 'long' wall SE panel</td>
<td>2-3</td>
<td>- Further vertical cracking at midspan (Figure 9-26f).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Development of minor diagonal cracking from midspan third-height of panel</td>
</tr>
<tr>
<td></td>
<td></td>
<td>to midspan top of S wall (Figure 9-26f).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Development of minor vertical cracking at W end of panel.</td>
</tr>
<tr>
<td>Internal pilasters</td>
<td>1-2</td>
<td>- Minor crushing at base of each pilaster (Figure 9-26c).</td>
</tr>
</tbody>
</table>

Note: *For description of damage grades see Table 9-3.
As with the u-shaped wall units, the shake table shut down prematurely during simulation S6 (S125%), due to shake table overload during the period of intense shaking (around time $t = 25-26$ seconds). This occurred after the peak displacement was imparted by the shake table, and is thus not considered to have affected the results significantly.

Figure 9-26 Model house 4A: damage from simulation S6 (S125%).
Structural response

Table 9-9 shows the peak absolute displacement and RMS of absolute displacement of each LVDT sensor, plus the ratio of this data with respect to the peak and RMS of the shake table displacement (ST) for simulation S6 (S125%).

### Table 9-9 Model house 4A: simulation S6 (S125%) - peak and RMS of absolute displacement, and ratio of peaks and RMS with respect to shake table displacement (ST).

<table>
<thead>
<tr>
<th>TOP</th>
<th>S wall</th>
<th>N wall</th>
<th>S wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SE</td>
<td>NE</td>
<td>Mid</td>
</tr>
<tr>
<td>L ST</td>
<td>L12</td>
<td>L1</td>
<td>L2</td>
</tr>
<tr>
<td>Peak</td>
<td>19.120</td>
<td>-</td>
<td>58.791</td>
</tr>
<tr>
<td>w.r.t. Peak (ST)</td>
<td>100%</td>
<td>-</td>
<td>307%</td>
</tr>
<tr>
<td>RMS</td>
<td>5.849</td>
<td>-</td>
<td>16.098</td>
</tr>
<tr>
<td>w.r.t. RMS (ST)</td>
<td>100%</td>
<td>-</td>
<td>275%</td>
</tr>
</tbody>
</table>

### THIRD-HEIGHT + BASE

<table>
<thead>
<tr>
<th>N wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>SE</td>
</tr>
<tr>
<td>L ST</td>
</tr>
<tr>
<td>w.r.t. Peak (ST)</td>
</tr>
<tr>
<td>w.r.t. RMS (ST)</td>
</tr>
</tbody>
</table>

**Notes:**
- Shake table shut down prematurely during simulation S6 (S125%), therefore RMS calculated for time $t = 0.26$ seconds.
- Data from L12 was erroneous for simulation S6 (S125%).

The key response features include:

- As expected, the greatest movement again occurred at the end of the E shear wall (L1, NE corner). For this simulation, the rocking of the wall was again significantly greater (confirmed by video footage, see Appendix A).
- The least movement was again experienced at each end of the W shear wall, with sensors L5 (NW corner), L13 (SW corner) and L10 (NW corner, third-
height) experiencing only a slight amplification of displacement (with respect to the shake table movement). This outcome was quite remarkable considering the number of severe simulations which the specimen had been subjected to.

- The response at L11 (midspan-base of the N wall) indicated minor sliding and/or rocking at the base, with a relative movement of up to 1.7mm.

- The large amplification of the response at L1 (top NE corner) and L3 (top midspan of N wall) is shown in Figure 9-27. The graphs show the premature shut down of the shake table after the period of intense shaking. The graphs also demonstrate the offset experienced by the wall at L1 (approx. -3.7mm) and L3 (approx. -2.2mm) due to damage of the wall.

- Despite substantial damage to the structure there was no noticeable change of frequency, with the frequency response at L1 and L3 following that of the shake table (Figure 9-27c).

- Figure 9-28 shows a snapshot of the relative horizontal and vertical flexure of the N wall. The snapshot corresponds with the peak response at L1, and clearly shows the significant movement at the E end of the wall, and the stability at the W end.
Figure 9-27 Model house 4A: absolute displacement of L1, L3 and ST (shake table) for simulation S6 (S125%).

[Peak displacements: L1: -58.791mm; L3: -38.710; ST: -19.120mm]
Figure 9-28 Model house 4A during simulation S6 (S125%): (a) horizontal flexure of top of N wall; (b) vertical flexure at midspan of N wall.

Time: 23.294 – 23.653s (Δt = 0.359s).
9.3.5 Simulation S7 (S100% repeated)

Observations

Table 9-10 shows the damage grades and observations for model house 4A during simulation S7 (S100% repeated).

Table 9-10 Model house 4A: observations from simulation S7 (S100% repeated).

<table>
<thead>
<tr>
<th>Location</th>
<th>Damage grade*</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>E shear wall</td>
<td>3-4</td>
<td>- Major rocking of specimen (see video footage, Appendix A).</td>
</tr>
<tr>
<td>(w/ door)</td>
<td></td>
<td>- Further cracking vertically + diagonally from ends of lintel above door (Figure 9-29b&amp;c).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Further damage around N and S shear connectors at base (Figure 9-30a&amp;b).</td>
</tr>
<tr>
<td>N 'long' wall</td>
<td>3</td>
<td>- Further horizontal cracking from lintel above window (esp. NE corner) (Figure 9-29c and Figure 9-30c).</td>
</tr>
<tr>
<td>NE panel</td>
<td></td>
<td>- Further diagonal cracking downwards from window (Figure 9-30c).</td>
</tr>
<tr>
<td>(w/ window)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>N 'long' wall</td>
<td>3</td>
<td>- Further diagonal cracking (running from base at midspan and linking with diagonal cracking in W panel) (Figure 9-30e&amp;f).</td>
</tr>
<tr>
<td>NW panel</td>
<td></td>
<td>- Further diagonal cracking from top NE side and linking with diagonal cracking from base at third-height (Figure 9-30e&amp;f).</td>
</tr>
<tr>
<td>W shear wall</td>
<td>2</td>
<td>- Further vertical cracking at NW corner.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Still no other damage (Figure 9-31a).</td>
</tr>
<tr>
<td>S 'long' wall</td>
<td>3</td>
<td>- Further diagonal cracking from lintel above door (Figure 9-31b)</td>
</tr>
<tr>
<td>SW panel</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(w/ door)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S 'long' wall</td>
<td>3</td>
<td>- Further vertical cracking at midspan (Figure 9-29a, Figure 9-31d&amp;e).</td>
</tr>
<tr>
<td>SE panel</td>
<td></td>
<td>- Further minor diagonal cracking from midspan third-height of panel to midspan top of S wall (Figure 9-31d&amp;e).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Further minor vertical cracking at W end of panel (Figure 9-31c&amp;d).</td>
</tr>
<tr>
<td>Internal pilasters</td>
<td>2</td>
<td>- Minor crushing at base of each pilaster (Figure 9-31c).</td>
</tr>
</tbody>
</table>

Note: *For description of damage grades see Table 9-3.
The instrumentation was removed from the model house prior to simulation S7 (S100\% repeated) to avoid the risk of damage to the delicate sensors. As a result, no displacement data is available for this simulation. Pre-existing cracks were enlarged during this intense simulation, and despite being severely damaged, collapse of the structure was still prevented. Interestingly, the W shear wall was still relatively undamaged during the simulation. Again, none of the vertical bamboo, horizontal wire or through-the-wall strings appeared to break during the simulation.

(a) S and E walls (exterior)

(b) E wall (exterior)

(c) NE corner (exterior)

Figure 9-29 Model house 4A: damage from simulation S7 (S100\% repeated).
Figure 9-30 Model house 4A: damage from simulation S7 (S100% repeated).
Figure 9.31 Model house 4A: damage from simulation S7 (S100% repeated).
9.3.6 Simulation S8 (‘shakedown’)

Simulation S8 ‘shakedown’ was undertaken to push the limits of both the model house and the shake table. It involved subjecting the specimen to a sine-sweep, covering a range of frequencies (1-20 Hz) and displacements (1-30 mm) in an effort to identify the resonant frequencies of the damaged house and shake the house to pieces. After intense shaking for over ten minutes, the shake table eventually shut down (due to overloading). Video footage of simulation S8 (‘shakedown’) is included in Appendix A.

Observations

Table 9-11 describes the damage grades and observations for model house 4A during simulation S8 (‘shakedown’).

<table>
<thead>
<tr>
<th>Location</th>
<th>Damage grade*</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>E shear wall (w/ door)</td>
<td>4</td>
<td>Severely damaged, especially above lintel and at base (Figure 9-32b&amp;c).</td>
</tr>
<tr>
<td>N ‘long’ wall NE panel (w/ window)</td>
<td>4</td>
<td>Severely damaged, with N offset of panel above lintel, and dislodgment of some bricks below window. Whole wall leaning N (Figure 9-32c, Figure 9-33a&amp;b).</td>
</tr>
<tr>
<td>N ‘long’ wall NW panel</td>
<td>4</td>
<td>Severely damaged, with severe diagonal cracking, and some brick units dislodging from wall. Whole wall leaning N (Figure 9-33c&amp;d).</td>
</tr>
<tr>
<td>W shear wall</td>
<td>4</td>
<td>Bottom course of bricks effectively ground away, leaving wall ‘hanging’ from the ring beam. Severely damaged at bottom corners (Figure 9-33e&amp;f).</td>
</tr>
<tr>
<td>S ‘long’ wall SW panel (w/ door)</td>
<td>4</td>
<td>Severely damaged, with severe diagonal cracking in panel W of door, and vertical and diagonal cracking above lintel (Figure 9-32a, Figure 9-33e).</td>
</tr>
<tr>
<td>S ‘long’ wall SE panel</td>
<td>4</td>
<td>Severely damaged, with severe diagonal and vertical cracking (Figure 9-32a).</td>
</tr>
<tr>
<td>Internal plasters</td>
<td>3</td>
<td>Significant damage at base.</td>
</tr>
<tr>
<td>Ring beam</td>
<td>3</td>
<td>Separation of ring beam joint at SE corner.</td>
</tr>
</tbody>
</table>

Note: *For description of damage grades see Table 9-3.
Again, for simulation S8 (‘shake down’) no instrumentation was used. The specimen was severely damaged during the prolonged shaking, with severe abrasion between damaged components increasing the crack width and generating large amounts of dust. Despite our best efforts we were unable to make the structure collapse. This outcome clearly shows that a mudbrick structure reinforced with bamboo, wire, string and a timber ring beam possesses a very high level of safety and integrity.

Figure 9-32 Model house 4A: damage from simulation S8 (‘shakedown’).
Figure 9-33 Model house 4A: damage from simulation S8 ('shakedown').
9.4 Comparative analysis of results from model house 4A

This section compares displacement results of simulations S2 - S6 (up to S125%) for model house 4A. It is not possible to compare the results of the model house 4A with the results of the u-shaped panel tests, because the boundary conditions and modes of failure were so different.

Table 9-12 shows the peak absolute displacement (mm) for all LVDT displacement sensors and the ratio of peaks with respect to peak shake table displacement (ST) for simulations S2 - S6.

Table 9-13 shows the RMS of absolute displacement (mm) for all LVDT displacement sensors and the ratio of RMS with respect to the RMS of the shake table displacement (ST) for simulations S2 - S6. For simulations S2 - S5 the RMS was calculated for time $t = 0.52$ seconds. For simulation S6 (S125%) the RMS was calculated for time $t = 0.26$ seconds, due to premature shutdown of the shake table at around time $t = 26.5$ seconds.

For these tables, a colour-coding system has been developed (in conjunction with the observations above) to signify the degree of movement experienced at each key location: (i) negligible response [green]; (ii) slight response [dark yellow]; (iii) moderate response [orange]; and (iv) severe response [red].

From Table 9-12 and Table 9-13 the following observations can be made:

- There was a strong correlation between the ratios of peak displacement and RMS (with respect to the shake table, ST) for all locations during the lower intensity simulations S2 (S25%) and S3 (S50%). For the higher intensity simulations S4 (S75%), S5 (S100%) and S6 (S125%) there was a strong correlation for those locations which experienced a minor relative response (e.g. L5, L10 and L13), whereas for those locations which experienced significant relative movement (e.g. L1, L2, L6 and L12) the ratio of peak displacements (w.r.t. peak ST displacement) was significantly higher than the ratio of the RMS. [The exception to this was simulation S6 (S125%), where the shake table shut down prematurely and the RMS was taken for an abbreviated time, $t = 0.26$ seconds.]
• Locations L1 and L12 experienced the same effective response. This suggests that the main response of the structure was severe racking in the east shear wall (also confirmed by observations and video footage), with little or no vertical separation of the walls at these corners.

• Locations L5, L10 and L13 experienced the same effective response, which was negligible. This indicates that the west shear wall was extremely stiff and undamaged during the simulations.

• The response of L11, midspan base of north ‘long’ wall, shows that the base experienced only slight to moderate movement (sliding) during the simulations.

The crack patterns and failure mechanisms resultant from these movements are discussed in the following section.
Table 9-12 Model house 4A: peak displacement (mm) for all LVDT displacement sensors and ratio of peaks with respect to peak shake table (L ST) displacement (shaded) for simulations S2 - S6.

<table>
<thead>
<tr>
<th>Simulation</th>
<th>Peak</th>
<th>L ST</th>
<th>L1</th>
<th>L2</th>
<th>L3</th>
<th>L4</th>
<th>L5</th>
<th>L6</th>
<th>L7</th>
<th>L8</th>
<th>L9</th>
<th>L10</th>
<th>Base N</th>
<th>Top of S wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>L ST</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(S25%)</td>
<td>w.r.t. L ST</td>
<td>100%</td>
<td>99%</td>
<td>103%</td>
<td>105%</td>
<td>104%</td>
<td>99%</td>
<td>109%</td>
<td>102%</td>
<td>101%</td>
<td>99%</td>
<td>107%</td>
<td>100%</td>
<td>103%</td>
</tr>
<tr>
<td>(S50%)</td>
<td>w.r.t. L ST</td>
<td>100%</td>
<td>108%</td>
<td>109%</td>
<td>105%</td>
<td>102%</td>
<td>99%</td>
<td>111%</td>
<td>101%</td>
<td>102%</td>
<td>99%</td>
<td>101%</td>
<td>101%</td>
<td>110%</td>
</tr>
<tr>
<td>(S75%)</td>
<td>w.r.t. L ST</td>
<td>100%</td>
<td>135%</td>
<td>128%</td>
<td>119%</td>
<td>106%</td>
<td>97%</td>
<td>114%</td>
<td>107%</td>
<td>105%</td>
<td>101%</td>
<td>102%</td>
<td>104%</td>
<td>141%</td>
</tr>
<tr>
<td>4A-S5</td>
<td>Peak</td>
<td>15.606</td>
<td>42.538</td>
<td>41.863</td>
<td>35.484</td>
<td>27.005</td>
<td>15.367</td>
<td>27.080</td>
<td>22.063</td>
<td>22.667</td>
<td>18.228</td>
<td>16.152</td>
<td>17.660</td>
<td>43.825</td>
</tr>
<tr>
<td>(S100%)</td>
<td>w.r.t. L ST</td>
<td>100%</td>
<td>273%</td>
<td>268%</td>
<td>227%</td>
<td>173%</td>
<td>98%</td>
<td>174%</td>
<td>141%</td>
<td>145%</td>
<td>117%</td>
<td>103%</td>
<td>113%</td>
<td>281%</td>
</tr>
<tr>
<td>(S125%)*</td>
<td>w.r.t. L ST</td>
<td>100%</td>
<td>307%</td>
<td>261%</td>
<td>202%</td>
<td>155%</td>
<td>106%</td>
<td>157%</td>
<td>115%</td>
<td>138%</td>
<td>122%</td>
<td>107%</td>
<td>109%</td>
<td>-</td>
</tr>
</tbody>
</table>

Notes:  
* Shake table shut down prematurely during simulation S6 (S125%).  
* Data from L12 during simulation S6 (S125%) erroneous.

Key:  
- Green: negligible response  
- Dark yellow: moderate response  
- Orange: major response  
- Red: severe response  
  
(up to 120%)  
(120-149%)  
(150-199%)  
(above 200%)
<table>
<thead>
<tr>
<th>Simulation</th>
<th>RMS</th>
<th>L ST</th>
<th>L1</th>
<th>L2</th>
<th>L3</th>
<th>L4</th>
<th>L5</th>
<th>L6</th>
<th>L7</th>
<th>L8</th>
<th>L9</th>
<th>L10</th>
<th>L11</th>
<th>L12</th>
<th>L13</th>
</tr>
</thead>
<tbody>
<tr>
<td>4A-S2</td>
<td>RMS</td>
<td>0.970</td>
<td>0.968</td>
<td>0.989</td>
<td>0.990</td>
<td>0.986</td>
<td>0.952</td>
<td>1.038</td>
<td>0.981</td>
<td>0.980</td>
<td>0.960</td>
<td>1.032</td>
<td>1.001</td>
<td>0.988</td>
<td>0.961</td>
</tr>
<tr>
<td>(S25%)</td>
<td>w.r.t. L ST</td>
<td>100%</td>
<td>100%</td>
<td>102%</td>
<td>102%</td>
<td>102%</td>
<td>98%</td>
<td>107%</td>
<td>101%</td>
<td>101%</td>
<td>99%</td>
<td>106%</td>
<td>103%</td>
<td>102%</td>
<td>99%</td>
</tr>
<tr>
<td>4A-S3</td>
<td>RMS</td>
<td>1.947</td>
<td>2.041</td>
<td>2.052</td>
<td>2.032</td>
<td>2.000</td>
<td>1.886</td>
<td>2.121</td>
<td>1.980</td>
<td>1.986</td>
<td>1.932</td>
<td>2.048</td>
<td>2.013</td>
<td>2.053</td>
<td>1.939</td>
</tr>
<tr>
<td>(S50%)</td>
<td>w.r.t. L ST</td>
<td>100%</td>
<td>105%</td>
<td>105%</td>
<td>104%</td>
<td>103%</td>
<td>97%</td>
<td>105%</td>
<td>102%</td>
<td>102%</td>
<td>99%</td>
<td>105%</td>
<td>103%</td>
<td>105%</td>
<td>100%</td>
</tr>
<tr>
<td>(S75%)</td>
<td>w.r.t. L ST</td>
<td>100%</td>
<td>126%</td>
<td>124%</td>
<td>118%</td>
<td>111%</td>
<td>101%</td>
<td>114%</td>
<td>107%</td>
<td>106%</td>
<td>101%</td>
<td>104%</td>
<td>99%</td>
<td>119%</td>
<td>99%</td>
</tr>
<tr>
<td>(S100%)</td>
<td>w.r.t. L ST</td>
<td>100%</td>
<td>217%</td>
<td>196%</td>
<td>174%</td>
<td>142%</td>
<td>102%</td>
<td>140%</td>
<td>126%</td>
<td>122%</td>
<td>108%</td>
<td>103%</td>
<td>101%</td>
<td>201%</td>
<td>99%</td>
</tr>
<tr>
<td>(S125%)*</td>
<td>w.r.t. L ST</td>
<td>100%</td>
<td>275%</td>
<td>241%</td>
<td>200%</td>
<td>156%</td>
<td>102%</td>
<td>154%</td>
<td>138%</td>
<td>130%</td>
<td>112%</td>
<td>104%</td>
<td>107%</td>
<td>-</td>
<td>100%</td>
</tr>
</tbody>
</table>

Notes:
- Shake table shut down prematurely during simulation S6 (S125%), therefore RMS calculated for time t = 0-26 seconds.
- Data from L12 during simulation S6 (S125%) erroneous.

Key:
- Green: negligible response
- Dark yellow: moderate response
- Orange: major response
- Red: severe response

(up to 115%)
(115-139%)
(140-170%)
(above 170%)
9.5 Analysis of crack patterns and failure mechanisms

The displacement data and observations (visual, video and photographs) from the tests clearly indicate the major rocking movement of the E shear wall (with door opening). In contrast, the opposite shear wall (W) experienced very little movement during the simulations. This large difference in movement is clearly shown in the displacement tables and flexure graphs for each simulation. Figure 9-34 summarises this information, and shows the maximum and minimum values (relative displacement) of the N ‘long’ wall for simulations S3 – S6. The graphs show the flexure along the top of the wall (horizontal), along the wall at third-height (horizontal), and at the midspan (vertical). In addition, Figure 9-35 shows the RMS of relative displacement of the N ‘long’ wall for simulations S3 – S6. Again, the flexure is depicted along the top of the N wall (horizontal), along the wall at third-height (horizontal), and at the midspan (vertical).

These graphs clearly show the large relative displacement experienced at the E end of the N ‘long’ wall. This large movement was due to rocking of the E wall, with the door opening forming a frame rather than a stiff shear wall. Even during the high intensity simulations (e.g. S4, S5 and S6) the solid W shear wall experienced very little movement. Such disparity in response between the E and W shear walls generated significant torsion in the structure. These torsions, in combination with the shear and flexure already acting on the structure, were responsible for the damage which occurred in the model house. This outcome clearly demonstrates the negative influence of penetrations in shear walls.

Even prior to testing, the E shear panel was identified as being particularly vulnerable. In addition to the discontinuities introduced by the door opening, the panel was also the least reinforced panel in the structure. When horizontal and vertical cracking occurred (around the lintel) the only resistance to rocking was provided by the timber ring beam and the orthogonal out-of-plane walls. In this case, however, the orthogonal walls experienced significant movement in their own right, due to the out-of-plane response to the ground motion.

The vertical flexure graphs for the N wall (Figure 9-34 and Figure 9-35) suggest that there was some bulging in the wall, which would have contributed to the diagonal
cracking in the out-of-plane wall panels, especially around the window opening in the N wall.

The combination of vertical and horizontal flexural response in the N and S ‘long’ walls generated 45° diagonal cracks in these walls. This was especially evident in the NW panel of the N wall, and below the window in the NE panel of the N wall. The 45° diagonal cracks indicate the presence of maximum tensile stresses on principal planes due to induced horizontal and vertical shear forces in these walls.

The vertical response at the midspan of the N wall was essentially symmetrical for movement in the positive (N) and negative (S) directions. This suggests that the internal pilasters at the midspan of the N wall had little effect in minimising inward movement (−/S direction). The separation at the base of the pilasters (during simulation S5) plus damage at the base (during simulations S6, S7 and S8) suggests a combination of base sliding and rocking of the pilaster. Because the outwards movement (+/N direction) was similar in magnitude to the inwards movement (−/S direction) it could be argued that the location (internal or external) of pilasters is not overly important. If this is the case, then internal pilasters represent a more viable option, because they require less protection from the elements (e.g. overhanging roof, protective render, etc), although they do reduce the internal, usable area of the house.

The presence of the reinforcement system (bamboo, wire, string and timber ring beam) worked to effectively restrain movement in the structure. The main failure patterns evident in unreinforced structures (vertical corner cracking and overturning of wall panels) did not occur in model house 4A. In fact, there was only evidence of slight vertical corner cracking in one corner (NW) which did not continue for the full height of the wall, but connected with the diagonal cracking in the NW panel of the N wall seven courses from the top. This suggests that the reinforcing system, as tested, served to effectively prevent the formation of severe vertical corner cracking, with seismic energy redistributed around the structure.
Figure 9.34 Model house 4A: maximum and minimum of relative displacement of N ‘long’ wall – along top (L1-L5, top-left graph); at third-height (L6-L10, bottom-left graph) and at midspan (L3, L8, L11, right graph) for simulations S3 – S6.
Figure 9-35 Model house 4A: RMS of relative displacement of N ‘long’ wall – along top (L1-L5, top-left graph); at third-height (L6-L16, bottom-left graph) and at midspan (L3, L8, L11, right graph) for simulations S3 – S6.
9.6 Summary

The following outcomes and general lessons were learned from the experimental testing of model house 4A:

- Test results indicate that a major improvement in the earthquake resistance of adobe mudbrick structures can be obtained by using external vertical bamboo reinforcement, external horizontal wire reinforcement and a timber ring beam. These additions, when securely tied together, create an integrated matrix which restrains movement and enhances the overall strength of the structure. The model house performed extremely well even during repeated high intensity shake table simulations, with catastrophic failure and collapse prevented in all cases. This reinforcement system can be more widely applied in the retrofit-strengthening of existing structures, as well as for new-build constructions.

- Results confirm that walls with penetrations are more vulnerable than walls without penetrations. Furthermore, it appears that in-plane shear walls with penetrations (which act as frames) are more vulnerable to damage than out-of-plane walls with penetrations. Interestingly, out-of-plane walls without penetrations are more vulnerable than in-plane shear walls without penetrations. Obviously, in reality it is not possible to accurately predict the direction of earthquake loading and to thus design a house with walls designated as shear walls or out-of-plane walls, so any design should consider the worst case scenario, which includes minimisation of openings, and provision of additional reinforcement around all penetrations.

- Test results confirm that additional reinforcement was required around the door opening in the E wall panel. This could have been practically achieved by placing an extra vertical bamboo pole on each side of the door. If possible, some form of diagonal reinforcement (wire or bamboo) could also be placed around penetrations. Furthermore, window and door frames would provide some additional resistance to racking, especially if the door or window is closed during an earthquake (although, obviously, this can not be assured).

- Results suggest there is significant value in undertaking racking tests to assess the in-plane shear capacity of wall panels, especially for panels with penetrations. For these experiments to be beneficial, it is important to ensure that the boundary conditions (base connection, corner connection, continuous ring beam) are appropriate and realistic.
10 MODEl HOUSE 4A: EXPERIMENTAL MODAL TESTING AND ANALYSIS (EMTA)

10.1 Introduction

Experimental Modal Testing and Analysis (EMTA) was undertaken for model house 4A to determine the dynamic characteristics both prior to, and during shake table testing. The following processes were undertaken:

- EMTA using an impact hammer to excite the model house to assess the influence of impact location and penetrations.
- EMTA using an impact hammer to excite the model house at key stages during the retrofit-strengthening process.
- EMTA using an impact hammer to excite the model house to determine the first natural frequency at the final strengthening stage (prior to shake table testing). The results were used to appropriately scale (with respect to time) the shake table input excitation to ensure dynamic similitude and induce damaging near-resonance conditions (described in detail in Chapters 5 and 9).
- EMTA using the shake table to excite the model house to assess the changes in dynamic characteristics during the series of shake table simulations.

A special testing procedure was adopted to capture the dynamic characteristics with the same general accuracy as for the u-shaped adobe wall units. The test procedure and instrumentation for both the impact hammer testing and the shake table testing are described in this chapter. The process of data acquisition, signal processing and modal analysis was the same as for the EMTA of the u-panels (detailed in Chapter 8).

Some additional information is included in Appendix D.
10.2 EMTA of undamaged model house 4A: impact hammer tests

10.2.1 Test procedure and instrumentation

The test procedure was similar to that described for the u-panel tests in Chapter 8 (excitation with a modally-tuned ICP sledge hammer; vibration responses measured by piezoelectric-type accelerometers). In order to capture the dynamic characteristics of the model house with the same accuracy (same general accelerometer mesh configuration) as the u-panels, a special testing strategy was developed to most effectively utilise the limited number of compatible accelerometers.

Testing was done by dividing the model house into two parts, representing the north (N) out-of-plane ‘long’ wall (Figure 10-1) and the south (S) out-of-plane ‘long’ wall (Figure 10-2). Each wall was further divided into halves, representing the same configuration and dimensions as the out-of-plane ‘long’ walls in the u-panels. Testing of each half was undertaken separately and the results later combined. This was done by mounting the accelerometers on the outer face of one half of the wall (e.g. left side of N wall, indicated by blue dots in Figure 10-1), with the wall excited at impact point A, followed by excitation at impact point B (on the adjacent half). The results for both impact points were recorded. Next, the accelerometers were moved to the adjacent half of the wall (i.e. right side of N wall, indicated by red dots in Figure 10-1), and the results recorded for both impact points A and B. The results for both halves were then combined to produce the overall dynamic characteristics of the N ‘long’ wall. The subdividing and combining of the wall results was possible since the data being analysed was in the frequency domain, which was independent of the force of the impact excitation. The same procedure was undertaken for the S ‘long’ wall, using impact points C and D (Figure 10-2).

This approach also allowed an assessment of the influence of different impact excitation points and the location of penetrations (window and door openings), as described in the next section.
Figure 10-1 Model house 4A, north wall: test set up for the impact hammer tests.

Figure 10-2 Model house 4A, south wall: test set up for the impact hammer tests.
10.2.2 Influence of impact point and penetrations

Analysis of the EMTA results for each impact point provides significant information about the influence of impact location and penetrations. The FRF graphs and first three mode shapes for each impact point for model house 4A prior to strengthening (stage ‘a’) are shown in Figure 10-3 to Figure 10-6.

The following features were noted:

- The door opening in the east (E) shear wall had a significant impact on the mode shape of Mode 1, which showed substantial deformation at the E end, and very little deformation at the rigid west (W) end. This feature was observed for all impact points and clearly demonstrates the vulnerability of the E shear wall.

- The door opening in the S wall had a notable impact on the mode shapes of the first three modes (Figure 10-5 and Figure 10-6). There was substantially more deformation of the S wall around the penetration than for the opposite solid panel on the N wall.

- For impact points A and D (east end of the model) the first mode dominated the FRF graphs (Figure 10-3 and Figure 10-6). This mode corresponds with the ‘first transverse translation’ (flexure) mode (discussed in greater detail below). For these impact points, the location of impact almost matched a node of the second mode, hence it was less dominant in the FRF graphs. For impact points B and C (west end of the model) the second mode dominated the FRF graphs. Despite these differences, the mode shapes and corresponding natural frequencies for each wall were closely matched (as expected).

The natural frequencies and damping ratios for each impact point at each of the strengthening stages are presented in Table 10-1.
Figure 10-3 Model house 4A, north wall – impact point A: dynamic characteristics at strengthening stage ‘a’ (unreinforced).

Figure 10-4 Model house 4A, north wall – impact point B: dynamic characteristics at strengthening stage ‘a’ (unreinforced).
Figure 10-5 Model house 4A, south wall – impact point C: dynamic characteristics at strengthening stage ‘a’ (unreinforced).

Figure 10-6 Model house 4A, south wall – impact point D: dynamic characteristics at strengthening stage ‘a’ (unreinforced).
10.2.3 Strengthening stages of model house 4A

In order to evaluate the influence of different interventions during the retrofit-strengthening of model house 4A, EMTA was undertaken at each key strengthening stage, as shown in Table 10-1. The table shows the results for both N and S ‘long’ walls, for each impact point, as described above.

Stage ‘a’ refers to the ‘as-built’ unreinforced model house (Figure 10-7a). Stage ‘b’ includes the string cross-ties passing through the wall and the timber ring beam, attached with wire loops and bamboo dowels, as described in Chapter 9 (Figure 10-7b). Stage ‘c’ refers to the specimen prior to testing, which includes the external vertical bamboo reinforcement, horizontal wire reinforcement, and wire connectors between the bamboo and the ring beam (Figure 10-7c).

![Figure 10-7 Model house 4A: strengthening stages.](image_url)
Table 10-1 Model house 4A: natural frequencies (f) and modal damping ratios (ζ) during the strengthening process.

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Impact point</th>
<th>Mode 1</th>
<th>Mode 2</th>
<th>Mode 3</th>
<th>Mode 4</th>
<th>Mode 5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>f₁ (Hz)</td>
<td>ζ₁ (%)</td>
<td>f₂ (Hz)</td>
<td>ζ₂ (%)</td>
<td>f₃ (Hz)</td>
<td>ζ₃ (%)</td>
</tr>
<tr>
<td>4A - a</td>
<td>North</td>
<td>A</td>
<td>23.98</td>
<td>0.91</td>
<td>30.49</td>
<td>1.31</td>
</tr>
<tr>
<td></td>
<td></td>
<td>B</td>
<td>24.02</td>
<td>0.96</td>
<td>30.34</td>
<td>1.31</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C</td>
<td>24.02</td>
<td>0.92</td>
<td>30.54</td>
<td>1.36</td>
</tr>
<tr>
<td></td>
<td></td>
<td>D</td>
<td>24.21</td>
<td>0.92</td>
<td>30.42</td>
<td>1.31</td>
</tr>
<tr>
<td>4A - b</td>
<td>North</td>
<td>A</td>
<td>24.13</td>
<td>1.14</td>
<td>31.63</td>
<td>0.28</td>
</tr>
<tr>
<td></td>
<td></td>
<td>B</td>
<td>24.25</td>
<td>1.09</td>
<td>31.43</td>
<td>1.81</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C</td>
<td>24.41</td>
<td>0.98</td>
<td>31.68</td>
<td>0.74</td>
</tr>
<tr>
<td></td>
<td></td>
<td>D</td>
<td>24.28</td>
<td>1.08</td>
<td>31.65</td>
<td>0.85</td>
</tr>
<tr>
<td>4A - c</td>
<td>North</td>
<td>A</td>
<td>24.23</td>
<td>0.64</td>
<td>31.33</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td></td>
<td>B</td>
<td>24.07</td>
<td>0.75</td>
<td>31.31</td>
<td>1.26</td>
</tr>
<tr>
<td></td>
<td>South</td>
<td>C</td>
<td>24.06</td>
<td>0.84</td>
<td>31.08</td>
<td>1.24</td>
</tr>
<tr>
<td></td>
<td></td>
<td>D</td>
<td>24.09</td>
<td>1.04</td>
<td>31.26</td>
<td>0.83</td>
</tr>
</tbody>
</table>
From Table 10-1 the following features were observed:

- The natural frequencies were closely matched for both N and S walls, irrespective of the impact point and configuration of penetrations. Theoretically, this outcome was expected, because the natural frequencies of the lower global modes are identical over the entire structure, as noted above and in Chapter 8. Figure 10-8 shows the natural frequency trends during the strengthening process.

- The natural frequencies for Modes 1, 2, 4 and 5 increased slightly during the strengthening process. The natural frequency for Mode 3, however, increased substantially between strengthening stages ‘a’ and ‘b’. This aspect is discussed in greater detail below.

- The first natural frequency of model house 4A was identified as 24.1 Hz, so a time scaling factor of 1.6 was applied, as presented in Chapter 9.

- As with the EMTA of the u-panels, the damping ratios varied considerably, with no clear trends evident.

**Figure 10-8** Model house 4A: natural frequency trends during the strengthening process.

Considering the consistency of dynamic characteristics (natural frequencies, damping ratios and mode shapes) for both N and S ‘long’ walls, further analysis was undertaken for the N ‘long’ wall. Figure 10-9 presents the FRF graphs and first three mode shapes for the N ‘long’ wall during the retrofit-strengthening process. (A complete suite of dynamic characteristics for each wall and impact point is presented in Appendix D.)
Figure 10-9 Model house 4A, north wall – impact point A: dynamic characteristics during strengthening stages.

[Orange arrows indicate location of detected modes]
As mentioned above, the natural frequency for Mode 3 increased substantially between strengthening stages ‘a’ and ‘b’.

The mode shape diagrams provide some insight into this feature. For stage ‘a’ the second mode represented the ‘first breathing mode’, and the third mode represented the ‘first torsion mode’ (as defined by Tolles and Krawinkler, 1990, and presented in Figure 10-10 below). For stages ‘b’ and ‘c’ the second mode represented the ‘first torsion mode’ and the third mode represented what appears to be the ‘first breathing mode’. In other words, the mode shapes for the second and third modes swapped between stages ‘a’ and ‘b’. This change can be attributed to the addition of the timber ring beam (strengthening stage ‘b’) which included a crossbeam running transversely between the midspan of each long wall, and diagonal braces at the corners (Figure 10-7b). These components had the effect of creating greater harmony between the opposite long walls, thus reducing the influence of the ‘first breathing mode’ (which refers to the opposite walls moving in opposite directions, Figure 10-10). Tolles and Krawinkler (1990) noted that for scale model houses with firmly anchored roof beams the ‘first breathing mode’ did not exist because the walls moved in-phase. In the case of model house 4A, the ‘breathing mode’ appears not to have been eliminated, which suggests that the timber ring beam used still allowed the walls to move partially out-of-phase. (It should be noted that because EMTA was undertaken on each wall separately, there is no information relating to the simultaneous modal deformation of each wall. The presence of the ‘breathing modes’ was identified by Tolles and Krawinkler, and seems reasonable in the case of model house 4A.) The changes in mode shapes between strengthening stages ‘a’ and ‘b’ are seen in the MAC trends presented in Figure 10-11 (MAC ratios calculated relative to the MAC at strengthening stage ‘c’). The mode shapes for Modes 1, 4 and 5 show a strong correlation, whereas there was a poor correlation for Modes 2 and 3.

As discussed in Chapter 8 (EMTA of the u-panels) the mode shapes and descriptions proposed by Tolles and Krawinkler (1990) have been adopted in this research project. Figure 10-10 shows the first five mode shapes for model house 4A, matched with the complete mode shapes and labels proposed by Tolles and Krawinkler. Comparison between the two sources clearly demonstrates the influence of door openings in the shear walls which generate asymmetry and torsion in the out-of-plane walls. The model houses tested by Tolles and Krawinkler (see Chapter 3) did not include a door opening in either shear wall, so the resultant mode shapes were relatively symmetrical.
Figure 10-10 Mode shapes and corresponding labels for impact hammer test prior to shake table testing (Model house 4A, and Tolles and Krawinkler, 1990).
Figure 10-11 Model house 4A: MAC trends for Modes 1 – 5 during the strengthening stages.
10.3 EMTA of model house 4A: shake table testing

10.3.1 Test procedure and instrumentation

The dynamic response of the N ‘long’ wall was recorded during the shake table testing. Only one wall was chosen due to the limited number of compatible accelerometers. The results from the impact hammer tests indicated that the dynamic characteristics of the N and S ‘long’ walls were closely matched, so recording of only one wall was deemed to be satisfactory. (Furthermore, instrumentation of the N wall left the S, E and W walls free of obstructions for clearer observation and video recording.)

Due to the limited number of compatible accelerometers a slightly different accelerometer mesh configuration was used for the shake table testing (Figure 10-12), compared with the impact hammer testing. The mesh configuration was determined according to the anticipated mode shapes of the structure (based on the impact hammer test results), plus the known dynamic behaviour of the u-panels during the shake table testing (Chapter 8). Accelerometers, therefore, were placed in the vicinity of the maximum anticipated deflections of the mode shapes of the lower modes. No accelerometers were placed at the base of the structure since only a minor response was expected at that location. The input excitation was measured by an accelerometer placed in the middle of the foundation frame attached to the shake table.

![Diagram of model house 4A, north wall: test set up for the shake table tests.](image)

Figure 10-12 Model house 4A, north wall: test set up for the shake table tests.
10.3.2 Dynamic characteristics

The dynamic characteristics of the N wall of model house 4A during the shake table testing are shown in Table 10-2, and Figure 10-13 to Figure 10-15. Observations and comments are noted after the figures.

Table 10-2 Model house 4A: natural frequencies (f) and damping ratios (ζ) for shake table testing.

<table>
<thead>
<tr>
<th>Simulation</th>
<th>Mode 1</th>
<th>Mode 2</th>
<th>Mode 3</th>
<th>Mode 4</th>
<th>Mode 5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>f₁ (Hz)</td>
<td>ζ₁ (%)</td>
<td>f₂ (Hz)</td>
<td>ζ₂ (%)</td>
<td>f₃ (Hz)</td>
</tr>
<tr>
<td>S1 (S10%)</td>
<td>27.29</td>
<td>1.29</td>
<td>35.62</td>
<td>0.80</td>
<td>41.60</td>
</tr>
<tr>
<td>S2 (S25%)</td>
<td>26.39</td>
<td>0.66</td>
<td>32.32</td>
<td>0.11</td>
<td>41.63</td>
</tr>
<tr>
<td>S3 (S50%)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>41.63</td>
</tr>
<tr>
<td>S4 (S75%)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>S5 (S100%)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Note:
From simulation S3 (S50%) onwards the mode stabilisations were unidentifiable (Figure 10-15).

![Amplitude vs Frequency](image1)

(a) Simulation S1 (S10%)

![Amplitude vs Frequency](image2)

(b) Simulation S2 (S25%)

Figure 10-13 Model house 4A: FRF stabilisation diagrams for simulations S1 (S10%) and S2 (S25%).

[Orange arrows indicate location of detected modes]
Figure 10-14 Model house 4A, north wall: mode shapes for simulations S1 (S10%) and S2 (S25%).
The following features were observed:

- There was a minor decrease in the frequency of Mode 1 from simulation S1 (S10%) to S2 (S25%). This minor decrease indicates a slight loss of stiffness of the structure. There was a more substantial decrease in the frequency of Mode 2.
- There was insufficient information to note any relevant trends in the damping ratio.
- In general, the FRF stabilisation diagrams were significantly 'messier' than for the impact hammer tests (which were clear, with well-defined mode peaks). This reflects the complex interactions and responses within the structure during the dynamic testing. Despite the 'noisy' FRF graphs, the FRF stabilisation diagrams showed clear and straight stabilisation lines for simulations S1 (S10%) and S2 (S25%) (Figure 10-13). From simulation S3 (S50%) onwards, clearly defined mode stabilisations were not identifiable, despite some random stabilisations shown in the FRF stabilisation diagrams (Figure 10-15). These isolated stabilisations indicate local higher modes.
- The mode shapes revealed some interesting features about the dynamic response of the structure (Figure 10-14). The mode shape descriptions for simulations S1 (S10%) and S2 (S25%) are shown in Table 10-3. Again, the descriptions refer to those proposed by Tolles and Krawinkler (1990), as shown in Figure 10-10. It is interesting to note that for the shake table testing the first mode corresponded with the 'first torsion mode', whereas for the impact hammer tests,
the first mode matched the ‘first transverse translation mode’. This is confirmed by the poor correlation of the MAC trends shown in Figure 10-16. This result shows that for the shake table testing, the asymmetry of the structure had a greater effect on the torsional response than the translational response.

Table 10-3 Model house 4A: mode shape descriptions for simulations S1 (S10%) and S2 (S25%).

<table>
<thead>
<tr>
<th>Mode</th>
<th>S1 (S10%)</th>
<th>S2 (S25%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>‘First Torsion Mode’</td>
<td>‘First Torsion Mode’</td>
</tr>
<tr>
<td>2</td>
<td>‘First Transverse Translation Mode’</td>
<td>‘First Transverse Translation Mode’</td>
</tr>
<tr>
<td>3</td>
<td>Most probably ‘Second Transverse Translation Mode’</td>
<td>‘Second Transverse Translation Mode’</td>
</tr>
<tr>
<td>4</td>
<td>‘Second Transverse Translation Mode’</td>
<td>Combination of translation and torsion modes</td>
</tr>
<tr>
<td>5</td>
<td>‘Second Torsion Mode’</td>
<td>(Not detected)</td>
</tr>
</tbody>
</table>

- For simulations S1 (S10%) and S2 (S25%) the mode shapes for the first three modes were closely matched (Figure 10-14). The mode shape of Mode 4 for simulation S2 (S25%) appeared to be a combination of translation and torsion modes, representative of the complex response of the structure. The correlation between the mode shapes for these simulations is shown in the MAC trends in Figure 10-17.
- The mode shapes of the ‘first torsion mode’ closely matched the flexure results presented in Chapter 9. This indicates that EMTA is a useful tool to predict and reflect the physical response of the structure.

As with the EMTA of the u-panels it was observed that shake table tests determined a higher first natural frequency than for the impact hammer tests. Given the clear and reliable FRFs for the impact hammer tests, compared with the ‘noisy’ shake table results, it is believed that the hammer test results are more accurate. It is recommended, therefore, that for future testing an impact hammer test be performed after each simulation if the dynamic properties at that state are of interest.
Figure 10-16 Model house 4A: MAC trends of Modes 1 – 5 for simulations S1 (S10%) and S2 (S25%), with respect to the MAC at strengthening stage ‘c’.

Figure 10-17 Model house 4A: MAC trends of Mode 1 to Mode 4 for shake table test stages S1 (S10%) and S2 (S25%), with respect to the MAC at S1 (S10%).
10.4 Summary

EMTA was successfully conducted both prior to and during the shake table testing for model house 4A. The following processes and outcomes were noted:

- EMTA using an impact hammer to excite the model house to assess the influence of impact location and penetrations. Results revealed that the impact location affects the form of the FRF graphs, but not the mode shapes and corresponding natural frequencies. Penetrations, however, were seen to have a significant effect on the mode shapes. This is especially the case for door openings in shear walls.

- EMTA using an impact hammer to excite the model house at key stages during the retrofit-strengthening process. Results revealed that the addition of a ring beam with midspan cross-beam changed the dominant second mode from the ‘first breathing mode’ to the ‘first torsion mode’. The natural frequencies (global stiffness) increased only slightly during the strengthening process.

- EMTA using an impact hammer to excite the model house prior to testing to determine the first fundamental frequency (24.1 Hz), which was then used to appropriately scale (with respect to time) the shake table input excitation to ensure dynamic similitude and induce damaging near-resonance conditions (described in detail in Chapters 5 and 9).

- EMTA using the shake table to excite the model house to assess the changes in dynamic characteristics during the series of shake table simulations. Clear results were obtained for the low intensity simulations (S10% and S25%). These results indicated that torsion was the dominant mode, most probably due to the asymmetry introduced by the penetrations. These mode shapes closely matched the flexure results presented in Chapter 9. The ‘messy’ results for the higher intensity simulations confirmed the complexity of structural responses when the specimen was subjected to dynamic shake table excitation.

EMTA is a useful tool to reflect the physical response of the structure, and evaluate the differences and changes in dynamic characteristics of adobe-mudbrick structures. Results provide greater insight into the structural behaviour than mere observations, and may be a practical tool for damage detection and condition monitoring of adobe structures.
11 WORLD ADOBE FORUM WEBSITE

11.1 Preamble

As an additional component of this PhD research project, the author has been responsible for the development of the World Adobe Forum, a website dedicated to the sharing of information about safer adobe. This chapter is a modified version of the concept paper / project launch presented at the SismoAdobe2005 conference in Peru (Dowling, 2005).

11.2 Introduction

The World Adobe Forum website is an exciting new medium for the sharing of information related to adobe mudbrick research and application, focusing on understanding and reducing seismic vulnerability. The web-based forum is being supported by the Earthquake Engineering Research Institute (EERI), the International Association for Earthquake Engineering (IAEE), the EERI-IAEE World Housing Encyclopaedia and a multi-disciplinary panel of experts and advocates from around the world.

At the time of writing this thesis (early 2006) a professional web developer had been contracted and the first reports were being prepared. The World Adobe Forum is expected to come on-line in late 2006. In order to gain a better understanding of the World Adobe Forum the reader is invited to view the preliminary/draft website, which is available on-line (follow the links to the World Adobe Forum from: www.eng.uts.edu.au/~ddowling).

The forum will include submissions from all areas related to adobe within a seismic context, including experimental testing and analysis, field research, and application/implementation. Contributions will encompass social and technical, academic and practical aspects of adobe, and focus on the processes and outcomes (inclusive of successes, lessons learned, problems and solutions). Submissions will be
initially welcomed in English and Spanish and will be peer-reviewed prior to posting on the website.

This chapter presents the concept and framework of the World Adobe Forum and describes the background, objectives, stakeholders, means and scope of the project. Sample templates are presented for three key subject areas: experimental testing/analysis; field research; and application/implementation (Table 11-1 to Table 11-3).

### 11.3 Background

Adobe houses have been the predominant form of low-cost housing in many parts of the world for many centuries. The vulnerability of traditional adobe housing to seismic forces is widely known; each decade, hundreds of thousands of adobe houses are damaged or destroyed in severe earthquakes around the world. Despite the necessity to undertake valuable research to reduce this vulnerability, the majority of global research resources are allocated to consider the performance of so-called 'modern' materials (e.g. steel, concrete, composites) and advanced systems (e.g. passive and active control). This resource allocation imbalance adds further impetus to the call to more effectively share information and act collaboratively in the quest to reduce the vulnerability of non-engineered construction.

In spite of this imbalance a variety of valuable research has been and is being conducted around the world, focusing on different methods to improve the seismic resistance of adobe structures (as presented in Chapter 3 and throughout this thesis). This research includes the testing and analysis of soils, bricks, prisms, walls and houses. Furthermore, a host of implementation projects have concentrated on promotion, training and construction of improved adobe houses, as well as the strengthening and repair of existing dwellings. Various adobe advocates have recognised the scattered and disjointed nature of this information, and this is seen as a major obstacle to the widespread dissemination of important findings and the further advancement in research and application. The World Adobe Forum endeavours to address this situation.
A recent opinion paper (Comartin et al, 2004) issued a challenge to earthquake engineering professionals to assume responsibility and take action to reduce the earthquake vulnerability in developing countries. The opinion paper identifies the problem (including the large disparity between losses in developed and developing countries), describes current efforts (including local initiatives and international networks) and proposes some future possibilities (including enhanced international collaboration, information exchange and recognition). The World Adobe Forum supports these future possibilities, whilst building upon and encouraging existing efforts.

Similar issues were raised in a report prepared by an ad-hoc committee of the International Association for Earthquake Engineering (Jain et al, 2004) which recommended a variety of initiatives for improving earthquake engineering practice worldwide, with particular reference to developing countries. The World Adobe Forum fits appropriately with the recommended actions, including raising global awareness towards earthquake mitigation issues, development of human resources, and dissemination of information.

The concept of the World Adobe Forum has been discussed and keenly welcomed at recent international conferences in San Francisco (8th National Conference on Earthquake Engineering, 17-21 April 2006), Los Angeles (Getty Seismic Adobe Colloquium, 10-13 April 2006), Peru (SismoAdobe2005, 16-19 May 2005) and Mexico (EERI Annual Meeting, 2-6 February 2005).

11.4 Objectives and benefits

The main objective of the forum is to provide a medium for the sharing of information related to improved adobe research and application. As this information becomes more widely available, a number of direct benefits are envisaged, including:

- Increased exposure and profile for adobe research and application.
- Improved exchange of ideas, questions, concerns, problems and solutions between the different stakeholders (beneficiaries, practitioners, supporters, researchers, etc).
• Greater consistency of testing, and therefore greater confidence and compatibility of results.
• Increased opportunity to learn lessons about which testing and construction methods are practical and effective, and which methods require further research and improvement.
• Enhanced capacity to more effectively identify future research needs and the resources required to respond to these needs.
• A foundation for the preparation of updated construction guidelines for safer adobe construction.

11.5 Scope

It is intended that the forum accept submissions in all areas related to adobe, including (but not limited to):

• Experimental testing and analysis (soils, bricks, prisms, wall units and houses; static, quasi-static and dynamic);
• Field research (reconnaissance, damage patterns and statistics);
• Application/implementation (promotion, training, construction, strengthening and repair).

Contributions will encompass social and technical, academic and practical aspects of adobe, and focus on the processes (the 'how'), not just the outcomes. A core component of each submission will be a discussion of the complexity, cost and seismic capacity of each system, which will be based on existing or newly developed rating classifications. Emphasis will be placed on objective accounts of activities, inclusive of successes, lessons learned, problems and solutions. The forum will provide a network linking proponents representing all aspects of adobe. This link is necessary to more effectively identify and address the challenges which exist and make positive advances.
11.6  Means and process

The forum will be a web-based interface, coupled with the existing EERI – IAEE World Housing Encyclopaedia (www.world-housing.net/). Authors will download a report template (see Table 11-1 to Table 11-3 for sample templates) which will provide a framework for report content and ensure consistency of format and presentation of key information. Authors will complete the relevant sections and submit their reports for review. Reports may range in length from 2-3 pages up to 10-15 pages, and authors will be invited to provide links to other related publications and resources. Submissions will be peer-reviewed and then posted on the website. Reports will be searchable and sortable. Contributions will be sought from around the world, with submissions initially welcome in both English and Spanish. (Other languages will be considered if demand and support are available.) As interest and involvement in the forum increases an online discussion board will be established to facilitate discourse related to submissions on the website, as well as other associated matters.

11.7  Stakeholders / participants

The World Adobe Forum is being designed to support the needs and contributions of a variety of stakeholders. Visitors to the site will be able to access practical, up-to-date information, as well as provide valuable feedback based on their own experiences. The forum will provide a direct link between different stakeholder groups (e.g. beneficiaries, practitioners, supporters, donors, researchers, etc) such that practical solutions can be effectively implemented and future research needs can be discussed and addressed. Stakeholders involved in implementation projects (e.g. home builders, non-governmental organisations, government agencies) will be largely interested in complete, practical and applicable solutions. Non-governmental organisations, government agencies and funding donors will be interested in the technical approach and outcomes which are a necessary component of assessing, justifying and endorsing a project. Researchers will be interested in promoting their research findings, as well as receiving feedback on the practical aspects of implementation and identifying future research needs. The World Adobe Forum is designed to address these interests and needs.
Support for the project is being provided by the Earthquake Engineering Research Institute (EERI), the International Association for Earthquake Engineering (IAEE) and the EERI-IAEE World Housing Encyclopaedia. Additional individual, organisational and financial support is being sought. A multi-disciplinary forum panel has been formed and is made up of experts and advocates from around the world. The role of the forum panel is to co-ordinate the development and operation of the World Adobe Forum, including defining the forum parameters and format, promoting the project, and reviewing and editing submissions.

11.8 Outcomes

It is envisaged that the World Adobe Forum will become the premier global adobe information hub, covering the full spectrum of adobe-related information from around the world. Interested stakeholders will be able to access up-to-date, relevant information, follow links to obtain further details about specific areas of interest, and engage in meaningful discussion with others. It is hoped that via the World Adobe Forum this valuable information will be more effectively transferred to application in the field, making inroads in the challenge to reduce the vulnerability of adobe houses in high risk areas around the world.

11.9 Summary

With support from EERI, IAEE and others, the concept of a web-based World Adobe Forum has been developed in some detail. A preliminary/draft website has been established, with templates for contributions from researchers, practitioners, etc. A multi-disciplinary panel has been formed to coordinate establishment and ongoing maintenance of the World Adobe Forum.

The World Adobe Forum is being planned as the premier global hub for information on adobe as a building material for residences in areas prone to seismic activity. The concept, scope, objectives, and progress to date were outlined in this chapter.
### EXPERIMENTAL TESTING / ANALYSIS REPORT TEMPLATE

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</thead>
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<tr>
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<td>Posted:</td>
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<tr>
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</tr>
<tr>
<td>(Options: experimental testing / numerical analysis / static / quasi-static / dynamic)</td>
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</tr>
<tr>
<td>Author/s</td>
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<tr>
<td>Affiliation</td>
<td></td>
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<tr>
<td>Contact</td>
<td></td>
</tr>
</tbody>
</table>

### Summary / abstract

### Keywords

### Aims
- What are / were the aims / goals / purposes of the research?

### Equipment / specimens
- **Equipment** (description, location).
- **Specimens** (quantity, properties, dimensions, boundary conditions, improvement systems, natural frequency)

### Method
- **Procedure**
- **Loading** (static / dynamic: loading rate, boundary conditions, input spectra, scaling factor, including justification).

### Results
Include discussion (with respect to both testing and potential implementation) of:
- Seismic improvement / performance / vulnerability (based on an appropriate rating system).
- Complexity (based on an appropriate complexity rating system).
- Cost (based on an appropriate cost rating system).

### Conclusions
- Lessons learned.
- Outcomes for application / implementation (least and most appealing aspects).
- Outcomes for further research.

### References

### Links
- Relevant publications, websites, other World Adobe Forum reports.
### Table 11-2 World Adobe Forum: sample template for field research.

**FIELD RESEARCH REPORT TEMPLATE**

<table>
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<td>Contact</td>
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</tr>
</tbody>
</table>

**Summary / abstract**

**Keywords**

**Location**
- Specific location (community / village / town, region, country).

**Stakeholders / Collaborators**
- Who was involved in the research (beneficiaries, participants, supporters, donors)?

**Aims**
- What are / were the aims / goals / purposes of the research?

**Process / method / approach**

**Research findings / results**
Include discussion of:
- Seismic improvement / performance / vulnerability (based on an appropriate rating system).
- Complexity (based on an appropriate complexity rating system).
- Cost (based on an appropriate cost rating system).

**Outcomes / Conclusions**
- Lessons learned.
- Outcomes for further application / implementation (least and most appealing aspects).
- Outcomes for further research.

**References**

**Links**
- Relevant publications, websites, other World Adobe Forum reports.
### APPLICATION / IMPLEMENTATION REPORT TEMPLATE

- **Title**
- **Report No.**
- **Focus Country**
- **Action** (Options: promotion / training / construction / strengthening / repair)
- **Author/s**
- **Affiliation**
- **Contact**
- **Key dates**
- **Submitted:**
- **Posted:**
- **Updated:**

#### Summary / abstract

#### Keywords

#### Location
- Specific location (community / village / town, region, country).

#### Stakeholders / Collaborators
- Who was involved in the project (beneficiaries, participants, supporters, donors)?

#### Aims
- What are / were the aims / goals / purposes of the project?

#### Resources
- Including personnel, materials, tools, time, money.

#### Process / method / approach
Include:
- Design / model.
- Seismic improvement / performance / vulnerability (based on an appropriate rating system).
- Complexity (based on an appropriate complexity rating system).
- Cost (based on an appropriate cost rating system).

#### Outcomes / Conclusions
- Lessons learned (ie. If you were going to do this project again what would you do [a] the same and [b] different?)
- Outcomes for further application / implementation (least and most appealing aspects.)
- Outcomes for further research.

#### References

#### Links
- Relevant publications, websites, other World Adobe Forum reports.
12 IMPLEMENTATION STRATEGY

12.1 Introduction

Implementation is where the rubber hits the road (or the bamboo hits the mud, in this case!)

This chapter presents the framework for a potential implementation program. The main objective of the proposed implementation strategy is to enhance the desire, capacity and confidence of local homeowners and builders to retrofit and/or construct safer adobe houses.

The following phases are presented:

- Location criteria for implementation program.
- Context assessment and feasibility study.
- Program planning.
- Promotion and awareness building.
- Simple brochure.
- Retrofit training.
- Extended support, mentoring and monitoring.
- Apprenticeships, accreditation and job opportunities.
- Design and construction of a new community building.
- Resource requirements.

For each phase the objectives, means and process, and resource requirements are discussed.

In spite of the extensive seismic adobe research and the development of numerous improved adobe construction guidelines and manuals since the 1970s (see Chapter 3), there are no clear examples of widespread and sustained implementation of these findings. This is because most of the proposed systems are too complicated and/or too costly to be used without ongoing external support. The implementation strategy proposed in this chapter has been developed by the author to address this issue. The suggestions in this chapter are based largely on the author’s experiences in Latin
America, and especially in El Salvador (as described in Chapter 2). It is imagined that many of the activities and processes described in this chapter could be applied in other parts of the world, although modifications to appropriately match local conditions will be required.

The strategy presented in this chapter incorporates many of the recommendations for reducing the vulnerability of non-engineered buildings proposed by Meli and Alcocer (2004). Their recommendations include:

- Encourage the participation of building owners and develop specific training programs for them;
- Promote the use of structural solutions akin to the local practice;
- Encourage the participation of local universities and professional associations;
- Take advantage of periods of hyperreceptivity immediately following severe earthquakes;
- Develop small-scale pilot studies;
- Disseminate the successes attained in similar areas and conditions;
- Implement economic incentives (suitable materials at reduced prices and temporary employment for masons); and
- Link vulnerability-reduction tasks to other actions aimed at improving housing habitability and durability, and at reducing poverty.

This chapter focuses on community-based implementation activities. It assumes that an appropriate level of funding has been attained and the implementation program is coordinated by an established local entity (non-governmental organisation and/or government agency) in collaboration with local communities.

12.2 Location criteria for implementation program

In reconstruction activities in the aftermath of an earthquake disaster, the focus of most international relief and development organisations and donors is the delivery of housing units and other essential services as quickly as possible. In this case, there is a strong tendency towards the use of 'modern' materials (e.g. concrete, fired brick, steel, plastics). Whilst these activities are an important part of restoring the dignity and
livelihoods of selected beneficiaries, in almost all cases there are simply not enough resources to provide the same level of support to all affected people. As a result, the most marginalised groups (due to poverty, landlessness, isolation, or other factors) are often left without any form of support. These are the populations who rebuild in the same traditional manner, and whose houses are the most vulnerable to future earthquakes. These are the communities who would be the most appropriate participants in a training program.

Although the post-earthquake reconstruction phase presents a good opportunity for the introduction of safer adobe construction techniques, there are a number of challenges associated with working in such a context. These relate mainly to the frenzied aid activity and competition with modern materials and provision of ‘free’ houses. Such competition would make it very difficult to introduce a skills building program requiring community participation for little immediate, modern and tangible benefit.

More appropriate locations for such an implementation program include:

- A region close to a severely affected area, where the type of construction and seismic hazard are similar, and the awareness of the risk is heightened. Examples of this include parts of Iran near Bam (December 26, 2003) and the countries in close proximity to earthquake-affected Pakistan (October 8, 2005), including Afghanistan, India, Nepal and Bangladesh.

- A region which suffered extensive destruction in a relatively recent earthquake, where designated earthquake reconstruction funding has been exhausted and unsupported communities are rebuilding in the same unsafe manner. An example is El Salvador four to five years after the devastating earthquakes of early 2001 (discussed in Chapter 2).

- A region which experienced a recent moderate earthquake that failed to attract significant international attention, but reminded residents of the earthquake risk and vulnerability of traditional adobe houses. Examples include recent earthquakes in Mexico, Peru and Iran.
12.3 Context assessment and feasibility study

Objective
Assess the context and feasibility of a project in a selected region.

Means and process
A scoping study, including:

- Identification of potential participant communities.
- Identification of key players in each local community. Ross-Jordan (2004) suggests that for the success of community-development initiatives “mobilising, training and monitoring community volunteers is the key.”
- Assessment of the attributes (strengths and weaknesses) of each community and the external factors (opportunities and threats) which may influence the delivery of an implementation project. This will include a needs assessment, and consideration of the social, economic and organisational capacity of the community (collectively and as discrete families). These aspects should be monitored for the duration of the project.

12.4 Program planning

Objective
To define the objectives, scope, means, process and resources of the program.

Means and process
Once the feasibility of a training program in a selected community has been determined, specific planning can commence. This involves working with the key local community representatives to define the program parameters and activities.
12.5 Promotion and awareness building

Objective

Raise the awareness of earthquake hazards, the vulnerability of traditional adobe houses, and the existence of simple and affordable techniques to reduce this vulnerability.

Means and process

Promotion activities should be tailored to the interests, needs and capacities of each community (as identified in the previous phases). The core component will be interactive and dynamic presentations and workshops, covering:

- The danger of earthquakes.
- The vulnerability of traditional adobe houses.
- The advantages and disadvantages of adobe housing.
- Debunking some common myths about adobe housing.
- Introduction of some simple improvement concepts.
- The importance of maintenance.

Some potential means include:

- A small theatre or puppet show.
- Video footage from earthquakes and research.
- Simple demonstrations (using models made of cardboard, timber, adobe).
- Complex demonstrations [e.g. construction and ‘testing’ of model houses on a rudimentary shake table, a technique which has been very successful in parts of India and Nepal (Upadhyay, 2004)].
- Distribution of a simple two-page brochure describing the steps to retrofit-strengthen an adobe house (Section 12.6 below).

Dudley (1993) identifies three factors which are critical to the success of a community development project:

- Does it make sense? Is the idea reasonable in terms of the intended beneficiary’s own rationale?
• **What is it?** Can the idea even be *recognised* – does it have a name and are its limits clearly defined?

• **Is it worthy of me?** Is the idea *respectable* – is it something which ‘people like us’ do?

These factors and the accompanying eight principles (see Dudley, 1993) should be considered in the development of the whole program, and especially in the delivery of the promotion phase.

Effective promotion increases the awareness of earthquake hazards and the interest in appropriate improvement systems. These factors are prerequisites for obtaining local support for any skills training, community construction projects.

Promotion activities are also an opportunity to further gauge the interest of the community in being involved in a training program (discussed below).

### 12.6 Simple brochure

**Objective**

A simple two-page brochure provides home owners with an easy reference guide which outlines the simple steps to strengthen their home. It is designed to be used in conjunction with a training program (described in Section 12.7 below) and serves as a reminder of the key stages in the retrofit-strengthening process.

A sample brochure is presented on the following pages. Naturally, the text would be translated to the appropriate language of the region. The sketches should also be modified to match local construction practices and to ensure understanding at the local level.

The brochure avoids prescriptive requirements for the quality and material properties of bricks and bamboo. This is because the focus of the brochure is to introduce an improved, holistic structural reinforcement system. Discussion of quality control should be included in the detailed training.
HOW TO MAKE YOUR HOUSE EARTHQUAKE SAFE

INSPECT YOUR HOUSE!

MAKE A PLAN FOR ACTION

MARK + DRILL HOLES THROUGH THE WALL

MINIMUM: 60CM

(YOU CAN USE AN ELECTRIC OR HAND DRILL OR MAKE A ‘DRILL’ OUT OF STEEL BAR)

DRILL HOLES AT THE TOP, MIDDLE + BOTTOM OF THE WALL, AND AROUND WINDOWS + DOORS
INSERT STRING

WET HOLES + FILL WITH MUD

TIE BAMBOO AGAINST THE WALL (AFTER THE MUD IS DRY)

YOU CAN ALSO USE CANE OR TIMBER POLES

CONNECT RING BEAM + BAMBOO WITH WIRE

USE GALVANISED WIRE (2MM MINIMUM)

ATTACH WIRE BETWEEN BAMBOO POLES (HORIZONTAL AT TOP, MIDDLE + BOTTOM OF HOUSE)

YOU CAN COVER THE BAMBOO + WIRE WITH MUD/LIME RENDER

THE WIRE MUST BE TIGHT AROUND THE HOUSE – LIKE A NET. USE PLIERS TO TIGHTEN THE WIRE.

EVERY 6 MONTHS CHECK THE CONDITION OF THE BAMBOO + WIRE... IF IT IS DAMAGED YOU NEED TO REPLACE IT...
12.7 Retrofit training

Objective

Develop the capacity and confidence of local homeowners and builders to retrofit-strengthen traditional adobe houses.

Means and process

A hands-on, interactive training program, with the foundation tenet: 'learn by doing'. A simple manual should accompany the training program. The training manual should be more detailed than the simple two-page brochure presented above. The following activities are proposed, with sample timeframes for each stage indicated in square brackets:

1) 'Why is this important?' Repeat of promotion and awareness raising workshop (as described above) in order to reiterate the danger of earthquakes, the vulnerability of traditional adobe houses, and the advantages and disadvantages of adobe housing, as well as introduce some simple improvement concepts. No technical details are given at this stage. The project scope and stages are outlined, and confirmation of the community's commitment to the project sought. [Half-day]

2) 'Let's look at our houses.' Working in the local community, the training group visit a number of existing adobe houses and objectively observe common features of each house (e.g. foundations, bricks, mortar joints, walls, openings, wall-roof frame connection, roof, floor, wall render, maintenance, drainage, etc). Judgments are avoided at this stage. [2 x half-days]

3) 'Why do houses fall down in earthquakes?' A simple 'classroom'\(^1\) workshop, including discussion of earthquake action, why do houses fall down in earthquakes, common deficiencies in houses and how these deficiencies affect seismic performance. The workshop will include a number of simple activities and demonstrations to reinforce the theory and ideas presented. [Half-day]

---

\(^1\) In this chapter, the term 'classroom' refers to a dedicated learning space, which could be a commonly used community meeting space (e.g. local community building, school, the village square, or simply in the shade of a large tree.)
4) 'Let's look at our houses again.' Revisit houses in the local community to undertake a more subjective review of common features, including identification of common deficiencies discussed in the previous workshop. The general vulnerability of each house will be assessed. This should lead to an awareness and acknowledgment that many local traditional adobe houses are very unsafe. [2 x half-days]

5) 'How do we make our houses safer?' Back to the 'classroom' to introduce a number of reinforcement systems, including the 'how does it work', plus the advantages, disadvantages, costs, complexity, and seismic capacity of each system. A number of common myths will be addressed (e.g. adding cement to anything will make it better). Practical demonstrations, combined with photographs and video footage will be used to convey the message. [Half-day]

6) 'Let's look at our houses yet again.' Revisit local houses to look at how some of the reinforcement systems may or may not be appropriate. This includes an acknowledgment that each house is unique, and may require a different solution. The participants are encouraged to undertake a process of observation, identification of deficiencies, and consideration of the appropriateness of different interventions. A retrofit plan will be developed for a number of local houses. (Depending on the literacy levels of the participants, the vulnerability assessment and retrofit plan may be verbal and/or written.) The retrofit plan will include identification of the steps involved and the resources required and available (materials, tools, labour, time, etc). (At this stage, the houses selected for retrofitting may be by allocated by ballot, or according to needs, or in consideration of those participants who have shown the most enthusiasm for and interest in the program. This selection process will be developed in collaboration with the local community to ensure transparency and fairness). [2-3 x half-days]

7) 'Let's retrofit some houses together.' Armed with the retrofit plan, the training groups work together to retrofit a number of houses in the community. The emphasis will be on 'learning by doing', with the opportunity to observe and participate in each step of the process. For each step the rationale and process will be explained, action taken and results reviewed. Any obstacles, challenges and potential solutions will be discussed. [As required, ~one week]
8) 'Let’s develop a plan to retrofit some more houses.' Participants are divided into small working groups (4-5 members) and tasked with assessing the vulnerability of other houses in the community (perhaps their own homes). They will develop a retrofit plan for each house. The small working groups will present these assessments and retrofit plans to the rest of the group for 'peer review', discussion and amendment, as required. [2-3 x half-days]

9) 'Let’s retrofit those houses.' The small working groups then enact the retrofit plans, with support available from other groups and the trainers. At each key stage a review of progress and future action will be undertaken, involving the small working group and the trainers. Self-review will be encouraged, and constructive feedback provided by the trainers. At the end of each day the working groups will come together to discuss progress, obstacles and lessons learned. Any common problems will be addressed through further group training. [As required, ~1-2 weeks]

10) 'What about new houses?' The training group will be introduced to a number of techniques applicable to the construction of new adobe houses. These include soil and site selection, brick making, house design, foundations, brick laying, reinforcement, and roof construction. The community will also be introduced to the possibility of a more comprehensive training program, including the construction of a community facility, as described in Section 12.10 below. [As required, ~1 week]

11) 'How do we maintain our homes?' The training group will be exposed to the importance of proper maintenance. The effects of inadequate maintenance will be highlighted, and techniques for regular maintenance will be presented. Where available, local houses requiring maintenance will be assessed and worked on by the group. [2 x half days]

12) Graduation and 'hand over'. Participants who have successfully completed the training program will be acknowledged at a 'graduation ceremony', where certificates will be issued and the program reviewed. The co-ordinating entity (NGO or government body) then 'hand over' the 'responsibility' of safer adobe houses to the community. The co-ordinating entity reconfirm their commitment to ongoing mentoring and support (described in greater detail below). [Half-day]
13) **Community action.** It is anticipated that the community then continue to retrofit-strengthen houses in the local area, under their own direction and means. [Ongoing…]

**Resources**

The training program should be co-ordinated by experienced trainers, who have technical and teaching proficiency. In addition to the simple two-page retrofit brochure issued during the promotion phase (Section 12.6), each participant will be supplied with a training manual / workbook which details each step of the process. These manuals will be developed with consideration of the literacy levels and cultural norms of the participant communities. ['Communicating Building for Safety' (Dudley and Haaland, 1993) is an excellent resource which presents some guidelines for effectively communicating technical information to local builders and householders.]

One aspect to be considered further is funding of resources. It is expected that the co-ordinating entity (NGO or government body) will cover the costs of the trainers and training resources. What is not determined is whether the community members pay for the tools and materials for the retrofit-strengthening of their homes, or whether these are covered, or subsidised, by the co-ordinating entity. The advantage of the co-ordinating entity providing the tools and materials is that progress will not be impeded by the potential delayed procurement of these materials by the participants. However, if the co-ordinating entity provides the tools and materials outright, then this contributes to the aid dependency which the program is designed to avoid! A balance could be the establishment of a ‘tool bank’ (for the loan or hire of tools) and provision of subsidised materials (which could be purchased by the participants using cash or through a simple micro-credit scheme). Another possibility is the introduction of ‘adobe credits’, which could be given to community members who participate in each stage of the training program. These ‘adobe credits’ could then be traded for tools and materials.

**Other considerations / ideas**

Depending on the interest and other commitments of the community participants a number of training groups may be formed. Training programs can be tailored to allow participants to maintain their existing work commitments. This may involve running
training segments in parallel (e.g. morning and afternoon sessions) and/or in series (consecutive programs).

### 12.8 Extended support, mentoring and monitoring

**Objectives**

- Ensure continuity of community-driven action (through periodic support and mentoring).
- Assess the effectiveness of the training program and learn lessons (monitoring).

**Means and process**

The co-ordinating entity periodically re-visits the community to troubleshoot problems, respond to questions, and encourage and support ongoing action. An appropriate timeframe for visits will be determined in consultation with the community. Immediately after completion of the training program, visits may be organised on a weekly or fortnightly basis. Over time, this may become a monthly event, and eventually a re-visit once or twice a year may be all that is necessary. Refresher training workshops should be provided as required. Ideally, the original trainers would perform the refresher training and re-visits, to maintain consistency and rapport with the local community. If a micro-credit scheme was introduced in the initial training program (to assist in materials procurement), it may continue in the post-training phase.

Additional mentoring is discussed further in the following section.

At the outset of the program a number of progress and performance indicators should be developed. These indicators will be monitored for the duration of the training program and at periodic intervals thereafter. Ross-Jordan (2004) suggests that community ownership and 'replication ability' are key indicators of sustainability. Lessons learned from the monitoring process will be used to modify subsequent projects, and be useful to determine the level of refresher training required in each community.
12.9 Apprenticeships, accreditation and job opportunities

Objectives

- Provide further opportunities for competent participants.
- Build a team of capable and competent trainers.

Means and process

Participants who demonstrate exceptional learning and skills development, and a strong aptitude for teaching, will be considered for ongoing training. This could involve an apprenticeship, whereby the participant works alongside an experienced trainer in the next community as part of a ‘train the trainer’ program. Mentorship, training and a stipend will be provided during this time. In due course, the theoretical and practical competency of the apprentice should be evaluated, and if certain minimum levels are attained the apprentice will be accredited. Accreditation will hopefully open the door to employment opportunities, in both community-based construction and larger-scale, agency-supported adobe initiatives.

12.10 Design and construction of a new community building

Objectives

- Support further community development.
- Develop the capacity and confidence of local builders to construct safer adobe buildings.

Means and process

As an additional component, communities may have an opportunity to apply for support for the construction of a community building (e.g. community hall, clinic, cultural centre, child-care centre). The project would be a collaborative venture between the local community and the co-ordinating entity. The community will be supported to prepare a proposal, which will include:

- Project description and justification.
- Project plan, timeline and resource requirements.
- Community contribution to the project (e.g. labour, materials, tools).
• Requested support from the co-ordinating entity (e.g. supervision, materials, tools, training resources).

Applications will be assessed according to a number of factors, including:

• The motivation, commitment and organisation of the community (as demonstrated through the retrofit training program and the preparation of the project proposal).
• The need for the project.
• The capacity of the community to contribute to the project, as proposed.
• The capacity of the co-ordinating entity to provide the required support.

Unsuccessful applications should be discussed with the community.

For successful applications, a project contract will be established between the community and the co-ordinating entity. The construction of a new community facility is an opportunity to train the participants in a variety of improved adobe techniques. Training will take the form of ‘classroom’ workshops combined with hands-on application. All aspects of constructing a new building will be presented, including planning, construction and maintenance. Special attention will be given to soil and site selection, brick making, house layout and design, foundations, brick laying, reinforcement, and roof construction.

12.11 Resource requirements

The key resource requirements for an implementation program such as that described in this chapter include:

• Experienced and trained social promoters / community development officers, who will work with the communities at each stage of the program.
• Competent technical builders and trainers, who will carry out the training, support the retrofit-strengthening activities, supervise any community construction project, and undertake the periodic re-visits and refresher training.
• Demonstration aids, props, video footage, and projector, etc, for promotion and training activities.
• A training manual, to be used in conjunction with the training program.
• Tools and materials (provided by the co-ordinating entity and the community).

12.12 Summary

This chapter presented a sample implementation program, which incorporated the following elements:

• Location criteria for implementation program.
• Context assessment and feasibility study.
• Program planning.
• Promotion and awareness building.
• Simple brochure.
• Retrofit training.
• Extended support, mentoring and monitoring.
• Apprenticeships, accreditation and job opportunities.
• Design and construction of a new community building.
• Resource requirements.

Any implementation program should acknowledge the capacity of the community participants to learn, innovate, adapt and adopt proposed reinforcement systems. The emphasis, therefore, should be on providing the relevant ‘tools’ (awareness, knowledge, confidence and skills) to allow participants to take appropriate action given their context and resource limitations.

Invariably, execution of the implementation activities will lead to the identification of further research needs, from both technical and social perspectives. Some further research needs are discussed in greater detail in Chapter 13.

Implementation program outcomes, lessons learned and further research needs should be communicated through publications in relevant conferences and journals and the World Adobe Forum website (described in Chapter 11).
13 FURTHER RESEARCH

13.1 Introduction

The reinforcement system developed as part of this PhD research is considered to be ready for implementation, as described in Chapter 12. It is expected that the implementation process will produce additional research questions. In any case, further research and development is required to enhance the current state of technical knowledge, improve the profile and acceptance of adobe, and understand some of the challenges and opportunities for widespread implementation of improved adobe. It is important that technical and social research activities be closely integrated, such that appropriate solutions are developed and implemented. A number of further research needs have been identified. These relate to both technical and practical areas, and include:

- Post-earthquake reconnaissance.
- Experimental testing.
- Numerical modelling.
- Parametric studies.
- Implementation and application.

This chapter discusses methodologies and scope for research in each of these areas. Particular attention is given to further research to develop a reliable numerical model, using finite elements, to represent the pre- and post-cracked behaviour of adobe structures. Some detailed background and a potential approach to conduct such modelling are proposed in this chapter. This research would rely on a combination of extensive experimental testing and numerical modelling, as detailed below.
13.2 **Post-earthquake reconnaissance**

There is particular value in evaluating the damages to adobe structures in the aftermath of a significant seismic event. The scope and value of reconnaissance investigations have been enhanced by recent technological advances in portable data collection devices, digital cameras, Global Positioning Systems (GPS), and Geographic Information System (GIS) mapping, and the creation of internet-based 'virtual clearinghouses'\(^1\). These resources have been used to effectively gather and disseminate detailed post-earthquake reconnaissance reports following recent major earthquakes in Iran, Pakistan and Sumatra.

Customised templates and protocols for reconnaissance focused on adobe structures could be developed based on the large number of existing reconnaissance procedures. These include procedures developed by Applied Technology Council (ATC), Consortium of Universities for Research in Earthquake Engineering (CUREE), Earthquake Engineering Research Institute (EERI), Federal Emergency Management Agency (FEMA), National Information Service for Earthquake Engineering (NISEE), and U.S. Geological Survey (USGS), as reported by Porter (2002).

Of particular interest would be a detailed database incorporating:

- Building features (age, location, orientation, configuration, dimensions, quality of construction and maintenance, reinforcing systems, etc).
- Building performance (cracking patterns, failure modes, structural deficiencies, etc).

Surveys should be undertaken for both affected and unaffected buildings and should include photographs of specific elements. This information would allow a greater understanding of the mechanisms of failure (in-plane, out-of-plane, and oblique-angle response of components and connections) and the role which different features and systems play in seismic performance.

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\(^1\) "The goal of the virtual clearinghouse is to facilitate information dissemination after major earthquakes. The clearinghouse is meant to be a centralized location for researchers of all disciplines to exchange data after particular earthquakes." (EERI) [http://www.eeri.org/ife/clearinghouse/](http://www.eeri.org/ife/clearinghouse/)
13.3  Experimental testing and numerical modelling

13.3.1  Introduction

The aim of this proposed research is to develop a robust nonlinear numerical model to accurately represent the pre- and post-cracked behaviour of adobe structures subjected to dynamic excitation.

Three key research activities have been identified:

- Extensive experimental testing, coupled with advanced Experimental Modal Testing and Analysis (EMTA) techniques to provide multi-stage updating of the dynamic characteristics of each test specimen at different damage states.

- Development of a reliable numerical model through nonlinear heterogeneous Finite Element (FE) modelling. The model should be developed and validated using the results from the experimental testing.

- Extensive parametric studies (using the validated FE model) to assess a broad range of design and construction variables and improvement systems, without the need for resource-intensive physical testing. These studies provide an opportunity to update current guidelines for the safe design, retrofit-strengthening and construction of adobe structures.

These activities are discussed in greater detail below.

13.3.2  Background

To date, adobe research around the world has tended to focus on destructive physical testing of specimens, as described in Chapter 3, and undertaken in this thesis. These activities are highly resource intensive (time, materials, equipment, labour) and do not adequately assess the broad spectrum of design and construction variables (e.g. wall height, width, length ratios; size and location of door and window openings; configuration of reinforcement; material variability; connection details, etc).

The limited numerical modelling of adobe structures has focused on the elastic pre-cracked behaviour. The initial, valuable contributions in this area were made at UNAM,
Mexico (Bazán et al., 1980) and Stanford University (Tolles and Krawinkler, 1990) using extensive experimental testing (described in Chapter 3) to develop and validate simple FE models. Since then significant advances in computing power and modelling software and techniques provide an opportunity to build on this initial research and develop more robust and reliable numerical models. More recently, Yamin et al. (2003) undertook some FE modelling of the pre-cracked behaviour of their 1:5 scale model houses (presented in Chapter 3). They reported good correlation between the FE models and experimental test results, although this appears to be based on qualitative data (observations) rather than detailed quantitative results (obtained through extensive instrumentation and EMTA). Cao and Watanabe (2004) developed an FE model using viscoelastic joint elements (described below) to “model the opening and closing behaviours of the interfaces between the bricks.” The model, however, has not been validated due to a lack of experimental data.

Despite the limited published research relating to the numerical modelling of adobe structures, valuable research focused on conventional fired-brick masonry provides a useful foundation for the development of numerical models for adobe structures (e.g. Gambarotta and Lagomarsino, 1996 & 1997a&b; Liu, 1997; Riddington and Jukes, 1998; Aoki, 2001; Han and Masia, 2004). Modelling strategies for masonry structures generally fall into two categories, known as homogeneous (macro) or heterogeneous (micro) modelling. The homogeneous (macro) method uses homogenisation techniques with no particular attention given to the position of mortar layers. It models the masonry structure as an anisotropic composite and smears the bricks, mortar and brick-mortar interface into an average continuum. An example of homogeneous modelling is the work of Riddington and Jukes (1998) in which a masonry structure was treated as a composite material and a two-stage homogenisation technique was used to model its structural response. By adopting this approach the computational effort necessary for standard nonlinear finite element (FE) analysis of large three dimensional masonry structures is considerably reduced and an accurate analysis becomes achievable using a standard workstation.

Heterogeneous (micro) modelling typically models individual mortar joints and brick units with continuum elements. This generally provides good accuracy since the elastic and inelastic properties of both bricks and mortar are taken into account, but extensive
computation is required when applied to full-scale wall and house units. With the introduction of interface elements, cracking and crack propagation can be modelled with heterogeneous models (Han and Masia, 2004; Cao and Watanabe, 2004).

As mentioned above, Cao and Watanabe (2004) proposed a heterogeneous finite element model using a viscoelastic joint element to model the interface between bricks. In principle this method is appropriate for early stage cracking, but without multi-stage updating (via experimental testing) to characterise the post-cracking behaviour of the structure, the model cannot be modified or validated. This heterogenous modelling approach requires extremely demanding computation power, and for large structures it is simply too onerous.

Under dynamic loading, the behaviour of a structure is dictated by its dynamic characteristics. The main shortcoming of previous research is the lack of dynamic updating and full-scale evaluation through systematic experimental work, especially for large three-dimensional structures. A correct model must possess the same dynamic characteristics as the physical specimen it represents. Moreover, as the behaviour of the structure changes with progressive damage during dynamic loading, modelling without considering changes of the dynamic characteristics of the structure will not be accurate. This aspect highlights the need to combine model development with extensive experimental testing, such that the dynamic progress can be accurately accounted for.

13.3.3 Approach

There are three key components to the development of a reliable numerical model: experimental testing, numerical modelling and parametric studies. Experimental testing and numerical modelling components should be interwoven throughout the research, with the results of each activity used to design subsequent activities. The final component involves parametric studies with the validated model to assess a variety of design and construction parameters. The details of each component are given below.

Experimental testing

Rigorous, systematic experimental testing of adobe blocks, prisms, wall units and model houses (static, quasi-static and dynamic testing). Extensive testing is required to
evaluate the relevant material properties and structural behaviour necessary for the
development of a reliable numerical model. The key stages of experimental testing include:

- Static testing and analysis of individual blocks, adobe mortar and multi-block
  masonry prisms for determination of characteristic structural properties
  (compressive, tensile, shear and bending strength, Young's Modulus and
  Poisson's Ratio). Research should be based on international standards for
  masonry testing and existing experimental test results (Chapters 3 and 4). A
  variety of parameters may be assessed, including different soil compositions,
  specimen dimensions, curing conditions, loading specifications, scaling factors,
  etc. Results would be used to develop a preliminary heterogenous (micro)
  numerical model.

- Quasi-static tests (cyclic) undertaken on I- or U-shaped adobe wall specimens to
  obtain stiffness and strain distribution relationships, as well as failure loads.
  Tests should cover combinations of 1:1 and 1:2 scale, in-plane and out-of-plane
  loading, effect of penetrations, and unreinforced and reinforced structures.
  Results would also be useful to develop an accurate scaling relationship for
  adobe panels.

- Dynamic shake table testing and analysis of I- or U-shaped wall units, and
  complete model houses to determine the pre- and post-cracked structural
  response of each specimen. As above, tests should consider different specimen
  scaling, load orientation, penetrations, roof loading, and reinforcement systems.
  As presented in this PhD research (Chapter 5), it is recommended that time
  scaling of the input excitation be undertaken to ensure dynamic similitude and
  induce damaging near-resonance conditions. In order to obtain valuable
  quantitative response data each specimen should be extensively instrumented
  (LVDT displacement sensors, accelerometers and strain gauges). EMTA should
  be conducted before, during and after shake table testing to gain an
  understanding of the changes in structural behaviour and to validate the
  numerical model. The research undertaken as part of this PhD thesis makes a
  substantial contribution to this stage.
Shake table testing is a vital component of the proposed research because after the development of large cracks, it is extremely difficult, if not impossible, to model high degrees of nonlinear behaviour of adobe structures without physical testing. The significance of shake table testing lies in not only providing the direct results in terms of dynamic response, strength and failure patterns, but more importantly to understand failure/cracking mechanisms as well as dynamic characteristics for progressively updating the nonlinear numerical model.

Numerical modelling

One of the challenges in modelling adobe structures is their inherently highly nonlinear post-cracked behaviour due to significant cracking and slippage. The correct modelling of this post-cracking behaviour is critically important because:

i) for brittle materials such as adobe masonry, cracking is common even at relatively low loads; and

ii) the major contributing factor to the loss of human lives is the collapse of cracked adobe structures.

An understanding of the post-cracked behaviour is the key to studying the failure of structures, evaluating strength, and enhancing the dynamic resistance of adobe buildings. Since both homogeneous and heterogeneous modelling have their inherent limitations in modelling large adobe structures, one possibility is to combine two modelling techniques through substructuring of the adobe structure and creation of super elements.

A 3-D nonlinear model can be created by combining:

i) heterogenous modelling techniques with interface elements for major cracked portions; and

ii) a two-stage homogenous modelling technique for minor cracked portions of the mudbrick structure.

A substructuring technique can be used to integrate the whole structure and reduce computational cost and effort.
Figure 13-1 Typical 3-D nonlinear FE modelling of mudbrick adobe house
(Cao and Watanabe, 2004).

In heterogenous modelling, the adobe brick is modelled with continuous solid elements, and the interface between the bricks is modelled with a viscoelastic joint element which simulates the opening and sliding behaviours of the interfaces (Figure 13-1). Each component possesses given constitutive properties determined from experimental testing (described above). The viscoelastic joint element is treated essentially like a solid element, but its thickness can be set at a very small value or zero, and its constitutive matrix can be altered using multi-stage updating from shake table testing. The most characteristic point is that the attenuation of vibration energy is considered in the element.

In two-stage homogenous modelling, minor cracking is allowed. First stage homogenisation considers modelling of mudbrick units, including the bed and perpend joints, into a homogeneous, orthotropic material. Second stage homogenisation is invoked when minor cracking of the mudbrick constituents (bricks or mortar) are detected. Here the position and orientation of the crack is calculated and the crack is subsequently smeared into the surrounding homogenised material.

Under dynamic loading, responses of two structures will be the same if their vibrational modes are identical. The approach here is that dynamic characterisation of the post-cracked physical structure provides insight into the changing nonlinear behaviour due to crack initiation/propagation, which can be used to update the FE model at each damage stage. This is only possible using in situ EMTA techniques for multi-stage updating.
during shake table testing, such that the nonlinear post-cracked behaviour of each structure can be accurately determined.

The ANSYS finite element analysis package and MATLAB development tool could be used to develop the numerical model. Strengthening systems could be modelled using standard line elements to represent reinforcement (e.g. bamboo, mesh, etc) with suitable properties. A series of sensitivity analyses should be undertaken to assess the robustness of the model. The updated FE model should be validated by shake table tests, as described above.

Parametric studies

A series of parametric studies using the validated FE model can be undertaken to assess a broad spectrum of design and construction variables (e.g. wall height, width, length ratios; size and location of door and window openings; configuration of reinforcement; material variability; connection details, etc). These results can then be used to update existing guidelines for safer adobe design and construction.
13.4 Implementation and application

There is large scope for further research focused on the implementation and application of improved adobe techniques. Potential research activities include:

- Evaluation of the short-term and long-term effectiveness of promotion and training activities aimed at introducing safer adobe construction and strengthening techniques.

- Appraisal of the attitudes of different stakeholder groups (homeowners, government agencies, NGOs and donors) towards seismic hazard, improved adobe initiatives, sustainable building practices, etc.

- The development of a detailed multi-criteria evaluation tool, building on the cost-complexity-seismic capacity matrix presented in Chapter 7. The evaluation tool could also incorporate other technical, social, cultural and institutional factors which will influence the design and realisation of any improved adobe construction project and implementation program. Factors to be considered may include:
  - Organisational capacity and resources of participating entities (e.g. NGO, government, community, homeowners).
  - Vulnerability of existing dwellings.
  - Levels of perceived and actual seismic risk.
  - Availability of resources (skills, time, labour, money, materials, tools).
  - Durability of materials available and preferred.
  - Skill levels of participants.
  - Cost, complexity and seismic capacity of proposed reinforcement systems.
13.5 Summary

Further research and development should focus on the enhancement of the current state of technical knowledge, while maintaining strong social links to ensure the investigation of appropriate solutions. Ongoing research and development is necessary in two broad areas:

- The technical aspects of adobe, including enhanced understanding of the response mechanisms, and the static and dynamic behaviour of adobe structures. This requires a combination of field research, experimental testing and numerical modelling.
- The social factors associated with adobe use, including promotion and training initiatives, and an awareness of the context, limitations and opportunities in a given location.

Progress will be made at local, national and international levels if these technical and social aspects can be linked to ensure the development of solutions which are appropriate to the current and future skills and resource capacities of beneficiary groups.

This additional research will lead to further updating and dissemination of guidelines for the design and construction of new adobe structures, as well as the retrofit-strengthening of existing dwellings. These activities are an important part of the overall objective of reducing the vulnerability of adobe houses to the force of earthquakes.
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14 SUMMARY AND CONCLUSIONS

14.1 General

Devastating earthquakes in Asia, The Middle East, Africa and Latin America have served as recent reminders of the vulnerability of non-engineered, low-cost dwellings to seismic forces. The loss of life and livelihood is often drastic, with millions of people in the poorest communities most severely affected. Adobe mudbrick housing is particularly vulnerable because of its inherently brittle nature, widespread use, generally poor construction quality and the limited awareness of concepts of aseismic design and construction. Despite this limitation, however, there is little doubt that adobe will continue to be the main construction material for the majority of the rural poor who simply cannot afford any alternative.

In Chapter 1, the main objective of this research project was stated:

To develop a low-cost, low-tech reinforcement system to improve the earthquake resistance of new and existing adobe mudbrick houses for application in developing countries without the need for ongoing external intervention.

This research project has successfully fulfilled this main objective. The result is an effective reinforcement system which is ready to be implemented, and can be truly regarded as low-cost and low-tech (ie. within the general financial and skill capacities of rural homeowners in developing countries). The reinforcement system developed and tested includes external vertical bamboo reinforcement, external horizontal wire reinforcement and a timber ring beam. The system can be applied to new-build constructions, as well as for the retrofit-strengthening of existing dwellings. Quantitative and qualitative shake table test results confirm the excellent seismic performance of the proposed reinforcement system. The reinforcement system delayed the onset of initial cracking, and reduced the severity of cracking during repeated high intensity shaking. Most importantly, collapse of the reinforced structures was prevented in all the experiments carried out.
The main research objective was realised through an extensive and multi-disciplinary research program, as described in this thesis. Research activities included:

- **Issue identification**: which involved identification of the core issues related to the seismic vulnerability of adobe houses. These issues were identified through field research and application projects in El Salvador, and a literature review covering seismic adobe research and improved adobe guidelines and manuals.

- **Experimental testing** of adobe prisms, wall units and a model house. The main focus was the shake table testing of eleven 1:2 scale u-shaped adobe wall panels and a 1:2 scale model house. Research activities included the design, construction, strengthening, instrumentation, testing and analysis of adobe specimens to evaluate the seismic capacity, costs and complexity of simple reinforcing systems.

- **Experimental Modal Testing and Analysis (EMTA)** of adobe structures prior to and during shake table testing to understand the key dynamic characteristics, and changes thereof, during testing.

- **Development of dissemination and implementation initiatives**, including a sample implementation program and the World Adobe Forum, a website dedicated to the sharing of information about safer adobe construction.

- **Identification of further research needs**, including research focused on post-earthquake reconnaissance, experimental testing, finite element modelling, and implementation and application initiatives.

The main processes and outcomes from each component of the research are presented in this chapter.

The main aspects which distinguish the research presented in this PhD thesis from other seismic adobe research around the world include:

- Specimen-specific time scaling of the input shake table motion to ensure dynamic similitude and induce damaging near-resonance conditions.

- Extensive experimental testing focused on low-cost, low-tech reinforcement systems.

- Detailed quantitative and qualitative data collection and analysis from the extensive shake table testing.
• A consideration of the cost and complexity of different reinforcement systems, in addition to an assessment of the seismic capacity.
• A multi-disciplinary approach, which relied on extensive practical field research to influence the development of technical solutions.
• Experimental Modal Testing and Analysis (EMTA) to determine the dynamic characteristics of adobe specimens both prior to and during shake table testing.
• Development of dissemination and implementation initiatives designed to transfer the research outcomes to the broader community.

14.2 Issue identification

In order to develop the objectives and framework for this research project, the key issues (related to reducing the vulnerability of adobe houses) were first identified. These issues cover technical, social, cultural and economic aspects, and include a variety of challenges and opportunities for improved adobe research and application. In order to appropriately identify these issues two main approaches were taken: (i) field research and application projects in El Salvador (Chapter 2), and (ii) a literature review covering seismic adobe research, and current guidelines and manuals on improved adobe design and construction (Chapter 3).

The field research and application projects in El Salvador included:
• Case study of adobe in El Salvador, including a discussion of the history and use of adobe housing, as well as some of the common features and deficiencies in traditional adobe houses.
• Evaluation of the features and effects of the 2001 El Salvador earthquakes, with a particular focus on the impacts to adobe housing. (More than 110,000 adobe houses were destroyed in the two earthquakes.)
• Review of the common damage patterns and failure mechanisms of adobe houses subjected to earthquakes.
• A ‘lessons learned over time’ investigation in El Salvador one year after the 2001 earthquakes, including a review of reconstruction activities and improved adobe initiatives (promotion, training and construction projects).
• The design and construction of a child-care centre in a small rural community in El Salvador. The project was designed as a hands-on training program in safer adobe construction and yielded some valuable lessons relating to the practical application of different improvement systems.

• A review of the general state of housing reconstruction and the housing deficit in El Salvador in 2005. A visit to an ‘improved adobe’ construction project revealed some significant deficiencies in the practices used.

• An assessment of the challenges and opportunities for the widespread implementation and acceptance of safer adobe construction and retrofit-strengthening techniques.

Field research undertaken in El Salvador has been a key component of this overall research project. The experiences in El Salvador have helped to develop an understanding of the core issues related to adobe housing, from both technical and practical perspectives. The field research has highlighted the importance of developing appropriate solutions, which are within the resource means (cost, skills, materials, tools) of the intended users. Such solutions have the opportunity to significantly reduce the vulnerability of adobe houses to earthquakes in El Salvador, and many other parts of the world.

The literature review covering seismic adobe research, and current adobe guidelines and manuals revealed the following general outcomes:

• Enhanced understanding of the different approaches and results of a variety of shake table tests of adobe wall units and model houses conducted in Peru, Mexico, the U.S.A. and Colombia. This included an appreciation of some of the strengths and limitations of different testing approaches, and the behaviour of adobe structures subjected to dynamic loading. Experimental testing to date has tended to focus on qualitative results (observations). Observations are useful to understand the general behaviour and relative performance of different adobe structures, but do not reveal some of the intricate response patterns detected by instrumentation (quantitative data).

• Most of the solutions that have been researched and presented in literature utilise at least some modern and costly materials (e.g. welded mesh, concrete, steel,
plastic mesh). These materials are often beyond the means of the rural poor, and thus external intervention is required for widespread implementation. Furthermore, many of the systems are too complicated to be incorporated without ongoing external support. There is generally a lack of discussion of the cost and complexity of the proposed initiatives in the literature.

- Many of the manuals and guidelines report systems which are based on testing undertaken in the 1970s, '80s and '90s. More recent research has highlighted the need to update these guidelines.
- The main emphasis of improved adobe manuals and guidelines has been the construction of new houses, with little attention given to the retrofit-strengthening of existing dwellings.

These research and dissemination activities have made a significant contribution to the current state-of-knowledge. Despite these efforts, however, there has been a distinct lack of large-scale application and community-level acceptance of these practices. The main reason for this lack of broadscale uptake is that most of the proposed systems are too complex and/or too costly to be widely used without sustained external intervention.

14.3 Experimental testing

Process

Extensive experimental testing was the core component of this research. Activities included:

- Fabrication of more than 8,000 adobe mudbricks (1:2 scale) for use in this project. The objective of this process was to make 'consistently average' bricks. [Chapter 4]

- Static testing of adobe prisms to determine characteristic material properties (compressive, shear and tensile bond strengths) and to assess the influence of different construction techniques on these properties. Evaluated techniques included using wet or dry bricks during construction, varying mortar thicknesses, and the application of compressive loads during curing. [Chapter 4]
• Selection, verification and modification of the input time history for the shake table testing. In this research the time history for the Mw 7.7 El Salvador earthquake of January 13, 2001 was used because it had a particularly devastating effect on adobe houses. A unique approach to time scaling the input spectra was undertaken. This involved the identification of the first natural frequency of each specimen in order to calculate the unique specimen-specific ‘time scaling factor’ which was used to modify the input spectra. This was done to ensure dynamic similitude (between the specimens) and induce damaging near-resonance conditions. Displacement-based intensity scaling of the input shake table motion was also undertaken in order to subject each specimen to a series of earthquake simulations of increasing intensity. [Chapter 5]

• Shake table testing of eleven 1:2 scale u-shaped adobe wall units. This stage included the design, fabrication, instrumentation and testing of the eleven wall units, which represented traditional and improved adobe structures. The reinforcement systems tested included: corner pilasters/buttresses, internal horizontal chicken wire mesh, external chicken wire mesh wrapping, internal vertical bamboo/timber poles, external horizontal bamboo poles, external vertical bamboo poles, external horizontal wire reinforcement and a timber ring beam. These improvements were tested individually and/or in combination with other interventions. [Chapter 6 and Appendix A]

• The construction, retrofit-strengthening, instrumentation and shake table testing of a 1:2 scale model house, which was reinforced with external vertical bamboo, external horizontal wire, and a timber ring beam. [Chapter 9 and Appendix A]

• Analysis of results from the shake table testing of the u-panel units and the model house. This included a review and comparative analysis of the qualitative results (observations, photographs, video footage) and quantitative results (displacement-time records, which showed the response of each specimen, including relative deformation, and vertical and horizontal flexure). The factors which caused damage, and the contributions of different reinforcing systems in ameliorating common failure modes were also considered. [Chapters 6, 7 and 9, and Appendix A]

• Development of a ‘specimen rating matrix’ which evaluates the seismic capacity, cost and complexity of each reinforcement system. [Chapter 7]
Outcomes

The main outcomes of the experimental testing were:

- Significant improvement in the shear and flexural bond strength of adobe masonry can be practically achieved by: (i) wetting the surface of each brick prior to laying; (ii) using a thin mortar joint; and/or (iii) applying a modest compressive load during curing. [Chapter 4]

- The variety of approaches to the shake table testing of adobe structures around the world highlights the need for better benchmarking and reporting of test parameters and characteristics. [Chapter 5]

- Test results confirm the importance of appropriate time scaling of input shake table motion to induce damaging conditions in a structure. Even an unreinforced adobe wall unit was undamaged during a 200% intensity simulation using the raw, unscaled (with respect to time) input motion. [Chapters 5 and 6]

- U-shaped adobe wall panels (with appropriate ‘wing’ wall restraint) exhibit classic failure patterns when subjected to shake table testing using a suitable input time history. Damages were consistent with damaged patterns observed in real structures subjected to real earthquakes. [Chapters 2 and 6]

- The main crack patterns in damaged adobe structures (vertical corner cracking, vertical midspan cracking, and horizontal and diagonal cracking) are due to combinations of overturning, vertical flexure and horizontal flexure. The most effective improvement systems reduce movement in the wall, which minimise these stresses, and thus delay the onset of initial cracking and the loss of strength of the structure. [Chapters 6, 7 and 9]

- The most effective systems which were tested incorporated external vertical bamboo reinforcement, external horizontal wire reinforcement, and a timber ring beam. These additions, when securely tied together, create an integrated system which restrains movement and enhances the overall strength of the structure. Effective reinforcement systems delay the onset of initial cracking, and reduce the severity of cracking during repeated high intensity shaking. Most importantly, collapse of reinforced structures was prevented in all cases. [Chapters 6, 7 and 9, and Appendix A]
• Major improvements in the seismic capacity of adobe structures can be achieved using low-cost and low-tech means. Such improvements are viable and effective for both new-build constructions and for the retrofit-strengthening of existing dwellings. [Chapters 6, 7 and 9]

• A continuous ring beam plays a key role in reducing the movement in the structure. The ring beam must be firmly connected to the wall (e.g. via dowels and external vertical bamboo) to be most effective. [Chapter 6, 7 and 9]

• Test results confirm that walls with penetrations are more vulnerable than walls without penetrations. This is especially the case for in-plane shear walls. Additional reinforcement should be placed around all penetrations. [Chapter 9]

• Test results question the widespread assumption that corner pilasters/buttresses will reduce the likelihood of vertical corner cracking. [Chapter 6]

• The use of internal (within the wall) vertical reinforcement appears to introduce discontinuities in the structure, which result in significant flexural movement, causing cracking, even at lower intensity simulations. The change of stiffness during the shake table testing was clearly detected by the Experimental Modal Testing and Analysis (EMTA) discussed below. [Chapters 6, 7 and 8]

• A ‘specimen rating matrix’ is a practical means of considering the relationship between the seismic capacity, complexity and cost of different reinforcement systems. These factors could be incorporated in a detailed multi-criteria evaluation matrix, which would be a useful tool in the planning and realisation of any construction and implementation project. [Chapters 7 and 13]

14.4 Experimental Modal Testing and Analysis (EMTA)

Process

Experimental Modal Testing and Analysis (EMTA) provides information on the key dynamic properties of a structure. These include the natural frequencies, damping ratios and mode shapes. EMTA was undertaken for the u-shaped adobe wall panels and model house 4A prior to and during shake table testing. The following activities were undertaken:
• EMTA using an impact hammer to excite each specimen prior to shake table testing to determine the first natural frequency. This information was then used to appropriately scale (with respect to time) the input excitation to ensure dynamic similitude and induce damaging near-resonance conditions, as described above. [Chapters 5, 6, 8 and 10]

• EMTA using an impact hammer to excite selected specimens (u-panels 3F, 3H, 3J and 3K, and model house 4A) prior to shake table testing. The multi-degree-of-freedom approach was used to determine the key dynamic characteristics (natural frequencies, damping ratios and mode shapes). [Chapters 8 and 10, and Appendix D]

• EMTA using an impact hammer to excite selected specimens (u-panel 3J and model house 4A) during the different stages of retrofit-strengthening to evaluate the influence of each strengthening intervention on the dynamic characteristics. [Chapters 8 and 10]

• EMTA using the shake table to excite selected specimens (u-panels 3H, 3J and 3K, and model house 4A) to assess the changes in dynamic characteristics during the series of shake table tests. [Chapters 8 and 10]

Outcomes

The main outcomes of the Experimental Modal Testing and Analysis (EMTA) were:

• Determination of the first natural frequency of each specimen (using an impact hammer to excite each specimen). This information was then used to appropriately scale (with respect to time) the input shake table motion, as described above. Results revealed that the u-panels with internal vertical reinforcement (specimens 3G and 3K) possessed the lowest first natural frequency (lowest stiffness), due to discontinuities introduced by the reinforcement. Specimen 3B had the highest first natural frequency due to the additional stiffness provided by the corner pilasters. [Chapters 5, 6, 8, 9 and 10]

• Determination of the key dynamic characteristics of selected u-panels prior to shake table testing (using an impact hammer to excite each specimen). The results confirmed the validity of the selected process (multi-degree-of-freedom EMTA), with the detected mode shapes matching those identified in previous adobe research. The addition of the ‘wing’ wall restraint on the u-panels was
seen to increase the first natural frequency (stiffness) and decrease the modal damping of the structure. [Chapter 8]

- Results from model house 4A (prior to shake table testing) revealed that the location of the impact hammer hit affects the form of the FRF graphs, but not the mode shapes and corresponding natural frequencies. Penetrations, however, were seen to have a significant effect on the mode shapes. This was especially the case for the shear wall with the door opening, which demonstrated a significant deformation tendency. The addition of a timber ring beam (with midspan cross-beam) during the retrofit-strengthening process changed the dominant second mode from a flexural ‘breathing’ mode to a torsional mode. The natural frequencies (global stiffness) of the model house increased only slightly during the strengthening process. [Chapter 10]

- Assessment of the changes in dynamic characteristics of selected u-panels during the series of shake table simulations. Results showed the progressive loss of stiffness, and general increase in modal damping, of the specimens as the level of damage increased. This was particularly evident for specimen 3K (internal vertical reinforcement) which experienced a steady loss of stiffness commencing at the low intensity simulation S4 (S20%). In contrast, the externally reinforced specimens 3H and 3J maintained integrity (stiffness) up to simulation S6 (S75%) after which damage to the structure became apparent. The mode shape of the dominant first mode (‘first transverse translation mode’) clearly matched the flexural response of the walls, as presented in Chapter 7. [Chapters 7 and 8, and Appendix D]

- Assessment of the changes in dynamic characteristics of model house 4A during the series of shake table simulations. Clear results were obtained for the low intensity simulations (S10% and S25%) only. These results indicated that torsion was the dominant mode, most probably due to the asymmetry introduced by the penetrations. These mode shapes closely matched the flexure results presented in Chapter 9. The ‘messy’ results for the higher intensity simulations confirmed the complexity of structural responses when the specimen was subjected to dynamic excitation. [Chapters 9 and 10]

EMTA was demonstrated to be a useful tool to reflect the physical response and changes in dynamic characteristics of adobe structures. Results provide greater insight
into the structural behaviour than observations alone, and may be a practical tool for
damage detection and condition monitoring of adobe structures.

14.5 Dissemination and implementation

The framework and preliminary development of two initiatives for the dissemination
and implementation of research findings were presented. These included:

- The World Adobe Forum: a website dedicated to the sharing of information
  about safer adobe construction. A detailed preliminary website has been
developed by the author of this thesis, in collaboration with a number of adobe
experts from around the world. The website incorporates key areas of research
and application, including experimental testing and analysis, field research, and
implementation. The World Adobe Forum is being supported by the Earthquake
Engineering Research Institute (EERI) and the International Association for
Earthquake Engineering (IAEE), and is expected to come on-line in late 2006.
The World Adobe Forum has the potential to become the premier global adobe
information hub, serving to effectively disseminate appropriate information
about safer adobe construction. This information transfer is a key component in
the challenge to reduce the vulnerability of adobe houses in high risk areas
around the world.

- A sample implementation program designed to increase the desire, capacity and
  confidence of local homeowners and builders to retrofit and/or construct safer
adobe houses. The core components in a potential community-based improved
adobe training program were outlined. These components include planning
(context assessment, feasibility study, resource requirements, etc), promotion
and awareness building, retrofit training (simple theory and practical
components), ongoing activities (extended support, mentoring and monitoring,
apprenticeships, accreditation and job opportunities) and the potential design and
construction of a new community building.

These dissemination and implementation initiatives are designed to transfer the
outcomes of this and other research projects to communities around the world where
people continue to live in vulnerable adobe houses.
14.6 Further research

A number of further research needs have been identified (Chapter 13). These relate to both technical and practical aspects of improved adobe, and include:

- Post-earthquake reconnaissance, including the development of a detailed database incorporating building features (age, location, orientation, configuration, dimensions, quality of construction and maintenance, reinforcing systems, etc) and building performance (cracking patterns, failure modes, structural deficiencies, etc). This information would allow a greater understanding of the mechanisms of failure (in-plane, out-of-plane, and oblique-angle response of components and connections) and the role which different features and systems play in seismic performance.

- Further experimental testing of adobe bricks, prisms, wall units and model houses (static, quasi-static and dynamic testing). This is necessary to enhance the understanding of relevant material properties and structural behaviour. Experimental testing, coupled with advanced EMTA techniques, provide important information for the Finite Element (FE) modelling of adobe structures.

- Development of a reliable numerical model through nonlinear heterogeneous FE modelling. The model should be developed and validated using data from experimental testing.

- Extensive parametric studies (using the validated FE model) to assess a broad range of design and construction variables and improvement systems, without the need for resource-intensive physical testing. These studies provide an opportunity to update current guidelines for the safe design, retrofit-strengthening and construction of adobe structures.

- Implementation and application activities, including an evaluation of the effectiveness of promotion and training programs, and an appraisal of the attitudes of different stakeholder groups towards seismic hazard, improved adobe initiatives, sustainable building practices, etc. The development of a comprehensive multi-criteria evaluation tool will be useful for the planning and realisation of any construction and implementation project.
14.7 Concluding comments

This thesis and research is one part in a long and iterative process, involving:

- Implementation projects, which include promotion, awareness-building and training.
- Ongoing review of implementation progress, successes, lessons learned and identification of future research needs (technical and social).
- Ongoing research (technical and social) and modifications to techniques.

It is hoped that this process, and the research outcomes presented in this thesis, will contribute to a substantial and sustainable reduction in the seismic vulnerability of adobe houses around the world.
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APPENDICES

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APPENDIX A. VIDEO FOOTAGE: UTS RESEARCH

A number of CDs and DVDs contain video footage from the UTS research. Table A-1 shows the contents of each CD/DVD.

<table>
<thead>
<tr>
<th>Disc</th>
<th>CD/DVD</th>
<th>Contents</th>
</tr>
</thead>
</table>
| Disc 1 | CD | Brick fabrication  
 | | U-panel 3J (construction)  
 | | U-panel 3K (construction)  
 | | Model house 4A (construction)  
 | | Model house 4A (strengthening)  |
| Disc 2 | CD | U-panels 3A – 3K (Best of tests)  |
| Disc 3 | CD | Model house 4A (Best of tests)  |
| Disc 4 | DVD | U-panels 3A – 3I (All tests)  |
| Disc 5 | DVD | U-panels 3J + 3K, Model house 4A (All tests)  |

A playlist has been created for each disc. This opens the relevant video clips in Windows Media Player. To play, open the disc in Windows Explorer (or equivalent), and double-click on the desired playlist.

A comprehensive list of the contents of each CD/DVD is included below.

(Discs 6 and 7 contain TV and radio profiles related to this research, as discussed in Appendix B.)
A.1 Video footage: u-panels

The configuration of the u-panels is shown in Figure A-1.

![Diagram of u-panels configuration]

**Figure A-1** U-panels: configuration.

Table A-2 shows the sequence of testing for each specimen.

**Key for Table A-2:**

- Shaded box = simulation was undertaken.
- The letters indicate which camera angles were captured and processed.
  
<table>
<thead>
<tr>
<th>Letter</th>
<th>Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>East</td>
</tr>
<tr>
<td>N</td>
<td>North</td>
</tr>
<tr>
<td>S</td>
<td>South</td>
</tr>
<tr>
<td>W</td>
<td>West</td>
</tr>
<tr>
<td>SE</td>
<td>South-east</td>
</tr>
<tr>
<td>NW</td>
<td>North-west</td>
</tr>
</tbody>
</table>

- The red letters indicate which footage is included in the “Best of Tests” CD (Disc 2)

For each specimen (except 3A) a ‘post-mortem’ video was taken. This involved a slow ‘walk around’ of the damaged specimen, recording any aspects of interest. The ‘post-mortem’ videos are included on Discs 4 and 5.
Table A-2 U-panel specimens: test sequence and video footage.

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity</th>
<th>3A</th>
<th>3B</th>
<th>3C</th>
<th>3D</th>
<th>3E</th>
<th>3F</th>
<th>3G</th>
<th>3H</th>
<th>3I</th>
<th>3J</th>
<th>3K</th>
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<tr>
<td>Unscaled</td>
<td>S1</td>
<td>U40%</td>
<td>E</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>S2</td>
<td>U100%</td>
<td>E</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>S3*</td>
<td>U150% / U200%</td>
<td>E</td>
<td></td>
<td></td>
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<td></td>
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</tr>
<tr>
<td></td>
<td>S4</td>
<td>S20%</td>
<td>E</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>S5</td>
<td>S50%</td>
<td>E</td>
<td>E</td>
<td>E</td>
<td>E</td>
<td>E</td>
<td>E</td>
<td>E</td>
<td>E</td>
<td>E</td>
<td>E</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td>S6</td>
<td>S75%</td>
<td>E</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
</tr>
<tr>
<td></td>
<td>S7</td>
<td>S100%</td>
<td>E</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
</tr>
<tr>
<td></td>
<td>S8</td>
<td>S125%</td>
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<td>E</td>
<td>E</td>
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<td>E</td>
<td>E</td>
<td>E</td>
<td>E</td>
<td>E</td>
<td>E</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td>S9 - S10</td>
<td>S75% (x2)</td>
<td>E</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
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<td>S</td>
<td>S</td>
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</tr>
<tr>
<td></td>
<td>S11 - S12</td>
<td>S100% (x2)</td>
<td>E</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
</tr>
<tr>
<td></td>
<td>S13 - S14</td>
<td>S100% (X2)</td>
<td>E</td>
<td>S</td>
<td>S</td>
<td>S</td>
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<td>S</td>
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<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
</tr>
<tr>
<td></td>
<td>S15</td>
<td>S120%</td>
<td>E</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
<td>S</td>
</tr>
<tr>
<td></td>
<td>‘Post-mortem’</td>
<td>N/A</td>
<td>S7+</td>
<td>S7+</td>
<td>S7+</td>
<td>S8+</td>
<td>S12+</td>
<td>S3+</td>
<td>S12+</td>
<td>S8+</td>
<td>S12+</td>
<td>S15+</td>
<td></td>
</tr>
</tbody>
</table>
A.2 Video footage: model house 4A

The configuration of the model house is shown in Figure A-2.

Figure A-2 Model house 4A: configuration.

Table A-3 shows the sequence of testing for model house 4A.
Table A-3 Model house 4A: testing sequence and video footage.

<table>
<thead>
<tr>
<th>Time</th>
<th>Simulation</th>
<th>Intensity</th>
<th>Model house 4A</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>S1</td>
<td>S10%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>S2</td>
<td>S25%</td>
<td>E S SE SW W</td>
</tr>
<tr>
<td></td>
<td>S3</td>
<td>S50%</td>
<td>E S SE SW W</td>
</tr>
<tr>
<td></td>
<td>S4</td>
<td>S75%</td>
<td>E S SE SW W</td>
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<td>S5</td>
<td>S100%</td>
<td>E S SE SW W</td>
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<td></td>
<td>S6</td>
<td>S125%</td>
<td>E S SE SW W</td>
</tr>
<tr>
<td></td>
<td>S7</td>
<td>S100%</td>
<td>E S SE SW W</td>
</tr>
<tr>
<td></td>
<td>S8</td>
<td>‘Shakedown’</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>NE SE</td>
</tr>
<tr>
<td>Post-mortem</td>
<td></td>
<td></td>
<td>S8+</td>
</tr>
</tbody>
</table>

**Key for Table A-3:**

- Shaded box = simulation was undertaken.
- The letters indicate which camera angles were captured and processed.
  - E = East
  - SE = South-east
  - S = South
  - SW = South-west
  - W = West
- The **red** letters indicate which footage is included in the “Best of Tests” CD (Disc 3)

Simulation S8 (‘shakedown’) runs for approximately 10 minutes. It is included on Disc 5.

After simulation S8 (‘shakedown’) a ‘post-mortem’ video was taken. This involved a slow ‘walk around’ of the damaged specimen, recording any aspects of interest. The ‘post-mortem’ video for model house 4A is included on Disc 5.
APPENDIX B. MEDIA EXPOSURE

B.1 ABC Catalyst (TV)

ABC Catalyst: 18 May 2006
http://www.abc.net.au/catalyst/stories/s1640933.htm

Adobe House
Reporter: Dr Maryanne Demasi
Producer: Maria Ceballos-Wallis
Researcher: Maria Ceballos-Wallis

Dom Dowling wants to make the world a better place. After witnessing first hand the devastation wrought by an earthquake in El Salvador, he set about trying to improve the design of Adobe mud brick houses, which had collapsed and killed so many of their inhabitants.

Whatever he came up with, this Engineering PhD student knew it had to be cheap, and able to be done by the locals using local materials. Back in Australia, he tried all sorts of options to keep the mud brick together.

While the structures had to be low-tech, the testing equipment was anything but. In fact, Dom had access to the latest earthquake simulation equipment at Sydney’s UTS.

Having chosen his retrofit earthquake proofing system (which uses string, bamboo and wire), Dom built his own half-size mud brick house and reinforced it.

Will it stand up to a major earthquake, like the one that devastated El Salvador?

Catalyst went along to find out.

The ABC Catalyst story is included on Disc 6 (DVD).

<table>
<thead>
<tr>
<th>Chapter</th>
<th>Contents</th>
<th>Start time</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Pre-introduction</td>
<td>00’00”</td>
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<td>Adobe house (Dom Dowling)</td>
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<td>Caffeine challenge cont’d</td>
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<td>7</td>
<td>Closure + credits</td>
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**B.2 ABC New Inventors (TV)**

ABC New Inventors: 2 August 2006 (Episode 25)

http://www.abc.net.au/newinventors/txt/s1699340.htm

Dom Dowling: episode winner (judges) and people's choice winner.

**Quake Safe**

by Dom Dowling

The Quake Safe is a frame made from string, bamboo and wire, which can be either retrofitted into an existing adobe (mud brick) house or incorporated into a new house as it's being built, in order to give it a much higher level of structural protection against earthquakes.

The frame is designed to be affordable to people who live in this type of house, particularly the poorer rural communities of Central America.

**Inspiration**

The Quake Safe was Dominic's recently completed PhD project. The inspiration for the invention came when he experienced the tragic results of an earthquake in El Salvador in 2001.

Dom was working as a volunteer aid worker in Nicaragua when the earthquake struck. When he arrived in El Salvador a week later he was shocked at the level of destruction the earthquake had wrought on people's homes and lives. The adobe mud brick structures, which were common feature of the area, had little resistance to the power of an earthquake, and many people had been caught inside their collapsing houses when the earthquake struck.

Dom felt that, using his engineering background, he must be able to come up with a way to give these buildings enough structure to keep them standing long enough for people to get out.

So this was the PhD task he set himself: to make a system which would greatly enhance the strength of mud brick buildings in the face of an earthquake. It must be able to be easily incorporated into the normal building methods used to make these houses. It must also be retrofitable to existing mud brick houses. It must use only materials which were commonly available in poorer rural areas and, above all, it must be as affordable as possible.

Five years later, he believes he has perfected the recipe.

Cont'd
How does it work?
The Quake Safe is a frame constructed of string, bamboo and wire which is designed to hold together an adobe (mud brick) house wall long enough in the event of an earthquake for the occupants to escape.

It can either be retro-fitted to an existing adobe house or incorporated into the construction of a new one. The aim of the invention is to use only materials which are readily and cheaply available in the poorer rural communities who often use this form of construction. It also requires very minimal additional tools to assemble.

The construction methods are slightly different depending on whether the frame is to be incorporated from the beginning of construction, or retro-fitted to an existing building.

The ABC New Inventors is included on Disc 6 (DVD).

<table>
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<tr>
<th>Chapter</th>
<th>Contents</th>
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<td>Filler: ‘Chindagu’</td>
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<td>7</td>
<td>Judging + presentation</td>
<td>23'01&quot;</td>
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B.3 Radio Australia (radio)

ABC Radio Australia
Innovations
3 October 2005
http://www.abc.net.au/ra/innovations/stories/s1473553.htm

Innovations
Host: Desley Blanch

Earthquake Resistant Mud-Brick House

About a third of the world's population lives in mud-brick houses and most of those are built in earthquake areas. When a big earthquake hits, tens of thousands of people die because the houses they live in are poorly built, but a simple and cheap modification could prevent mass fatalities.

The ABC Radio Australia interview is included on Disc 7 (CD).
Duration: 0'00" - 13'30"
B.4 Radio National (radio)

ABC Radio National
Saturday Breakfast – Home Affair
15 October 2005
http://www.abc.net.au/rn/talks/saturday/stories/s1482607.htm

Home Affair: Earthquake resistant houses
Host: Geraldine Doogue
Producer: Kerry Stewart

Home Affair is the part of the program where we explore new trends in our domestic environment. After the devastation in Pakistan and India, we’re looking at a new and resilient way of building that could be used in earthquake prone areas. Engineering PhD student Dominic Dowling is in the process of writing up his extensive research into a way of constructing houses that could stop them collapsing after an earthquake of very high magnitude.

The ABC Radio National interview is included on Disc 7 (CD).
Duration: 0'00" – 7'34"

Technology

String, mud and bamboo make a quake-proof house

When an earthquake strikes, it is not just modern concrete or brick buildings that are damaged or destroyed. Traditional adobe houses which shelter around a third of the world's population, are just as vulnerable - witness the300,000 deaths that hit El Salvador in just over a month destroying more than 100,000 adobe houses.

Now a team led by Brian Samali of the Centre for Built Infrastructure Research at the University of Technology in Sydney Australia have developed a fibre-reinforced mud adobe that can be used to repair and strengthen existing adobe houses to keep them standing during quakes.

Adobe is an ancient building material that has wide use in many countries. Its success relies on its ability to absorb and distribute the force of earthquakes.

The new material is made by mixing straw and mud in a 2:1 ratio. The mixture, which is packed into moulds shaped to the size of the walls, is then left to dry for two days before being taken out and cured for another six days. Once cured, it is left to dry for another six days to reach the final density of a solid concrete brick.

The material is said to be stronger than the traditional adobe and is less likely to crack during an earthquake. The team tested the material by subjecting it to simulated earthquakes and found that it was able to withstand the force of a 7.5 magnitude quake.

The new material has been patented and is now being marketed to the construction industry. The team is also working on a way to make the material more resistant to fire and water.

"We are confident that our material can be used to strengthen existing adobe houses," said Samali. "This will help to protect millions of people who live in areas prone to earthquakes."

The team plans to test the material in real-life conditions by building a prototype house and subjecting it to a simulated earthquake.

No escape for the oil slick cheats

The oil slick crisis that has plagued the Persian Gulf for months shows no signs of abating. The latest estimate from the International Maritime Organization (IMO) puts the total amount of oil spilled into the Gulf at 11.5 million tonnes, of which 3.5 million tonnes is still unaccounted for.

The IMO estimates that the amount of oil remaining in the Gulf is equivalent to the annual production of 35 million barrels of oil. The organization is calling for an international effort to clean up the area and prevent further contamination.

The oil slick crisis has sparked a wave of concern among the international community, with many calling for stronger action to address the issue. The United Nations is expected to hold an emergency meeting on the matter in the coming weeks.

The crisis has also drawn attention to the need for better regulation of the shipping industry, with many calling for stricter rules to prevent accidents and spills.

In the meantime, theIOR (International Oil Routes) is working to clean up the area and prevent further contamination. The organization has launched a new project to clean up the most heavily polluted areas of the Gulf.

The project is expected to cost around $1 billion and will involve the use of advanced technologies to remove oil from the water. The IOR is also working with local governments to establish long-term plans for the prevention of oil spills and the protection of the environment.

The oil slick crisis has also had a significant economic impact, with the cost of cleaning up the area estimated at $30 billion. The crisis has also caused a drop in oil prices, with some analysts predicting a further decline in the coming months.
APPENDIX C. EL SALVADOR EARTHQUAKE: RECORDING STATION CHARACTERISTICS

Table C-1 Earthquake recording station characteristics:
Hospital Santa Teresa, Zacatecoluca, El Salvador (COSMOS, 2001).

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<th>Station</th>
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<tr>
<td>Site Geology</td>
<td>Soil</td>
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<td>Earthquake</td>
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<td></td>
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<td></td>
<td>Vertical: +0.2325g / -0.2754g</td>
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<tr>
<td>Status</td>
<td>Uncorrected data</td>
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Figure C-1 E-W component of the January 13, 2001 El Salvador earthquake, recorded at Hospital Santa Teresa, Zacatecoluca, El Salvador (COSMOS, 2001). Time: 0-60s.
Figure C-2  E-W component of the January 13, 2001 El Salvador earthquake, recorded at
Hospital Santa Teresa, Zacatecoluca, El Salvador (COSMOS, 2001). Time: 15-25s.

Table C-2  Earthquake recording station characteristics:
Santiago de Maria, El Salvador (COSMOS, 2006).

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APPENDIX D. EXPERIMENTAL MODAL TESTING AND ANALYSIS (EMTA)

D.1 Introduction

This appendix contains the following information:

- General theory of Experimental Modal Testing and Analysis (EMTA), including data acquisition, signal processing (including pre-trigger delay and windowing), parameter identification (including SDOF and MDOF methods), validations (including frequency response functions, mode identification functions, stabilisation diagrams and modal assurance criteria), and modal parameters (natural frequency, damping ratios and mode shapes).

- Complete results for the EMTA (impact hammer testing) of selected specimens prior to shake table testing (3F, 3H, 3J and 3K).

- Complete results for the EMTA (impact hammer testing) of specimen 3J during the strengthening stages (prior to shake table testing).

- Complete results for the EMTA (shake table testing) of selected specimens during shake table testing (3H, 3J and 3K).

- Complete results for the EMTA (impact hammer testing) of model house 4A during the strengthening stages (prior to shake table testing).

The information in this appendix complements the summaries and general discussion in Chapters 8 and 10.
D.2 Experimental Modal Testing

D.2.1 Data acquisition

For this research project the main data acquisition system consisted of a Hewlett Packard state-of-the-art Vxi system equipped with leading software from LMS (LMS CADA-X). The system consisted of two HP Vxi 16 channel 51.2 kHz digitisers with anti-aliasing filter and a DSP (digital signal processing) on board in a C-size frame. The frame was equipped with a controller and high speed Mxi bus, connecting it to the HP workstation. The digitiser had an implemented DSP and a 4-32MB FIFO (file input, file output) digital anti-aliasing filter.

D.2.2 Signal processing

Signal processing methods are commonly used to improve the quality of time signals. For this project, a pre-trigger delay, plus force and exponential windowing were applied to increase accuracy. The methods are described below

Pre-trigger delay

The impulse signal exists for a very short period of time, so it is important that the analyser begins acquiring data prior to impact excitation (Schwarz and Richardson, 1999).

Windowing

Windowing functions (force window and exponential window) are used to adjust discontinuities of the signal and to ensure that the signal is zero at the start and the end of the sampling period. For the excitation channel (impact hammer) a ‘force window’ was applied to remove stray noise from the impulse signal. Ideally, the impulse signal is non-zero for the small time period of the excitation (hammer hit) and zero for the remaining time. Therefore, any non-zero data following the impulse signal was assumed to be noise and thus, considered to be zero (Figure D-1a). For the response channels (accelerometers) an ‘exponential window’ was applied to ensure that the transient signal decayed sufficiently at the end of the sampling period (Figure D-1b).
This was done by introducing artificial damping into the measurement data (Schwarz and Richardson, 1999).

(a) Force window  
(b) Exponential window

Figure D-1 Window functions.
D.3 Experimental Modal Analysis: theory

Experimental Modal Analysis (EMA) is an established and reliable vibration analysis tool, providing information on the dynamic characteristics of a structure and its excitation (Samali et al, 2002). By processing the excitation-response data obtained through experimental modal testing it is possible to determine the key dynamic features of the structure: the natural frequencies, the modal damping and the mode shapes. The process is depicted in Figure D-2.

The data sampled during the model testing was analysed using the ‘Modal Analysis Module’ of the software LMS CADA-X. First, the time signals (amplitude versus time) were converted (transformed) into frequency spectra (amplitude versus frequency) using Fourier transform. Then, the Fourier transform signals of the accelerometers (output) were divided by the Fourier transform signal of the hammer impact (input), resulting in the ‘Frequency Response Function’ (FRF). The FRF determines how much acceleration response a structure has per unit of force excitation. This is the most important measurement in the experimental modal analysis, as it provides information about the modal parameters of the structure, namely, the natural frequencies, modal damping and the mode shapes (Avitabile, 2001). The relationship between the FRF and the modal parameters is given in Equation D-1 (Samali et al, 2002).

![Figure D-2 Phases of the modal analysis.](image-url)
\[ h_{ij}(\omega) = \sum_{k=1}^{N} \left( \frac{r_{ijk}}{\omega - \lambda_k^*} + \frac{r_{ijk}^*}{\omega - \lambda_k^*} \right) \quad D-1 \]

Where:

\( h \) = unit impulse response function

\( \omega \) = frequency (rad/sec)

\( N \) = number of modes of vibration that contribute to the structure's dynamic response within the frequency range under consideration

\( r_{ijk} \) = residue value for mode \( k \)

\( \lambda_k \) = pole value for mode \( k \)

* designates complex conjugate

### D.3.1 Parameter identification

The FRF can be presented in rectangular coordinates (real part vs. frequency; and imaginary part vs. frequency) or in polar coordinates (amplitude vs. frequency; and phase vs. frequency). At resonance, in the rectangular presentation, the imaginary part is maximum and the real part is zero. In the polar system, the magnitude reaches a maximum and the phase lags by 90° at resonance. This is shown for a single-degree-of-freedom (SDOF) system in Figure D-3. These characteristics enable the identification of the modal parameters of a structure from the FRF.

LMS CADA-X software comprises different parameter estimation techniques. Depending on the data to be analysed and the objectives of the analysis, the appropriate method must be chosen in order to achieve the best modal model. Figure D-4 shows a chart summarising the available techniques for parameter estimation using LMS CADA-X software.
Figure D-3 FRF graphs in rectangular and polar coordinates for a single-degree-of-freedom system.

![FRF graphs](image)

**Figure D-4 Available parameter estimation techniques in LMS CADA-X (1997).**

### LMS CADA-X Modal Analysis

- **Single Degree of Freedom (SDOF)**
  - # Peak picking
  - # Mode fitting
  - # Circle fit

- **Complex Mode Indicator Function (Complex MIF)**
  - # Complex Mode Indicator Function

- **Time Domain Multiple Degree of Freedom (MDOF)**
  - # Least Squares Complex Exponential
  - # Least Squares Frequency Domain

- **Frequency Domain Multiple Degree of Freedom (MDOF)**
  - # Frequency Domain Direct Parameter Identification

**SDOF methods**

In the early days of EMA, modal parameter estimation was typically performed using single-degree-of-freedom (SDOF) methods. The SDOF methods peak picking and cycle-fitting, which are part of the LMS CADA-X 'Modal Analysis Module', are described below.
The peak picking method is the simplest approach to estimate the modal parameters of a structure. The natural frequencies are simply taken from the observation of the peaks of the graphs of the frequency response data. “The damping ratios are calculated from the sharpness of the peaks and the mode shapes are calculated from the ratios of the peak amplitudes at various points in the structure” (Maia and Silva, 1997). This method is one of the most popular techniques for estimating modal parameters. It is easy to use, very fast and requires minimal computing resources. It is, however, sensitive to measurement noise and effects from adjacent modes. Hence, this method is best suited for structures with light damping and well separated modes where modal coefficients are essentially real valued (Agilent Technologies, 2000.)

The circle fitting method extracts the modal parameters by fitting a circle to the real and imaginary part of the frequency response data by minimising the error between the radius of the fitted circle and the measured data. The modal coefficients are determined from the diameter of the circle, and the phases are determined from the positive or negative half of the imaginary axis in which the circle lies (Agilent Technologies, 2000). The circle fitting method works well for the majority of situations and even for highly complicated structures. Similar to the peak picking method, it works best for lightly damped structures with widely separated modes. However, the method is very time consuming and therefore often disregarded (Maia and Silva, 1997).

SDOF methods are best used when the data is not too noisy and the natural frequencies of the system being tested are well separated. They are most useful for troubleshooting problems; however, only where it is not necessary to create a modal model and time is limited (Agilent Technologies, 2000).

The parameter estimation of the u-panels 3A, 3B, 3C, 3D, 3E, 3G and 3I was executed using the peak picking method. These specimens were tested in an early stage of the research project where a detailed modal analysis was not performed. Only the fundamental frequencies were determined, which were necessary for the scaling of the earthquake input excitation (described in Chapter 5).
MDOF methods

For the u-panels 3F, 3H, 3J and 3K, a comprehensive modal analysis was conducted. Hence, the multi-degree-of-freedom (MDOF) ‘least squares complex exponential’ method combined with the least square frequency domain’ was used to identify the modal parameters. This method is less sensitive to noise and can be applied to structures with closely spaced modes. The method utilises curve-fitting algorithms and calculates first the system poles in the time domain. “The response can be expressed in terms of modal parameters in the time domain in the form of the least squares complex exponential” (Kelley et al., 1996), as shown in Equation D-2:

\[ H(t) = \sum_{k=1}^{m} \frac{1}{m_k \omega_{\text{d}_k}} e^{-\lambda_{\text{d}_k} t} \sin \omega_{\text{d}_k} t \]  \hspace{1cm} \text{D-2} \\

Where:
\[ \omega_{\text{d}_k} = \text{damped natural frequency of mode} \ k \]
\[ m_k = \text{modal mass of} \ k\text{th mode} \]

Once the system poles are identified, the modal parameters are estimated in the frequency domain using Equation D-3, which is the general equation for the frequency response matrix in terms of modal parameters (Kelley et al., 1996):

\[ [H(j\omega)] = \sum_{k=1}^{m} \left[ \frac{\{U_k\} \cdot \{L_k\} + \{U^*_k\} \cdot \{L^*_k\}^*}{(j\omega - \lambda_k) \cdot (j\omega - \bar{\lambda}_k)} \right] - \left[ \frac{\text{LR}}{\omega^2} \right] + [UR] \]  \hspace{1cm} \text{D-3} \\

Where:
\[ [H(j\omega)] = \text{frequency response matrix} \]
\[ m = \text{number of modes in database} \]
\[ \{U_k\} = \text{mode shape vector for the} \ k\text{th mode} \]
\[ \{L_k\} = \text{row vector of modal participation factors} \]
\[ \text{LR} = \text{lower residual term} \]
\[ \text{UR} = \text{upper residual term} \]
\[ \cdot = \text{complex conjugate symbol} \]
\[ \lambda_k = \text{complex pole value for the } k\text{th mode, as defined in Equation D-4:} \]

\[ \lambda_k = -\left(\xi_k \omega_{nk}\right) + j\omega_{nk} \sqrt{1 - \xi_k^2} \quad \text{(D-4)} \]

Where:
- \( \xi_k \) = damping ratio for mode \( k \)
- \( \omega_{nk} \) = natural frequency of mode \( k \)

### D.3.2 Validations

When a modal model is established it is essential to validate the model. Validation of a modal model is usually accomplished by several mode identification tools which assist in identifying the number of modes in a given frequency band. Among the most common of these techniques are the FRF summation function, the mode indicator function and the stabilisation diagram (Kelley et al, 1996). All of these techniques were utilised in this project.

**FRF summation function (or enhanced frequency response function)**

The simplest of the mode indication tools is the FRF summation function (also known as the enhanced frequency response function). By summing the FRF’s of all acceleration responses, the modal peaks in the data are accentuated and appear as maxima in the summation of the imaginary components. This can be done since the mode peaks of all FRF’s of a structure coincide (Kelley et al, 1996).

**Mode indicator function**

The mode indicator function (MIF) is a frequency domain function that exhibits local minima for each of the structure’s natural frequencies (LMS CADA-X, 1992). It takes advantage of the real component of the response vector being a minimum at resonance. It is a tool available in many commercial software packages to aid in the identification of modes in measured data (Kelley et al, 1996). An example of a MIF is shown in
Figure D-5. The black graph is the FRF, the letters are the symbols from the stabilisation diagram (see below) and the green graph presents the MIF.

![Diagram](image)

**Figure D-5 Example of a mode identification function (MIF).**

**Stabilisation diagram**

"The stabilisation diagram is a tool used during the least squares complex exponential pole estimation process. The diagram identifies the stability of a pole as the order of the model is increased" (Kelley et al, 1996). This incremental approach starts from a model with size ‘m’, which gives ‘m’ solutions for frequency, damping and vector parameters. In the next step, the model with size ‘m+1’ gives ‘m+1’ solutions. Each pole of the ‘m+1’ model is then compared with the ones from the ‘m’ model. If the value of a frequency, damping or vector parameter is smaller than a designated tolerance value a pole solution is found. Depending on which parameter (frequency, damping or vector) falls below the tolerance, different pole stabilisations are obtained (LMS CADA-X, 1997). The pole stabilisation notation and colours used in the FRF stabilisation diagrams are shown in Table D-1. The ‘s’ poles are the ones which correspond to the real resonance phenomena for the physical system."
An example of a stabilisation diagram is shown in Figure D-6. Here, three modes were clearly detected as being stable for frequency, modal damping and pole vector. Even the second mode, which is shown as a very small bump in the FRF, is clearly determined to be a stable mode.

For both the MIF and stabilisation diagrams the amplitude (vertical axis) describes the ratio between the response acceleration (m/s²) and the input force (N). The absolute values of the amplitude are irrelevant, as they do not influence the modal parameters: natural frequencies, damping ratios and mode shapes. (These modal parameters have been automatically calculated from the FRFs by the LMS software.) The relative values (within each FRF graph), however, are important because they provide information about the dominant response of the modes. Therefore, the pattern of the graphs and frequencies detected are the main focus of the FRF graphs.

Minor fluctuations in the graphs may be attributed to digital signal processing errors (leakage and aliasing), noise (from equipment, such as power supply, cables, etc), calibration or operation error (complete system calibration, transducer calibration) and data processing errors (modal analysis). This is especially evident in the results from the shake table testing, seen later.
Modal assurance criterion (MAC)

The modal assurance criterion is commonly used to assess the degree of correlation between any two vectors (Kelley et al., 1996), and is formulated using Equation D-5:

$$MAC\left(\phi_q, \phi_r\right) = \frac{|\phi_q^T \phi_q^T|^2}{\left|\phi_q^T \phi_q^T \phi_q^T \phi_r^T\right|}$$  \hspace{1cm} \text{D-5}

Where:

- $\phi_q$ = mode shape vector for mode $q$ of dataset A
- $\phi_r$ = mode shape vector for mode $r$ of dataset B

A MAC value close to 1.0 (100%) indicates that two modes are well correlated and a value close to 0 (0%) is indicative of uncorrelated modes. In this research the MAC value was used to compare the mode shapes at different stages of damage.
D.3.3 Modal parameters: natural frequencies, damping and mode shapes

The modal parameters (natural frequencies, modal damping ratios and mode shapes) are determined by analysing the response of a structure to an excitation, via EMTA. Each structure can be represented by a discrete parameter model as illustrated for a single-degree-of-freedom (SDOF) system in Figure D-7.

![Figure D-7 SDOF discrete parameter model.](image)

"The idealised elements are mass, spring, damper and excitation. The first three elements describe the physical system. Energy is stored by the system in the mass and the spring in the form of kinetic and potential energy, respectively. Energy enters the system through excitation and is dissipated through damping" (Agilent Technologies, 2000). The idealised elements of the physical system can be described by the equation of motion (Equation D-6):

\[
m\ddot{u}(t) + c\dot{u}(t) + ku(t) = F(t)
\]

\[D-6\]

Where:

- \(m\) = mass
- \(c\) = damping coefficient
- \(k\) = stiffness
- \(u(t)\) = displacement
- \(\dot{u}(t)\) = velocity
- \(\ddot{u}(t)\) = acceleration
- \(F(t)\) = external force
This equation relates the effects of mass, stiffness and damping, and leads to the calculation of natural frequency and damping ratio, as shown in Equations D-7 and D-8, respectively (Agilent Technologies, 2000):

\[ \omega^2 = \frac{k}{m} \] \hspace{1cm} \text{D-7}

\[ \xi = \frac{c}{\sqrt{2km}} \] \hspace{1cm} \text{D-8}

Where:
- \( \omega \) = natural frequency (rad/sec)
- \( \xi \) = damping ratio
- \( m \) = mass
- \( c \) = damping coefficient
- \( k \) = stiffness

For a linear multi-degree-of-freedom (MDOF) system, the number of natural frequencies is equal to the number of degrees-of-freedom. Each natural frequency is associated with a unique mode shape and a modal damping ratio (Fertis, 1995).

**Natural frequency**

If a system is excited with a force that coincides with one of the natural frequencies of the system, resonance occurs and the amplitude of vibration will approach infinity with time in the absence of damping. In practice, however, the amplitude of vibration may exceed allowable values in a short period of time, with the subsequent loss of structural integrity (damage to the structure) (Fertis, 1995). The natural frequency can be formulated as circular frequency \( \omega \) [rad/s] that describes the number of vibration cycles per \( 2\pi \) seconds, or as cyclic frequency \( f \) [cycles/s = Hz] that describes the number of vibration cycles per second. The natural frequency is also called resonance frequency, characteristic frequency or normal frequency.
For the analysis of the u-panels, the first three modes (Mode 1, Mode 2 and Mode 3) were examined in greater detail. The mode numbers describe the order of the modal frequencies. Subsequently, the first mode (Mode 1) has the lowest frequency, the second mode (Mode 2) has the next lowest, and so on. (Tolles and Krawinkler, 1990).

**Damping**

Damping (ζ) is a force that counteracts the motion of a system by dissipating energy. The energy dissipation equals the work done by the damping force. In the case of free vibration the presence of damping results in a continuous decay of the force amplitude. It is present in all practical systems and is, among other causes, due to the hysteresis properties of the materials from which they are made. Another common source of energy dissipation in structures, and thus of their damping, is the friction which exists in joints between components of the structure. These effects may be macro slip between adjacent parts or, more commonly, micro slip in the areas of connection between them. Figure D-8 shows different types of damping.

![Damping Diagram](image)

**Figure D-8 Different types of damping (Bachmann, 1995).**

The damping capacity can be defined by the ratio shown in Equation D-9 (Ewins, 2000):

\[
\text{Damping capacity} = \frac{\text{Energy loss per cycle}}{\text{Maximum energy stored}} \quad \text{D-9}
\]
The damping ratio can also be represented as a percentage of critical damping – the damping level at which the system experiences no oscillation. This is the more common understanding of modal damping. Although there are three distinct damping cases (underdamped, critically damped and overdamped conditions), only the underdamped case (\(\xi<1\)) is important for structural dynamics applications. A system with underdamped conditions oscillates around the zero-crossing while the maximum amplitude constantly decays (Kappos, 2002).

Although the theory on damping is well understood for simple linear structures, little is known about the real damping mechanisms in complex structures (Carr, 2005). The estimation of the damping ratios of the tested u-panels with the LMS software is not very robust or reliable and, therefore, questionable. This is due to the noise pollution of the data, especially from the shake table testing, plus the complexity of the materials and structural interactions. In the analysis of the u-panels, damping has been presented for the sake of completeness, although not examined in greater detail.

Mode shapes
A mode shape is a deflection-pattern associated with a particular modal frequency. It represents the relative displacements of all the parts of a structure for that particular mode. Normal mode shapes are characterised by the fact that all parts of the structure are moving either in-phase, or 180° out-of-phase, with each other. They can be thought of as standing waves with fixed node lines. (Døssing, 1988)

The LMS software generates the mode shape diagrams in one of two phases mentioned above (\(\alpha\) and \(\beta\)). The polarity of the mode shapes is irrelevant in this case, and the different phases of the mode shapes represent the same mode.

For the analysis of the u-panels in this project the first three modes were considered. They refer to the ‘first transverse translation’ mode (Mode 1), the ‘first torsion’ mode (Mode 2) and the ‘second transverse translation’ mode (Mode 3), as defined by Tolles and Krawinkler (1990), and shown in Figure D-9. (The work by Tolles and Krawinkler (1990) was undertaken on model adobe houses, however the modes shown in Figure D-9 match those identified during the EMTA of the u-panels in this research project,
thus the descriptions and titles proposed by Tolles and Krawinkler have been adopted in this project also.] The first transverse translation mode clearly matches the flexural response of the u-panels, as presented and discussed in Chapter 7.

<table>
<thead>
<tr>
<th>First Transverse Translation Mode</th>
<th>First Torsion Mode</th>
<th>Second Transverse Translation Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Mode 1)</td>
<td>(Mode 2)</td>
<td>(Mode 3)</td>
</tr>
</tbody>
</table>

(a) Tolles

(b) Phase α

(c) Phase β

Figure D-9 Mode shapes and descriptions for Modes 1, 2 and 3 in different phases (α + β).
D.4 **EMTA of undamaged u-panels: impact hammer tests**

D.4.1 Dynamic properties of undamaged u-panels 3F, 3H, 3J & 3K

This data complements Section 8.4.2 in the main body of the thesis.

**Table D-2** Specimens 3F, 3H, 3J and 3K: natural frequencies ($f$) and modal damping ratios ($\zeta$) prior to testing.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Mode 1</th>
<th></th>
<th>Mode 2</th>
<th></th>
<th>Mode 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_1$</td>
<td>$\zeta_1$</td>
<td>$f_2$</td>
<td>$\zeta_2$</td>
<td>$f_3$</td>
</tr>
<tr>
<td>3F</td>
<td>33.70</td>
<td>1.01</td>
<td>NC</td>
<td>NC</td>
<td>61.05</td>
</tr>
<tr>
<td>3H</td>
<td>33.33</td>
<td>1.19</td>
<td>47.91</td>
<td>0.30</td>
<td>55.99</td>
</tr>
<tr>
<td>3J</td>
<td>33.82</td>
<td>1.08</td>
<td>42.19</td>
<td>0.44</td>
<td>59.98</td>
</tr>
<tr>
<td>3K</td>
<td>26.95</td>
<td>0.19</td>
<td>35.48</td>
<td>0.13</td>
<td>48.85</td>
</tr>
</tbody>
</table>

*Note:* NC — not captured. See discussion in Chapter 8 for explanation.

[The orange arrows in the FRF graphs indicate the locations of the identified modes]

![Figure D-10 Specimen 3F: dynamic characteristics prior to shake table testing.](image)
Figure D-11 Specimen 3H: dynamic characteristics prior to shake table testing.

Figure D-12 Specimen 3J: dynamic characteristics prior to shake table testing.
Figure D-13 Specimen 3K: dynamic characteristics prior to shake table testing.
D.4.2 Dynamic properties during strengthening of u-panel 3J

This data complements Section 8.4.3 in the main body of the thesis.

Table D-3 Specimen 3J: natural frequencies (f) and damping ratios (ζ) during strengthening process.

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Mode 1</th>
<th>Mode 2</th>
<th>Mode 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>f₁ (Hz)</td>
<td>ζ₁ (%)</td>
<td>f₂ (Hz)</td>
</tr>
<tr>
<td>3J - ‘a’ (internal horiz. mesh)</td>
<td>29.99</td>
<td>2.74</td>
<td>54.84</td>
</tr>
<tr>
<td>3J - ‘b’ (mesh + wing wall restraint)</td>
<td>33.60</td>
<td>1.40</td>
<td>42.70</td>
</tr>
<tr>
<td>3J - ‘c’ (mesh + restraint + ring beam)</td>
<td>33.62</td>
<td>1.27</td>
<td>42.40</td>
</tr>
<tr>
<td>3J - ‘d’ (mesh + restraint + ring beam +</td>
<td>33.82</td>
<td>1.08</td>
<td>42.19</td>
</tr>
<tr>
<td>external bamboo + wire)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

[The orange arrows in the FRF graphs indicate the locations of the identified modes]

Figure D-14 Specimen 3J: characteristics at strengthening stage ‘a’ (mesh only).
Figure D-15 Specimen 3J: characteristics at strengthening stage ‘b’ (+ restraint).

Figure D-16 Specimen 3J: characteristics at strengthening stage ‘c’ (+ ring beam).
Figure D-17 Specimen 3J: characteristics at strengthening stage ‘d’ (+ bamboo + wire).
D.5  **EMTA of u-panels 3H, 3J & 3K: shake table testing**

This data complements Section 8.5 in the main body of the thesis.

D.5.1  **Specimen 3H**

Table D-4  Specimen 3H: frequencies (f) and damping (ζ) for shake table simulations.

<table>
<thead>
<tr>
<th>Simulation</th>
<th>Mode 1</th>
<th>Mode 2</th>
<th>Mode 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>f_1 (Hz)</td>
<td>ζ_1 (%)</td>
<td>f_2 (Hz)</td>
</tr>
<tr>
<td>S1 (U40%)</td>
<td>39.54</td>
<td>0.31</td>
<td>52.26</td>
</tr>
<tr>
<td>S2 (U100%)</td>
<td>38.08</td>
<td>0.97</td>
<td>52.69</td>
</tr>
<tr>
<td>S3 (U150%)</td>
<td>37.55</td>
<td>0.21</td>
<td>52.59</td>
</tr>
<tr>
<td>S4 (S20%)</td>
<td>37.28</td>
<td>0.35</td>
<td>52.15</td>
</tr>
<tr>
<td>S5 (S50%)</td>
<td>36.48</td>
<td>0.88</td>
<td>50.96</td>
</tr>
<tr>
<td>S6 (S75%)</td>
<td>29.18</td>
<td>0.08</td>
<td>NC</td>
</tr>
<tr>
<td>S7 (S100%)</td>
<td>14.81</td>
<td>1.38</td>
<td>NC</td>
</tr>
<tr>
<td>S8 (S125%)</td>
<td>4.30</td>
<td>NC</td>
<td>NC</td>
</tr>
</tbody>
</table>

Note: NC – not captured. See discussion in Chapter 8 for explanation.

Figure D-18  Specimen 3H: frequency trend of Modes 1, 2 and 3 for shake table tests.
Figure D-19  Specimen 3H: dynamic characteristics for simulation S1 (U40%).

Figure D-20  Specimen 3H: dynamic characteristics for simulation S2 (U100%).
Figure D-21 Specimen 3H: dynamic characteristics for simulation S3 (U150%).

Figure D-22 Specimen 3H: dynamic characteristics for simulation S4 (S20%).
Figure D-23 Specimen 3H: dynamic characteristics for simulation S5 (S50%).

Figure D-24 Specimen 3H: dynamic characteristics for simulation S6 (S75%).
Figure D-25 Specimen 3H: dynamic characteristics for simulation S7 (S100%).

Figure D-26 Specimen 3H: dynamic characteristics for simulation S8 (S125%).
D.5.2 Specimen 3J

Table D-5 Specimen 3J: frequencies ($f$) and damping ($\zeta$) for shake table simulations.

<table>
<thead>
<tr>
<th>Simulation</th>
<th>Mode 1</th>
<th></th>
<th>Mode 2</th>
<th></th>
<th>Mode 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_1$ (Hz)</td>
<td>$\zeta_1$ (%)</td>
<td>$f_2$ (Hz)</td>
<td>$\zeta_2$ (%)</td>
<td>$f_3$ (Hz)</td>
</tr>
<tr>
<td>S1 (U40%)*</td>
<td>NR</td>
<td>NR</td>
<td>NR</td>
<td>NR</td>
<td>NR</td>
</tr>
<tr>
<td>S2 (U100%)</td>
<td>38.40</td>
<td>0.17</td>
<td>41.27</td>
<td>0.41</td>
<td>59.39</td>
</tr>
<tr>
<td>S3 (U150%)</td>
<td>35.22</td>
<td>0.29</td>
<td>41.00</td>
<td>0.40</td>
<td>59.76</td>
</tr>
<tr>
<td>S4 (S20%)</td>
<td>35.04</td>
<td>0.18</td>
<td>41.23</td>
<td>0.48</td>
<td>62.86</td>
</tr>
<tr>
<td>S5 (S50%)</td>
<td>34.77</td>
<td>0.84</td>
<td>41.49</td>
<td>0.12</td>
<td>59.53</td>
</tr>
<tr>
<td>S6 (S75%)</td>
<td>32.58</td>
<td>0.20</td>
<td>NC</td>
<td>NC</td>
<td>53.13</td>
</tr>
<tr>
<td>S7 (S100%)</td>
<td>14.35</td>
<td>1.34</td>
<td>NC</td>
<td>NC</td>
<td>34.76</td>
</tr>
<tr>
<td>S8 (S125%)</td>
<td>9.88</td>
<td>NC</td>
<td>NC</td>
<td>NC</td>
<td>NC</td>
</tr>
</tbody>
</table>

Notes: NR = Not recorded. NC = Not captured.

Figure D-27 Specimen 3J: frequency trends of Modes 1, 2 and 3 for shake table tests.

[The orange arrows in the FRF graphs indicate the locations of the identified modes]
Figure D-28 Specimen 3J: dynamic characteristics for simulation S2 (U100%).

Figure D-29 Specimen 3J: dynamic characteristics for simulation S3 (U150%).
Figure D-30 Specimen 3J: dynamic characteristics for simulation S4 (S20%).

Figure D-31 Specimen 3J: dynamic characteristics for simulation S5 (S50%).
Figure D-32 Specimen 3J: dynamic characteristics for simulation S6 (S75%).

Figure D-33 Specimen 3J: dynamic characteristics for simulation S7 (S100%).
Figure D-34 Specimen 3J: dynamic characteristics for simulation S8 (S125%).
D.5.3 Specimen 3K

Table D-6 Specimen 3K: frequencies (f) and damping (ζ) for shake table simulations.

<table>
<thead>
<tr>
<th>Simulation</th>
<th>Mode 1</th>
<th>Mode 2</th>
<th>Mode 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>f₁ (Hz)</td>
<td>ζ₁ (%)</td>
<td>f₂ (Hz)</td>
</tr>
<tr>
<td>S4 (S20%)</td>
<td>27.41</td>
<td>1.45</td>
<td>35.87</td>
</tr>
<tr>
<td>S5 (S50%)</td>
<td>20.77</td>
<td>1.59</td>
<td>NC</td>
</tr>
<tr>
<td>S6 (S75%)</td>
<td>12.20</td>
<td>1.77</td>
<td>NC</td>
</tr>
<tr>
<td>S7 (S100%)</td>
<td>8.16</td>
<td>2.80</td>
<td>NC</td>
</tr>
<tr>
<td>S8 (S125%)</td>
<td>6.28</td>
<td>NC</td>
<td>NC</td>
</tr>
</tbody>
</table>

Notes:
* For specimen 3K simulations S1 – S3 were not undertaken, as explained in Chapter 6.
  NC = Not captured.

Figure D-35 Specimen 3K: frequency trends of Modes 1, 2 and 3 for shake table tests.

[The orange arrows in the FRF graphs indicate the locations of the identified modes]
Figure D-36 Specimen 3K: dynamic characteristics for simulation S4 (S20%).

Figure D-37 Specimen 3K: dynamic characteristics for simulation S5 (S50%).
Figure D-38  Specimen 3K: dynamic characteristics for simulation S6 (S75%).

Figure D-39  Specimen 3K: dynamic characteristics for simulation S7 (S100%).
Figure D-40 Specimen 3K: dynamic characteristics for simulation S8 (S125%).
D.6  EMTA of model house 4A: impact hammer tests

This data complements Section 10.2 in the main body of the thesis.

There is no additional information for the EMTA undertaken during the shake table testing.
Table D-7 Model house 4A: frequencies ($f$) and damping ($\zeta$) during the strengthening process.

<table>
<thead>
<tr>
<th>Configuration</th>
<th>(Wall)</th>
<th>Impact point</th>
<th>Mode 1</th>
<th>Mode 2</th>
<th>Mode 3</th>
<th>Mode 4</th>
<th>Mode 5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$f_1$ (Hz)</td>
<td>$\zeta_1$ (%)</td>
<td>$f_2$ (Hz)</td>
<td>$\zeta_2$ (%)</td>
<td>$f_3$ (Hz)</td>
</tr>
<tr>
<td>4A - a</td>
<td>North</td>
<td>A</td>
<td>23.98</td>
<td>0.91</td>
<td>30.49</td>
<td>1.31</td>
<td>32.18</td>
</tr>
<tr>
<td>(unreinforced)</td>
<td>B</td>
<td>24.02</td>
<td>0.96</td>
<td>30.34</td>
<td>1.31</td>
<td>32.12</td>
<td>1.09</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C</td>
<td>24.02</td>
<td>-</td>
<td>30.54</td>
<td>1.36</td>
<td>31.95</td>
</tr>
<tr>
<td></td>
<td>South</td>
<td>D</td>
<td>24.21</td>
<td>0.92</td>
<td>30.42</td>
<td>1.31</td>
<td>32.23</td>
</tr>
<tr>
<td>4A - b</td>
<td>North</td>
<td>A</td>
<td>24.13</td>
<td>1.14</td>
<td>31.63</td>
<td>0.28</td>
<td>37.11</td>
</tr>
<tr>
<td>(+ ring beam + string)</td>
<td>B</td>
<td>24.25</td>
<td>1.09</td>
<td>31.43</td>
<td>1.81</td>
<td>37.56</td>
<td>1.86</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C</td>
<td>24.41</td>
<td>0.98</td>
<td>31.68</td>
<td>0.74</td>
<td>37.06</td>
</tr>
<tr>
<td></td>
<td>South</td>
<td>D</td>
<td>24.28</td>
<td>1.08</td>
<td>31.65</td>
<td>0.85</td>
<td>37.22</td>
</tr>
<tr>
<td>4A - c</td>
<td>North</td>
<td>A</td>
<td>24.23</td>
<td>0.64</td>
<td>31.33</td>
<td>0.86</td>
<td>35.20</td>
</tr>
<tr>
<td>(+ ring beam + string + bamboo + wire)</td>
<td>B</td>
<td>24.07</td>
<td>0.75</td>
<td>31.31</td>
<td>1.26</td>
<td>36.23</td>
<td>1.96</td>
</tr>
<tr>
<td>[Prior to shake table testing]</td>
<td></td>
<td>C</td>
<td>24.06</td>
<td>0.84</td>
<td>31.08</td>
<td>1.24</td>
<td>36.59</td>
</tr>
<tr>
<td></td>
<td>South</td>
<td>D</td>
<td>24.09</td>
<td>1.04</td>
<td>31.26</td>
<td>0.83</td>
<td>35.04</td>
</tr>
</tbody>
</table>

Note: NC = Not captured.
Figure D-41 Model house 4A, north wall – impact point A: dynamic characteristics at strengthening stage ‘a’ (unreinforced).

Figure D-42 Model house 4A, north wall – impact point B: dynamic characteristics at strengthening stage ‘a’ (unreinforced).
Figure D-43 Model house 4A, south wall – impact point C: dynamic characteristics at strengthening stage 'a' (unreinforced).

Figure D-44 Model house 4A, south wall – impact point D: dynamic characteristics at strengthening stage 'a' (unreinforced).
Figure D-45 Model house 4A, north wall– impact point A: dynamic characteristics at strengthening stage ‘b’ (+ ring beam + string).

Figure D-46 Model house 4A, north wall– impact point B: dynamic characteristics at strengthening stage ‘b’ (+ ring beam + string).
Figure D-47  Model house 4A, south wall—impact point C: dynamic characteristics at strengthening stage ‘b’ (+ ring beam + string).

Figure D-48  Model house 4A, south wall—impact point D: dynamic characteristics at strengthening stage ‘b’ (+ ring beam + string).
Figure D-49 Model house 4A, north wall – impact point A: dynamic characteristics at strengthening stage ‘c’ (+ ring beam + string + bamboo + wire).

Figure D-50 Model house 4A, north wall – impact point B: dynamic characteristics at strengthening stage ‘c’ (+ ring beam + string + bamboo + wire).
Figure D-51 Model house 4A, south wall – impact point C: dynamic characteristics at strengthening stage ‘c’ (+ ring beam + string + bamboo + wire).

Figure D-52 Model house 4A, south wall – impact point D: dynamic characteristics at strengthening stage ‘c’ (+ ring beam + string + bamboo + wire).
# APPENDIX E. CDs AND DVDs

<table>
<thead>
<tr>
<th>Disc</th>
<th>CD/DVD</th>
<th>Contents</th>
<th>Appendix</th>
</tr>
</thead>
<tbody>
<tr>
<td>Disc 1</td>
<td>CD</td>
<td>Brick fabrication</td>
<td>A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>U-panel 3J (construction)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>U-panel 3K (construction)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Model house 4A (construction)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Model house 4A (strengthening)</td>
<td></td>
</tr>
<tr>
<td>Disc 2</td>
<td>CD</td>
<td>U-panels 3A – 3K (Best of tests)</td>
<td>A</td>
</tr>
<tr>
<td>Disc 3</td>
<td>CD</td>
<td>Model house 4A (Best of tests)</td>
<td>A</td>
</tr>
<tr>
<td>Disc 4</td>
<td>DVD</td>
<td>U-panels 3A – 3I (All tests)</td>
<td>A</td>
</tr>
<tr>
<td>Disc 5</td>
<td>DVD</td>
<td>U-panels 3J + 3K, Model house 4A (All tests)</td>
<td>A</td>
</tr>
<tr>
<td>Disc 6</td>
<td>DVD</td>
<td>ABC Catalyst (TV)</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ABC New Inventors (TV)</td>
<td></td>
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<tr>
<td>Disc 7</td>
<td>CD</td>
<td>Radio interviews</td>
<td>B</td>
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</table>