Proceedings of the Getty Seismic Adobe Project 2006 Colloquium

The Getty Conservation Institute

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Getty Center
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Front cover: Examples of earthen sites from around the world.
Clockwise from upper left: Cathedral of Ica after the 2007 earthquake,
Ica, Peru; Aït-Benhaddoud, Ouarzazate province, Morocco;
La Purisima Mission State Historic Park, Lompoc, California, USA;
Casa Riva Agüero, Lima, Peru; Hakka clan houses, Fujian, China.
Photos: Claudia Cancino, Gail Ostergren, and Neville Agnew.

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The Getty Conservation Institute works internationally to advance conservation practice in the visual arts—broadly interpreted to include objects, collections, architecture, and sites. The GCI serves the conservation community through scientific research, education and training, model field projects, and the dissemination of the results of both its own work and the work of others in the field. In all its endeavors, the GCI focuses on the creation and delivery of knowledge that will benefit the professionals and organizations responsible for the conservation of the world's cultural heritage.

Contents

Foreword

It is my pleasure to present the *Proceedings of the Getty Seismic Adobe Project 2006 Colloquium*. The GCI's commitment to the preservation of earthen architectural heritage worldwide has generated training programs, conferences and symposia, research, and field projects that have deepened the understanding of earthen architecture and its particular vulnerabilities and have explored new strategies for its conservation. Research and laboratory testing carried out in the 1990s under the Getty Seismic Adobe Project (GSAP) have advanced the understanding of how adobe buildings perform during earthquakes and have led to the development of a set of protective measures that could be taken to minimize earthquake damage to these structures.

In recent years, destructive earthquakes in regions with significant earthen architectural heritage—particularly the earthquakes in Bam, Iran (2003), and Al-Hoceima, Morocco (2004)—focused renewed attention on the vulnerability of earthen structures during earthquakes. The GCI took particular interest in the outcome of these natural disasters as it sought to evaluate the impact of the GSAP research and to understand why seismic stabilization has not been widely adopted to protect earthen buildings located in seismic zones. In April 2006, the GCI convened a colloquium at the Getty Center in Los Angeles to address these issues.

The Getty Seismic Adobe Project 2006 Colloquium brought together an interdisciplinary group of sixty specialists from around the world to discuss traditional seismic-resistant building techniques and modern retrofit methods appropriate for historic and new earthen buildings. The colloquium provided a forum for the presentation of recent work and for in-depth discussion of key issues and future research needs. The meeting offered the GCI an opportunity to gauge the effects of GSAP principles on the field of seismic retrofitting of historic earthen structures and to discuss where and how the GSAP guidelines have been implemented outside the state of California. It also allowed participants to articulate factors that may have prevented wider acceptance and application of these guidelines. The ultimate goal of the colloquium was to identify gaps in our current knowledge, as well as further areas of research to address these gaps.

Much of the world's earthen architecture remains vulnerable to seismic damage. The implementation of existing traditional construction practices and modern retrofit methods, such as those developed by GSAP, can greatly improve the capacity of earthen buildings to withstand earthquakes. The challenge to the conservation community is to disseminate this vital information to those entrusted with safeguarding earthen heritage around the world and to support efforts to implement these practices before another devastating earthquake. We hope that the publication of the *Proceedings of the Getty Seismic Adobe Project 2006 Colloquium* is a step in this direction.

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Introduction

By Mary Hardy, Claudia Cancino, and Gail Ostergren

Across the ages and around the globe, people have constructed buildings of earth. Our earthen architectural heritage is rich and varied—ranging from ancient archaeological sites to living cities, the vernacular to the monumental, individual buildings to large complexes—and the challenges of preserving this precious legacy are equally diverse. Earthen construction is widespread in many seismic regions of the world; consequently, the vulnerability of earthen structures to damage or destruction by earthquake is of great concern. For many years, the Getty Conservation Institute (GCI) has taken a leading role in setting the standards for proper methodologies for the conservation of earthen sites, including seismic retrofitting.

Earth is a nonductile material, and structures built of earth are unable to withstand the tremendous lateral loads imposed by strong earthquakes unless they are properly constructed with walls sufficiently thick to avoid overturning, or are reinforced in a manner that adds tensile strength to the structure. Historic structures that have been subjected to inappropriate interventions or that have been abandoned through time are in particular danger of collapse during an earthquake. To address this issue, the GCI conceived the Getty Seismic Adobe Project (GSAP, 1990–2002), a multidisciplinary research effort that designed, tested, and advocated less-invasive, stability-based retrofit programs for historic earthen structures located in seismic regions.

GSAP addressed the vulnerability of unreinforced historic adobe buildings by analyzing their structural properties and proposing technologically feasible and minimally invasive retrofitting techniques. The project proposed stability-based retrofit interventions that enhance existing structural properties of historic adobe structures, such as the thickness of their existing walls. This enhancement is achieved by anchoring together the roof, walls, and floors; by adding flexible bond beams at the top of the walls; and by applying flexible straps to both sides of the wall—or, alternatively, by placing small-diameter vertical rods in the centers of walls (center coring). These measures are far less invasive than the more commonly used strength-based retrofit methods, which introduce independent structural frames of reinforced concrete or steel and require the removal of large amounts of historic material.

The work developed during GSAP is documented in three GCI publications: *Survey of Damage to Historic Adobe Buildings after the January 1994 Northridge Earthquake* (1996); *Seismic Stabilization of Historic Adobe Structures: Final Report of the Getty Seismic Adobe Project* (2000); and *Planning and Engineering Guidelines for the Seismic Retrofitting of Historic Adobe Structures* (2002), which are available in PDF format on the GCI's Web site (http://www.getty.edu/conservation/). A Spanish translation of the final volume and a brief video of the GSAP seismic shake table testing program are also available on the Web site.

The Colloquium

The Getty Seismic Adobe Project 2006 Colloquium was held at the Getty Center on April 11–13, 2006. The meeting brought together a group of professionals with expertise in earthen conservation, building standards, and earthquake engineering to discuss the current state of knowledge and the challenges of preserving our earthen cultural heritage in active seismic zones. The colloquium was primarily an opportunity to evaluate the impact that the GSAP research and guidelines have had on the field locally and internationally and to discuss the feasibility of implementing the GSAP guidelines in other contexts. It also allowed for the exchange of information and the prioritization of future work in the field of retrofitting historic earthen sites.

The three-day program, which included formal talks, panel discussions, and site visits to several local retrofit projects, was designed to provide maximum opportunity for information exchange among participants representing different disciplines. Formal sessions addressed stability-based earthquake resistant design, appropriate testing methods, and building codes and standards specific to earthen architecture. The case studies presented included examples of traditional earthquake resistant design and repair techniques from around the world, as well as recent retrofit projects. On the final day, roundtables and panel discussions were designed to help participants identify the best steps to further advance the field in the areas of earthen building codes, research and testing for seismic retrofitting, and information dissemination and training. An optional, two-day, post-colloquium tour took participants to nine historic Southern California adobe sites, where they viewed a variety of architectural typologies and examined a range of retrofitting techniques.

Further Dissemination

In order to disseminate the outcomes of the GSAP colloquium more broadly and to raise local awareness of the GSAP guidelines, the GCI sponsored a symposium, "New Concepts in Seismic Strengthening of Historic Adobe Structures," in September 2006. Developed in partnership with the California Preservation Foundation, the California State Office of Historic Preservation, and US/ICOMOS, the symposium was directed at practitioners, managers of historic properties, and government officials responsible for developing and enforcing public safety regulations and building codes. The two-day event, which was attended by more than seventy individuals, took place at the Getty Center and at Rancho Camulos in Ventura County. A well-attended public lecture by architectural historian Stephen Tobriner, entitled "The Quest for Earthquake Resistant Construction in Europe and the Americas, 1726–1908," introduced the topic to the general public.

About This Publication

Much of the value of the GSAP colloquium was a consequence of the discussions and interactions among participants representing many different professional disciplines. This publication is an effort to record and share the core content of these exchanges. It contains a selection of papers presented during the event, as well as several subsequent submissions by colloquium participants. Its intent is to reflect developments in the field of retrofitting historic earthen structures, to document successfully retrofitted sites, and to present colloquium conclusions designed to advance the field. The colloquium papers presented here have been organized into four sections: Research and Testing, Building Codes and Standards, Case Studies, and GSAP Implementation. The decision to make these proceedings a Web-based publication recognizes the need to disseminate this important information as widely as possible in order to safeguard the greatest number of earthen structures and the lives of their inhabitants.

Part one includes four papers on the research and testing of appropriate retrofitting methods and materials for extant earthen structures, as well as seismic design criteria and materials for new earthen construction. Perhaps the most active research institution for seismically resistant new earthen buildings is the School of Engineering at the Catholic University of Peru (PUCP) in Lima. Here, PUCP researchers present the static and dynamic testing programs carried out there over the past thirty years, including the effective use of polymer mesh as an external reinforcement technique.

The added cost of reinforcing new construction or retrofitting existing earthen buildings is a serious deterrent in many of the seismically active regions where earthen architecture is abundant. Papers from researchers at Saitama University in Japan and from Australia's University of Technology, Sydney, address this problem. The first identifies low-cost methods for strengthening adobe blocks, while the second reviews the design and dynamic testing of low-cost and low-tech methods for reinforcing new and existing earthen buildings using alternative and locally available materials. It should be emphasized that the materials, costs, and techniques designed to allow for minimal intervention and loss of fabric in historically significant earthen buildings are different from those for new or existing, nonhistoric vernacular earthen structures.

This section concludes with a summary of the GSAP project, presented by the project's primary research engineer. The paper recaps the shake table testing program and explains the stability-based retrofit measures developed as part of the project.

The papers in part two highlight the importance of incorporating earthen materials and building techniques into building codes and standards. These legal codes establish earth as a legitimate construction material and serve as specific enforceable guidelines that help assure building safety in seismic regions while preserving existing historic structures. Codes can be important educational devices, instructing those who would build using earthen materials, as well as those who inspect and enforce the code, about proper design, construction, and maintenance of such structures.

The first three papers in this section review the development of building codes or national standards that specifically address earthen architecture. First, researchers from PUCP describe the development and content of the highly influential Peruvian Adobe Building Code, which has informed guidelines for earthen construction in other countries. A second paper describes the New Zealand Earth Building Standards, which are organized in three volumes, each focused on specific aspects of earthen design or construction and directed at particular user groups with different needs and technical skill levels. A third paper argues for the inclusion of traditional earthen building materials and techniques in the Moroccan seismic building code; currently pending Moroccan codes require the use of steel or reinforced concrete. The final paper in this section provides an overview of *The Secretary of the Interior's Standards for the Treatment of Historic Properties*, historic structure reports, and project regulatory review processes in California and discusses the ways in which these documents are applicable to best practice in the conservation and seismic retrofit of historic adobe structures.

Part three of these proceedings comprises five case studies that discuss structural interventions to enhance earthquake resistance at historic sites in a number of countries. Throughout history, the designers and builders of earthen structures in seismic regions have exhibited a remarkable understanding of earthquake forces and intuitive structural design solutions. The damage inflicted by each past major earthquake has increased the understanding of how buildings behave under seismic loads. Well-designed historic interventions, such as buttresses constructed following an earthquake, instruct us on the ways in which traditional structural reinforcement improves the ability of earthen sites to withstand future earthquakes.

Since severe and destructive earthquakes occur infrequently, the cumulative memory of lessons learned over generations is often lost as regional building traditions are modified, eliminated, or forgotten. The first three papers of this section are investigations into the continuing use of traditional and historic construction techniques that improve the overall seismic performance of structures in Turkey, Central Asia, Trans-Himalaya, Western China, and India.

The last two papers in this section highlight the importance of conducting thorough structural studies in order to prioritize areas of intervention and to facilitate high-quality design for retrofit proposals. In the first, a study of the historic earthen architecture of the Kathmandu Valley in Nepal, the relationship between construction details and the buildings' seismic vulnerability was analyzed in order to devise suitable strengthening strategies to reduce seismic risk. In the final paper in this section, preliminary guidelines based upon structural assessments were designed for the seismic retrofitting of seventeenth-century earthen churches located in the central Andes of Peru.

Part four of this publication is dedicated to the implementation of the GSAP guidelines at nine historic California adobe sites. The three papers detail plans addressing different retrofitting challenges and architectural typologies at sites as varied as Rancho Camulos, Mission San Miguel, and the Las Flores Adobe. The retrofit designs are described, along with the rationales behind the designs and the selection of materials for implementation. Most important, the papers speak to the difficult challenge of simultaneously meeting engineering requirements and conservation principles.

These proceedings conclude with a summary of discussions detailing the colloquium participants' recommendations for advancement of the field. As a final remark, we wish to emphasize that the GSAP colloquium provided the opportunity for a multidisciplinary group of professionals to compare experiences and discuss the state of the art of the conservation of historic earthen structures located in seismic areas, and for the GCI to assess the implementation of the GSAP guidelines in the United States and abroad. The case studies in particular promoted dialogue on the suitability of the guidelines in situations where local materials and traditional techniques must be used in their implementation. The ultimate objective of this publication is to capture the content and energy of the colloquium and to point the direction toward future areas of research that will further the application of seismic retrofit techniques to preserve the world's earthen architectural heritage.

Acknowledgments

The Getty Seismic Adobe Project (GSAP) 2006 Colloquium and this volume of proceedings are the result of the efforts of a large number of individuals, whom we would like to acknowledge here. First and foremost, the editors would like to thank all of the colloquium speakers and participants for their serious work and open-minded collegiality during the three-day event. We are especially indebted to the final day's moderators—Milford Wayne Donaldson, Melvyn Green, Khalid Mosalam, and Douglas Porter—and to the rapporteurs—Stephen Farneth, Bruce King, Stefan Simon, and Jeffrey Cody—who did a tremendous job of guiding and summarizing the results of four wide-ranging roundtable discussions. The information shared throughout the colloquium and the recommendations that the group produced collectively will not only influence the GCI's future research agenda, field projects, and training programs but will guide the efforts of other institutions and individuals working to conserve the world's rich and diverse earthen architectural heritage. We also wish to thank the speakers at and participants in the "New Concepts in Seismic Strengthening of Historic Adobe Structures" symposium for their work in disseminating this information to a wider professional audience.

Site managers and staff at a number of Southern California earthen heritage sites shared generously of their time and expertise and welcomed colloquium and symposium participants for site visits. We would especially like to thank Ellen Calomiris, who hosted the entire group of colloquium participants at Rancho Los Cerritos in Long Beach, California, in April 2006, and Hillary Weireter at Rancho Camulos near Piru, California, who allowed us to use the historic adobe and grounds as the September symposium's mobile workshop site.

We are grateful to those who served on the steering committees for both the April colloquium and the September symposium. These individuals gave generously of their time and expertise to design the programs, and many also served as speakers or moderators. A complete list of steering committee members is found in appendix A. We also want to acknowledge the good work of GCI staff members Chris Seki and Virginia Horton, who helped to plan, coordinate, and implement the two events.

In addition to our thanks to the authors of the papers included in this volume, we wish to acknowledge those who worked on its publication, including Frederick A. Webster for technical reviews and William S. Ginell for technical expertise and reviews and for guidance in shaping this publication. GCI staff members Amila Ferron and Elise Yakuboff reviewed submissions and coordinated the authors and editors. Thanks also to Valerie Greathouse, Judy Santos, and Cameron Trowbridge of the GCI Information Center for their work with the reference lists, and to Angela Escobar, GCI publication specialist, who oversaw the publication process.

Finally, we are indebted to our respected colleagues who envisioned and carried out the Getty Seismic Adobe Project from 1990 to 2002. Their vision and scientific research provided the foundation for significant progress in the field of structural retrofitting and earthquake resistance of historic earthen structures and sites.

The GSAP 2006 Colloquium was dedicated to the memory of Edna Kimbro, a key member of the original

GSAP team, whose contribution to the planning and design of the 2006 colloquium was vital. Sadly, Edna died on June 26, 2005. As preservation specialist on the GSAP research team, she played an important role in developing retrofit solutions respectful of historic fabric. An admired and well-loved colleague, Edna is remembered for her enthusiasm and spirit and for the substantial contributions she made to the preservation of California's historic adobe architecture.

PART ONE

Research and Testing

Earthquake Resistant Design Criteria and Testing of Adobe Buildings at Pontificia Universidad Católica del Perú

Daniel Torrealva, Julio Vargas Neumann, and Marcial Blondet

Abstract: *Research work on the seismic resistance of earthen buildings started in Peru at the beginning of the 1970s with the occurrence of the devastating May 31, 1970, Huaraz earthquake. The Pontificia Universidad Católica del Perú (PUCP), or Catholic University of Peru, among other institutions, began a program to investigate the seismic behavior of earthen buildings using a tilt-up table where full-scale models 4 × 4 m (about 13 × 13 ft.) in plan could be built and tested. The main outcome of these initial tests was identifying the need for using continuous, compatible reinforcement inside the adobe walls, such as that provided by the choice of round and split bamboo cane as an appropriate reinforcing material. In the 1980s a shake table was installed at the PUCP Structures Laboratory for testing similar models using seismic unidirectional simulation. These dynamic tests corroborated the results obtained using the tilt-up table. In the 1990s the research program focused on reducing the vulnerability of existing buildings through the use of reinforcement techniques that could be applied externally on the wall surface—mainly welded steel mesh covered with a sand-cement stucco. Since 2003 dynamic testing on full-scale adobe models has focused on the use of polymer mesh as a reinforcement material. This appears to be compatible with earthen buildings up to high levels of seismic acceleration. Continuing work on the seismic resistance of earthen buildings carried out by the Catholic University over the last thirty-five years has provided valuable input to the seismic design criteria stated in the various Peruvian adobe building codes. In all versions of the code (1977, 1985, and 2000), the elastic criterion has been used to design for initial strength, and the limit state design concept is present in the reinforcing systems required to avoid collapse.*

Introduction

Peru is located in the Pacific Ocean Ring of Fire, where most of the world's earthquakes occur. Seismic activity has been frequent and intense in the coastal areas of Peru throughout its history. Peru is also a repository of a long tradition of earthen building construction, from pre-Inca times through the colonial period and up to the present. Many examples of monumental and vernacular earthen architecture have survived that show the degree of technical expertise of the ancient builders. At present, earthen building construction is mostly used in the rural areas, with a diminishing quality of construction either because of the workmanship or because of changes in the architectural layout, such as the imitation of modern brick masonry architecture, which has negative consequences for the building's seismic resistance. The occurrence of a number of strong earthquakes between the years 1940 and 1978 sparked a systematic research project in several Peruvian universities, among them the Catholic University, which began studies on the seismic resistance of earthen buildings in 1972. The results obtained during this continuous research period have provided invaluable input to the three versions of the Peruvian Adobe Building Code. This paper summarizes the contribution to the knowledge regarding earthen buildings obtained from the experimental research projects carried out at the PUCP.

Beginning of Research at PUCP: 1972–80

It is widely recognized that analysis of the response of earthen buildings is particularly complex when they are subjected to static testing. Because of their large mass, weakness in tension, and brittleness, it is difficult to apply concentrated loads to earthen models. The first tests carried out at the Catholic University were performed with a tilt-up table that simulated the inertial earthquake forces with the inclined component of its own weight. With this testing technique, several reinforcement procedures using wood, bamboo cane, and steel wire were tested on full-scale models (Corazao and Blondet 1973). Nevertheless, static tests were also performed on full-scale walls subjected to horizontal shear and flexure, in order to study the mechanical characteristics of adobe masonry (Blondet and Vargas 1978). The most efficient reinforcement procedure at this stage was found to be placement of whole bamboo canes in the interior of the walls at a spacing of one and a half times the wall thickness. The canes were cross tied with horizontal split canes placed every four layers. Initial monotonic tests of this reinforcement showed that this technique provided an important increase in the deformation capacity of adobe walls.

Initial Dynamic Testing: 1980–90

In 1984 the first seismic simulation tests using the unidirectional shake table of the Structures Laboratory at PUCP were performed within the framework of a cooperative project with the financial support of the United States Agency for International Development (USAID) (Vargas et al. 1984). Full-scale adobe building models without roofs, and with and without internal cane reinforcement, were tested by subjecting them to several seismic motions of increasing amplitude. The main conclusion was that in the event of a severe earthquake, the internal cane reinforcement together with a wooden ring beam located in the upper part of the wall prevents wall separation and consequent out-of-plane collapse. In a subsequent research project, models with a roof and several alternative methods of cane reinforcement, including one model reinforced with only vertical canes, were subjected to similar seismic simulation tests. It was concluded that in order to maintain the integrity of the adobe walls, both horizontal and vertical reinforcements are necessary. These tests were performed using a displacement command signal derived from the longitudinal component of the May 31, 1970, Huaraz earthquake. The signal was then filtered for low and high frequencies in order to meet the table capabilities.

Focus on the Vulnerability of Existing Houses: 1990–2000

In accordance with the International Decade for the Reduction of Natural Hazards, a joint research project between the Centro Regional de Sismología para América del Sur (CERESIS), the German Agency for International Development (GTZ), and PUCP focused experimental work on existing houses, with the objective of reducing the seismic vulnerability of earthen buildings. Natural fiber ropes, wood, chicken wire, and welded steel wire mesh placed at critical points were tried as reinforcement materials (Zegarra et al. 1997). The best solution found was the use of welded steel mesh applied on both faces of the wall, vertically at the corners and horizontally at the top of the walls, simulating columns and beams. The tests were performed on U-shaped walls to increase the number of directional effects obtained in each seismic simulation. As a practical complement to the experimental research program, rural houses in several parts of Peru were reinforced using this technique (Zegarra et al. 1999).

Dynamic Testing on Retrofitting Techniques: 2003

In 2003 a strong earthquake hit the southern part of Peru, causing extensive damage in all types of buildings. Among them, thousands of earthen houses in the coastal and Andean areas were affected. The houses retrofitted with steel mesh and sand-cement plaster in 1999 withstood the effects of this earthquake without damage, becoming a model for a reconstruction project of several hundred houses in the area (Zegarra et al. 2001). In order to corroborate the effectiveness of this reinforcement, three model houses with a geometrical layout similar to the one built in the reconstruction project were tested dynamically (Zegarra et al. 2002).

The first model (URM-01) was built without any reinforcement in order to serve as a baseline for the reinforced models. The second model was reinforced on both sides of the wall with horizontal and vertical bands of welded wire mesh protected with a cement mortar (RM-SM). Vertical bands were placed at all corners, and the horizontal band was placed at the top of the walls, simulating a ring beam. The third model (RM-RC) was similar to the previous one, but a reinforced concrete ring beam was added that was anchored to the walls with shear connectors at all corners. All models were subjected to several seismic motions of increasing intensity. The seismic performance of the unreinforced model was used to establish a relationship between the table displacement and the Modified Mercalli intensity scale (MMI).

The results showed that for strong motions, equivalent to intensity MMI = X, partial collapse and global instability are not avoided with this reinforcement technique. The reinforced mortar bands are much stiffer than the adobe walls and tend to absorb most of the seismic forces until the elastic resistance is reached and a fragile rupture occurs.

Introduction of Polymer Mesh as a Compatible Reinforcing Material: 2003–6

Since 2003 polymeric materials were used in the experimental work as an alternative for reinforcement in earthen buildings. The advantage of this material lies in the compatibility with the earthen wall deformation and its ability to provide an adequate transmission of tensile strength to the walls up to the final state. In the first experimental program (Blondet et al. 2005), I-shaped adobe walls with several reinforcing techniques were subjected to cyclic static tests. Among them, internal and external polymer mesh was used as wall reinforcement (see fig. 1).

The results showed that external polymer mesh confines the adobe wall up to high levels of horizontal displacement, allowing a great amount of energy dissipation in comparison with the unreinforced wall and with the wall reinforced with stiff steel mesh and sand-cement plaster.

In 2004 a joint project between the PUCP and the Getty Conservation Institute (GCI) aimed to corroborate dynamically the effectiveness of external compatible reinforcement using natural and industrial meshes. Two model houses with geometrical characteristics similar to the CERESIS-GTZ-PUCP project were tested with external reinforcement. One of them (RM-NM) was reinforced with natural materials using whole bamboo cane as vertical reinforcement and ropes as horizontal reinforcement (see fig. 2). The reinforcement was placed at both sides of the wall and connected with a small cabuya thread through a hole previously drilled in the wall. The second model (RM-PM100) was reinforced with a polymer mesh (geogrid) completely covering the walls on both sides. The mesh was connected with plastic thread through holes previously drilled in the walls spaced 40 cm (15.6 in.) in two orthogonal directions. In both models, mud stucco was applied to half of the structure in

FIGURE 1 Cyclic static test results.

FIGURE 2 Natural materials (RM-NM).

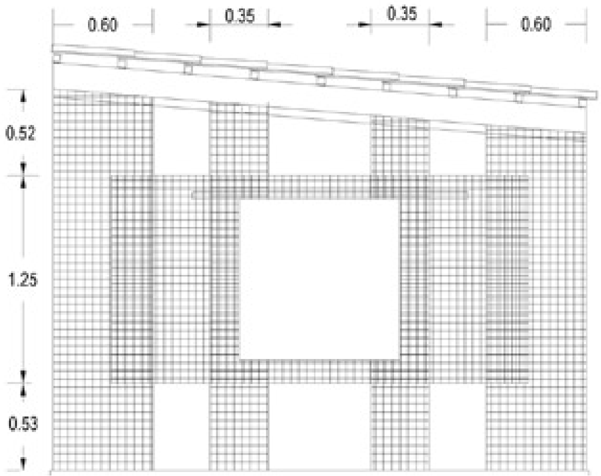

FIGURE 3 Reinforcement distribution for RM-PM75 (dimensions are given in meters).

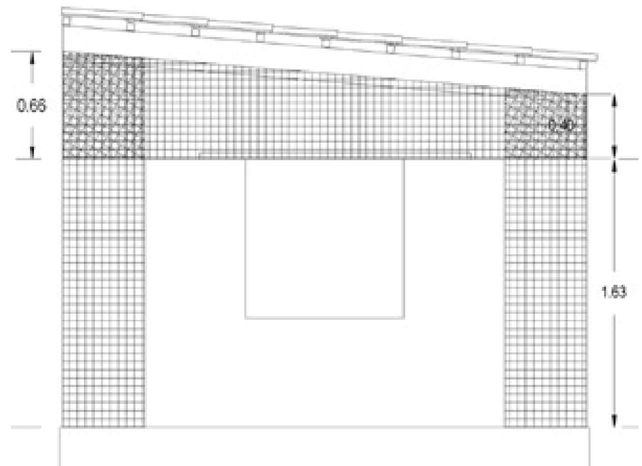

FIGURE 4 Reinforcement distribution for RM-PM50 (dimensions are given in meters).

order to study the effect of stucco on the effectiveness of reinforcement (Torrealva and Acero 2005).

The results showed that placing an external natural or industrial mesh on both sides and connecting through the thickness of the adobe wall is an effective way to avoid partial or total collapse of adobe buildings, even in severe earthquakes. If the mesh is not covered with mud stucco, the initial strength is the same as the plain, unreinforced wall, and the mesh becomes effective after the wall is cracked. After the cracking, the mesh confines the different sections into which the wall is broken, thus preventing partial or total collapse. In both cases, the mud plaster over the mesh greatly increases the initial shear strength and the stiffness of the wall, controlling the lateral displacements and preventing the cracking of the wall to a great extent. This is particularly notable in the case of the polymer mesh.

Based on these results, the polymer mesh reinforcement placed over the entire wall can be considered the upper limit of the amount of external reinforcement. The natural alternative, on the contrary, can be considered as near the lower limit of the external reinforcement, because of the bigger spacing between horizontal and vertical elements.

After the GCI-PUCP project, additional dynamic testing was performed on models with the same geometric characteristics, but with varying amounts and quality of polymer mesh, in an attempt to reduce the overall cost of the mesh technique. Three additional models were tested using the same geometric characteristics and seismic motions as in the two previous projects. Model RM-PM75 was reinforced by covering 75% of the wall surface with polymer mesh (see fig. 3), model RM-PM50 covered 50% of the wall surface (see fig. 4), and model RM-LCM was reinforced at 100% on one longitudinal wall and at 70% on the parallel wall but with a low-cost polymer mesh (see fig. 5).

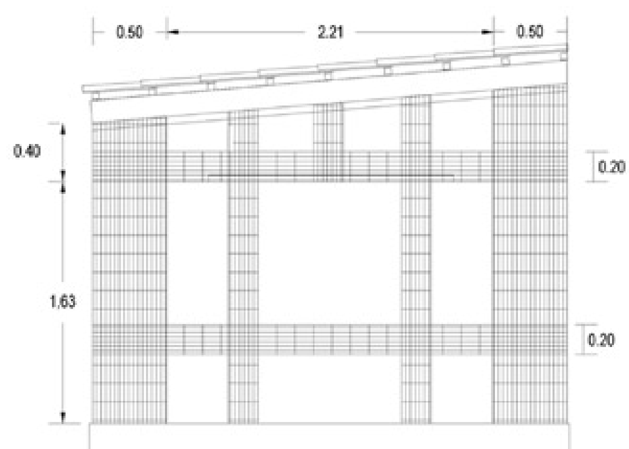

FIGURE 5 Reinforcement distribution for RM-LCM (dimensions are given in meters).

The results of this last group of dynamic tests showed that the amount of mesh placed on the walls is more important than the resistance of the mesh. The wall, fully plastered and reinforced with low-cost mesh, had a better seismic performance than the models reinforced with stronger mesh at 75%. In all cases, the testing also confirmed the beneficial effect of having the plaster cover the mesh.

Application of Polymer Mesh Reinforcement to Vaulted Models

Between December 2005 and February 2006, two vaulted models were subjected to seismic simulation tests using the same earthquake signal, for the sake of comparison with the models with traditional occidental architecture. The models were designed by the Program for the Enhancement of the Modernization of the Health Sector in Rural Areas (AMARES), a nongovernmental organization (NGO) working on implementation of health infrastructure in the Andean areas of Peru, with the technical advice of architects from the University of Kassel in Germany.

Model URV was unreinforced, and model RV-PM100 was fully reinforced with polymer mesh on both sides of the wall. The results showed that the unreinforced adobe vault was very vulnerable and collapsed at lesser motion intensity than did the unreinforced traditional houses (see fig. 6a). The fully reinforced vault, on the contrary, performed well even in the final phases of testing at the maximum acceleration intensities of table shaking (see fig. 6b).

Seismic Performance of Models

Almost all seismic simulation testing performed in the Structures Laboratory at PUCP has been done using a table command signal derived from the longitudinal component of the May 31, 1970, Peruvian earthquake. In addition to this, the last three experimental projects have tested identical models while varying the amount and type of reinforcement. This fact makes it possible to compare the seismic performances of the different models tested through the years. Table 1 shows a list of all models tested in the last four years along with their reinforcing characteristics: unreinforced models, models reinforced with welded wire mesh, and models reinforced with polymer mesh placed in several configurations.

For the purpose of comparison, a range of table-induced damage was established: ND means no damage; LD means light damage, with small cracks; HD stands for heavy damage, with large cracks and some structural instability; and C signifies total or partial collapse. The seismic performance of all these models is depicted in table 2.

(a)

(b)

FIGURES 6A AND 6B Test results showing the collapse of unreinforced model URV (a); the fully reinforced vault RV-PM100 (b) fared better.

Table 1 Reinforcement description for models

Model	Reinforcement description
URM-01	Nonreinforced—traditional
RM-SM	Welded wire mesh with cement plaster, vertically at corners and horizontally at top on both sides of wall
RM-RC	Welded wire mesh as RM-SM, plus reinforced concrete ring beam with shear anchors to the wall at corners
RM-NM	Natural mesh with vertical whole cane and horizontal fiber rope placed externally on both sides of wall
RM-PM100	Polymer mesh covering the wall at 100% on both sides
URM-02	Nonreinforced—traditional
RM-PM75	Polymer mesh covering the walls at 75% on both sides
RM-PM50	Polymer mesh covering the walls at 50% on both sides
RM-LCM	Low-cost polymer mesh covering half of the model at 100% and the other half at 70%
URV	Nonreinforced vaulted model
RV-PM100	Vaulted model with polymer mesh covering the model completely on both sides

Table 2 Seismic performance of models (2003–6) (ND = no damage; LD = light damage with fine cracks; HD = heavy damage with wide cracks; C = total or partial collapse with instability)

Maximum table displacement D_0 (mm)	Associated intensity (MMI)	CERESIS-GTZ-PUCP (2003)			GCI-PUCP (2005)			PUCP (2005-2006)			AMARES vaults (2006)	
		URM-01	RM-SM	RM-RC	RM-NM	RM-PM100	URM-02	RM-PM75	RM-PM50	RM-LCM	URV	RV-PM100
≤ 30	< VI	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
$30 < D_0 \leq 70$	VII	LD	ND	ND	LD	LD	—	—	—	—	LD	LD
$70 < D_0 \leq 90$	VIII	HD	LD	LD	HD	LD	HD	HD	HD	LD	C	HD
$90 < D_0 \leq 110$	IX	C	HD	HD	HD	HD	—	—	—	—	—	HD
$110 < D_0 \leq 135$	X	—	C	C	HD	HD	C	HD	C	HD	—	HD

Table 2 shows that the general conclusion is that continuous external reinforcement is necessary to avoid collapse, and this reinforcement has to be compatible with the deformations of earthen building. In this sense, stiffer bands, such as welded wire mesh with sand-cement stucco, prevent cracking at higher levels of seismic intensity but do not work jointly with an adobe wall for severe seismic motions, and they show brittle final behavior. On the contrary, polymer and natural flexible meshes embedded in a mud mortar work together with adobe walls up to high levels of seismic intensity without collapse. In addition, it can be said that the polymer mesh is also appropriate for any type of architectural configuration.

Evolution of Adobe Building Codes and Design Criterion

From the beginning of the research program in 1972 until now, the focus of the design has been placed on avoiding the collapse of the earthen structures (ultimate state criterion) in addition to providing adequate elastic resistance. The first Adobe Building Code, in

1977 (Oficina de Investigación y Normalización 1977), established the basic architectural configurations that should be used in order to obtain an adequate seismic behavior. Such considerations have been improved in the subsequent versions of the code. The use of natural cane as reinforcement is also indicated in a general way as a means of preventing failure. The 1987 version of the code (Instituto Nacional de Investigación y Normalización de la Vivienda 1987), based on the experimental work at PUCP, established a specific procedure for using cane as internal reinforcement. The elastic criterion and the ultimate state criterion are maintained in this version. The 2000 code (Ministerio de Transportes, Comunicaciones, Vivienda y Construcción 2000), influenced by the work performed in the Getty Seismic Adobe Project (GSAP), introduced the concept of stability based on the slenderness of the wall (Tolles, Kimbro, and Ginell 2002). Another important inclusion was the shift in the target structures from new to existing buildings, with alternative design criteria influenced by the use of stronger and stiffer reinforcing materials. Even though the use of external reinforcement is mandatory for existing buildings, it has been quickly demonstrated that flexible and compatible materials perform much better than stiffer and stronger materials in large earthquakes. On the other hand, the present version of the code maintains the architectural layout recommendations regarding wall thickness, plan dimensions, and location of openings that were present in all previous versions of the Peruvian code.

Therefore, it can be said that the general criterion that governs the design of earthen buildings is based on the seismic performance for small, medium, and strong earthquakes. For small earthquakes the aim is to minimize the wall cracking by following certain architectural configurations and by a simple calculation of shear forces in the elastic range. For medium and strong earthquakes, the target performance is to avoid both partial and total collapse by the use of continuous, flexible, and compatible external or internal reinforcement. In the case of external reinforcement, it is recommended that it be embedded in a mud plaster.

Conclusion

The work performed over the last thirty-five years at PUCP has confirmed the basic engineering principles that can be applied in developing reinforced earthen buildings to resist earthquakes. The next step is to determine the technical specifications necessary to design earthquake resistant earthen buildings.

Adobe walls must work jointly with the compatible reinforcements embedded in the walls. This is obtained by the application of mesh-type reinforcement either internally or externally. In the case of external reinforcement, it has to be applied on both sides of the wall and connected by natural or industrial threads in holes through the wall. The plaster mortar has to have a minimum thickness to assure the integrity of the reinforcement with the wall and to provide protection from the environment. Mud mortar mixed with fibers should be used as plaster to allow moisture transfer between the wall and the environment.

Polymer mesh has proven to be an adequate material for reinforcing earthen buildings because of its compatibility with the earth material, because of its resistance to biological and chemical agents, and because its tensile strength can be transferred to the wall where it is applied.

The solution found so far for traditional, occidental architectural configurations can also be applied to other architectural typologies around the world where earthen construction is used.

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Seismic Response of Fiber-Reinforced and Stabilized Adobe Structures

Mohammad Shariful Islam and Kazuyoshi Iwashita

Abstract: *Most losses of life and wealth in developing countries during earthquakes are due to the collapse of adobe houses. In spite of this, after considering different socioeconomic reasons and the availability of other alternate solutions, it is expected that these types of structures will continue to be built for the decades to come, especially in developing countries. Seismic deficiencies of adobe structures are caused by their inelastic and brittle behavior and by weakness of the mortar. Reinforcement for adobe structures should be inexpensive, locally available, and easy to construct. In this context, hemp, jute, and straw have been selected to improve the seismic resistance of adobe block. Cement has been selected to improve the strength characteristics of the mortar. Uniaxial test results showed that jute and straw effectively incorporate ductility in the adobe, but hemp is not effective to incorporate ductility in adobe. However, the strength of straw-reinforced adobe is significantly lower than that of jute-reinforced adobe. It means that jute is the best option among these fibers to improve the seismic performance. Adobe reinforced with 2% jute is the most effective to improve the seismic performance of adobe block. Jute length should be 1–2 cm (0.4–0.8 in.) for the best seismic performance. With the use of jute or jute and cement together, the strength of the mortar can be increased. Jute fiber is also effective to reduce cracking in the mortar. Shake table test results also showed that jute-fiber-reinforced adobe structures have the maximum seismic resistance.*

Introduction

Historically the use of adobe construction has many advantages, including low cost, easy availability, easy construction, low energy requirements, environmental friendliness, and comfort. It is estimated that about 50% of the population in developing countries lives in earthen houses (Houben and Guillaud 1994). This type of structure is common in developing countries such as Afghanistan, Bangladesh, Guatemala, India, Iran, Pakistan, Peru, and Turkey. Under favorable weather conditions (in climates of extreme dryness), these earth structures can be extremely durable. Unfortunately, they are very vulnerable to earthquakes. The February 22, 2005, Zarand earthquake and the December 26, 2003, Bam earthquake, both in Iran, bear ample testimony to this fact. While adobe structures cause most losses of human lives, relatively few published technical papers deal with this type of building. It is evident that technical solutions have to be developed to improve the seismic resistance of adobe structures.

Seismic behavior of adobe buildings is commonly characterized by a sudden and dramatic failure. From historical earthquake events it is estimated that the collapse of adobe structures is mainly due to the following three reasons: (1) adobe is a brittle material and has practically no tensile strength; (2) poor construction practices often decrease the bond between adobe and mortar, so that mortar partly or totally disintegrates under a few cycles of a moderate earthquake; (3) they are massive and heavy and thus they are subject to high levels of seismic force. Additionally, architectural concepts of the past have changed, and at present the typical thickness of adobe walls has been greatly reduced to make them externally similar to brick masonry. These factors, together with lack of maintenance, contribute greatly to increased adobe vulnerability.

Possibilities of using concrete beams, wooden beams, anchored roof beams, horizontal steel rods, welded wire mesh, steel mesh with cement mortar, and tensile steel bars to improve the seismic resistance of adobe structures have been investigated by various researchers (e.g., Torrealva Davila 1987; Scawthorn and Becker 1986; Tolles and Krawinkler 1990; Tolles et al. 2000). These methods were found to be effective to improve the seismic resistance of adobe structures; however, they can be expensive and they require skilled design and construction. In this context, natural fibers, such as straw, jute, and hemp, were selected as reinforcing materials to improve the seismic resistance of adobe block. Cement has been selected to improve the strength characteristics of the mortar. This paper describes the effectiveness of the proposed reinforcing materials to improve the seismic resistance of adobe structures. The seismic response of fiber-reinforced and cement-stabilized adobe structures is also presented.

Selection of Soils

Adobe can be made with many types of soil. Old adobe from Iran and Bangladesh was collected and the grain size distributions of the samples determined, in order to try to match the grain size distribution. The adobe from Iran was provided by the Iran Cultural Heritage Organization. The sample was taken from the ziggurat at Al-Untash-Napirasha, which was the capital city of the Elamite king Untash-Naprisha (ca. 1260–35 BC). The adobe from Bangladesh was collected from a fifty-year-old adobe building situated in the Comilla district of Bangladesh. Locally available Japanese soils were selected to prepare the adobe in the present research. Acadama clay, Toyura sand, and bentonite have been mixed with a ratio of 2.5:1.0:0.6 by weight. This mixture is called "soil-sand mixture" in this study. Grain size distribution of the soil-sand mixture along with those collected from Iran and Bangladesh are presented in figure 1. It is seen that the grain size distributions of the soil-sand mixture are similar to those of old adobe from Iran and Bangladesh. More details about the soil selection are available in Islam (2002).

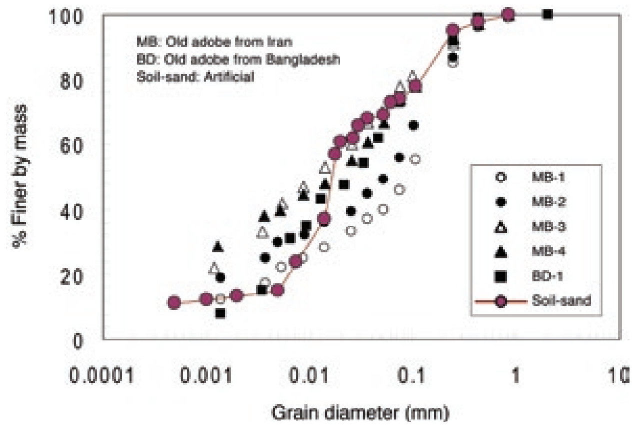


FIGURE 1 Grain size distribution of the soil-sand mixture.

Uniaxial Test

Uniaxial tests were conducted on several groups of cylindrical block and cylindrical sandwich specimens to investigate the effectiveness of the proposed reinforcing material on adobe block and mortar respectively. For each group, three specimens were tested to check the repeatability of the test results. Specimen preparation, characteristics of fiber-reinforced adobe, and the effect of fiber content and fiber length on adobe are presented in the following sections.

Preparation of Specimens

Specimens were prepared from soil-sand mixture, fiber, and cement. At first, water and soils were mixed vigorously so that a homogeneous mix was formed. The mix was then poured into a steel mold 5 cm (2.0 in.) in diameter and 10 cm (3.9 in.) in height in three layers. Each layer was compacted to remove the entrapped air. After that, the mold with the sample was kept in an oven at 60°C for three days. Finally, specimens were taken out from the mold and returned to the oven at the same temperature for three more days. Figures 2a–c show the details of specimen preparation using an oven.

Characteristics of Fiber-Reinforced Adobe

The effect of three different fibers—hemp, jute, and straw—on the seismic resistance of adobe material was investigated. In all cases, specimens were prepared by mixing the soil-sand mixture with 1.0% fiber (by weight) of 1.0 cm (0.4 in.) in length. Final water content and dry

FIGURES 2A–C Preparation of adobe specimens: soil-sand slurry (a); steel mold for specimen preparation (b); and oven used for drying specimens (c).

Table 1 Characteristics of unreinforced and reinforced adobe

Reinforcement	Final water content (%)	Dry density (g/cm³)	Comp. strength (kPa)	Toughness (kPa)
Unreinforced	4.3–5.2	1.16–1.17	1177.8	10.09
Straw	5.3–5.6	1.05–1.11	585.6	8.26
Hemp	3.5–4.5	1.09–1.14	1058.3	8.48
Jute	5.3–5.8	1.14–1.15	996.3	15.93

density of the specimens are presented in table 1. From the table, it is seen that final water content and dry density of the specimens varied from 3.5% to 5.8% and from 1.05 to 1.17 g/cm^3 (65.5 to 73.0 lb./ft.^{3}), respectively. Figure 3 presents typical stress-strain relationships of reinforced and unreinforced adobe. It is observed that failure of unreinforced and hemp-reinforced adobe is brittle. But the failure of jute- and straw-reinforced adobe shows ductile behavior. However, straw-reinforced adobe has significantly lower strength than jute-reinforced adobe. More details about straw reinforcement for adobe are available in Islam and Watanabe (2001).

Toughness is a measure of the total energy that can be absorbed by a material before failure. To compare, toughness has been calculated using the area under the stress-strain curve under uniaxial test up to failure. Failure point was defined corresponding to the 2/3 q_u (where q_u is the compressive strength). Average compressive strength and toughness of the reinforced and unreinforced adobe are also presented in table 1. It is seen that jute-reinforced adobe has the maximum toughness. Thus, jute fiber is the best option among these three fibers for improving the seismic resistance of adobe material. Figures 4a and 4b show unreinforced and jute-reinforced specimens at failure, respectively.

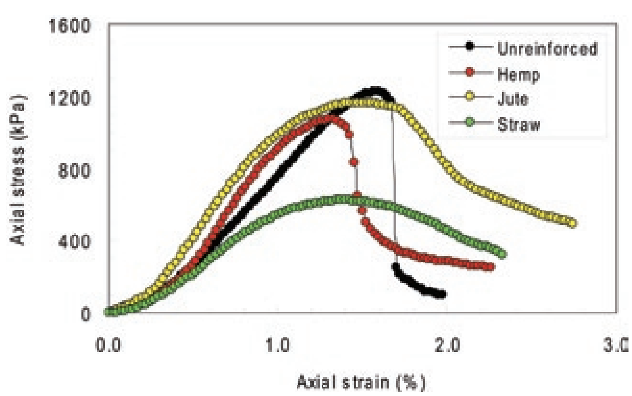


FIGURE 3 Stress-strain relationships of adobe.

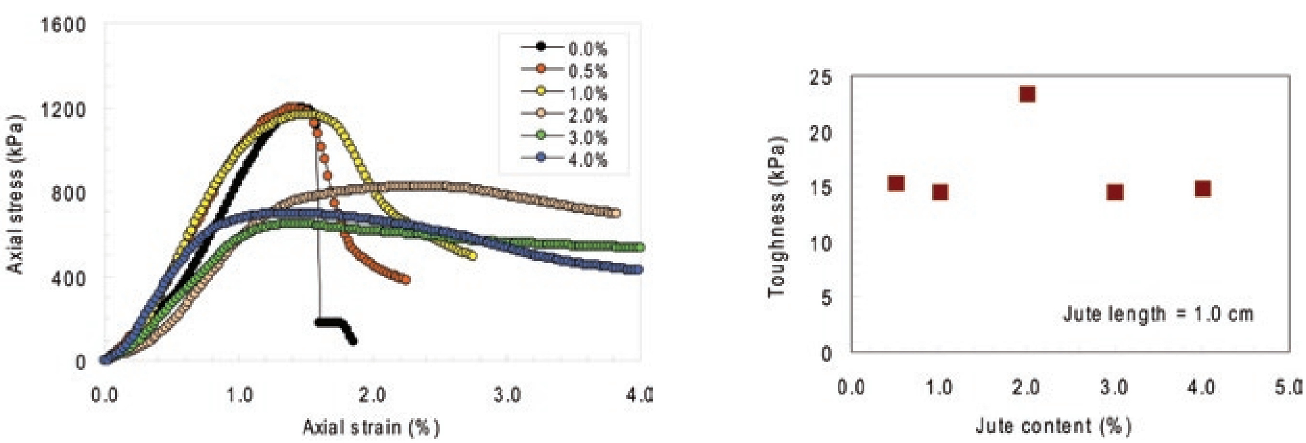
(a) (b) (c)

FIGURES 4A–C Failure pattern of reinforced and unreinforced adobe: failure of unreinforced adobe (a); failure of jute-reinforced adobe (b); detail of failure plane of jute-reinforced adobe (c).

Figure 4c is a detail of the failure plane of the jute-reinforced specimens. In this photograph, the action of the fiber can be seen clearly. It is clear that fiber resists the brittle failure of the adobe material.

Effect of Fiber Content

To investigate the effect of fiber content on adobe, specimens were prepared using 1.0 cm (0.4 in.) long jute by varying the jute content from 0.5% to 4.0% by weight. Final water content and the dry density of the specimens varied between 3.2% to 4.6% and 0.93 to 1.15 g/cm^3 (58.1 to 71.8 lb./ft.^{3}), respectively. Typical stress-strain relationships of jute-reinforced adobe have been presented in figure 5a. It is seen that the compressive strength of the specimens containing 2% to 4% jute is significantly lower than that of specimens containing 0% to 1% jute. But while the failure of the specimens containing jute up to 1% is brittle, the failure pattern of adobe reinforced with jute from 2% to 4% shows ductile behavior. Variation of toughness with jute content has been presented in figure 5b. It is observed that adobe reinforced with 2% jute fiber has the maximum toughness. Results indicate that 2% fiber is optimal for improving the seismic resistance of adobe material.



(a) (b)

FIGURES 5A AND 5B Typical stress-strain relationships of jute-reinforced adobe (a), and variation of toughness with jute content (b).

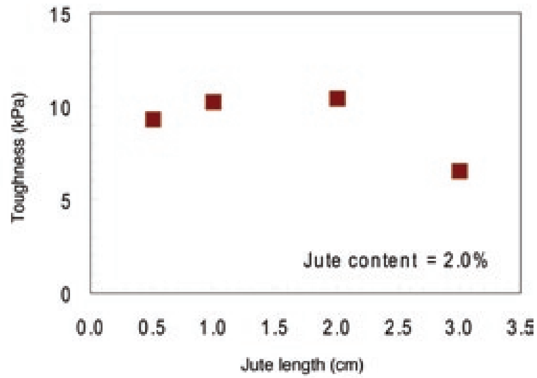


FIGURE 6 Variation of toughness with jute length.

Effect of Fiber Length

To investigate the effect of jute length on adobe, specimens were prepared using 2% jute and varying the jute length from 0.5 to 3.0 cm (0.2 to 1.2 in.). The variation of toughness with jute length is presented in figure 6. In this case, the toughness has been calculated using the area under the stress-strain curve until peak. It is observed that toughness of the material is almost the same in all cases, except in the case of 3.0 cm (1.2 in.) long fiber. Toughness of the specimens reinforced with 3.0 cm long fiber is significantly lower than that of other cases. From figure 6, it is also evident that jute length should be 1–2 cm (0.4–0.8 in.) to obtain the best seismic performance.

Mortar Strength

Past earthquakes showed that mortar is the weakest part of adobe structures. Cylindrical sandwich specimens were prepared to investigate the effectiveness of selected reinforcing material in improving mortar characteristics. Sandwich specimens were prepared cutting cylindrical specimens into two pieces at 60° to horizontal, since failure of specimens under uniaxial compression showed that specimens failed at 60°–70° to horizontal. Mortar of about 0.5 cm (0.2 in.) thickness was inserted between the two parts. The sandwich specimens were then kept in an oven at 60°C for three to four days for drying. Figures 7a–c show the details of the sandwich specimen preparation.

Composition of the sandwich specimens is presented in table 2. It is seen that in groups C-2 through C-4, the jute content was 1%, while the specimens of the C-5 group contain 2% jute in both the block and the mortar part. Table 2 also presents the mean compressive strength (q_u) and failure strain (ε_f) of the sandwich specimens. It is observed that the strength of the adobe material with mortar is significantly lower than that of the specimen without mortar (see tables 1 and 2). It is seen that mortar strength can be increased from 33.2 to 129.7 kPa (4.8 to 18.8 psi) using 1% jute in both the block and mortar. By using 1% jute in the block and 1% jute and 9% cement together in the mortar, the strength of the mortar can be increased from 33.2 to 196.1 kPa (4.8 to 28.4 psi). But in all of these cases, the strength is

(a) (b) (c)

FIGURES 7A–C Making of sandwich specimens: cutting of cylindrical specimens at 60° to horizontal (a); two parts of specimen after cutting (b); and a sandwich specimen (c).

Table 2 Characteristics of sandwich specimensw

Specimen designation	Reinforcement		Jute content (%)	Comp. strength q_u (kPa)	Failure strain ε_f (%)
	Block	Mortar			
C-1	—			33.2	1.10
C-2	Jute		1.0	68.1	2.13
C-3	Jute	Jute	1.0	129.7	2.71
C-4	Jute	Jute and cement	1.0	196.1	2.47
C-5	Jute	Jute	2.0	527.0	0.50

W

significantly lower than that of the adobe block. From the test results of the group C-5, it is observed that by using 2% jute both in the block and the mortar, the strength can be significantly increased, up to 527.0 kPa (76.4 psi).

Figures 8a–e show the failure patterns of the mortar specimens. It is seen that in all cases, separation has occurred between the two parts during failure. However, in unreinforced cases, the mortar also failed. It is also seen that unreinforced mortar has many cracks. But mortar reinforced with cement and fiber does not have any cracks. These results indicate that jute, or jute and cement together, are effective in preventing cracks in mortar. Cracks in the mortar might be the reason for the low strength of the unreinforced sandwich specimens.

Shake Table Test

In the preceding sections, uniaxial compression test results have been presented to describe the effectiveness of the selected reinforcing material in improving the seismic resistance of adobe block and mortar. Shake table tests were also conducted to investigate the seismic performance of the fiber-reinforced and stabilized adobe structures. Shake table test results are provided below.

Construction of Models

Preparation of Adobe Block

For constructing models, adobe blocks were made first. Materials were mixed in dry condition, then water was added and the mixture was mixed vigorously by hand. The mix was poured into a steel mold 20 cm (7.8 in.) in length, 9 cm (3.5 in.) in width, and 10 cm (3.9 in.) in height. Blocks were kept in the steel mold to reduce the water content, so that blocks can stand without any support. After that, blocks were taken out of the mold and kept in the natural weather condition for approximately seven to ten days. Once the blocks were strong enough to handle, they were placed in an oven at 40°C for two

(a) (b) (c) (d) (e)

FIGURES 8A–E Sandwich specimens after failure; samples C-1 (a), C-2 (b), C-3 (c), C-4 (d), and C-5 (e) are shown.

(a) (b)

FIGURES 9A AND 9B Procedure for adobe model making. Models are placed in the oven (a), resulting in a finished adobe model (b).

days. Finally, the temperature of the oven was raised to 60°C until the blocks were dry.

Construction of Model

Each model was constructed using four blocks. At first, mortar of about 1–2 cm (0.4–0.8 in.) thickness was inserted between blocks. After that, the model was kept in an oven at 40°C for two days. Then the models were kept in the oven for two more days at 60°C. Details of adobe model making have been presented in figures 9a and 9b.

Description of Models

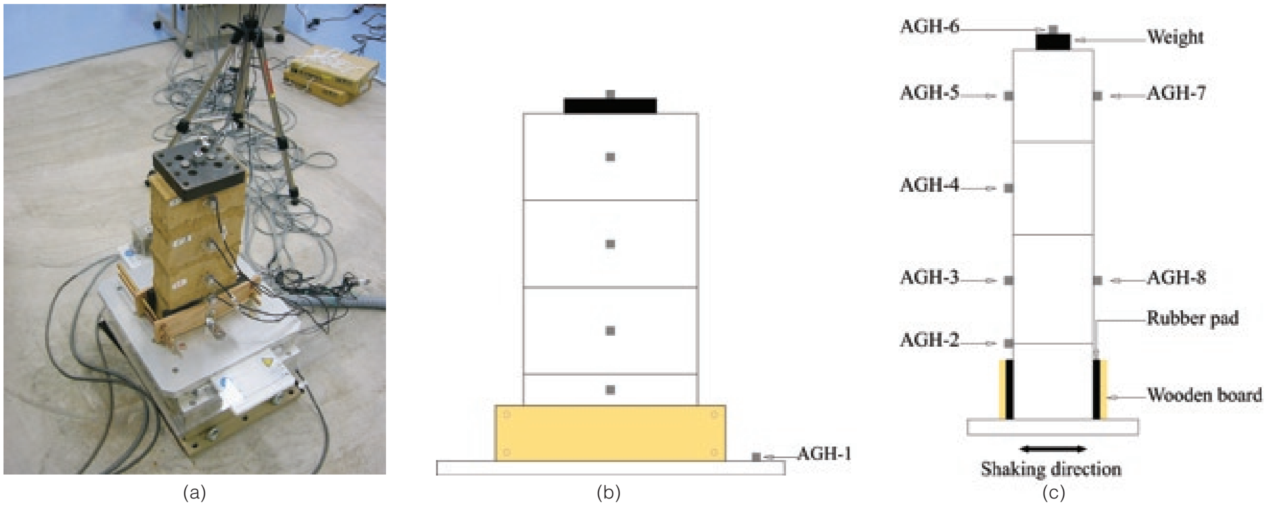
Five models were tested to check the effectiveness of the fiber and cement on the mortar characteristics. Dimensions of each model were about 20 cm (7.8 in.) in length, 9 cm (3.5 in.) in width, and 40 cm (15.6 in.) in height. Composition of the blocks and mortar of the models is presented in table 3. Soil composition of the blocks was soil-sand mixture (i.e., Acadama clay, Toyura sand, and bentonite mixed at the ratio of 2.5:1.0:0.6 by weight). From table 3, it is seen that all of the blocks of the models M-1, M-2, M-3, and M-4 were reinforced with jute. Blocks of all four models M1 to M4 contained 2% jute of 3.0 cm (1.2 in.) length; the blocks of the model M-5 did not have any fiber. Mortars of models M-1 and M-5 are unreinforced. Mortar of models M-2 and M-3 were reinforced with 2% jute and 9% cement, respectively, while the mortar of model M-4 was reinforced with 2% jute and 9% cement together.

Table 3 Composition of block and mortar of test models

Model	Block	Mortar
M-1	Jute	Unreinforced
M-2	Jute	Jute
M-3	Jute	Cement
M-4	Jute	Jute and cement
M-5	Unreinforced	Unreinforced

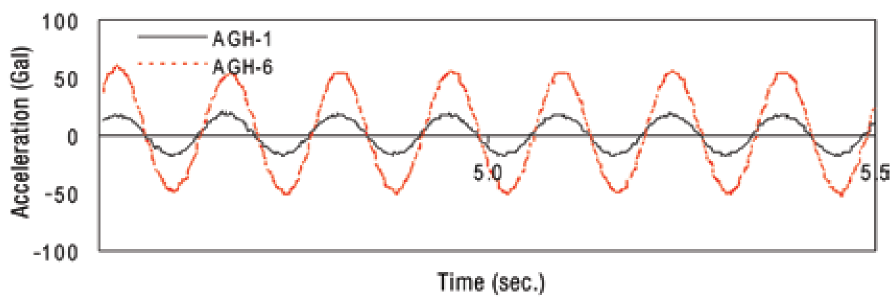
Instrumentation

The shake table available at the Vibration Engineering Laboratory of Saitama University, near Tokyo, Japan, was used to shake the models. The shake table has the capacity to give acceleration up to 1170 Gal (1.193 g). The maximum weight that can be shaken by this table is 20 kg (44 lb.). The frequency range of the table is 0.5–20 Hz. The maximum force that can be applied by the table is 294 N (30 kgf). Figures 10a–c show the instrument setup for the shake table test. Eight accelerometers of piezoelectric type were used to record the acceleration of the shaking models. Positions of the accelerometers (named AGH-1 to AGH-8) on the models are presented in figures 10b and 10c. AGH-1 was used to record the base acceleration. AGH-6 was put on the top of the model to record the acceleration at the top. An external weight of 4.0 kg (8.8 lb.) was fixed on the top of the model to represent the load on the wall. The base of the model was fixed to the table using a rubber pad, bolts, and wooden board, as shown in figures 10a and 10c. Models were shaken parallel to the shorter dimension. Figure 11 presents a typical recording of acceleration at the base and its response at the top of the model. Models were shaken using a sinusoidal wave of 7.0 Hz for 10.0 sec., with variance of the input base acceleration until failure, as shown in figure 11.



FIGURES 10A–C Instrument setup for shake table test: model with instruments (a); schematic diagram of front (b); and schematic diagram of side (c).

FIGURE 11 Typical input base acceleration and its response at the top.



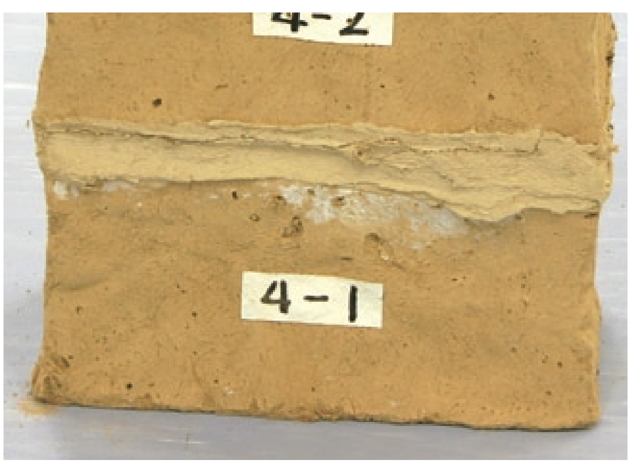


FIGURE 12 Model M-4 after failure.

Test Results

Description of Failure

All the models failed at the same level, at the top of the first mortar layer and the bottom of the second block. A photograph of the model M-4 after failure is presented in figure 12; the crack line can be seen clearly. Base accelerations that were observed at failure for each model have been presented in table 4. Description of failure and photographs of the failure surface are also presented in the table.

From table 4, it is seen that model M-1 failed at the base acceleration of 55.0 Gal (0.056 g). Separation occurred between the top of the first mortar layer and the bottom of the second block. Some parts of the mortar also failed. In the photograph, it is seen that there are many cracks in the mortar. However, there was no crack or damage to the blocks.

Table 4 Comparison among model performances

	Reinforcement				
Model	Block	Mortar	Accel. (Gal)	Description of failure	View of failed surface
M-1	Jute	Unrein-forced	55	Many cracks in the mortar; first mortar layer failed; no damage to the block; poor bonding between block and mortar in comparison to model M-5	
M-2	Jute	Jute	630	Separation between block and mortar; failure initiated from the second block; no significant damage to the block and mortar; strong bonding between block and mortar; failure plane is curved	
M-3	Jute	Cement	120	Cracks in the mortar; separation occurred; first mortar layer failed; no damage to the block; moderate bonding between the block and mortar	
M-4	Jute	Jute and cement	305	Separation between block and mortar; failure initiated from the second block; no significant damage to the block and mortar; good bonding between block and mortar	
M-5	Unrein-forced	Unrein-forced	180	Many cracks in the mortar; separation between block and mortar; bonding between block and mortar is moderate	

Model M-2 failed when the input base acceleration achieved 630.0 Gal (0.642 g). Failure initiated from the second block, as seen from the photograph in table 4, and failure was at the top of the first mortar layer. However, there was no damage to the block or mortar. There was no crack in the mortar of this model. This result indicates that jute fiber is effective in preventing cracking in the mortar.

Model M-3 failed at the top of the first mortar layer when the input base acceleration reached 120.0 Gal (0.122 g). Some parts of the mortar also failed, and there were many cracks in the mortar. However, cracks were fewer than in models M-1 and M-5.

Model M-4 failed at the top of the first mortar layer, as did the other models. This model failed at a base acceleration of 305.0 Gal (0.311 g). In this case, failure was also initiated from the second block. There were no cracks in the mortar. This indicates that jute and cement together are also effective in preventing cracks in mortar.

Model M-5 failed at the top of the first mortar layer when the base acceleration reached 180.0 Gal (0.184 g).

The failure occurred at the top of the first mortar layer and at the bottom of the second block. Separation between block and mortar occurred, and some parts of the mortar also failed. There were many cracks in the mortar.

Comparison Between Model Performances

From table 4, it is seen that the base acceleration at failure for model M-1 was 55 Gal (0.056 g), while for model M-5 it was 180 Gal (0.184 g). The difference between these two models is in the composition of the block only. The block of model M-1 contains jute, while the block of model M-5 is unreinforced. The shrinkage in the mortar and in the block was not the same, since the block had fiber but the mortar did not have any fiber. For this reason, there might be some gap between the block and mortar. Bonding between the block and mortar was poor. That is why the cohesion between the block and mortar of model M-1 is not as high as that of model M-5.

Model M-2 is the strongest among all the models. It contains fiber in both the block and the mortar. The bonding between the block and the mortar is very good. In the case of the sandwich specimens, it was observed that both the mortar and the block reinforced with 2% fiber were the strongest.

Model M-3 has cement in its mortar. It is stronger than model M-1 but weaker than the unreinforced one. In this case the bonding is not as good as in the case of model M-2. However, as the model was dried using an oven, the time might not be enough for the hydration of cement. This might be one reason for the lower strength of the model M-3.

Model M-4 failed at the base acceleration of 305 Gal (0.311 g). It is stronger than models M-1, M-3, and M-5. This means that the bonding between the block and mortar is better than that of these three cases. It is weaker than model M-2. It indicates that the use of jute alone is better than mixing jute and cement together. Also in this case, lack of hydration of cement might be one reason for lower strength.

The statistical uncertainty of using one sample of each type of model should be considered. Significant statistical uncertainty is inherent in any test when only one specimen is used, especially for soil materials. Another factor is that all the models were prepared using an oven, which constitutes a variation from the natural condition.

Estimation of Design Strength

Strength obtained from the uniaxial compression and shake table tests cannot be used directly for design purposes, because real structures are different in several ways—an example being openings in the wall construction. It is necessary to estimate the strength of the adobe material that can be used for design purposes.

Using the Mohr-Coulomb failure criterion, cohesion of an adobe model can be determined (assuming angle of internal friction, $\varphi = 0$) as stated in equation 1.

$$c = \tau = \frac{\sigma}{2} \tag{1}$$

where c is cohesion, τ is shear strength, and σ is axial stress, which can be determined as follows.

$$\sigma = \frac{F}{A}$$

where F is force, and A is the cross-sectional area of the failure surface. Force can be determined from equation 2.

$$F = mk \tag{2}$$

where $k = \frac{\alpha_f}{g}$ and m is the mass of the model above the failure line (see fig. 13).

Here, α_f is the base acceleration of the model at failure; g is acceleration due to gravity.

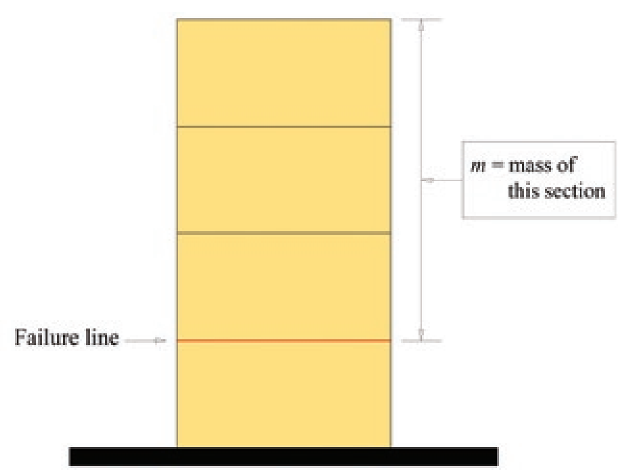


FIGURE 13 Description of mass used for calculating cohesion.

Table 5 Estimated design earthquake intensity of adobe walls

Model	Estimated base acceleration (Gal)	Earthquake intensity (JMA scale)	Earthquake intensity (MMI scale)
M-1	14.0	3	IV
M-2	161.0	5 Low–5 High	VII–VIII
M-3	30.5	4	IV–V
M-4	78.0	4–5 Low	VI–VII
M-5	44.7	4	V

Assuming failure of an adobe wall at the same level as indicated in figure 13, the design base acceleration (α_i) for the wall can be estimated from equation 3, using the cohesion value calculated from equation 1.

$$\alpha_i = \frac{2gcA}{m_i} \tag{3}$$

where c is cohesion, m is mass of the wall above the failure line, and A is the cross-sectional area of the wall.

Estimated base accelerations of a 2.4 m (7.9 ft.) high wall for five cases are presented in table 5. From the table, it is seen that an unreinforced adobe wall can survive an earthquake of the intensity of 4 according to the Japanese Meteorological Agency (JMA) scale, and an intensity of V according to the Modified Mercalli intensity (MMI) scale. Adobe walls reinforced with jute fiber, in both the block and the mortar, can survive an earthquake of 5 Low to 5 High on the JMA scale, or VII to VIII on the MMI scale.

Cost of Reinforcement

The reinforcement cost for a two-room, typical adobe house (6.1 × 9.15 × 2.90 m, or 20 × 30 × 9.5 ft.), as described by Coburn and colleagues (1995), has been estimated. If an adobe house of this dimension is reinforced with 2% jute fiber in both block and mortar, the total cost of the reinforcement will be about thirty U.S. dollars. The unit price of jute was taken from the local market price in Bangladesh, where adobe houses are being used on a large scale and jute is also locally available.

Conclusion

Natural fibers and cement were selected as reinforcing material for improving the seismic resistance of adobe structures. From the uniaxial and shake table test results, the following conclusions can be drawn:

- Jute is effective for improving the ductility and toughness of adobe material. However, there is an optimal jute content (i.e., 2%) for the best performance. Jute length should be in the range of 1.0 to 2.0 cm (0.4 to 0.8 in.) for the best seismic performance of adobe material.
- The strength of adobe with mortar is very low. By adding 1% jute, the strength of the mortar can be increased from 33.2 to 129.7 kPa (4.8 to 18.8 psi). Again, using 1% jute and 9% cement together, the strength of the mortar can be increased from 33.2 to 196.1 kPa (4.8 to 28.4 psi). But by using 2% jute in both the block and mortar, the strength can be increased significantly, from 33.2 to 527.0 kPa (4.8 to 76.4 psi). Many cracks were observed in the unreinforced mortar. This might be the reason for the low-strength, unreinforced sandwich specimens.
- All the shaking models failed at the same level, at the top of the first mortar layer and the bottom of the second block. Shake table test results also showed that jute is the most effective among the selected reinforcing materials for improving the seismic resistance of adobe structures. A strong bond between the mortar and block in the case of the jute-reinforced sample is the reason for its best seismic performance. Adobe walls reinforced with 2% jute in both block and mortar can survive an earthquake up to VII–VIII on the MMI scale. In unreinforced cases, there are many cracks, and bonding between the block and mortar is poor. This might be the reason for poor seismic performance of unreinforced adobe walls.

Finally, it can be concluded that 2% jute is effective to improve the seismic resistance of adobe structures. The cost of jute reinforcement is about thirty U.S. dollars for a standard, two-room adobe house. Gross national income (GNI) data indicate that this cost of reinforcement can be afforded by the dweller of developing countries such as Afghanistan, Bangladesh, India, and Pakistan. However, in the current research, the adobe specimens, blocks, and models were dried in an oven—a factor that varies from natural conditions. This fact must be considered in the design strength.

Acknowledgments

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Low-Cost and Low-Tech Reinforcement Systems for Improved Earthquake Resistance of Mud Brick Buildings

Dominic M. Dowling and Bijan Samali

Abstract: *Traditional, unreinforced adobe mud brick structures are highly susceptible to damage and destruction during seismic events. This vulnerability is evident in historic adobe structures around the world, as well as in traditional adobe homes in developing countries where severe earthquakes repeatedly cause drastic losses of life and livelihood. Adobe research at the University of Technology, Sydney (UTS), Australia, is focused on the development of low-cost, low-tech reinforcement systems for adobe structures. To date, ten U-shaped adobe wall panels and one full model house (1:2 scale) with different reinforcing systems have been subjected to transient dynamic loading using a shake table to evaluate the response to seismic forces. Time-scaled input spectra have been used to ensure dynamic similitude and impart sufficient energy to each structure to induce damaging conditions. The force-displacement characteristics and failure mechanisms of each structure have been studied to determine the resistance capacity of each system. Results indicate that a major improvement in structural performance can be achieved by using stiff external vertical reinforcement (e.g., bamboo), external horizontal reinforcement (e.g., bamboo or wire), and a timber ring/crown beam. This integrated matrix acts to restrain movement and enhance the overall strength of the structure. Tests have shown this system to effectively delay the onset of initial cracking and prevent collapse, even during severe shaking. The proposed system is effective, simple, affordable, and widely adaptable to a variety of materials and local conditions. It can be used for the retrofit-strengthening of existing structures, as well as in new construction. It shows tremendous promise for application in developing countries and for the protection and preservation of historic adobe structures around the world.*

Introduction

Traditional adobe mud brick structures are highly vulnerable to the effects of earthquakes, a problem that is particularly acute in rural housing in developing countries and in historic adobe buildings worldwide. Influencing factors include the inherently brittle nature of the material itself, its widespread use, its generally poor construction quality, limited awareness of concepts of aseismic design and construction, and limited resources to address the issue.

Adobe research at UTS is focused on developing and assessing methods to reduce the vulnerability of adobe housing to extreme dynamic loading such as caused by earthquakes (Dowling 2006). This research combines traditional building techniques, inexpensive reinforcement systems, and state-of-the-art facilities, including the UTS shake table, to investigate low-cost, low-tech solutions for application in developing countries.

Earlier Studies

The most notable shake table testing of adobe structures has been undertaken in Peru (Bariola et al. 1989; Zegarra et al. 1999; Quiun et al. 2005), Mexico (Hernández et al. 1981; Flores et al. 2001), the United States (Tolles and Krawinkler 1990; Tolles et al. 2000), and Colombia (Yamin et al. 2004).

U-shaped wall panels and model houses ranging in size from 1:6 scale to 3:4 scale have been subjected to uniaxial shake table testing. A number of reinforcement systems for adobe houses have been proposed and tested. These include external reinforcement (e.g., corner pilasters, timber boards, rope, wire, wire mesh, welded mesh, nylon straps, Geogrid mesh) and internal reinforcement (e.g., bamboo, chicken wire mesh, wire). Past research has made a significant contribution to the understanding of the behavior of adobe structures when subjected to earthquake forces. Furthermore, it 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. The results have been used to develop a number of design and construction manuals and guidelines (e.g., International Association for Earthquake Engineering 1986; Blondet, Garcia, and Brzev 2003). Large-scale implementation of the solutions, however, has not occurred. While a wide range of factors contribute to this lack of local implementation (e.g., cultural attitudes, resistance to change, lack of resources available for training, supervision, materials and tools, etc.), it seems that the development of a practical solution that is within the resource and skill levels of the rural poor is a critical initial step in the challenge of generating sustainable change.

In addition to the practical limitations of previously proposed systems, research to date has tended to focus on qualitative performance (observations) rather than on the collection and analysis of quantitative response (displacement, acceleration, dynamic amplification, etc.). Quantitative data provide important objective information about the behavior of specimens at a microscale, as well as increase the accuracy of comparative studies among different specimens and different tests. The collection of detailed quantitative data is also an important step toward developing a reliable finite element model for adobe structures.

Research at UTS endeavors to advance both academic studies (vis-à-vis the collection and study of qualitative and quantitative data) and the development of practical solutions for field application.

Testing Methodology

Description of Specimens

Research at UTS has included static and dynamic testing of adobe prisms and structures. This paper focuses on the dynamic testing of adobe structures. To date, ten U-shaped adobe wall panels and one full model house (1:2 scale) have been subjected to transient dynamic loading using a shake table to evaluate the response to seismic forces.

U-Shaped Wall Panels

It is widely known that the predominant failure modes of common adobe houses subjected to earthquake loads are vertical corner cracking at the intersection of orthogonal walls, and horizontal, vertical, and diagonal cracking due to out-of-plane flexure (Tolles and Krawinkler 1990; Flores et al. 2001). This often leads to overturning of walls and collapse of the roof. Improvement systems or techniques that are designed to reduce damage and destruction of adobe structures should primarily address these main failure modes. 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 wall units was undertaken at UTS (fig. 1). A variety of reinforcement systems and configurations were tested separately and/or collectively, as shown in table 1.

For each specimen, a downward restraining force was applied to the tops of the short 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 (fig. 1). This restraint acts to effectively transfer the bulk of the seismic loading to

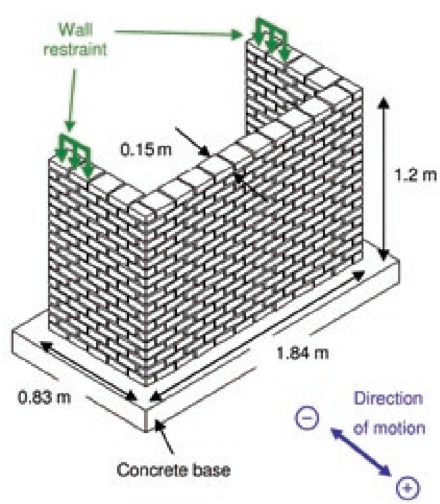


FIGURE 1 Specimen configuration and dimensions. Wall width = 0.15 m (5.9 in.), except for specimen 3H (= 0.10 m, or 3.9 in.); mortar joints, 12–13 mm (0.5 in.).

Table 1 Specifications of U-shaped adobe wall panels

Specimen	Reinforcement	1st natural frequency, f_1 (Hz)	Time scaling factor
3A	Unreinforced, common	29.6	2.0
3B	Corner pilasters/buttresses	34.1	2.3
3C	*Horizontal:* chicken wire mesh (internal)	33.0	2.2
3D	Chicken wire mesh (external wrapping) + timber ring beam	32.8	2.2
3E	*Horizontal:* chicken wire mesh (internal) + timber ring beam; *vertical:* bamboo (external)	30.8	2.1
3I	*Horizontal:* chicken wire mesh (internal) + bamboo (external) + timber ring beam; *vertical:* bamboo (external)	31.6	2.1
3H	*Horizontal:* chicken wire mesh (internal) + timber ring beam; *vertical:* bamboo (external). Thin wall (= 0.10 m)	33.0	2.2
3F	Retrofit. *Horizontal:* fencing wire (external) + timber ring beam; *vertical:* bamboo (external)	33.7	2.2
3J	Optimized. *Horizontal:* chicken wire mesh (internal) + fencing wire (external) + timber ring beam; *vertical:* bamboo (external)	33.0	2.2
3K	*Horizontal:* chicken wire mesh (internal) + timber ring beam; *vertical:* timber poles (internal)	27.0	1.8

the areas of main interest: the critical corner connections and the vulnerable out-of-plane long wall. The use of this applied restraining force constitutes a significant difference between this research and other dynamic tests of U-shaped adobe wall panels, which do not include any shear wing wall restraint and thus neglect the additional stiffness and restraint contributed by the shear walls (Zegarra et al. 1999; Quiun et al. 2005).

Model House

A model house (1:2 scale) was constructed, retrofitted, and tested on the UTS shake table (fig. 2). Its dimensions were 3.53 × 1.84 m (11.6 × 6.0 ft.), with a wall thickness of 0.15 m (5.9 in.) and a height of 1.2 m (3.9 ft.). The model house featured two doors and one window; the direction of shaking was north–south, perpendicular to the long walls.

The sequence of construction and retrofitting of the model house was: (1) construct unreinforced house and allow to dry; (2) mark and drill holes in rows at top, middle, and bottom of each wall; (3) insert polypropylene string loops through holes; (4) fill holes with mud, allow to dry; (5) place timber ring/crown beam on top of wall and connect with bamboo dowels and wire; (6) place external vertical bamboo (inside and outside house), tied with through-wall polypropylene string ties; (7) place and tension galvanized fencing wire horizontally between bamboo poles (top, middle, and bottom); and (8) connect bamboo poles and ring beam with wire loops.

FIGURE 2 Model house 4A prior to testing; south and east walls are visible.

If desired, the bamboo, string, and wire could be easily covered with a mud or lime render to provide an attractive finish, as well as afford protection from weathering.

Description of Equipment and Input Time History

The dynamic testing was undertaken on the 10 tonne capacity, 3 × 3 m (9.8 × 9.8 ft.) MTS Systems uniaxial shake table at UTS. A series of accelerometers and dynamic LVDT displacement transducers was used to record the dynamic response at key locations on each specimen and the shake table during the series of simulations. Of main interest was the response of the midspan-top of the out-of-plane long wall in relation to the ground motion (shake table displacement).

In this study the input time history from the January 13, 2001, El Salvador earthquake (M_w 7.7) was used (station, Hospital Santa Teresa, Zacatecoluca, La Paz; site geology, soil; epicentral distance, 51.2 km, or 31.7 miles [COSMOS 2006]). This earthquake, in combination with an M_w 6.6 earthquake on February 13, 2001, in the same area, caused the destruction of over 110,000 adobe houses (Dowling 2004).

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; see table 1), was identical for each specimen prior to testing.
- Ensure damaging near-resonance conditions, which are achieved when the pretest natural frequency of each specimen (U-panels and model house) is matched with the dominant frequency range of the input spectrum.

Given the variation in first natural frequencies of each specimen prior to testing, the input spectra were uniquely time-scaled for each individual specimen, using the time scaling factor (table 1). This was done to meet the above objectives and to ensure that consistent and sufficient energy was imparted to each structure to induce damaging conditions and allow comparative studies among specimens.

In addition to the time scaling of the input spectra, scaling of the intensity was undertaken. This was achieved by scaling the displacement component of the displacement time history. Intensity scaling 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 behavior), as well as for severe damaging conditions (postelastic behavior).

Results

U-Shaped Wall Panels

Each specimen was first subjected to three simulations using the raw, unscaled (with respect to time) input spectra, ranging in intensity from 40% to 200% of the displacement time history. In each case, no damage was observed, even for the unreinforced specimen 3A. Each specimen was then subjected to a series of time-scaled shake table tests of increasing intensities (20%, 50%, 75%, 100%, 125%, 75%, 75%, 100%, 100% of the displacement time history). The results from the time-scaled tests confirmed the destructive nature of ground motions containing sufficient energy and possessing dominant frequencies in the region of the natural frequencies of the wall units. This outcome clearly demonstrates the importance of appropriately time scaling the input motion during laboratory tests to ensure that sufficient energy is imparted to structures to induce damaging conditions, thus allowing a detailed study of the response and performance of different reinforcement systems.

In this paper, the behaviors of specimens 3A, 3F, and 3K are presented in detail, considering both qualitative results (observations, photographs) and quantitative results (displacement–time graphs)

Specimen 3A

Specimen 3A represented a common, unreinforced adobe structure. Sudden, brittle failure occurred during the moderate, 75% intensity simulation, S6 (figs. 3a and 3b). The primary failure modes for the unreinforced

(a)

(b)

FIGURES 3A AND 3B Specimen 3A after simulation S6 (75%). The in-plane wall and corner connection (a) and the out-of-plane wall (b) are shown.

specimen 3A (and lightly reinforced specimens 3B, 3C, and 3D) were: (1) vertical corner cracking at the intersection of orthogonal walls; (2) midspan vertical cracking in the out-of-plane long wall; and (3) horizontal and diagonal cracking in the out-of-plane long wall, with a propensity for overturning of the affected panel. These damage patterns are consistent with common damage to real houses subjected to real earthquakes. This feature confirms that the selected specimen configuration, boundary conditions, and input spectra are an acceptable means of assessing the seismic capacity of different reinforcement systems for adobe structures.

Specimen 3F

Specimen 3F was built as a common, unreinforced structure (as was specimen 3A); it was then retrofit-strengthened with external vertical bamboo poles, external horizontal fencing wire, and a timber ring beam (using the same procedure as described above for the model house). Specimen 3F performed extremely well. The reinforcement system was observed to delay the onset of initial cracking and reduce the severity of cracking, even during the series of high-intensity simulations (figs. 4a and 4b).

Specimen 3K

Specimen 3K included internal vertical reinforcement (25 mm [1 in.] diameter timber broom poles), plus internal horizontal chicken wire mesh reinforcement. This method and derivations thereof have been widely promoted as an effective earthquake strengthening

(a)

(b)

FIGURES 4A AND 4B Specimen 3F after simulation S12 (100% repeated). The out-of-plane wall (a) and the in-plane wall and corner connection (b) are shown.

(a)

(b)

FIGURES 5A AND 5B Specimen 3K after simulation S12 (100% repeated). The in-plane wall and corner connection (a) and the out-of-plane wall (b) are shown.

technique for new adobe houses (e.g., International Association for Earthquake Engineering 1986; Blondet, Garcia, and Brzev 2003). This system, however, has a number of deficiencies, which have limited its widespread acceptance and use. The main problem is that the method is complex and time-consuming and requires continuous involvement by skilled masons.

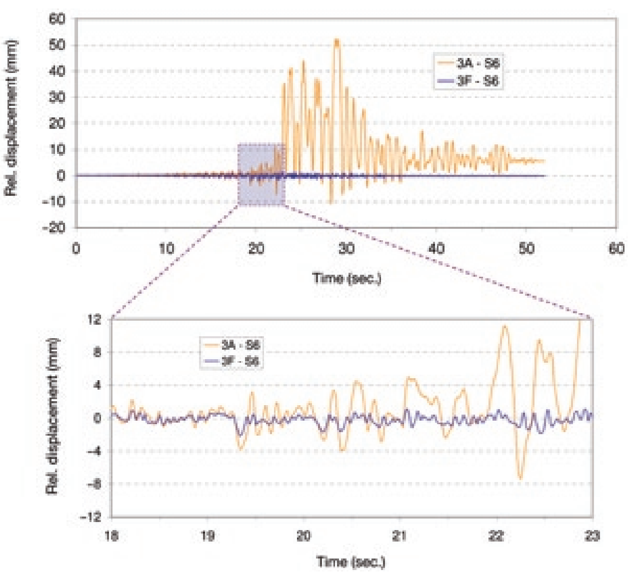


FIGURE 6 Specimens 3A and 3F: displacement (relative to shake table) at midspan-top of wall for simulation S6 (75%).

Concerns also exist about the durability of the natural materials commonly used as internal vertical reinforcement (e.g., bamboo, reeds, timber), which cannot be checked or replaced when encased in the structure. Despite these practical limitations, specimen 3K performed well during testing (figs. 5a and 5b). The reinforcement acted to reduce the severity of cracking, although it was observed that the internal vertical poles introduced discontinuities to the structure, evidenced by vertical cracking concentrated around the location of the vertical poles. This feature may be attributed to a difference in dynamic response between the stiff adobe wall and the flexible timber poles, as well as the reduced cross-sectional area of the wall around the poles.

Relative Displacement

Figure 6 shows the displacement of the midspan-top of the "long" wall relative to the shake table displacement for specimens 3A and 3F during simulation S6 (75%). The major difference in relative displacement between the unreinforced specimen 3A and the reinforced specimen 3F is evident. Initial cracking of specimen 3A appears to have occurred around t = 19.3 sec., with significant cracking occurring around t = 22 sec. The peak relative displacement of specimen 3A (52.34 mm, or 2.0 in.) was 24 times that of specimen 3F (2.16 mm, or 0.1 in.) for simulation S6 (75%).

Figure 7 shows the relative displacements of the midspan-top of the long wall for specimens 3F and 3K for simulation S7 (100%). For specimen 3K the amplification of the response was much larger than for specimen 3F, even in the initial stages of the simulation when there was relatively little ground motion. This confirms the progressive damage and loss of stiffness of specimen 3K, even from the low-intensity simulations (most probably due to discontinuities and cracking around the internal vertical reinforcement). The peak relative displacement of specimen 3K (10.81 mm, or 0.4 in.) was 1.6 times that of specimen 3F (6.81 mm, or 0.3 in.) for simulation S7 (100%).

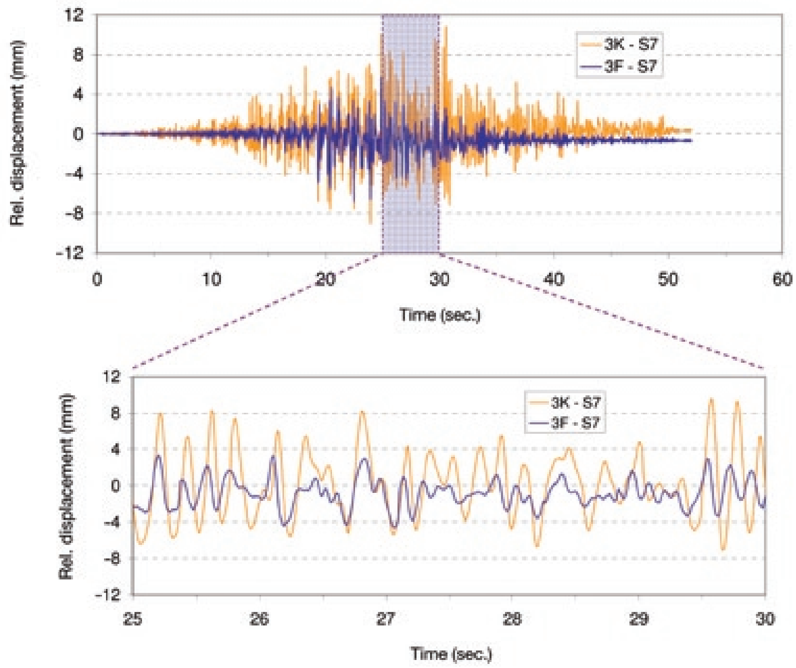
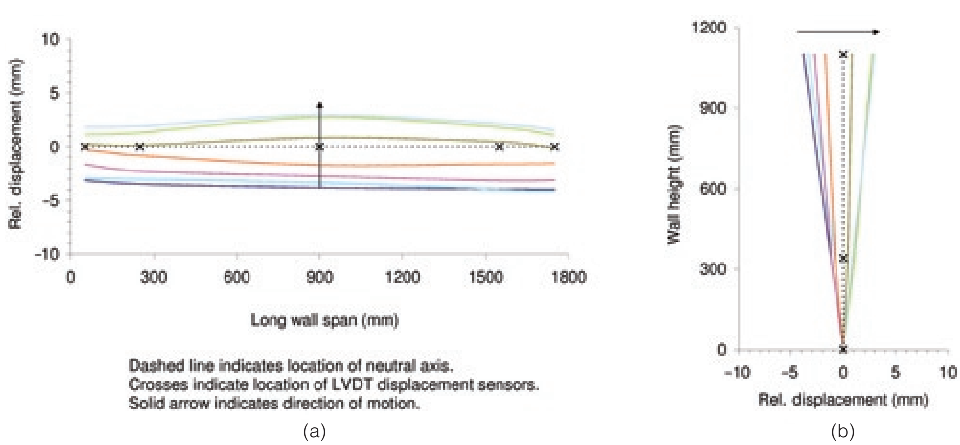


FIGURE 7 Specimens 3K and 3F: displacement (relative to shake table) at midspan-top of wall for simulation S7 (100%).

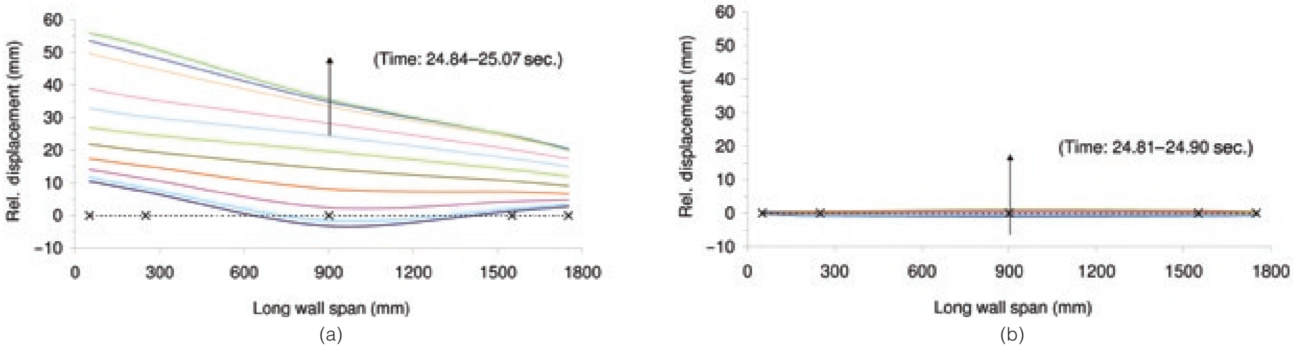
Flexural Response

Snapshots of the flexural response of specimens 3A, 3F, and 3K during simulation S6 (75%) and simulation S8 (125%) are shown in figures 8–10. The precracked behavior of specimen 3A is shown in figures 8a and 8b, which reveal a moderate flexural response. This response is significantly different from the post-cracked snapshot (fig. 9a), which shows the large flexure of the wall, in particular on the left-hand side, where the main vertical corner cracking occurred (fig. 3). By comparison, the flexural response of reinforced specimen 3F (fig. 9b) at the same approximate time (~24.9 sec.) shows the contribution of the reinforcement system in reducing the flexure of the wall, thus delaying the onset of initial cracking.

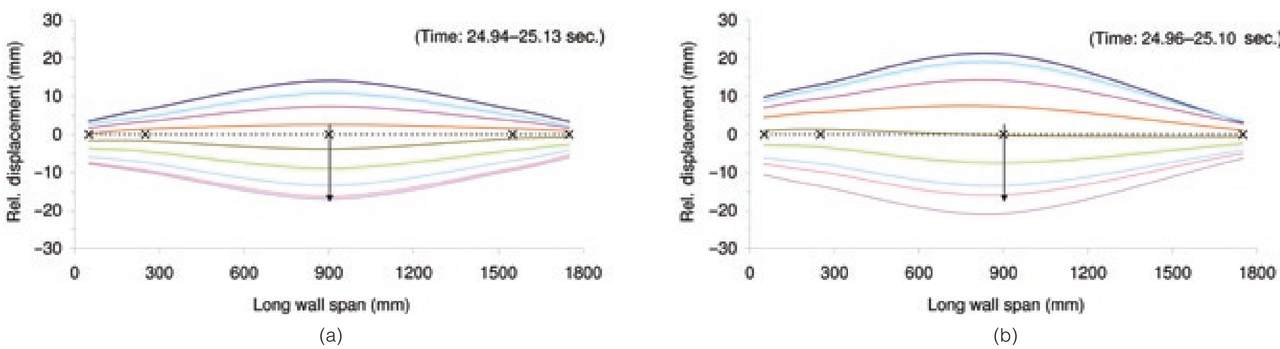
Figures 10a and 10b show the horizontal flexure of specimens 3F and 3K during simulation S8 (125%). The graphs show the larger flexural response of specimen 3K, which, when matched with the results presented in figure 7, confirm the effectiveness of the external reinforcement matrix (specimen 3F) at reducing movement, even during high-intensity simulations.



FIGURES 8A AND 8B Specimen 3A during simulation S6 (75%): horizontal flexure of top of “long” wall (a), and vertical flexure at midspan of long wall (b) (time: 19.35–19.47 sec.).



FIGURES 9A AND 9B Specimens 3A and 3F during simulation S6 (75%): horizontal flexure of top of long wall for specimen 3A (a) and specimen 3F (b).



FIGURES 10A AND 10B Specimens 3F and 3K during simulation S8 (125%): horizontal flexure of top of long wall for specimen 3F (a) and specimen 3K (b).

Failure Mechanisms

The observed damage patterns can be attributed to the following factors:

a. Flexure induced in the out-of-plane long wall, causing a splitting-crushing cycle at the midspan of the wall and the intersection with the orthogonal shear wing walls (see figs. 8–10).
b. Large relative displacement between the "flexible" out-of-plane long wall and the stiff in-plane shear wing wall, leading to tearing failure of both the mortar-brick interface and the individual brick units at the corner intersection (vertical corner cracking).
c. Vertical flexure and overturning of the out-of-plane long wall leading to horizontal cracking, and contributing to diagonal cracking, in combination with the horizontal flexure.

Model House 4A

Model house 4A was subjected to a series of time-scaled shake table tests of increasing intensities (10%, 25%, 50%, 75%, 100%, 125%, 100% of the displacement time history); this procedure was followed by a "shakedown," which involved subjecting the specimen to approximately ten minutes of sinusoidal shake table motions, covering a range of frequencies (1–20 Hz) and displacements (1–30 mm, or 0.04–1.17 in.), in an effort to identify the resonant frequencies of the damaged house and shake the house to pieces.

Observations

Initial, minor cracking occurred during simulation S4 (75% intensity), with hairline cracking evident above the lintel in the east shear wall. (Recall that the unreinforced U-shaped wall panel 3A was severely damaged during a 75% intensity simulation—see fig. 3.) Damage of model

(a) (b)

FIGURES 11A AND 11B Model house 4A: damage after simulation S8 (shakedown). The south wall (a) and the east wall (b) are shown.

house 4A increased during subsequent simulations. Figures 11a and 11b show the condition of the house after simulation S8 (shakedown). Despite being severely damaged, the structure resisted collapse, even after the series of severe earthquake tests.

The reinforcement system acted as a netting to contain the structure, even after significant damage. Cracking was distributed around the structure, with major damage occurring around the window and door openings. Vertical cracking at the corner intersections was largely prevented. This represents a major positive outcome, as one of the main failure modes of adobe houses is vertical corner cracking, which often results in the overturning of the walls and the collapse of the roof (as discussed above). There was no evidence of failure or breaking of the bamboo, string, or wire during testing.

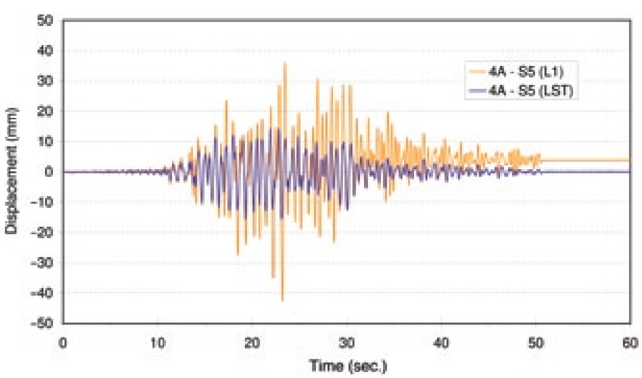


FIGURE 12 Model house 4A: absolute displacement of top NE corner (L1) and ST (shake table) for simulation S5 (100%) (peak displacements: L1, 58.79 mm, or 2.29 in.; ST, 19.12 mm, or 0.75 in.).

Displacement

Figure 12 shows the response at the top northeast corner (L1) of the model house, plus the movement of the shake table (LST) during simulation S5 (100% intensity). The graph clearly shows the amplification of the response at L1, due largely to the presence of the door in the east shear wall.

Flexural Response

Figure 13 shows a snapshot of the relative horizontal and vertical flexure of the north wall. The snapshot corresponds with the peak response at L1 (top, northeast corner) and clearly shows the significant movement at the east end of the wall and the stability at the west end. This large difference is due to the influence of the door opening in the east shear wall, which was significantly less stiff than the opposite west shear wall (without penetration). This difference in response had a significant effect on the entire structure, with the introduction of severe warping (combination of horizontal and vertical flexure) in the house.

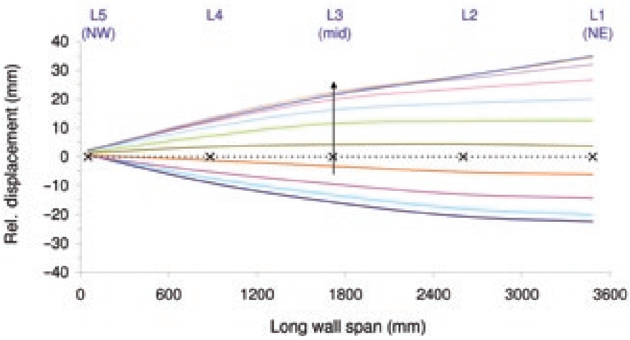


FIGURE 13 Model house 4A during simulation S5 (100%): horizontal flexure of top of north wall (time: 26.72–26.95 sec.).

Conclusion

The successful testing and analysis of ten U-shaped adobe wall units revealed the following general outcomes:

a. 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.
b. Test results confirm the importance of appropriate time scaling of input time history to induce damaging conditions in a structure. Time scaling is also necessary to ensure dynamic similitude among specimens, such that accurate comparisons may be made among the performances of different specimens.
c. The dynamic testing and assessment proved both reinforcement systems (3F and 3K) to be effective at improving the seismic capacity of adobe mud brick U-panels. Although significantly damaged after the rigorous testing program, both wall units resisted collapse. Overall, specimen 3F performed significantly better, maintaining dynamic stiffness at lower-intensity simulations and exhibiting less relative wall movement and more even distribution of cracking. By contrast, the loss of stiffness of specimen 3K at the lower-intensity simulations, plus the major failure in the wing wall, indicate a generally weaker and more vulnerable structure. In addition to the superior dynamic performance of specimen 3F, a major advantage of the system is the relative simplicity of construction, which makes it a more appealing reinforcement alternative.

The dynamic testing of model house 4A confirmed the efficacy of the reinforcement system used in U-shaped wall panel 3F. Results indicate that a major improvement in the earthquake resistance of adobe mud brick 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 that 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. The proposed system is effective, simple, affordable, and widely adaptable to a variety of materials and local conditions. It can be used for the retrofit-strengthening of existing structures, as well as for new-build construction.

The proposed reinforcement system was recently incorporated in an existing adobe dwelling in rural El Salvador. The retrofit-strengthening procedure was undertaken by two people in one week, with material costs of fifty U.S. dollars and equipment costs also totaling fifty U.S. dollars. This represents a substantial improvement on previously proposed reinforcement systems and paves the way for wide-scale implementation.

Acknowledgments

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Getty Seismic Adobe Project Research and Testing Program

E. Leroy Tolles

Abstract: *During the 1990s the Getty Conservation Institute (GCI) funded the Getty Seismic Adobe Project (GSAP), a multidisciplinary research effort to develop effective seismic retrofit measures that have a minimal impact on the significant historic fabric of historic adobe buildings.*

The early stages of the research included a field survey of common architectural types of historic adobe buildings, a survey of common practices of retrofitting historic adobe buildings in the United States, a review of technical literature, and a review of studies of the damage to historic adobe buildings. The major lack of basic information was in the area of documentation of the details of actual earthquake damage to historic adobe buildings.

The 1994 Northridge earthquake was a significant opportunistic event for this research project. During that event, the historic adobe buildings near Los Angeles suffered significant damage. The damage to more than a dozen historic adobe buildings was documented and published in 1996. This field study also included an overview and analysis of the typical types of seismic damage that occur in historic adobe buildings in this region.

A substantial portion of the research effort was dedicated to shake table testing of reduced-scale models of adobe walls and adobe buildings. Nine small-scale buildings (1:5 scale) were tested at Stanford University, in Palo Alto, California. Two large-scale models (1:2 scale) were tested at the research facility of the Institute of Earthquake Engineering and Engineering Seismology (IZIIS), University "SS. Cyril and Methodius" in Skopje, Republic of Macedonia. The testing program was used to evaluate the effectiveness of a range of seismic retrofit measures, including vertical and horizontal straps, vertical center core rods, anchorage at the roof and floor line, and the use of bond beams.

The final part of the project was to develop engineering design guidelines for the retrofit of historic adobe buildings. The engineering guidelines were combined with planning guidelines and published together as part of the final publication of the GSAP.

Introduction

The Getty Seismic Adobe Project was a multiyear project of the Getty Conservation Institute to develop structurally effective seismic retrofitting strategies for historic adobe buildings that have minimal and, to the extent possible, reversible impacts on historic fabric. This project included a survey of historic adobe buildings in California, preparation of planning guidelines for retrofitting historic adobe buildings, performance of tests of model adobe buildings on an earthquake simulator, a survey of damage to historic adobe buildings after the 1994 Northridge earthquake, and preparation of engineering guidelines for the retrofit of historic adobe buildings. Additional large-scale earthquake simulator tests were performed at IZIIS.

Background Research

The goal of this research program was to determine means of seismic retrofitting for historic adobe structures that have a minimal effect on the historic fabric of the buildings. Two issues form the theoretical basis for this research program:

1. *Stability-based measures:* The seismic performance of unreinforced adobe buildings can be greatly improved by the use of minor restraints and elements of continuity that inhibit the relative displacements of cracked wall sections and prevent the principal modes of failure.
2. *Slenderness ratio and wall thickness:* The slenderness (height-to-thickness) ratio (S_L) is of fundamental importance in determining the behavior of unreinforced masonry in general and adobe in particular. The slenderness ratio will affect the susceptibility of an adobe building to damage and affect the type of retrofitting measures that may be appropriate.

Both of these issues have been addressed by shake table tests on reduced-scale models, by studies of observed damage to adobe buildings after the Northridge earthquake, and by testing performed on large-scale model buildings.

Earthquake Damage Assessment of Historic Adobes

The damage to more than a dozen historic adobe buildings that resulted from the 1994 Northridge earthquake was studied and documented as part of the overall GSAP program (Tolles et al. 1996). Each of the buildings was studied, and the type of damage was documented. The overall set of damage was itemized to characterize the types of damage that may occur to historic adobe buildings, as shown in figure 1.

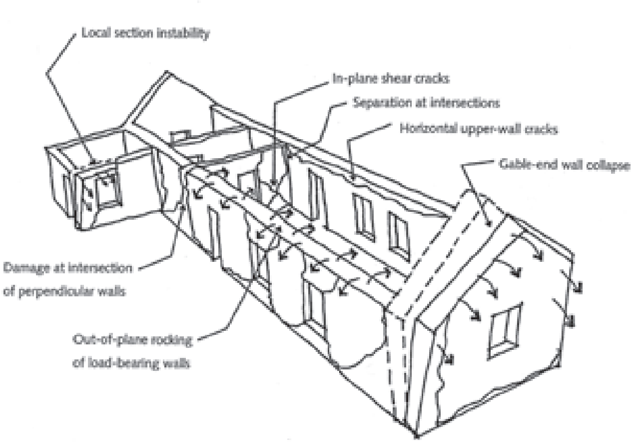

FIGURE 1 Typical damage to historic adobe buildings, as observed after the 1994 Northridge, California, earthquake.

Laboratory Research

The GSAP laboratory research included both small-scale and large-scale models tested on dynamic earthquake simulators. The ground motion for the tests was based on actual earthquake records from historic earthquakes recorded in California. The small-scale tests were carried out on 1:5 scale adobe buildings, and the large-scale tests were performed on 1:2 scale adobe buildings. Test results were published in 2000 (Tolles et al. 2000).

Small-Scale Models

Three 1:5 scale adobe models (group A: models A1, A2, and A3) were tested during 1992–93. The tests on these models were designed to address the first issue—i.e., the effectiveness of stability-based retrofit techniques. These tests clearly demonstrated that the use of stability-based retrofit measures can dramatically improve the seismic performance of an adobe building. Each model was subjected to a series of up to ten shake table motions, in which each test was approximately 30% larger than the previous test, as listed in table 1. A listing of all model buildings tested is presented in table 2.

Models 4, 5, and 6 were designed to address the second issue, the effects of wall slenderness. The walls of model 4 had a slenderness ratio of 5. The walls of models 5 and 6 had slenderness ratios of 11.

The results of the tests on models 1 through 6 indicated that the thickness of adobe walls has an effect on the seismic performance but that it is of secondary importance compared to the improved performance provided by the implemented stability-based retrofitting measures.

Model 7 was the first building designed as a complete building, with gable-end walls and floor and roof framing. The retrofitting measures were designed to address many of the issues that may occur in an actual building and to assess the performance of a larger, more

Table 1 Simulated earthquake motions for testing model buildings (prototype dimensions). EPGA = estimated peak ground acceleration (Tolles and Krawinkler 1990)

Test level	Maximum EPGA (g)	Maximum displacement (cm)	(in.)
I	0.12	2.54	1.00
II	0.18	5.08	2.00
III	0.23	7.62	3.00
IV	0.28	10.16	4.00
V	0.32	12.70	5.00
VI	0.40	15.88	6.25
VII	0.44	19.05	7.50
VIII	0.48	25.40	10.00
IX	0.54	31.75	12.50
X	0.58	38.10	15.00

complex building with the application of stability-based measures.

The plan layout of model 7 was similar to the layout of the first six models. In model 7, floor joists were added at the level of the tops of the walls of models 1 through 6. The basic layout of model 7 was based on typical *tapanco*-style adobe construction. The load-bearing walls of *tapanco* style adobe buildings extend approximately 3 ft. (1.0 m) above the attic floor, and there are gable-end walls at the non-load-bearing ends (east and west) of the building.

In models 7, 8, and 9, the walls extended four courses (approximately 2 ft., or 0.65 m, in prototype dimensions) above the attic floor. The load-bearing (north and south) walls have a door and a window in each wall. The gable-end (east and west) walls extend above the north and south walls at a slope of 6:12 vertical to horizontal. The wall elevations of model 9 are shown in figure 2.

The retrofitting measures used on model 7 were based upon the more successful measures tested in models 1 through 6, with the addition of partial diaphragm measures used on the attic floor and roof system. The retrofitting system used consisted primarily of the following: (1) horizontal and vertical straps applied to the walls, and (2) partial wood diaphragms applied to the attic floor and roof. The remainder of the retrofit system consisted of connection details.

A combination of vertical and horizontal straps was applied to all the walls. As had been implemented on previous models, the retrofitting strategy is slightly different on the west and south walls, compared to that implemented on the east and north walls.

Two horizontal straps were placed on each of the four walls. The upper horizontal strap was located at the attic floor line, and the lower horizontal strap was located just below the bottom of the window. The strap at the attic floor line ran around the perimeter of the building and was attached to the floor system. The attachments to the floor system are shown in the details in figures 3 and 4. The lower horizontal strap was located on both sides of each of the walls. Smaller straps were used as cross-ties to connect the straps on both sides of the wall.

On the west wall, no vertical straps were added. The west and east walls had no door and window openings, except for small attic windows. The south wall had only one vertical strap located at the center of the pier between the door and the window. The north and east walls had vertical straps at regular intervals (see fig. 2).

The vertical straps were located on both sides of each wall. The straps went over the tops of the walls and through

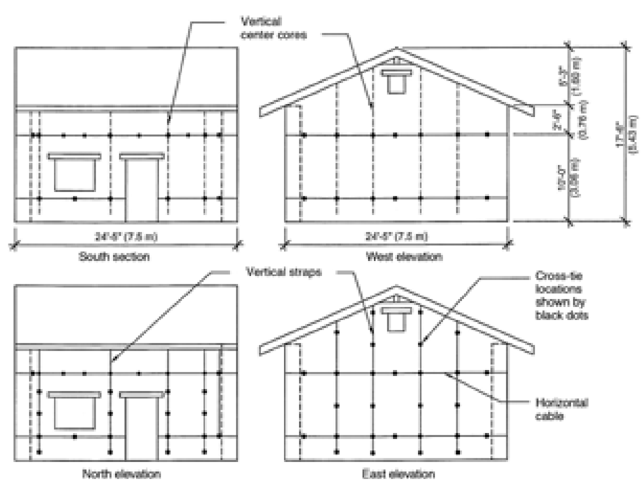

FIGURE 2 Wall elevations of model 9, with vertical straps on the north and east walls and center core rods on the south and west walls. Note that dimensions are prototype dimensions.

Table 2 Description of models tested. Models 10 and 11 were tested at IZIIS; all other models were tested at Stanford University

Model no.	Slenderness	Scale	Walls	Description of retrofit strategy
1	7.5	1:5	NE	Upper horizontal strap
			SW	Upper and lower horizontal straps
2	7.5	1:5	NE	Bond beam and center cores
			SW	Bond beam plus vertical and horizontal straps
3	7.5	1:5	NE	Bond beam, center cores, and saw cuts
			SW	Bond beam, center cores, and lower horizontal straps
4	5	1:5	NE	Upper strap
			SW	Upper and lower horizontal straps
5	11	1:5	NE	Unretrofitted control model for model 6
			SW	Unretrofitted control model for model 6
6	11	1:5	NE	Bond beam, lower horizontal straps, and vertical straps
			SW	Bond beam, lower horizontal straps, and local ties
7	5	1:5	NE	Partial diaphragm applied on attic floor and roof framing and lower horizontal and vertical straps
			SW	Same as the NE walls, except vertical straps placed only on the piers between the door and window on the north wall
8	7.5	1:5	NE	Vertical straps on north and east walls only
			SW	Vertical center core rods in south and west walls only
			Both walls	Partial diaphragms applied to the attic floor and roof framing. Horizontal strap at the floor line anchored to floor diaphragm. Lower horizontal straps.
9	7.5	1:5		Unretrofitted control model for model 8
10	7.5	1:2		Unretrofitted control model for model 11
11	7.5	1:2	NE	Vertical straps on north and east walls only
			SW	Vertical center core rods in south and west walls only
			Both walls	Partial diaphragms applied to the attic floor and roof framing. Horizontal strap at the floor line anchored to floor diaphragm. Lower horizontal straps.

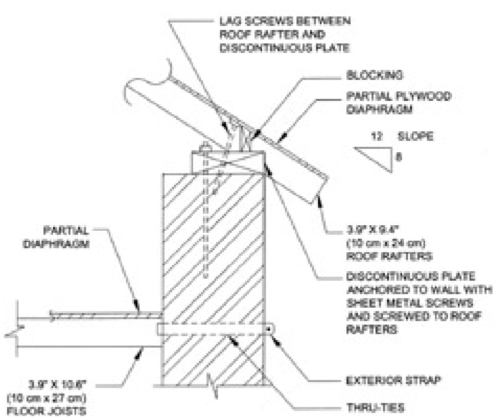

FIGURE 3 For models 7 and 9, connection and roof and floor partial diaphragms at load-bearing walls. Note that dimensions are prototype dimensions.

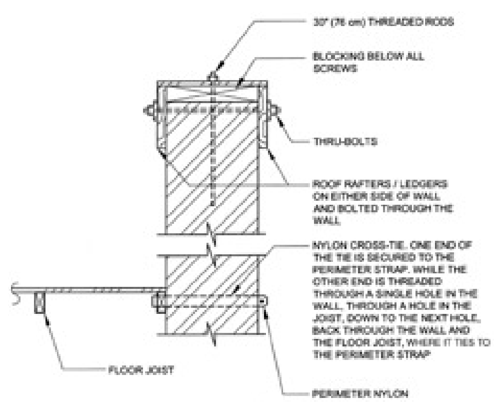

FIGURE 4 For models 7 and 9, connection between walls and diaphragms at non-load-bearing walls. Note that dimensions are prototype dimensions.

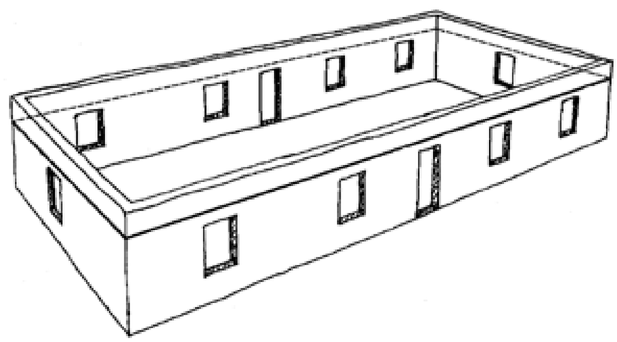

FIGURE 5 The upper wall element of a typical retrofit system.

drilled holes at the base of the walls. Small-diameter nylon cord was used for cross-ties on these straps, similar to those used with the lower horizontal straps.

Partial wood diaphragms were added to the attic floor and roof. The width of the diaphragm on the attic floor was approximately 8 in. (0.20 m), equivalent to the spacing between the floor joists. Additional straps were added to the attic diaphragm for continuity, as shown in figure 5. The width of the partial roof diaphragm was approximately 6 in. (0.15 m).

On the load-bearing walls (see fig. 3), the bearing plates on the tops of the wall were discontinuous so that they did not act as bond beams. These bearing plates were cut into four sections on top of the north and south walls. The bearing plates were anchored to the walls with 3 in. (7.6 cm) sheetrock screws. The roof rafters were anchored with screws to the bearing plates, and blocking was placed between each of the roof rafters.

The floor joists were anchored to the walls with small-diameter cord. This cord went through a hole drilled through the center of the floor joist. The cord went through the adobe wall on either side of the joist and attached to the horizontal strap on the exterior face of the wall.

On the non-load-bearing walls (see fig. 4), the roof rafters were placed directly on either side of the wall and tied together with bolts through the wall. The partial roof diaphragm was attached to the tops of the roof rafters. Six-inch (15.2 cm) screws extended through the roof diaphragm and the blocking below the diaphragm and extended into the wall. The purpose of these details was to tie the tops of the gable-end wall to the roof system. These connections worked well and did not fail during the tests.

Overall, the performance of model 7 and the behavior of the retrofit measures was extremely good. From observation of the videotapes, it appeared that substantial sections of the models might have collapsed during test level VI or VII. Instead, model 7 performed well through test level X. Only a lightly retrofitted section of the south wall collapsed during the first repetition of test level X. The out-of-plane performance of both gable-end walls was particularly impressive, as neither end collapsed.

The important aspects of the performance of model 7 are as follows:

1. The model behaved very well and generally as expected, based upon the results of the previous six model tests. The retrofitting system used on this model was clearly a success.
2. The cracking pattern was generally as predicted. The vertical and horizontal straps with cross-ties at regular intervals behaved well, even when cracks did not occur where they were expected.
3. The roof diaphragm was sufficiently stiff to prevent out-of-plane collapse of the gable-end walls. Large displacements occurred at the tops of these walls because of the flexibility of the diaphragm system, but the restraint was sufficient to prevent collapse. The roof diaphragm was particularly flexible because of the break in the diaphragm that occurred at the ridge line.
4. The horizontal diaphragm at the attic level was considerably stiffer than the roof diaphragm. The through-wall connections performed well during the tests. Horizontal cracks developed in the two gable-end walls because of the out-of-plane motions of these walls.
5. Permanent displacements of 1–2 in. (2.5–5.1 cm) occurred at the horizontal cracks in the east and west walls during tests 8, 9, and 10. The retrofitting system was sufficient to prevent collapse of these walls but not to prevent this amount of displacement.
6. The lower horizontal straps worked effectively to prevent the deterioration of the piers under

the windows. In most of the models and as expected, diagonal cracks extended from the lower corner of the windows to the corner of the building, but the straps prevented substantial widening of these cracks.

Models 8 and 9 were constructed to be nearly the same as model 7, except that the S_L ratio was 7.5. Model 9 was unretrofitted. Both gable-end walls collapsed during test 6. The west gable end would have collapsed during test 5, except for the moderate restraint provided by the roof system. Model 8 was retrofitted similarly to model 7. The retrofitting schemes on the north and east walls were nearly identical to those of model 7, with both vertical and horizontal straps. The south and west walls of model 8 had ¼ in. fiberglass center core rods (0.25 in., or 0.6 cm) placed in epoxy grout in 0.375 in. (1.0 cm) holes. Model 8 behaved well through test 10, with substantial damage but with limited offsets and no collapsed sections. The walls with fiberglass center cores behaved particularly well and sustained only minor damage.

Large-Scale Models

The two large-scale models (models 10 and 11) tested in IZIIS in Macedonia were built for direct comparison to models 8 and 9, tested at Stanford. These two models were one-half the size of the prototype building. The first model tested at IZIIS (model 10) was an unretrofitted building, and the second model (model 11) was retrofitted as the smaller scale model (model 8), using a combination of partial diaphragms, horizontal cables, vertical straps, and vertical center core rods.

Guidelines

The final product of the GSAP research effort in the 1990s was the *Planning and Engineering Guidelines for the Seismic Retrofitting of Historic Adobe Structures* (Tolles, Kimbro, and Ginell 2002).

Before plunging into the retrofit design process, the design team must devote some effort to identifying the goals that can be attained by retrofitting. Decisions must be made about the goals of the retrofitting system and how those goals might be achieved. The minimum level of intervention must provide for life safety in and around a building, but other goals for structural performance may be considered. The design may be geared toward preventing collapse or other life-safety hazards during the largest seismic events, but it may also be directed toward the minor damage that may occur during more moderate earthquakes.

Global Design

The starting point in the design process is an understanding of the basic elements necessary for global performance. Restraint at the tops of walls to prevent out-of-plane collapse is the first consideration of a retrofit design. A flexible diaphragm or other measures that prevent out-of-plane failure may be all that is necessary to prevent the collapse of many thicker-walled adobe buildings. Vertical wall elements (center cores or straps) may also be considered, to prevent collapse of thinner walls; vertical wall elements can also add ductility or strength to any adobe wall. Lower wall elements can add additional tensile capacity to an adobe wall for protection against progressive types of failures. Figure 5 is a diagram of an adobe building with upper wall cables; the drawing could also represent a partial or flexible roof or floor diaphragm. Figure 6 shows the addition of vertical straps to the retrofit system.

Crack Prediction

Schematic diagrams of a building with possible variations of cracks that are likely to occur during seismic events or that may occur from foundation settlement

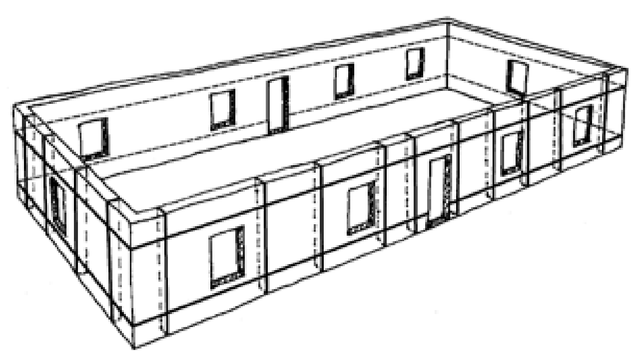

FIGURE 6 Vertical straps on the two adjacent walls, in addition to upper and lower horizontal straps.

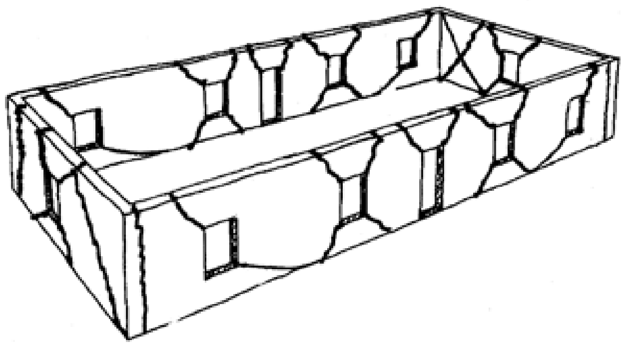

FIGURE 7 Typical predictive crack pattern for identification of wall sections that will require stabilization during strong ground motions.

can be a very useful tool in determining the possibilities of local wall failures. Each section of wall bounded by cracked elements can be a potential hazard to building occupants. Therefore, each cracked wall section should be stabilized by the retrofit system. Possible cracked wall elements are shown in figure 7.

Specific Retrofit Measures

The specifics of retrofit measures should address many different issues. These issues include out-of-plane design of the adobe walls, in-plane design, diaphragm design, and connection details. Connections are particularly difficult in adobe buildings because the low strength of the adobe material makes connections likely sources of failure. Connections should be designed such that the local failure of the adobe does not cause complete failure of the material.

Tension anchors are good examples of the type of connection that should *not* be used with a material as weak as adobe. A tension anchor such as that shown in figure 8 should *not* be used in adobe construction. A much more ductile connection can be achieved that avoids the tensile requirements on the adobe material; this can be done by designing the connection to anchor to a horizontal element, such that the adobe material will be compressed. The method shown in figure 9 will create compression on the adobe material and will be extremely unlikely to cause failure, even though there may be some crushing of the local adobe material.

Conclusion

The GSAP research effort at the Getty Conservation Institute was the single largest effort that has yet been made with regard to the study of the seismic behavior of adobe buildings and, more specifically, historic adobe buildings. Aside from just the scope of the effort, the

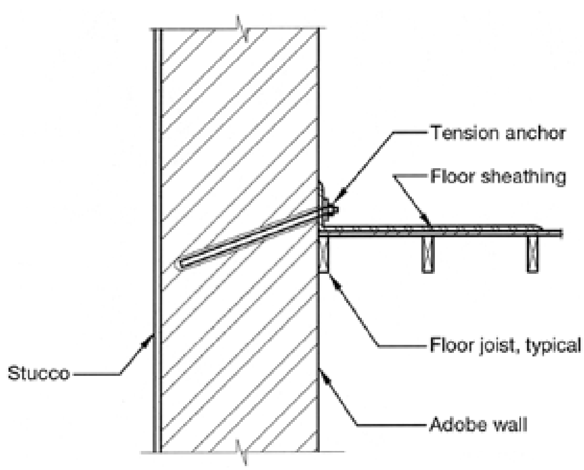

FIGURE 8 A tension anchor. Such anchors are typically not recommended for adobe buildings.

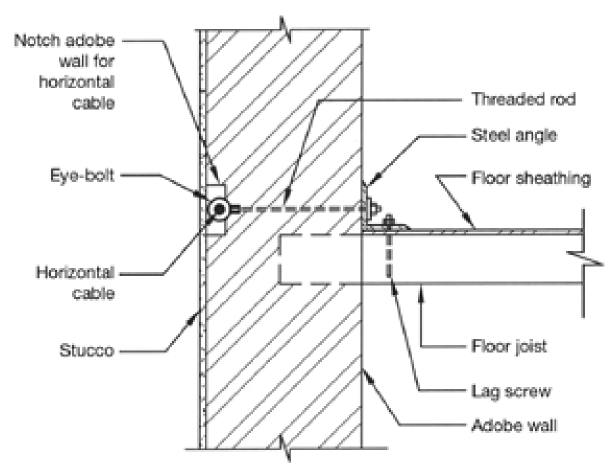

FIGURE 9 Connection between floor framing and a horizontal cabling system. This system will prevent failure due to the poor tensile capacity of the adobe material.

multidisciplinary character of the research also made this work unique in the general efforts with regard to the seismic retrofitting of historic adobe buildings.

But this work could not have been accomplished had earlier research not been conducted around the world—more specifically, in Mexico, Peru, and California. There are continuing efforts to improve the knowledge of adobe structures, and there are clearly research needs with regard to understanding the dynamic behavior of adobe buildings.

Nevertheless, the largest needs for the seismic retrofit of adobe buildings are the application and dissemination of information on a worldwide basis.

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PART TWO

Building Codes and Standards

The Peruvian Building Code for Earthen Buildings

Julio Vargas Neumann, Marcial Blondet, and Nicola Tarque

Abstract: *Every time a strong earthquake occurs in areas where earthen buildings are common, there is widespread damage, economic losses, and death caused by the collapse of earthen houses. In some cases, as in Peru, the academic and professional communities have reacted against this dreadful situation by conducting research to find adequate seismic reinforcement alternatives for earthen buildings, and the resulting solutions have been implemented in a building code.*

This paper discusses the effects of earthquakes on earthen dwellings and the technical solutions for seismic reinforcement developed at the Pontificia Universidad Católica del Perú (PUCP) (Catholic University of Peru). The Peruvian Adobe Code is then briefly described, with critical comments on some design considerations. Finally, the authors share some thoughts and reflections on the usefulness of building codes that define requirements for construction of earthen building in developing countries.

Introduction: Earthen Houses in Seismic Areas of Developing Countries

In many developing countries, earthen dwellings are a traditional housing solution because appropriate soils are abundant and inexpensive. Unfortunately, because earthen houses are built informally, every time an earthquake occurs, many of these buildings collapse, causing considerable economic losses and regrettable casualties. The earthquakes that occurred in Huaraz, Peru (1970), and in Bam, Iran (2003), caused the tragic deaths of thousands of people who were crushed under their own earthen houses.

The academic and professional communities have not remained passive in the face of this critical situation. For example, in Peru research on earthen construction in seismic zones has been performed for more than thirty years. Simple techniques have been developed to reinforce earthen buildings, and they have shown their effectiveness both in full-scale laboratory tests of adobe houses and in the field during moderate earthquakes. The principal research results have been incorporated in the Peruvian Adobe Seismic Design Code (Ministerio de Transportes y Comunicaciones 2000).

This paper describes the effects of earthquakes on earthen buildings and the technical solutions developed at the PUCP.[1] It then critically discusses important issues of the seismic design requirements provided in the current Peruvian Adobe Code and finally makes some suggestions to the most impoverished peoples of the world on the usefulness of earthquake resistant code provisions for building safe adobe houses.

Effects of Earthquakes on Earthen Buildings

Earthen houses are warm during the winter and are fresh and cool during the summer because dry soil has excellent thermal properties. However, the adobe walls have adverse seismic properties because they are heavy, weak, and brittle. Colonial earthen houses that still survive have thick walls with small openings. Currently, the land for house construction is scarce in urban areas and new adobe houses are built with slender walls, imitating the architectural configurations of “modern” masonry houses. In Peru most adobe houses are very vulnerable

FIGURE 1 Seismically vulnerable adobe house in Peru.

because they are built in imitation of the architectural features of clay brick masonry houses that have large openings, long and slender walls, and very heavy roofs (fig.1).

During earthquakes the ground shakes in all directions and generates inertial forces that earthen materials should be able to withstand. Since the compressive strength of adobe is much higher than its tensile strength, significant cracking starts in the regions subjected to tension. Seismic forces perpendicular to the walls produce out-of-plane rocking. Cracking starts at the lateral corners of the walls, where the tensile stresses are higher. Large vertical cracks that separate the walls from one another are thus produced. Front walls that overturn into the adjacent street are usually the first to collapse in an earthquake.

Lateral seismic forces acting within the plane of the walls generate shear forces that produce diagonal cracks, which usually follow stepped patterns along the mortar joints. The diagonal cracks often start at the corners of doors and windows because of stress concentration at these locations (fig. 2). If the seismic movement continues after the adobe walls have cracked, the wall breaks into separate pieces, which may collapse independently. In most cases, the adobe walls can sustain the seismic stresses due to vertical shaking. During superficial earthquakes, however, the strong vertical seismic forces may weaken walls and roofs and hasten the structural collapse. If the walls are wet, the strength of adobe masonry is drastically reduced, and the seismic vulnerability of the house increases accordingly.

Traditional adobe houses are extremely vulnerable to earthquakes. Because adobe is brittle, failure is always sudden, and the inhabitants do not have enough time to leave their houses. It is vital, therefore, to provide additional reinforcement to prevent sudden collapse during earthquakes.

Seismic Reinforcement Systems: PUCP Contribution

Initial research at the PUCP was oriented toward the experimental study of different reinforcement alternatives using locally available materials. A reinforced-concrete tilting platform was used to test full-scale adobe models (figs. 3a and 3b), where the seismic force was represented by the lateral component of the weight of the models (Corazao and Blondet 1973). The failure mode was very similar to that observed after an earthquake had occurred (fig. 3b). An internal reinforcing system within the walls—consisting of vertical cane rods anchored to the foundation, combined with horizontal crushed cane strips placed within the mortar every four layers—was quite effective in providing additional strength and deformation capacity to the walls of adobe houses (Vargas 1978; Vargas et al. 2005) (fig. 3a).

To test the effectiveness of the interior cane mesh, full-scale seismic simulation tests of adobe

FIGURE 2 Seismic cracks of unreinforced adobe house.

(a)

(b)

FIGURES 3A AND 3B Full-scale adobe models over tilting platform: reinforced adobe model with horizontal crushed cane strips showing at the corners (a), and unreinforced adobe model after the seismic simulation test (b).

dwellings were performed. The interior reinforcement, combined with a wooden *viga collar*, or bond beam, at the tops of the walls was very effective because cane and adobe masonry are compatible materials. During the most severe seismic movements, the internal mesh prevented the separation of the walls at the corners, thus maintaining the integrity of the structure (Ottazzi et al. 1989).

Unfortunately, the use of interior cane mesh has the following shortcomings: (1) to build adobe walls with internal reinforcement requires significantly more labor than to build traditional adobe walls without reinforcement; (2) cane is not available in all regions, and even in areas where cane is available, it is practically impossible to obtain the required quantity for a massive construction or reconstruction program; and (3) it cannot be used in existing houses.

In 1996 the PUCP began an experimental project to develop reinforcement techniques for existing adobe buildings. U-shaped walls were tested on the seismic simulator with different reinforcement materials, such as wooden boards, rope, chicken wire mesh, and welded wire mesh. The best results were obtained with welded wire mesh nailed with metallic bottle caps against the adobe walls and covered with cement-sand mortar. The mesh was placed in horizontal and vertical strips, simulating beams and columns. After successful testing of four full-scale models on the seismic simulator, this solution was applied to the reinforcement of existing adobe houses located in different regions of Peru (Zegarra et al. 1997). In 2001 an earthquake occurred in Arequipa, in southern Peru, and destroyed most adobe houses in the affected region. The reinforced houses, however, suffered no damage and were used as shelters (Zegarra et al. 2001). The external wire mesh reinforcement thus proved to be successful for protection during moderate earthquakes.

External reinforcement with welded wire mesh, however, also has some disadvantages: (1) it costs around two hundred U.S. dollars for a typical one-floor, two-room adobe house, an amount that exceeds the economic capacity of most Peruvian adobe users; (2) because of economic reasons, the reinforcement is only placed on wall edges, which means that it does not cover the entire wall surface; and (3) the postelastic behavior of these walls shows stiffness and strength degradation, which could lead to sudden and brittle failure during a severe earthquake.

A research project to study the feasibility of using industrial materials for the seismic reinforcement of adobe houses is being developed at the PUCP. Encouraging results have been obtained as a result of cyclic tests on both reinforced and unreinforced adobe walls (Blondet et al. 2005). Currently, several shake table tests have been performed, and the data are being processed.

Even though effective technical solutions have been developed to reduce the seismic vulnerability of adobe houses, the real problem is far from being solved, mainly because adobe builders do not accept these new construction techniques as their own. The people who build traditional, unreinforced adobe houses are reticent to change, especially if change implies higher skills, more labor, and higher cost. Consequently, it is urgent to explore ways to raise consciousness of the seismic risk among the adobe dwellers, to develop effective training techniques, and to implement programs for the safe construction of earthen buildings, in order to develop a national culture of disaster prevention.

The Peruvian Adobe Seismic Design Code

A seismic design code is an official document that contains technical specifications for the structural design and construction of buildings in seismic areas. Conventional earthquake resistant design philosophy states that buildings must not suffer any significant damage during frequent, small earthquakes, should suffer only repairable damage during moderate earthquakes, and should not collapse during severe earthquakes.

The seismic design philosophy of earthen buildings should recognize that the material is heavy, weak, and brittle. It must be accepted, therefore, that significant cracking may occur even during moderate earthquakes. However, to prevent loss of life, the building should be reinforced to prevent brittle collapse during moderate and severe earthquakes.

The first Peruvian Adobe Code was approved in 1985 as an integral part of the National Building Code (Instituto Nacional de Investigación y Normalización de la Vivienda 1987). This code has been used to develop general guidelines to generate seismic codes (International Association for Earthquake Engineering 1986; Programa Iberoamericano de Ciencia y Tecnología para el Desarrollo 1994) and used as a crucial reference for the development of seismic codes in other vulnerable countries such as India and Nepal.

The current version of the Peruvian Adobe Code (Ministerio de Transportes, Comunicaciones, Vivienda y Construcción 2000) has a rather typical format. First, it presents a declaration of scope, general requirements, and definitions of structural elements and components. Then it describes the seismic behavior of adobe buildings, gives the expression for the calculation of the seismic design force, and provides specifications for the dimensioning of the structural systems. Finally, it defines allowable stresses for the masonry and gives specifications for the design of adobe walls. Adobe buildings should be dimensioned by rational methods based on principles of mechanics and with elastic behavior criteria. However, it also recommends placing reinforcement in slender walls to improve their behavior during the inelastic phase.

The seismic action is represented by a lateral force, $H = SUCP$, where C is the percentage of weight that must be applied laterally as seismic load. C depends on the seismic zone where the building is located. In the highest seismicity zones, C is equal to 0.20. The soil factor, S, is 1.00 if the soil is good (rock or very dense soil) and 1.20 when the soil is soft or intermediate. The use factor, U, is 1.00 for houses and 1.20 for buildings such as schools or medical facilities. The weight P must include 50% of live load. Therefore, an adobe house located at a place of high seismicity with intermediate soil conditions must be designed to elastically withstand a lateral force

$$H = SUCP = 1.20 \times 1.00 \times 0.20 \times P = 0.24\,P$$

or almost one-fourth of its total weight.

Past earthquakes have shown that adobe buildings suffer much more damage when located on soft, rather than on stiff, soils. Hence, it seems to be necessary to review the Peruvian Adobe Code in order to increase the soil coefficient for adobe buildings on intermediate soils, to allow earthen construction only on rock or very dense soils.

In the Peruvian code, the country is divided into three seismic zones. The coastal region has the highest seismicity (zone 3), and construction of two-story adobe houses is not allowed there. Two-story adobe houses are only allowed in the zones of lower seismic hazard: zone 2, located in the Andean mountains, and zone 1, within the Amazon jungle, as long as the second story is built with a lightweight material such as *quincha* (wooden frames filled with crushed cane and plastered with mud).

Some general recommendations for good seismic behavior are that adobe houses must have sufficient wall density in both principal directions, with floor plans as symmetric as possible. Wall openings should be small and centered, and reinforcement should be provided to tie the walls together. Foundations and plinths should be built with cyclopean concrete (unreinforced concrete mix, made with medium and large stones) or stone masonry.

The adobe walls must be designed to elastically withstand seismic forces and to transmit them to the foundation. The allowable stresses are: (1) compressive strength of adobe blocks, f_o = average strength of 6 cubes, or $f_o = 12$ kgf/cm^2 (170.7 psi); (2) compressive strength of adobe masonry, $f_m = 0.25\, f'_m$, where f'_m is the compressive strength of adobe masonry piles, or $f_m = 2$ kgf/cm^2 (28.4 psi); (3) crushing strength of adobe masonry = $1.25\, f_m$; (4) shear strength of adobe masonry, $V_m = 0.40\, f_t$, where f_t is the ultimate strength of small walls tested under diagonal compression, or $V_m = 0.25$ kgf/cm^2 (3.6 psi). Adobe blocks are usually rectangular or square in plan. The cubes required for compression tests are made by cutting adobe blocks in such way that the size of the cube is the thickness of the block. Masonry piles are made with four or five adobe blocks joined with mortar and placed vertically on top of one another. Diagonal compression specimens are small square walls. Their side measures approximately the length of one and a half blocks. The specimens are tested by applying a compressive force along their diagonal.

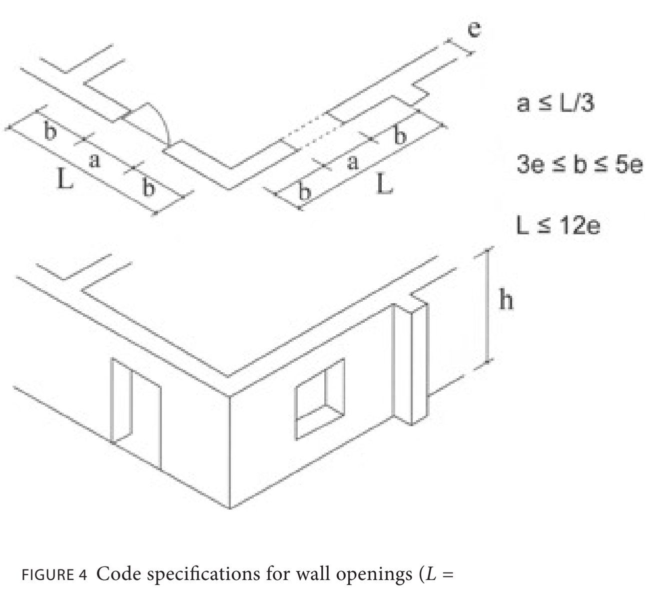

FIGURE 4 Code specifications for wall openings (L = length of wall, a = opening length, b = distance from opening to corner of reinforcement, e = wall thickness, and h = wall height).

All adobe walls must be adequately braced by transverse walls, buttresses, or reinforced concrete columns. Horizontal braces can be provided by wooden or concrete bond beams. The code provides geometric specifications to guarantee reasonable seismic behavior. Maximum wall length between braces is twelve times wall thickness. Openings must be centered and short (fig. 4).

The presence and amount of reinforcement required depend upon wall slenderness, λ (ratio of wall height over wall thickness, λ = h/e) (see fig. 4). The reinforcement of adobe walls can be made out of cane, welded wire mesh, or concrete.

The code requires the use of bond beams on the tops of all adobe walls. This requirement is reasonable because it is consistent with experimental evidence that shows that the bond beam integrates the walls and helps to delay collapse of the walls after they have developed vertical cracks at the corners. Additionally, the use of bond beams contributes to a more effective distribution of the weight of the roof over the walls and includes the roof in the overturning control of exterior walls.

Table 1 shows that walls with slenderness ratios of $\lambda \leq 6$ can be built without reinforcement. This specification contradicts field and laboratory observations that walls without reinforcement show brittle failure (though not collapse) after they have cracked in response to the seismic action. For walls with slenderness ratios between 6 and 8, the code requires horizontal and vertical reinforcement elements only at wall intersections. However, the collapse of heavily cracked adobe walls that have separated into independent pieces can only be avoided by having a continuous reinforcement configuration along the entire wall. The code also allows the construction of slim walls, with slenderness ratios between 8 and 9 (and up to 12 with technical validation), that must be integrally reinforced. It would seem too risky to build such slender walls in zones of high seismic hazard. It seems, therefore, that these code specifications are not conservative and are unsafe. Continuous reinforcement should be mandatory for all adobe walls, independent of their slenderness, at least for zones of high seismicity and where collapse of adobe houses has been reported. The maximum slenderness requirements should depend on the seismicity of the building site.

TABLE 1 **Reinforcement specifications for adobe walls (slenderness, λ, is the height-thickness ratio)**

Slenderness	Mandatory reinforcement	Minimum wall thickness (m)	Maximum wall height (m)
$\lambda \leq 6$	Bond beam	0.4–0.5	2.4–3.0
$6 \leq \lambda \leq 8$	Bond beam + horizontal and vertical reinforcement elements at wall joints	0.3–0.5	2.4–4.0
$8 \leq \lambda \leq 9$	Bond beam + horizontal and vertical reinforcement elements along wall length	0.3–0.5	2.7–4.5

Conclusion

The aim of any building code for earthquake resistant design of adobe buildings should be to disseminate the construction knowledge that will guarantee safety and be economical for the users.

The earthquake resistant code provisions for adobe are addressed to professionals involved in the design and construction of adobe buildings. In most countries, only certified professionals are legally allowed to approve and sign off on design projects, and these professionals belong to the formal system; very few people live in earthen houses designed in accordance with the code. Most of the people that build and live in adobe houses do not know or use the code; therefore, most adobe codes for seismic areas do not fulfill their aim because they do not reach the users that they should benefit.

In order to take care of the needs of the majority of people who do not know or use the code, it is necessary to use tools that complement the code, such as construction manuals and booklets, as well as educational campaigns carried out through popular organizations, local governments, and the media. This will help disseminate the basic concepts of earthquake resistant construction for adobe houses.

The code provisions should faithfully collect the acquired knowledge from research programs and from observation of the effects of past earthquakes. This knowledge must be translated into simple and direct recommendations that can be implemented by the dwellers, who have limited technical support. The acquired knowledge should not be distorted with less-demanding requirements in an effort to reach a greater number of users. This would be a serious blunder, which would be equivalent to lowering the quality of medicine in order to make it more affordable to more patients.

The contents of the code are the result of considerable research efforts to reduce the consequences of earthquakes, especially in highly populated areas. A well-conceived code is an indispensable tool to guide the professional community in the design and construction of affordable and safe earthen buildings.

It is clear that, in order to succeed, any massive dissemination and implementation program on safe earthen construction must have political support from the government. The professional community, however, has the responsibility of disseminating among adobe builders the knowledge to mitigate the risk of damage and loss of life in earthen houses in seismic areas, which today has reached unacceptable levels.

Note

1 A version of this paper was previously published as Marcial Blondet, Julio Vargas, and Nicola Tarque, “Building Codes for Earthen Buildings in Seismic Areas: The Peruvian Experience,” in *Proceedings for the First International Conference Living in Earthen Cities—Kerpic ’05, 6–7 July 2005, ITU-Istanbul, Turkey* (Istanbul: Istanbul Technical University Faculty of Architecture, 2005).

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New Zealand: Aseismic Performance-Based Standards, Earth Construction, Research, and Opportunities

Hugh Morris

Abstract: *New Zealand has a combination of owner-built earth buildings and high-value earth houses built by contractors. This paper outlines the historical context of earth construction, comments on the status of conservation, gives an overview of the development of the New Zealand earth building standards, and identifies research opportunities.*

Introduction

In 1998 a suite of three limit state earth building standards was published in New Zealand for Design, Materials and Workmanship, and Earth Buildings Not Requiring Specific Design. The standards were developed by a committee of architects, engineers, and builders to cover adobe, rammed earth, and pressed brick construction.

A modest amount of research was undertaken to confirm parameters for the standards. Tests included in-plane testing for a range of reinforcement types, bond testing of unstabilized abobe, and durability. Concepts in the standards that relate to out-of-plane performance using an energy method are outlined, as is the need for further theoretical research, testing, and review.

Reinforcement is required within the walls of adobe buildings in most parts of New Zealand and is predominantly of steel and plastic geogrid. This has been successfully implemented, but further research and testing of geogrid-reinforced walls are needed. Other key aseismic features are timber diaphragms and bond beams. Developments are continuing on fiber-reinforced earth wall panels and thermal performance of earth materials and buildings. Reliable low-cost test methods to predict durability are needed, but reliable verification procedures also need to be developed. A method of measuring surface erosion of existing houses with 3-D stereophotogrammetry is showing promise. Verification of low-cost durability evaluation methods is a major research need.

New Zealand Seismicity

New Zealand is on the boundary of the same Pacific tectonic plate as is the western seaboard of America and has a similar seismic hazard level to that of California. The tectonic context is the Pacific Plate subducting the Indo-Australian Plate to the north and east, and the plates shearing along the length of the South Island, as shown in figure 1 (National Earthquake Information Center 2003). South of the country, the reverse subduction occurs, with the Pacific Plate overriding the Indo-Australian Plate. The surface evidence of the tectonic movement consists of the substantial mountain peaks and the Alpine Fault along the west coast of the South Island, and a lower-elevation mountain range that continues to the eastern corner of the North Island.

Earthquakes and Their Influence on Construction

New Zealand's first human inhabitants were Maori from Polynesia, who arrived around AD 1200 and lived in houses predominantly made of timber and reeds (Best 1974). The Maori passed on an oral history of major

FIGURE 1 New Zealand seismicity, 1900–2002, and major earthquake events and volcanoes. Detail of USGS poster (National Earthquake Information Center 2003). Credit: U.S. Geological Survey, Department of the Interior.

earthquakes; however, their lightweight housing was not seismically vulnerable.

Captain James Cook landed in New Zealand in 1769, and slow British settlement followed, with housing ranging from crude huts made of reeds to two-story timber-framed houses, including some of stone and brick. Settlement accelerated in the 1840s, and when the town of Wellington had reached a population of forty-five hundred, in 1848, citizens experienced high-intensity shaking from a major earthquake of M_L7.5 (magnitude based on the energy released similar to the Richter scale) centered across the straits in the upper South Island. This was soon overshadowed by MMI X intensity (Modified Mercalli intensity in Roman numerals, based on the severity of damage) caused by the M_L 8.2 earthquake of 1855, with an epicenter at a distance of about 20 km (12.4 miles). Most masonry buildings were damaged by this quake, so from that time, timber houses were usually constructed (Downes 1995; Grapes, Downes, and Goh 2003).

There were other significant earthquakes, but it was a major event in 1931 (M_L 7.9, intensity MMI X) in Hawkes Bay, on the east coast of the North Island, that resulted in New Zealand's largest natural disaster. This quake killed 256 people and demolished a number of buildings in Napier; the center of town was devastated by the resulting fire. The New Zealand Standards Institution was then formed, and the first building code was published in 1935 (Conly 1980). Building standards for all significant structures have been enforced by territorial authorities (i.e., councils) since that time.

History of Earth Buildings

A large number of temporary earth buildings were built during the gold rush days in the 1860s, but few remain because roof materials were removed for reuse, and the walls degraded quickly in the damp climate (Allen 1990). Of the more permanent buildings, approximately 121 earth houses constructed between 1840 and 1870 still exist; an additional 168 survive from 1870 to 1910. There was little activity from that time until the 1940s, when a number of houses were built of cement-stabilized earth with technical support from P. J. "Pip" Alley (Alley 1952), an enthusiastic academic at Canterbury University who was involved in a short burst of activity to cover materials shortages that followed World War II. Earth housing declined again until the late 1980s, when growing interest in environmentally friendly and sustainable buildings led to an upsurge of earth building construction (Allen 1997). Some 30 to 40 earth buildings are now built each year, which corresponds to approximately 0.15% of new houses countrywide. In some localities over 1% are constructed of earth.

Of the extant older earth buildings, those of adobe and cob construction are the most common. The main forms of earth construction at present in New Zealand are adobe, rammed earth, and pressed brick. Adobe bricks usually use straw in the mix, and there is a range of construction, from small owner-built houses up to luxury homes built by specialist contractors. Some mid-range adobe houses are shown in figures 2 and 3.

Conservation

New Zealand was not significantly populated by Europeans until the 1840s, so its heritage buildings are very recent when compared internationally. There was little conservation expertise until the 1980s, when

FIGURE 2 Typical midrange adobe home in New Zealand.

FIGURE 3 Typical midrange adobe home in New Zealand.

serious conservation plans were written and ICOMOS (the International Council on Monuments and Sites) was established in New Zealand, with a conservation charter completed in 1993.

The Historic Places Trust was established by an act of Parliament in 1954; the trust actively preserved historic buildings, purchased several dozen (mostly timber) structures, and placed many more on a register. However, no restriction was put on many lower-ranked historical buildings. Considerable damage was done to buildings as they were repaired by well-meaning amateurs. A more serious problem is particularly evident on Auckland's main street. During the 1970s and 1980s, a number of beautiful old buildings were largely demolished. Only their historical frontage was retained, then incorporated into new buildings in a superficial attempt at conservation.

Much of the very early New Zealand settlement occurred in the far north of the country, where seismicity is low. Two of the prominent historical buildings still standing are a tannery and a bookbindery that were developed in French provincial style for Roman Catholic bishop Pompallier. A French architect directed the construction of lower walls of pisé (rammed earth) fabricated from local soils and crushed shells, with earth panels within the timber-framed second story. In 1967 Pompallier House came under the Historic Places Trust, and in 1990 a major conservation effort was undertaken by a local enthusiast who managed reconstruction of a major wall section using original materials and methods. The decision to return the building to its original form has been the subject of differing conservationist opinions.

Another notable example of an 1850s earth building that has survived three major earthquakes (MMI VII or greater) is Broadgreen House, near Nelson in the upper South Island (fig. 4). The apparent factors that account for the good performance of this large two-story cob building are the low height-to-thickness ratio of the earth walls, the relatively few openings, sufficient earth bracing walls in each direction, the first floor acting as a structural diaphragm, and relatively good-quality earth wall construction. The 50 cm (19.5 in.) thick earth walls of the ground floor reach 2.7 m (8.9 ft.) to the first floor, giving a height-to-thickness ratio of 5.4, which complies with present design criteria for unreinforced earth walls in New Zealand.

FIGURE 4 Broadgreen House, with lower-story cob walls. Photo: Richard Walker.

Design Guidelines and Standards

In the 1980s considerable earth construction was undertaken in Nelson, a seismically active area at the northern end of the South Island. Two engineers became actively involved and did most of the design work. Engineer Gary Hodder wrote a design guide (Hodder 1991) that allowed prospective owners and architectural designers to do preliminary designs before seeking approval from him or another engineer for verification and sign-off. Hodder recognized that more work was needed, and in 1991 the Earth Building Association (EBANZ) took the initiative to develop guidelines for earth buildings that paralleled the New Zealand building standards for other materials (www.earthbuilding.co.nz; current membership of 275).

In 1993 the project was formally adopted jointly by Standards New Zealand and Standards Australia. A valuable exchange of experience and technical expertise came from the collaboration. However, the difficulty of satisfying the national requirements of both New Zealand and Australia led to a disbanding of the joint effort and the formation of separate committees in 1997. A major point of difference was the regulatory environment around house construction in New Zealand, which is partly due to New Zealand's higher seismic risk. The final years of writing were completed as a Standards New Zealand project. Standards Australia went on to support the publication of *The Australian Earth Building Handbook* (Walker and Standards Association of Australia 2002), and the Earth Building Association of Australia went on to produce *Building with Earth Bricks and Rammed Earth in Australia* (Andrews and Gales 2004).

New Zealand Building Legislation

The New Zealand *Building Act 2004* (New Zealand 2004) established a framework of building controls and construction that must comply with the mandatory New Zealand Building Code. Approved documents provide methods of compliance with the Building Code, and New Zealand Standards are one way to comply with the code.

The first such approved document for nonengineered construction was NZS 3604, *Code of Practice for Light Timber Frame Buildings Not Requiring Specific Design* (Standards New Zealand 1978). Timber is used in over 90% of New Zealand house construction, so this established the precedent for this type of document. Earth building standards have needed to provide a comparable level of detail to satisfy the territorial authorities and builders familiar with NZS 3604. The latest version, *NZS 3604: 1999 Timber Framed Buildings* (Standards New Zealand 1999) now has four hundred pages with numerous tables and well-drawn diagrams that allow builders and architectural designers to design houses to resist earthquake and wind loads.

New Zealand Earth Building Standards

Three comprehensive performance-based standards for earth-walled buildings were published in 1998. Substantial documents were needed for design and construction that used a performance-based approach to comply with the general standards framework. These have been approved as a means of compliance with the New Zealand Building Code. The standards were prepared by a joint technical committee of engineers, architects, researchers, and builders and were developed over a period of seven years. These documents have made a significant contribution to the increased acceptance of earth building in New Zealand.

The standards are described below, and some of the supporting research follows in a subsequent section.

Engineering Design of Earth Buildings

NZS 4297: Engineering Design of Earth Buildings (Standards New Zealand 1998a) specifies design criteria, methodologies, and performance aspects for earth-walled buildings and is intended for use by structural engineers.

Limit state design principles were used in the formulation of this standard, so that it would be consistent with other material design standards. Earthquake loads are more critical than wind loads for most earth buildings in New Zealand, and earth wall heights are limited to 6.5 m (21.3 ft.) in this standard. The design methodologies are discussed in more detail later in this paper.

Materials and Workmanship for Earth Buildings

NZS 4298: Materials and Workmanship for Earth Buildings (Standards New Zealand 1998b) defines the material and workmanship requirements to produce

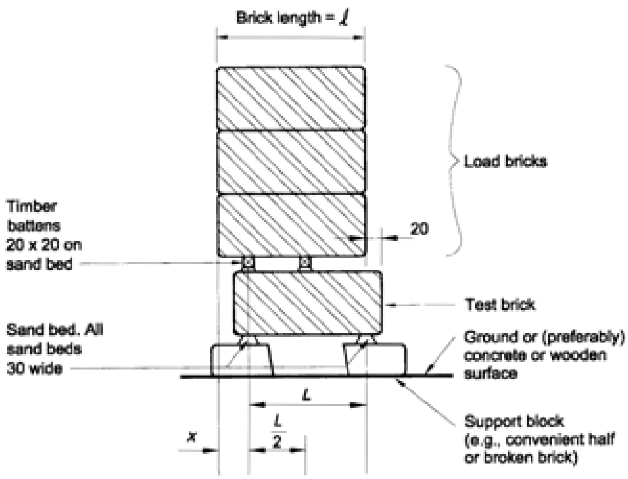


FIGURE 5 Stacked brick modulus of rupture test (measurements are in millimeters). Originally published in NZS 4298 (Standards New Zealand 1998b, 66). Content reproduced from NZS4299/NZS4297/NZS4298 with the permission of Standards New Zealand under License 000738. www.standards.co.nz.

earth walls, which, when designed in accordance with NZS 4297 or NZS 4299 (Standards New Zealand 1998c), will comply with the requirements of the New Zealand Building Code. Requirements are given for all forms of earth construction—but more specifically for adobe, rammed earth, and pressed brick.

The suite of standards is primarily intended for small-scale construction and includes a number of simple, low-cost test procedures that are defined in the materials and workmanship standard. This testing can be done by the person responsible for the construction of the building in the presence of the owners or the controlling building authority, as required.

Compression or simplified modulus of rupture tests are specified for determining the strength of the earth wall materials. Compression tests require a laboratory, but two simple field procedures are detailed for modulus of rupture tests of earth bricks, including the stacked brick test (fig. 5). A brick drop test is also specified for simple field testing of earth bricks.

Two grades of earth wall material are covered within the standard:

- Standard Grade, with a design compressive strength of 0.5 MPa (72.5 psi), which can be obtained by low-strength materials with a minimal amount of testing.
- Special Grade, which requires more testing to reasonably predict the characteristic strength. Earth stabilized with cement may achieve strengths of up to 10 MPa (1450.4 psi). More complex engineered structures would be of Special Grade.

Further technical details are available elsewhere (Walker and Morris 1998; Morris and Walker 2000). NZS 4298 also includes durability requirements, which are significant in the temperate New Zealand climate.

Earth Buildings Not Requiring Specific Design

NZS 4299: 1998 Earth Buildings Not Requiring Specific Design (Standards New Zealand 1998c) provides methods and details for the design and construction of earthen-walled buildings not requiring specific engineering design. The document will be mainly used for designing houses, and users will include those in the earth building industry, such as builders, architects, engineers, students, and building authority staff.

This standard covers buildings with single-story earth walls and a timber-framed roof, or single lower-story earth walls with timber second-story walls and a light timber framed roof. The scope is limited to footings, floor slabs, earth walls, bond beams, and structural diaphragms. The design of the timber roof structure would be covered by *NZS 3604: 1999 Timber Framed Buildings* (Standards New Zealand 1999), or specific design could be undertaken by a certified professional engineer.

NZS 4299: 1998 Earth Buildings Not Requiring Specific Design (Standards New Zealand 1998c) is the earth wall construction equivalent of NZS 3604, with a similar methodology. It is intended to provide a means of compliance with the New Zealand Building Code. Earth buildings covered by this standard resist horizontal wind and earthquake loads by load-bearing, earth bracing walls that act in-plane in each of the two principal directions of the building. A simple design methodology uses tables in terms of "bracing units" for determining the "bracing demand" required for the building; the "bracing capacity" is provided by the nominated bracing walls, as shown in figure 6. This methodology is familiar to designers and builders, almost all of whom are using the same approach with NZS 3604 for timber-framed buildings (Standards New Zealand 1999).

Many construction details that have been proved in earth buildings constructed in New Zealand during the past twenty years are included in the standard. Specific details from the standard are shown in figures 7 and 8. Figure 9 shows the reinforcement being placed during construction.

Design Approach

Design methodologies for earth buildings in New Zealand have been adapted from existing masonry and concrete standards. The approach in the standards is based on reinforced concrete design theory and uses limit state design principles for both elastic and limited ductile response. The structural ductility factor was taken as 2.0 for reinforced earth walls, 1.25 for the narrower Cinva brick walls, and 1.0 (equivalent to elastic response) for unreinforced and partially reinforced earth walls.

In *NZS 4299: 1998 Earth Buildings Not Requiring Specific Design* (Standards New Zealand 1998c), the earth walls were designed as spanning between the reinforced concrete foundation at the bottom of the wall and the top plate or bond beam at the top of the wall. Loads from the tops of walls, roofs, and timber second stories were assumed to be distributed by concrete or timber bond beams or structural ceiling, roof, or first-floor diaphragms to transverse earth bracing walls.

Out-of-Plane Loads

Ultimate strength reinforced concrete theory is cautiously used as the basis for designing reinforced earth walls. Generally, vertical reinforcing is considered to provide the tensile force for reinforced earth wall panels to work in flexure against out-of-plane face loading.

An energy method is used for assessing the ultimate limit state seismic out-of-plane resistance of unreinforced walls spanning vertically. Rather than elastic strength at first cracking, the energy approach is based on the collapse mechanism when the displacement of the wall moves beyond stability. The method is described with some questions in the out-of-plane analysis section near the end of this paper.

Using the energy method, unreinforced earth walls for low-earthquake zones (zone factor $Z \leq 0.6$) were found to be satisfactory for the maximum wall heights permitted in the standard. For example, the failure of a 2.7 m (8.9 ft.) high and 28 cm (10.9 in.) thick wall was calculated to occur at 178% of the calculated demand requirement with $Z \leq 0.6$.

In-Plane Loads

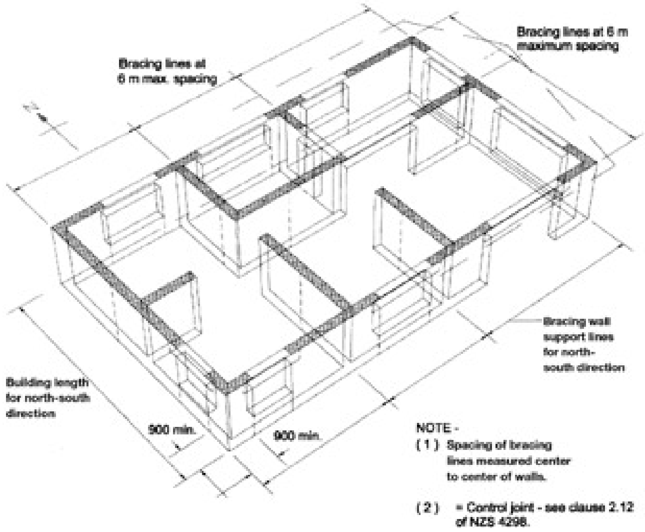
Earth bracing walls provide seismic load resistance in each principal direction of the building. Reinforced earth walls are reinforced vertically and horizontally to provide some in-plane ductility and to develop extra shear strength.

The reinforcement permits the use of smaller seismic design loads when a planned ductile failure mode is designed for the structure. The designed failure mode is in-plane bending of the earth bracing walls with yielding of vertical reinforcing at each end of the wall. Shear failure of these walls is prevented typically by the use of well-distributed horizontal reinforcing. Vertical reinforcement is kept to a reasonable minimum, to limit in-plane shear loads and foundation forces. Unreinforced walls provide considerably less bracing capacity without the vertical and horizontal reinforcement. Shear failure is prevented solely by the shear strength of the earth.

The maximum bracing capacity provided by a reinforced earth wall 2.4 m long, 2.4 m high, and 28 cm thick (7.9 ft. long, 7.9 ft. high, and 10.9 in. thick) with typical details in accordance with the standard (see fig. 8) was calculated to be 30 kN (6744 lb.). The bracing capacity provided by a similar-sized unreinforced earth wall in a low earthquake zone was calculated to be 10 kN (2248 lb.).

Statistics for Testing

Because users may undertake tests to establish the earth material strength, some simple statistics are required to establish the characteristic values. Soils used in earth building are quite variable, but the compressive strengths of dried or compressed earth materials usually have a coefficient of variation (C_v) between 0.15 and 0.3. No sets of test data large enough to establish the underlying statistical population distribution were found.



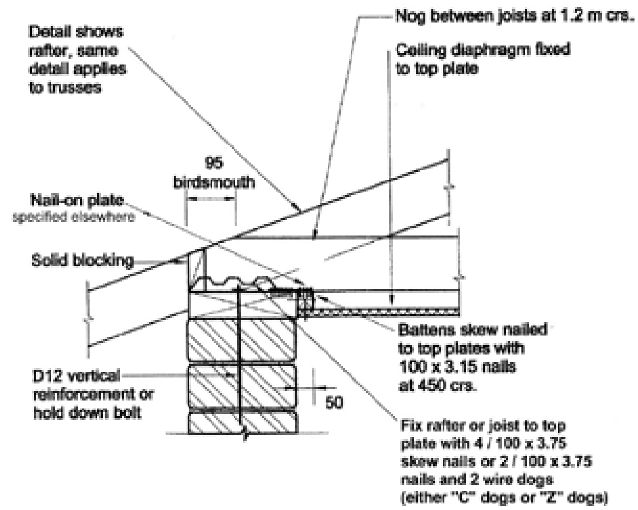
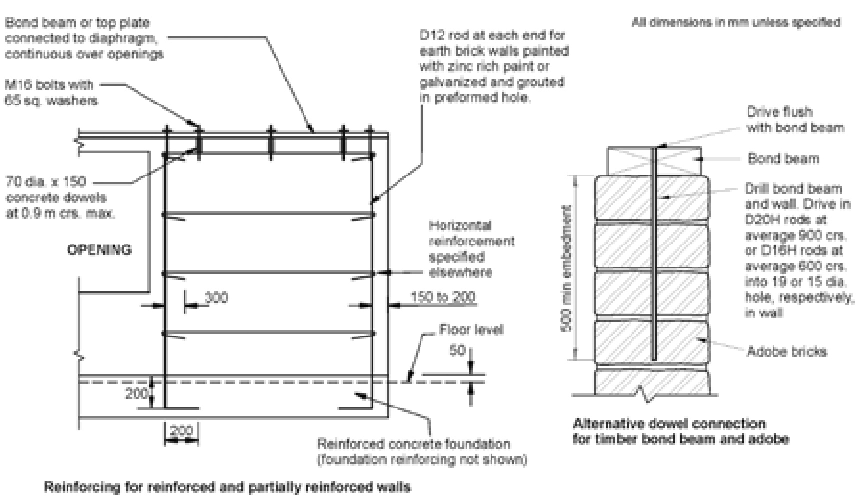


FIGURE 6 Bracing line method of assessing lateral resistance. Originally published in NZS 4299 (Standards New Zealand 1998c, 47). Content reproduced from NZS4299/NZS4297/NZS4298 with the permission of Standards New Zealand under License 000738. www.standards.co.nz.

FIGURE 7 Diaphragm ceiling detail from NZS 4299 (note that the illustrated steel connector has now been replaced with other nailed and "wire dog" details). Originally published in NZS 4299 (Standards New Zealand 1998c, 71). Content reproduced from NZS4299/NZS4297/NZS4298 with the permission of Standards New Zealand under License 000738. www.standards.co.nz.



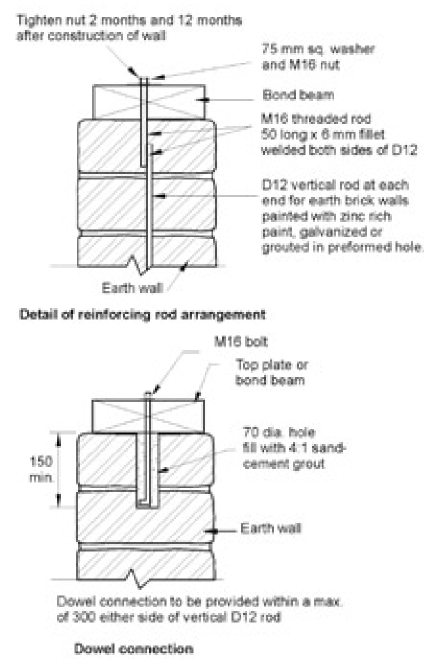


FIGURE 8 Typical NZS 4299 reinforced wall detail—polypropylene geogrid or steel used horizontally. Originally published in NZS 4299 (Standards New Zealand 1998c, 57). Content reproduced from NZS4299/NZS4297/NZS4298 with the permission of Standards New Zealand under License 000738. www.standards.co.nz.

FIGURE 9 Geogrid reinforcement at corner joint. Photo: Richard Walker.

The Australian Masonry Standard AS 3700 (Standards Association of Australia 1991) determines the characteristic strength from 30 specimen tests. This is not viable for a simple house because of the effort to construct specimens and the cost of testing. A 5-specimen simplified approximation is used to determine the characteristic strength

$$f' = \left(1 - 1.5\frac{x_s}{x_a}\right)x_1 \quad (1)$$

where x_1 is the lowest of the five results, x_s is the standard deviation, and x_a is the mean. The standard includes the more reliable Ofverbeck power method (Hunt and Bryant 1996) for sample sizes of 10 to 29. This method, which is presented in a simplified form, is not dependant on knowing the population distribution to determine the characteristic strength.

An example from NZS 4298 is given in table 1, where the lowest three values of a series of ten results are used to determine the characteristic strength. If there were between 20 and 29 samples, then the lowest four values would be used to determine the characteristic strength. Coefficients would be selected from a similar table with different values.

Research in Support of the Earth Building Standards

There were many contributors to the earth building standards, as well as a depth of knowledge based on local experience. This gave access to informal literature based on personal experimentation and results of laboratory testing associated with previous buildings. The standards committee also compiled the best of the literature we could locate. For my part, there were a range of practitioners who suggested research and contributed to a variety of experimentation that gave a feeling for the materials and an overview of the problem.

Some of the tests undertaken under my supervision were:

Table 1 Determination of characteristic compressive strength for earth material using a series of ten specimen tests (from Standards New Zealand 1998b, 52). Content reproduced from NZS4299/NZS4297/NZS4298 with the permission of Standards New Zealand under License 000738. www.standards.co.nz

For the number of test specimens in the sample, n, between 10 to 19, the characteristic strength is:

$f' = x_3^{1-\varepsilon}\,(x_2 x_1)^{\varepsilon/2}$ where, for n = 10–19, ε is given by:

n	10	11	12	13	14	15	16	17	18	19
ε	3.31	3.12	2.96	2.80	2.66	2.53	2.41	2.29	2.19	2.08

Example: For a series of 10 test results for which the lowest values are 1.45, 1.75, and 1.84. For n = 10, the ε value is 3.31;

therefore $f' = x_3^{1-3.31}\,(x_2 x_1)^{3.31/2} = 1.84^{-2.31}\,(1.75 \times 1.45)^{1.66} = 1.14$

Note that x_1, x_2, x_3, x_4 are the lowest, second lowest, third lowest, and fourth lowest test results.

- In situ testing of parts of a rammed earth house in Wellington prior to demolition.
- Approximate modulus of rupture testing of small soil-cement beams.
- Flexural tests on 35 × 35 cm (13.7 × 13.7 in.) soil-cement beams with longitudinal pretensioning.
- Investigations of the performance of soil-cement, comparing compaction, cement contents, and strength.
- Determination of the approximate tensile strength of soil-cement using the diametral tensile strength method to compare with compressive strength.
- Plotting of stress-strain curves to determine the approximate elastic modulus of soil-cement.
- Evaluation of height-to-width ratios for compression tests.
- Influence of wetting time and mortar thickness on mortar bond.
- Drip test and spray test comparisons.
- Development of the surface soak test.
- Diagonal compression tests on 1.2 m (3.9 ft.) wall panels with differing reinforcement.

Student work on the bond strength and similar work on rammed earth was reported at the SimsoAdobe conference in Peru (Morris 2005).

Shabani Gurumo (Gurumo 1992) did the 1.2 m (3.9 ft.) adobe wall panel tests with differing reinforcement regimes. The results clearly indicated that diagonal compression with reinforcement carried almost twice the load of unreinforced adobe.

Gurumo also tested out-of-plane flexural bond strength with a simple bond wrench, giving variable but extremely low bonds of around 50 kPa (7.25 psi). This may have been due to the experience of the masons with adobe and to inadequate soaking of the bricks, but it has led to a conservative expectation for the standards.

A near full-scale 1.8 × 1.8 m (5.9 × 5.9 ft.) adobe wall panel was quasi-statically earthquake tested, with horizontal slowly reversing in-plane loads applied to the top edge of the wall. Subsequent to this, a 1.2 × 1.8 m (3.9 × 5.9 ft.) wall panel was similarly tested by student Bernard Jacobson. Figure 10 is a plot of the load deformation performance of the top of the

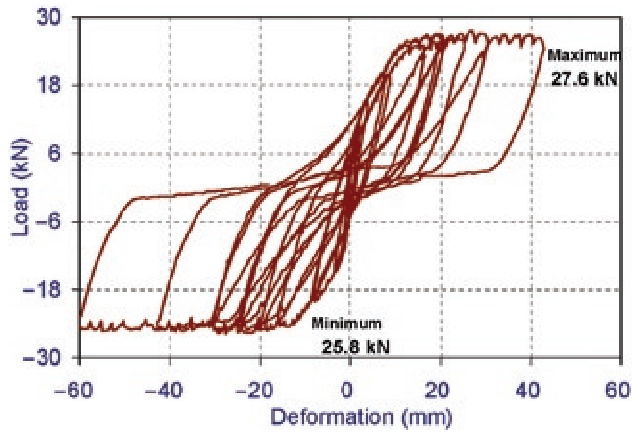


FIGURE 10 Cyclic load performance of a 1.2 × 1.8 m (3.9 × 5.9 ft.) adobe wall.

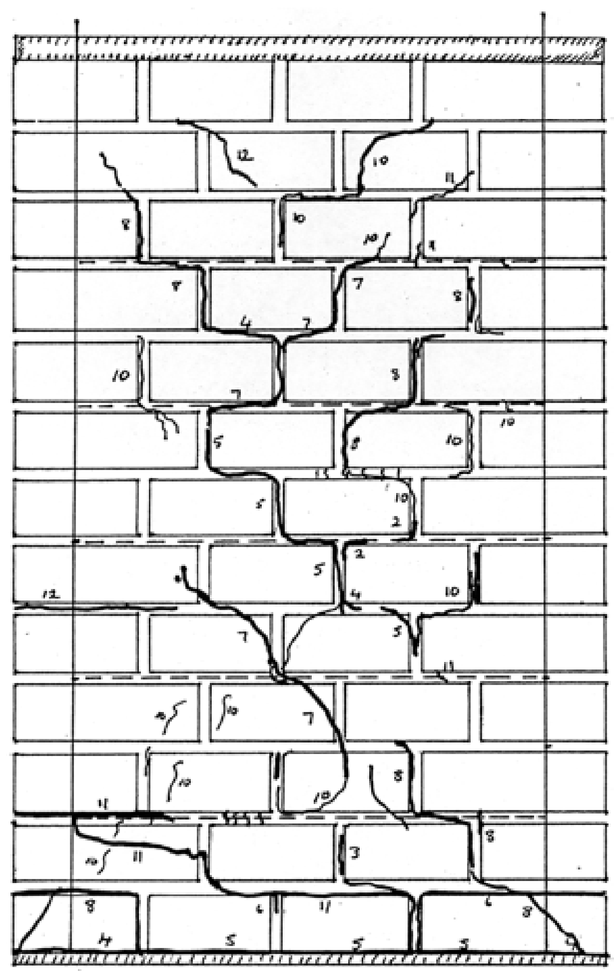


FIGURE 11 Crack pattern of an adobe wall, showing the load progression.

FIGURE 12 Detail of reinforcement, with vertical rods in holes through the adobes and horizontal reinforcement wrapped around the vertical rods. Now geogrid is more typically used for horizontal reinforcement.

wall. This graph shows that slipping in the mortar planes provided effective ductility to the wall system (Morris 1993). Figure 11 illustrates the crack patterns in a wall with both horizontal and vertical reinforcing, as observed by Jacobson. It shows the crack growth progression of the wall as the reversing loads were applied; the load to the right is recorded as positive. Figure 12 shows the reinforcing detail during demolition following the wall tests.

Soil-cement rammed earth walls were tested and carried much higher loads, but they required reinforcement to prevent brittle failure. These adobe walls with internal reinforcing behaved in a ductile manner in-plane, but they are low in strength. This requires most walls within a structure to be available to provide the needed bracing strength.

None of the above testing was definitive, but it did give indicative performance in setting values for the standards. The most significant need is for tests of a large enough number of specimens to establish a proper statistical basis for what is a quite variable material.

Statistics for Out-of-Plane Wall Strength

Some statistical simulation was done to establish a suitable parameter to take into account the averaging effect of multiple blocks acting together. This is significant, given the high coefficient of variation for earth materials.

The reliability of wall strengths can be considerably higher than the characteristic strength of one brick (often the 5 percentile value). If one brick from a row of bricks is weaker than the others, then there will be load sharing with the adjacent, stronger bricks. A Monte Carlo simulation of the strengths of individual blocks according to the coefficients of variation was run to determine the reliable strength for different numbers of bricks in layers. The 15% increase in strength (k_m factor of 1.15) is permitted for the normal range of coefficients of variation (C_V). For a higher C_V the characteristic strength will be lower, as a proportion of the average, so when enough tests establish the C_V with enough reliability, a k_m of 1.3 is allowed where more than ten bricks are working together in a row.

Recent Research and Future Development

Natural Fiber Reinforced Soil-Cement

Recent research work in Auckland has involved the use of native flax fiber (similar to sisal fiber) to reinforce soil-cement to make monolithic walls. This offers the possibility of thinner walls but raises the issue of thermal performance. With the building regulations for thermal performance focused on insulation, this presents a challenge to prove the effectiveness and value of thermal mass. Existing earth buildings have been monitored for thermal performance, and this has been used to check the calibration of a thermal performance model (Tenorio et al. 2006). This will allow the evaluation of various thicknesses and configurations.

Durability

The durability of earth walls is of concern in both temperate and tropical climates. A need exists for a test approach that is simple and low in cost. The New Zealand standards have two tests modified from those developed in Australia. The accelerated spray test uses an expensive standard nozzle and sprays a very severe jet, as shown in figure 13. This can cause the failure of otherwise satisfactory adobe materials. This severe test is complemented by a very simple drip test, where water drops 40 cm (15.6 in.) onto the surface. This technique was checked by some simple laboratory experiments that considered raindrop energies, but the test needs field verification.

FIGURE 13 Accelerated degradation water spray test.

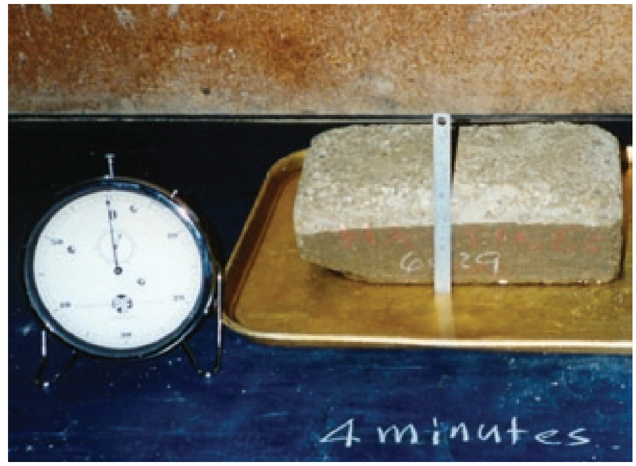


FIGURE 14 Surface soak test. Note that the moisture has nearly penetrated the soil-cement brick after four minutes.

With guidance from members of the New Zealand Earth Building Standards Committee, I also developed a surface wetting and drying test in which moisture penetration and surface effects from a single soaked surface are observed. The brick sits 1.5 mm (0.06 in.) off the bottom of a tray, and the water depth is maintained at 10 mm (0.4 in.) (as shown in fig. 14) for a fixed time, after which the deterioration is evaluated. This test is repeatable because of its ease of setup, and it has a number of empirical visual checks that indicate suitability; but only a limited number of trials were undertaken.

Kevan Heathcote (2002) was involved in experimentation on soil-cement blocks for a number of years, and he proposed the use of a different nozzle for the spray test. This approach does not simulate the effects of wetting and drying or account for thermal effects. Kerali did an excellent analysis of the erosion process for stabilized earth blocks (Kerali 2001) and proposed a slake test (Kerali and Thomas 2004). These only partially represent the erosion criteria he identified and will be much too severe for adobe. Hall looked into the soil constituents and proposed a wick soakage test for pore suction, based on a masonry approach, but this does not simulate rain erosion (Hall 2004).

To be able to accurately develop testing approaches for durability, an absolutely key requirement is a test rig that can represent accelerated climate conditions, so that various tests can be calibrated. From initial investigations it is clearly necessary to develop test equipment to simulate repeated rain strike with wind and cyclic temperature effects, to be able to define and calibrate low-cost procedures.

Another approach is to monitor weather conditions precisely and to measure surface degradation on real buildings as a function of time. A number of methods for obtaining a surface mold were attempted, but all damaged the surface of adobe. John Morris of the University of Auckland Department of Computer Science has recently supervised experimental work using stereophotogrammetry to give precise, noninvasive measurements of surface degradation (Lin 2006; Lin, Morris, and Govignon 2007). Figures 15 and 16 show the laboratory test camera arrangement, which has the capability of precise adjustment. The data projector is used to create a Gray code line shift pattern of light for calibration.

Figures 17 and 18 show a photograph and 3-D surface model of an adobe brick. We intended to set up a weather station adjacent to existing buildings and create contour plots of surface degradation by photographing and plotting sample wall areas each six months.

Out-of-Plane Analysis

The standards need review or further development in the area of unreinforced out-of-plane performance. Background information on the out-of-plane procedures in *NZS 4297: 1998 Engineering Design of Earth Buildings* (Standards New Zealand 1998a) is discussed below.

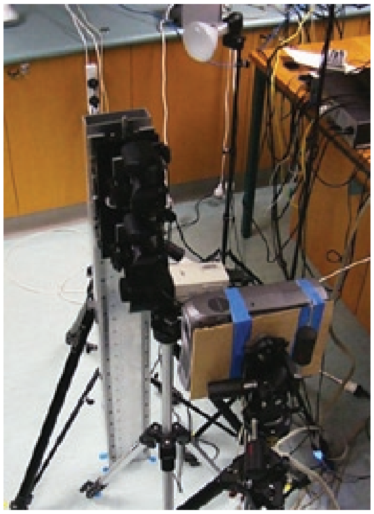


FIGURE 15 Stereo cameras set up with light projector and video for calibration. Photo: John Morris, University of Auckland.

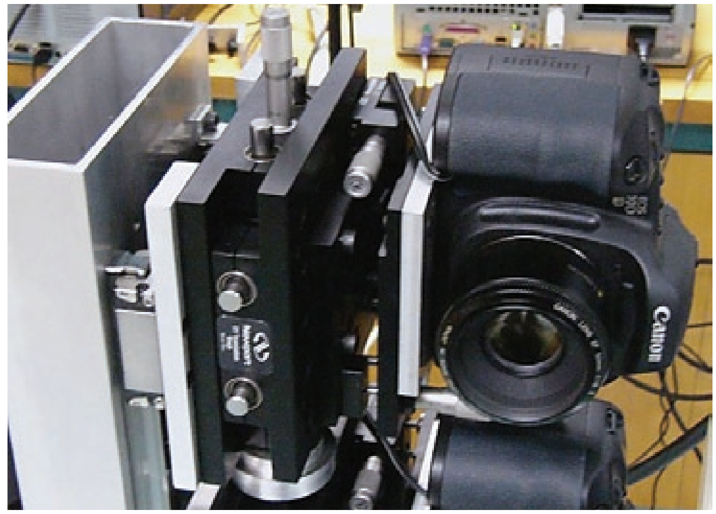


FIGURE 16 Close-up of a camera and adjustment apparatus. Photo: John Morris, University of Auckland.

Peter Yttrup (Yttrup 1981) recognized that when the strength of the earth material is exceeded, causing a horizontal crack in the wall, this is not the critical condition for wall collapse due to very high wind forces. He proposed that the full overturning equilibrium be considered, to more realistically determine the wind resistance of thick-wall earth buildings. Later Priestley proposed an energy method for determining earthquake instability as a criterion to take into account the collapse mechanism in unreinforced masonry (Priestley 1985). A procedure was developed and was published in *Guidelines for Assessing and Strengthening Earthquake Risk Buildings,* issued as a draft in 1995 (New Zealand National Society for Earthquake Engineering 1995). This procedure was slightly refined and incorporated in NZS 4297 for out-of-plane calculations for unreinforced earth brick or adobe walls (Standards New Zealand 1998a).

FIGURE 17 Three-dimensional extrusion of photograph of adobe block.

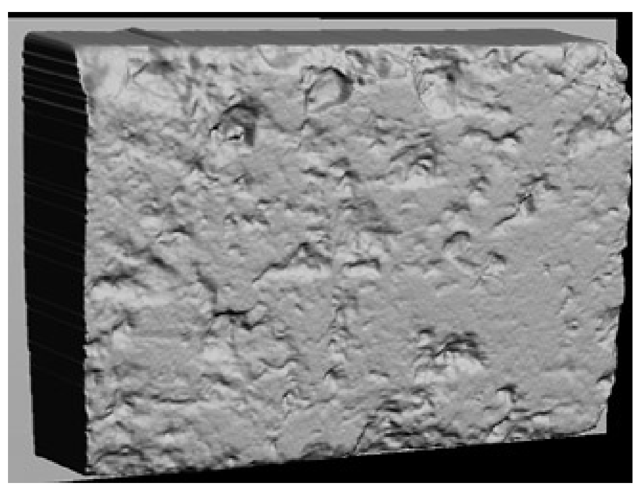
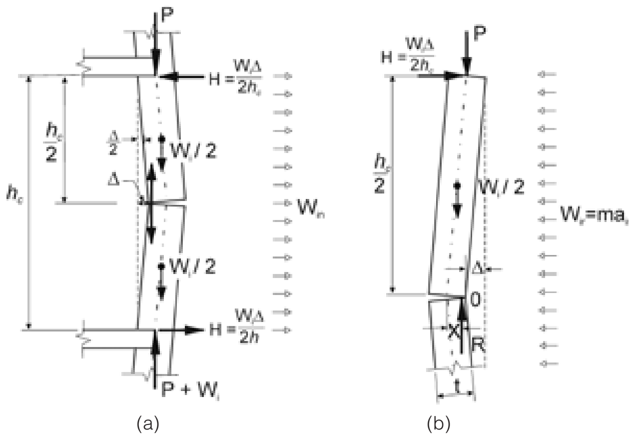


FIGURE 18 Three-dimensional surface model derived from stereophotographs.



FIGURES 19A AND 19B Moment equilibrium parameters for determining the out-of-plane performance of unreinforced walls in low-earthquake zones. Forces on face-loaded wall, including lateral reactions (a), and moment equilibrium for face-loaded wall (b) are shown (P = gravity load per unit length at top of wall; W = self-weight of wall under investigation; Δ = displacement at center of wall; h = height of wall between horizontal restraints; R = vertical reaction at crack; t = wall thickness). (Originally published in *NZS 4297* [Standards New Zealand 1998a, 53].) Content reproduced from NZS4299/NZS4297/NZS4298 with the permission of Standards New Zealand under License 000738. www.standards.co.nz.

NZS 4297 is the first publication of this procedure within design standards, and while it had been through some review prior to the draft documents, there was very little comment at the time the standards were published. Another revision of the earthquake society guidelines was recently released, and the procedure has been updated (New Zealand Society for Earthquake Engineering 2006). However, the new procedure needs to be evaluated for earth buildings. The procedure is based on the assumption that the out-of-plane wall segments need to reach an unstable failure point for collapse to occur. Figures 19a and 19b, from *NZS 4297: 1998 Engineering Design of Earth Buildings* (Standards New Zealand 1998a), set the parameters for this calculation.

Blaikie and Davey have further developed this concept using time history analyses, and they challenge some of the earlier ideas as being nonconservative (Blaikie and Davey 2002; 2005). This concept is still rather simplistic, and the Blaikie approach only represents the vertical direction of span. More sophisticated modeling is required to represent spanning in both directions and to determine at what point vertical cracks at the edges of wall panels will allow spans to act only in the vertical direction.

It is important to investigate the actual rocking performance of structures with little tensile strength. Analytical models can be tuned to give very realistic model responses, but the material parameters that produce the realistic performance are usually incorrect. The critical material characteristics need to be identified and understood for proper analysis. There is a need for shake table testing on stacked adobe blocks to establish the performance in this most simple situation, to identify key parameters for analysis. This will provide initial data for full analyses, which should be followed by verification using full-scale shake table tests.

Strength Determination

A major concern is that there needs to be consistency in testing procedures used, so that results are comparable among researchers. In much of the literature on adobe and earth buildings, there is no definition of the height-to-width ratio, moisture content, or loading rate of compression specimens at the time of testing. This makes an enormous difference, and if combined, the effects of the lowest height-to-width ratio and low moisture content could theoretically produce results that are two times that of a tall specimen with high moisture content. Moisture content in a dry wall in service may be in the range of 4%–8%, whereas the great difficulty of drying materials to exactly the right moisture content means that during testing it could be as high as 10%–15%, or even oven dried. Standard reporting procedures, loading rates, platen constraints, and specimen preparation are needed if researchers and practitioners are to be able to compare results. In the longer term, earth specimens should be conditioned under standard temperature and humidity for a fixed period before testing. If it is not possible to undertake testing in a standard manner, this practice would at least allow differences to be understood if specimen size and orientation, moisture content at the time of test, and loading rate are defined with the results.

Conclusion

New Zealand has a small number of earthen buildings within a highly seismic area, and conservation of historical buildings in New Zealand has only recently been undertaken with scientific rigor. The application of the comprehensive suite of earth building standards has worked well in the New Zealand context and facilitated the adoption of this environmentally suitable technology in a tightly regulated environment. The analysis method for out-of-plane performance of unreinforced earth brick walls in the New Zealand earth building standards has been progressive, but it would benefit from further verification and revision. Many parameters reported in the literature are not well specified, and standardization of measurement is needed even for a parameter as simple as compressive strength.

There was a range of research carried out on adobe to obtain indicative strengths for the standards, but testing with large numbers of samples for statistical reliability is needed. Research is under way to investigate the thermal performance of rammed earth and fiber-reinforced soil-cement to determine the acceptable limits on wall thickness. Durability is a major issue for earthen structures exposed to moist environments. Also needed are methods for testing, laboratory calibration for testing, and measurement of existing structures.

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Reflecting on Materials and Structure: Building Cultures and Research Methodology in the Project of a Seismic Building Code for Traditional Materials in Morocco

Mohammed Hamdouni Alami and Stefania Pandolfo

Abstract: *This paper presents the project of a Building Code for Traditional Materials conceived and developed in Morocco with international cooperation in the aftermath of a ruinous earthquake that struck the city of Al-Hoceima in February 2004. Several actions have been executed within an organized and phased framework. To date, the implementation of the project is reaching the second of a three-phase schedule.*

We will first give a brief and broad overview of the project. Then we will discuss the necessity of developing a better knowledge of local structural systems and the theoretical hypotheses about a local building culture that helped us in defining the related architectural and structural typologies. Here, building culture is not perceived simply as a surplus "topping" of rites and know-how but as a theoretical view of materials, form, and structure, an ontology and related epistemology. The views of master masons about earthen material, workmanship, and construction techniques are presented in connection with architectural forms. On the basis of these views and other observations, a preliminary definition of architectural and structural typologies was made, the corroboration of which was sought by way of an extensive survey conducted in three discrete regions of the country.

Introduction

In February 2004 the northern Moroccan city of Al-Hoceima was struck by a powerful earthquake that caused hundreds of casualties and triggered heated debates about disaster relief and intervention, and about the safety of rural and urban housing, illegality, and the need for regulation. Soon after, a new law was issued that requires all new buildings, in both urban and rural areas, to comply with earthquake safety regulations within the coming five years. Construction permits, which until now have not been required in rural areas, will become a legal obligation, and compliance with the seismic building code will be required all over the country.

The law known as Loi 04-04 was approved by the Moroccan government (Conseil des Ministres) and is currently pending approval by the parliament. The existing Moroccan building code, known as R.P.S. 2000, provides seismic regulations exclusively for steel and reinforced concrete. A related law, also prepared by the Ministry of Housing and City Planning in 2004, Le Règlement Général de Construction (General Building Code), similarly ignores all the possible uses of traditional construction materials. This means that if nothing is done before the five-year deadline, all traditional construction techniques—already strongly challenged by the concurrent use of industrial materials—will be outlawed.

The social and cultural consequences of such a situation would be catastrophic. Over 70% of the construction in rural areas is of earth or stone masonry, and the inhabitants do not have the means to move to different housing. This problem is dramatically coloring the aftermath of the Al-Hoceima earthquake. Almost two years after the event, many families were still living in tents, according to a survey we conducted in the region of Al-Hoceima in winter 2006. The lack of resources has caused the reconstruction process to be characterized by the simultaneous presence, on the same sites, of tents, thatch, and wattle dwellings, and the reinforced concrete foundations of the projected new homes. It should be noted that despite galloping urbanization (more than half of Morocco's population now lives in urban areas [Morocco 2004]) and the massive introduction of modern building techniques (notably the use of reinforced concrete—in many cases of poor quality), vast and environmentally significant regions still display high-quality vernacular architecture. Keeping this vernacular architecture alive is critical for the preservation of Moroccan building traditions and cultural patrimony, as well as for the possibility of building legal and safe affordable housing in rural areas. At the macroeconomic level, it is also a means of saving on imports by using local materials.

To meet that goal, one of the authors, Mohammed Hamdouni Alami, then professor at the École Nationale d'Architecture of Rabat, Morocco (ENA-Rabat), initiated a research project with CRATerre and the UNESCO Chair of Earthen Architecture, Building Cultures and Sustainable Development. To the initial partners of the project were added the Engineering School of Casablanca (École Hassania des Travaux Publics, EHTP), the Laboratoire Public d'Etudes et d'Essais (LPEE), and the Getty Conservation Institute (GCI). The project became fully operational when an international scientific advisory committee met in Rabat in May 2005.

Prior to the meeting, a research team composed of the authors; Mary Comerio, Department of Architecture, University of California, Berkeley; Khalid Mosalam, Department of Civil and Environmental Engineering, University of California, Berkeley; and Mel Green, Mel Green and Associates, wrote an initial research project. This document was further developed by Abdelkader Cherrabi, Casablanca School of Engineering; Hubert Guillaud, CRATerre; and author Mohammed Hamdouni Alami, and it was submitted and discussed with participants at a May 2005 workshop in Rabat.

The project aims to study techniques of earthen construction that have acceptable performance for earthquake safety regulations and are adequate to the needs of low-cost housing in developing countries. Our methodological premise is that research on such techniques should focus on preservation of the local building culture, and that preference should be given to the use of vernacular materials, such as stone, earth, wood, textile fibers, and other locally available solutions. Consistent with these premises, our project engages in a dialogue with a local Moroccan tradition of building and does so at the level of its core conceptual formulation. It seeks to promote a creative exchange between contemporary engineering modes of analysis and solutions and other traditional construction methods, taking seriously that tradition's different understanding of structures and materials, and relying on the active engagement of engineers with that tradition and its conception of structure and materials, as well as on the mobilization of local know-how to come up with original solutions. Yet—and inasmuch as we think that both approaches are necessary and complementary—we see our intervention as an attempt at preserving and transforming local building techniques while, at the same time, taking into account the requirements and solutions made available by contemporary engineering.

In recent years some research and systematic improvement of building techniques has been carried out in several countries (see "Earthquake Resistant Design Criteria and Testing of Adobe Buildings at Pontificia Universidad Católica del Perú" and "New Zealand: Aseismic Performance-Based Standards, Earth Construction, Research, and Opportunities," both in the present volume). These studies have applied modern engineering concepts to traditional building materials such as earth. They would benefit by integrating knowledge of local building traditions into their methodologies.

Whereas most of the research conducted thus far has been focused on the study and improvement of materials rather than on traditional building techniques and structural typologies, the objective of our research and educational efforts is to develop and implement a methodology to evaluate and to improve the reliability of adobe buildings subjected to seismic forces. Our work takes its lead from the possibility of improving upon the highly sophisticated vernacular techniques that are found in Morocco and in many other countries in order to make them seismically safe. Indeed, some of the ancient techniques developed in highly seismic regions—such as Central Asia, the Himalayan Mountains, and Anatolia—demonstrate their effectiveness by their survival over long periods of time (see "Observing and Applying Ancient Repair Techniques to Pisé and Adobe in Seismic Regions of Central Asia and Trans-Himalaya" and "Seismic Rehabilitation Study in Turkey for Existing Earthen Construction," both in the present volume).

The improvement of traditional building methods through laboratory experiments and cutting-edge scientific analysis would provide populations with the seismic safety techniques required by modern building codes. It would also make possible the promotion of the social image of earth as a competitive building material vis-à-vis industrial ones and facilitate its re-adoption by local populations. This would be a major achievement because it would provide a large part of the world's population with seismic safety, as well as aid in the struggle against poverty and the deterioration of the environment. The production of an earthen architecture for the very rich in Marrakech and the widely recognized disastrous climatic performance of modern materials in low-cost housing are strong arguments in favor of traditional materials. Our own survey in southern Morocco has shown that people who moved from the traditional earthen structures to concrete and cement block homes experienced hardship in the summer due to extreme heat, and they often returned to spend the summer in the old villages when the earthen structures were still standing.

Fostering the development of that awareness is part of this project. Among the actions we consider as our goals are (1) the introduction of seismic-engineering-based design in architectural education; (2) seismic engineering training of practicing engineers and architects; (3) training contractors and master masons to correctly use the seismic techniques required and described in the projected building code; (4) informing municipal administrations and *agences urbaines* of the content and the philosophy of the projected building code, including training of their architects and engineers; and (5) providing public information about seismic building techniques and maintenance of earthen structures. This last goal is an important one because the code concerns self-help construction, without an architect or an engineer, and because the workmanship in building and maintenance is partly provided by the people who own or live in the buildings. All of these aspects make the project a long-term one.

Defining Contemporary Architectural Typologies and Structural Systems

The writing of a building code adapted to the local building tradition requires in-depth acquaintance with the architectural and structural typologies of the country and of the building tradition itself. This also means that local typologies should be approached and understood in their cultural context as both cultural forms and productive forms of knowledge. Consistent with this view, we take the notion of a building tradition as a theoretical view of materials, form, and structure, as well as the embodied practice of a particular know-how related to a way of being in the world. In our understanding—and far from its reduction to a superficial or cosmetic addition of rites and workmanship—such a notion of building tradition is critical to any understanding of structure and must be engaged by engineers. For this reason, the issue of architectural typologies and structural systems has been addressed in terms of an anthropology of science and on the basis of anthropological fieldwork conducted in Morocco in recent years.

To begin, it should be noted that the question of typology is twofold. On the one hand, it has to do with existing structures and historic buildings, and with the retrofitting measures that may be applied in order to make them seismically safe. On the other hand, it is related to future buildings and the seismic design safety principles they should comply with. For existing structures, the question of architectural and structural typologies is mainly a question of defining the criteria established on the basis of earthquake engineering and the principles of architectural preservation, and it relies on direct observation in the field. For future buildings, defining architectural and structural typologies not only has to do with engineering criteria and with the observation of recently built structures, it *must also imagine future architectural evolutions* based on social, technical, and economic factors. Our case research has shown that the highly sophisticated and structurally complex traditional architectural types of the Moroccan pre-Saharan regions have been progressively and irreversibly abandoned, at least at this point in time (fig. 1). To address this issue, we dedicated our project's first workshop (May 2005, in Rabat) to the analysis of typological and sociocultural research on the evolution of spatial forms and to the impact current typological and structural transformations have on earthen buildings' seismic resistance.

In our second workshop (September 2005, Rabat) we tried to further our reflection on the relationship between traditional typologies and building techniques,

FIGURE 1 Most of the residents of Ait Ben Haddou have moved from the old village (lower) to the new village (top).

and on the evolution of that relationship in contemporary building practices. We focused on the detailed study of one region in the Moroccan south, where some of us had previously conducted research and where further research has been and will be conducted (Pandolfo 1997; Hamdouni Alami and Bahi 1992). We were able to show that in their contemporary use, traditional building techniques had lost their symbolic and epistemic foundations. The housing building process had transformed from what local master builders described as an interconnected vertical growth, to a horizontal development of discrete elements from which connections and all form of juncture or "attachment" were excluded (between walls, stairs, and between walls and roof). This technical change was of great significance, for the "vertical structure" was technically conceived and phenomenologically experienced as a coming into being of form and life (as a network of articulated joints) (fig. 2).

The issue is twofold. On the one hand, the spatial exodus from the walled, multi-story, and highly dense *qsar* village form that is characteristic of this region (*qsar*, plural *qsour*, are walled earthen village/town settlements) toward multiple scattered settlements composed of independent housing units reflects a typological and sociocultural mutation that cannot be overlooked—and one that is related to the experience of modernity in this peripheral region. The architecture and building technique of these new homes is novel, and the use of earthen or hybrid building materials is the only vestige of a local building tradition. Studies of seismic resistance and vulnerability of earthen buildings have to take this mutation into account.

On the other hand, any attempt to identify seismic resistant structural typologies and techniques must also seriously engage with the historic architectural forms and structural typologies, and with the highly sophisticated building traditions that are today being abandoned because the *qsar* village form is perceived as uninhabitable. As documented in our architectural and structural survey, it is unquestionable that the new typologies of scattered housing units are less seismically resistant than the old typologies. This is because the structures are built incrementally and without a preconceived plan, and all successive additions are adjoined without any structural connection (unpublished preliminary survey conducted in 2005, and general survey conducted in winter 2006). New houses outside the walls are no longer attached and interconnected with everything else. The issue, however, is not to return to a connectedness that today is perceived as uninhabitable from a cultural/social point of view, as well as in its technical sense. The question is instead that of the creation of something else that might newly interpret some key formal principles that might have rendered the *qsar* constructions more structurally stable and spatially dynamic.

In our first workshop, we described the evolution of the architectural typology of housing in the southern valleys of Morocco and discussed the social factors that

FIGURE 2 Aerial view of new houses across the river from the historic settlement at Ait Ben Haddou.

FIGURE 3 *Qasba* of Amerdil, Skoura.

determine this evolution. We also tried to show that this evolution led to the creation and adoption of new types, including the *qasba* (small castle) and the Ecochard housing unit (the so-called *habitat économique*, after the name of its original designer, Michel Ecochard, head architect of the French Protectorate in Morocco from 1947 to 1953).

Starting from observations in the field, and from a limited but interesting literature (Jacques-Meunié 1962; Chorfi 1991; Ben el-Khadir and Lahbabi 1989), we have attempted to show that this evolution started very early, perhaps with the building of homes in the midst of the cultivated gardens in the seventeenth through the nineteenth centuries. As D. Jacques-Meunié suggested in her *Architectures et habitats du Dadès* (1962), very early, powerful families started leaving the *qsour*, the collective settlements, and settled outside, either just next to these *qsour* or in the gardens (on agricultural land). Designed as improved and fortified garden houses, the first *qasbas,* or small castles, were single-family homes with or without patios, depending on the altitude of the site (fig. 3). In the first half of the twentieth century, around 1920–30, the typology of the *qasba* with patio was sometimes abandoned in favor of an urban typology, that of Marrakech *riyads,* or houses with large internal gardens.

Because of the safer and freer nature of present days, the exodus from the collective settlements accelerated after Moroccan independence, in particular in the 1980s. Many of the *qsour* of the region are now being abandoned (fig. 4), and people are rebuilding and settling outside in completely different spaces, from the point of view of architecture and urban design. Seeking more spacious houses, most families are leaving the overcrowded old settlements. An outspoken longing for "freedom" is driving everyone out of the traditional housing forms. More strikingly, traditional housing and even traditional materials are identified with the old way of life, with the overcrowded homes and lack of freedom that resulted from living with the extended family in constricted spaces. Whereas the traditional agricultural way of life in the region was organized around a seasonal rhythm, with daytime in the gardens and nighttime at home, the shift from an agricultural economy to a migration economy, which relies on remittances from migrant workers, put an end to the importance of gardening. In the new context, old housing forms are no longer perceived as shelters for nighttime and hard times, but as oppressive "family prisons," where only the poor are constrained to remain.

Moving out of the *qsour,* to be near the road and public services (the weekly market, the school, the infirmary), whatever little these services may appear to us, has become the driving force of the settlements and townscape of the region. With the new urban and village fabric, the housing typologies changed as well, for better and for worse.

FIGURE 4 An abandoned *qsar* (a walled earthern town settlement), Tineghir, province of Ouarzazate.

These observations and the Earthquake Engineering Research Institute (EERI) World Housing Encyclopedia Report form (see www.world-housing.net/) were the basis on which a preliminary survey of architectural typologies and structural systems was conceived. Some of the Getty Seismic Adobe Project publications, particularly the *Survey of Damage to Historic Adobe Buildings after the January 1994 Northridge Earthquake* (Tolles et al. 1996), have been a model for us, especially in attracting our attention to certain specific vulnerabilities of earthen buildings in terms of their seismic behavior (such as water erosion and basal conditions of walls). The preliminary survey was conducted in the region of Ouarzazate, southern Morocco, in October 2005 (it was carried out by two architects from CRATerre and one from ENA-Rabat, following an anthropologically informed training seminar in Rabat). It confirmed our previous findings concerning the loss of symbolic values, particular techniques, and connections of structural features, and of know-how as well. The practical knowledge lost concerns the capacity to recognize different qualities of earth, good preparation of earth bricks, appropriate proportions and compacting of rammed earth, and efficient maintenance techniques. It also confirmed our preliminary hypotheses concerning the loss of structural connectedness in new buildings and related increased vulnerability from architectural and structural evolution.

All of the issues mentioned above were discussed in the particular geographical context of the southern valleys. However, since our research aimed at drafting a national code, those issues had to be addressed nationwide. Were our conclusions also valid for other parts of the country? Are the contemporary architectural and structural types of the south comparable to those of other regions? Of course, the problem takes different forms for existing and historic structures, and for future and contemporary types.

Perhaps we should first explain why, given the large number of documents and studies of Moroccan regional architectural typologies available to us, we decided to conduct new surveys on the subject. Indeed, Moroccan regional architecture has been an object of interest for scholars as well as for the Moroccan state. The Department of Housing and City Planning has commissioned many studies that resulted in official reports on vernacular architecture. Most of these studies were conceived within the same intellectual mold and were inspired by French colonial literature on typology and morphology. One such study, *Les architectures régionales du Centre Sud* (Chorfi 1991), defines architectural typologies on the basis of the six following criteria:

1. The site: because architectural objects are always viewed in their natural or urban environment
2. Urban fabric, or *formes de groupements*: because urban fabric determines the access to buildings, their relation to streets, their relations to one another, and their visibility
3. Housing organization: groups of functional spaces, their two-dimensional characteristics, and their relations to outside spaces
4. External morphology: external volumes, facades, surfaces, colors
5. Internal morphology: internal spaces both as colored and lit volumes and living spaces
6. Materials, techniques, and building systems (*systèmes constructifs*)

Despite mention of the phrase *systèmes constructifs,* or building systems, as a component of the sixth criterion, a systemic approach to structures is absent. Structural elements are barely described, and connections between them are completely ignored. The approach is mainly concerned with materials, and it remains totally separate from earthquake engineering. When compared to the American approach, it simply reveals the absence of a preoccupation with structural systems. For instance, the approach of the U.S. National Park Service (NPS) to buildings, as set forth in *Guidelines for Preserving, Rehabilitating, Restoring, and Reconstructing Historic Buildings* (Weeks and Grimmer 1995), is based on a set of features that reveals a different view of architectural typology. Among these are:

- Building exterior
  - Materials: masonry, wood, architectural metal
  - Features, roofs (shapes, cupolas, chimneys, vaults), windows, entrances, porches (general forms and shapes, volumes)
- Building interior
  - Structural systems

- Spaces, features, and finishes (interior floor plan, the arrangement and sequence of spaces, primary and secondary spaces defined according not only to their function but also to their features and finishes)
- Mechanical systems

In the NPS *Guidelines,* the preoccupation with structural systems is stated clearly. This is perhaps a consequence of the American culture of earthquake preparedness. On the other hand, the French model, which was the implicit referent for studies of Moroccan regional architecture, was not developed with regard to any logic of earthquake preparedness.

Consequently, our survey was inspired by the American model, with recourse to the World Housing Encyclopedia Report form. Not only did we introduce the notion of structural systems as a criterion of architectural typology, we also adopted a distinction between architectural and structural typologies. If in vernacular architecture the rule is that an architectural type is almost always associated with a structural system, we have noted that in the contemporary context, an architectural typology can be associated with different structural types. The traditional Moroccan urban house can be built with earth in Marrakech, with sandstone masonry in Fez, and with concrete frame in the two cities and elsewhere. A contemporary rural house can be built with traditional materials or with reinforced concrete. Thus, in today's experience an architectural type is no longer connected to a particular material. As Le Corbusier said, "Architecture has emancipated itself from technique. It is the [building] technique which must now bend to the architecture" ("L'architecture s'est émancipée de la technique. La technique doit maintenant se plier à l'architecture") (*Vers une architecture* [Paris: Cres, 1923], 37). Because of this contemporary mutation, the survey was conducted with the aim of characterizing architectural typologies independently from structural systems or types, which had to be characterized on their own. Different entries were devoted to each typology.

Reflecting on Materials and Structure in Local Building Tradition

During our second workshop, traditional housing types and building techniques were presented from an anthropological perspective. The discussion of the earthen construction process and of spatial organization was drawn from Stefania Pandolfo's research in the Wâd Draʿ region of southern Morocco during the 1980s and early 1990s (Pandolfo 1997), with additional information from trips in 2004 and 2005. It was based on the observation of buildings and *qsar* formations; on in-depth interviews with residents on their histories, memories, and poetry (from written documents, property and inheritance deeds, and oral accounts); and, most prominently, on interviews with master builders (*mʿaallem,* plural *mʿaallmin,* in colloquial Arabic and in Tashelhit)—one in particular, Brahim Dagdid, who was still practicing at that time. It is also based on Pandolfo's experience living in one such space in the late 1980s and witnessing the process of the resettlement of a *qsar* community in the desert area outside of its perimeter walls, which was newly constructed in a sprawling, scattered architectural style consistent with contemporary changes in the perception of built space, family structures, cultural reconfigurations, and traumatic disconnections as they are reflected in the growth of new architectural typologies in this area.

Conversations with *mʿaallem* Brahim Dagdid took place in 1985, 1986, and 1989, and they resumed in 2004 and 2005–06 in the context of the present research. These conversations were primarily with author Stefania Pandolfo. Author Mohammed Hamdouni Alami met with several master builders between 2004 and 2006, in the regions of Zagora as well as Tazzarine, in the pre-Saharan region. Of particular interest were master Ait Zayd from the *qsar* of Tamnugalt and master Ourzazi from Tazzarine. These conversations focused on the technique and process of building, on the form of the house, and on the nature of materials, including their transformation and dynamic workings. In spite of their plastic potential, houses inside the *qsar* have a sophisticated structure, conceived according to a specific geometry expressed in the symmetry, interconnection, and internal articulation of the buildings. From the anthropological point of view—but also arguably in order to appreciate the flexibility of these structures in terms that might be of interest to the engineers (who look at structures for their objective properties in order to assess their seismic vulnerability)—it is important to pay attention to the local (technical/symbolic) conception underpinning the process of building, which is expressed in the Arabic and Berber technical vocabulary and in the exegeses of

FIGURE 5 New houses sprawling away from an old *qsar,* Tissergat, province of Ouarzazate.

local master builders. Many knowledgeable *m'aallmin bennay* (craftsmen/master builders) are still alive and are the recipients, if not necessarily the transmitters, of a sophisticated know-how that is practical but also theoretical. In this project on seismic vulnerability and prevention testing and evaluating, in the last instance, are the prerogatives of the engineer, yet the intermediate stages should take other knowledge into account.

Traditional housing types and building techniques were presented on the basis of detailed notes taken by Stefania Pandolfo during and after conversations with *m'aallem* Brahim Dagdid in Beni Zouli, Zagora, in 1985 and 1986. At that time, the *m'aallem* was still working, and people were still living inside the old *qsar*. However, the communal land outside the walls and in the direction of the open, arid pre-Saharan plateau, where there is an alternation of cultivated and irrigated oasis and rocky wasteland (away from the river and the gardens), was in the process of being divided and allocated; in the space of less than four years, the *qsar* would be abandoned, quickly transformed into a ruin. A new village was built in the open land at a small walking distance from the old *qsar*, and a new architectural style was introduced and quickly spread over the entire territory (a grid plan had been brought in by the local authorities and had been adapted to the needs of the community). As mentioned above, this process is not specific to the *qsar* in question but is characteristic of a general transformation in the southern valleys and in Morocco at large (fig. 5).

In telling about the technique of construction, the *m'aallem* Brahim Dagdid was referring to other houses in the *qsar* of Beni Zouil, to houses he has built or visited in other *qsour,* and to the house in which he was living. He was also contrasting that technique to the new ways of building outside the walls, the techniques developing in the "new village."

Dagdid's description and interpretation of his art and technique, which he had practiced for more than fifty years, took place over many conversations and was composed of four parts:

1. A description of the qualities and properties of different types of soils appropriate for specific tasks—such as making mortar, adobe, or rammed earth, or waterproofing—and the preparation of the *l-'ajina dyel at-tub,* or earth dough.
   a. Preparation and construction of *l-luh* (*l-luh*—literally, board—refers at the same time to the wooden formworks and to the rammed earth wall itself).
   b. Preparation of *at-tub,* or adobe bricks, and construction techniques.
2. The structure of the house inside the *qsar* and, in particular:
   a. The *as-swari,* or columns, as constituting the structural space within which the house will grow (fig. 6).
   b. The *'tabi,* or wooden beams, that connect different elements of the structures, and the connection with the ceiling and the floor of the upstairs.
3. The *sallum,* or staircase, as an elevated, growing structure articulated and intertwined with wood. This is one of the permanent elements of the house, as are the well and the bearing columns, which may be incorporated into some spatial modification but cannot be eliminated. Each of these "fixed" places is marked by a sacrifice (*debiha*) involving the ritual slaughter during construction of a sheep or, in certain cases, a cock.
4. The relationship between the structure of the house and the structure of the *qsar*, superimposing and merging one into the other.

FIGURE 6 Remains of a four-column structure.

In the master's view there is a *tashbih* (a structural parallel or an analogy) between the transformation of matter/materials and the articulation of structure in the construction technique of the house. Both have to do with coming into being from an inanimate prior state: the coming into being of *ar-ruh* (breath, soul—but also simply the articulation that generates movement and life). Such metaphysical understanding is actually central for the understanding of the building culture, as well as for the logic underpinning its structural dynamics.

In the preparation of the earth dough of which adobes are made, the key point is fermentation, or rotting. The earth used for *l-luh* (rammed earth) is not the same as that used for *at-tub* (adobe). The first is taken from the building site. It is usually coarse and contains a small amount of rocks that need to be removed by hand or by sifting. In contrast, the earth used for adobes is very fine. It is dug out of selected regions inside the gardens and palm groves, and it is usually earth that contains clay and sometimes sand; in traditional practice, the proportion of clay to sand is determined by the touch of the hand. In both cases a shallow pit is dug, water is placed in the pit, and earth is added and left to rest. In the case of *at-tub,* straw (*t-ben*) is added to increase the rotting. The point of this process is to cause swelling and "transformation" in the original material, which is no longer just earth but *modified earth*, subject to chemical modification through fermenting or rotting—a modification that gives birth to "form." Swelling is related to the coming into being of *ar-ruh,* a "soul" or vital principle.

The *m'aallem* went into much detail in explaining the technique of *at-tub* made with rotten earth dough formed in a mold and then dried in the sun, and in explaining the technique of *l-luh*, where the mold is the *luh* itself, the wooden board inside which earth is pressed. But while the *at-tub* bricks are left to dry in the sun until they are ready, the *al-luh* is pressed and beaten down or compressed. While the technique of fabricating adobe bricks is relatively solitary, the technique of *l-luh* is collective—or at least a group task. In the old days when a construction in *l-luh* was begun, the *qsar* would call *hadd as-saym*—those of fasting age—to participate in collective works, such as construction tasks or the clearing of irrigation canals.

According to the *m'aalem,* inside the old house the parallel is between the coming into being of form through rotting (as in the earth dough) and the production of form through articulation, via a systematic network of imbrications and articulations. Indeed, there is a concept in Arabic central to the argument of the master builder that vividly summarizes this central point, the concept of *t'shkal*. Linguistically related to the term *shakl,* or shape, it means making connections through a creation of forms.

The structure of a house inside a *qsar* is vertical, and it is conceived and described by masons as growing around a central void or opening (*'ayn ddar, rahba*—the eye/source of the house) generated by the work of bearing columns (*sariya*, plural *swari*). In the course of our survey in the region of Zagora, the engineer Khalid Mosalam from the University of California, Berkeley, observed that the bearing structure resembles an elevator shaft. A second vertical element is the staircase (*sallum*), which is conceived in a similar style and enacts verticality. The staircase is an articulated wood-and-earth structure. It exemplifies a crucial principle of construction in these houses, which is the articulation and reciprocal imbrications of wooden elements and earthen bricks, and the making of a knot between two elements (the bearing beams of successive flights of steps), which

FIGURE 7 Interior of a house, Tanmougalt.

creates “breathing” (*ruh*), a dynamic articulation and a “soul.” It also exemplifies the “movement” of the house itself, growing away from the ground and developing in a standing position. The main difference from the *riyads* in Marrakech is that these are conceived as structures surrounding a garden, whereas the house we are describing in the *qsar* is conceived and built as a structure springing from the bearing columns, which literally generate the space (fig. 7).

When built with traditional materials, the new houses located outside the perimeter walls are built exclusively in *l-luh* (rammed earth), and the manufacture of mud bricks, used in the old houses, is declining because of the change in typology. The new typologies no longer use two of the main structural elements that require bricks—that is, bearing columns and the staircase. When still made, bricks are bigger and less cohesive because the technique of fermentation of the earth dough (*l-ʿajina dyel at-tub*) has become less rigorous. There is a general perception that *at-tub,* or mud bricks, belong to the world of the old *qsar* and to the structural requirements of building vertically within a contained and yet connected space, where everything was *mshebbek,* or interconnected with everything else. Not just the mud bricks, but the system of *ʿtabi,* or horizontal articulations (wooden beams or thresholds), and of the columns themselves was associated with the representation and perception of the old *qsar*. The result is that new houses located outside the walls likely have less stability and flexibility because they are no longer attached and interconnected with everything else. The issue, however, is not to return to a connectedness that today is perceived as uninhabitable—and this from a cultural/social point of view as well as in its technical sense. The question is rather that of the creation of something else, which might newly interpret some key formal principles that might have rendered the *qsar* constructions more stable and dynamic.

In the light of the description of the building practices we observed, the structural typologies of the southern valleys could be defined as follows:

1. A traditional structural typology (*dar dyel bkri,* “the house from the past.” *Bkri* is a remote past, a past perceived today as distant, even if from a few years ago, a past perceived as severed from the speaker). This is the model found in the vertical structure of houses inside the *qsour*. It is conceived and described by masons as growing around a central void or opening (*ʿayn ddar*, *rahba*) generated by the work of

FIGURE 8 Interior of a house, Tanmougalt.

FIGURE 9 New house, Ait Bouguemmaz.

bearing columns (*sariya*, plural *swari*), with structural components that are interconnected, *mshebbek* (fig. 8).

2. The new structural typology with traditional local materials. This is the model described as horizontal structures where components are no longer attached and interconnected (fig. 9).
3. A mixed typology that is structurally similar to the new typology but makes use of new conventional materials, in particular reinforced concrete (fig. 10).

FIGURE 10 House in Zagora.

In both the north and the south of the country, houses are built around patios. The dimensions of the patio vary with rainfall variations, snow, and altitude. They are larger in the dry lowlands and smaller in humid and snowy places. The classical description made by Jacques-Meunié (1962), according to which the size of the patio decreases down to nothing with higher altitude, applies all over the country. The same observation was made in the northern Rif Mountains by Ben el-Khadir and Lahbabi (1989). There exist, however, differences in the roofing systems. Traditional horizontal roofs observable everywhere are replaced with sloped and gabled roofs in the humid areas of the northern mountains. Structural typologies are similar in different regions, with, of course, a diversity of local materials (wood and stone) and variations in the quality of those materials and workmanship.

To validate these typologies, a preliminary survey was conducted in the regions of Ouarzazate (south center) and Al-Hoceima (northeast) in October and November 2005. Following this preliminary survey and the corroboration of the chosen classification of typologies, a second and more complete survey was conducted in three selected regions, Ouarzazate, Tadla-Azilal (located east of Marrakesh in the High Atlas Mountains), and Taroudant (to the south of Marrakesh), in January and February 2006.

Throughout these regions, and in terms of a more in-depth survey, our hypothesis of the existence of three main typologies was confirmed: an "old" structural typology characterized by a spatial compactness and the tight articulation of structural elements; a "new" structural typology characterized by incremental additions and a loose articulation of discrete elements producing the effect of sprawling structures and settlements; and a mixed typology structurally similar to the latter but making use of a combination of conventional and earthen materials. While the "old" typology, found in old earth-and-wood structures, is no longer in use, the "new" typology maintains the use of local materials and somewhat degraded traditional techniques that highly increase the vulnerability of buildings. In the course of our survey, however, we saw a small number of extremely well built structures at sites where master masons were motivated to perfect their technique, for the know-how is still alive. The structures that make recourse to a mixed typology are paradoxically the least resistant, in spite of the use of concrete. They are less resistant precisely because concrete is perceived as evidence of strength and social success but is used without respect for the rules of good practice (for example, salty or muddy waters are used in making the concrete, concrete beams are constructed without sufficient iron bars, and posts are installed without a footing or foundation). Slight variations were found in the size of openings, the width of walls, the quality of the workmanship, and the inclusion of stone masonry in the construction, particularly at the level of the ground floor. Variations were also found in the technical vocabulary, but generally the typological transformation is quite consistent over the regions we surveyed.

FIGURE 11 Structure exhibiting a mix of materials that reveals ignorance of rules of good practice.

Conclusion

While visiting a construction site in Tanmougalt, a *qsar* in the upper Wâd Draʿ region of southern Morocco, on February 26, 2006, the *mʿaallem* Ayt Zayd, who has forty years of experience, was interviewed by author Mohammed Hamdouni Alami. In this interview he indicated an old wall and said,

> Look at that wall, it is a hundred years old yet it looks in great shape. Now look at this one, it is not more than four or five years old, and it is already badly eroded. It is the same earth, the same material. . . . New buildings do not last because they are badly made. Workmanship is not good nowadays. In the old days the rammed earth was watered during eight to ten days, until it fermented enough, until it was ready. Only then was it worked in the *l-luh,* and it was worked well, well compacted until it became hard like stone. The test was to leave the mold of the newly rammed portion of the wall and to cover it with water. If the water was still there the next day, the rammed earth was said to be good and building could proceed, but if the water was absorbed by the rammed earth, that meant that this was not properly compacted. That portion of the wall had to be destroyed and rebuilt anew. . . . A master and his aides could make only two or three *luh* (blocks of rammed earth) per day then. The master permanently had to check the quality of his rammed earth. Nowadays it is different. No tests of quality are performed. Masters think in monetary terms. They see their work as a cash flow: thirty Dhs [about three U.S. dollars] per *luh.* To make more money, they have to work faster. Today a master with his aides makes up to eight or even ten *luh* per day. He doesn't care about compacting well enough or tying adjacent walls or anything of the sort. He sees his work through money. In the old days money was not an issue, what people sought in the work was quality, work that had stood the test.

The *mʿaallem* kept explaining the fatal downslide of workmanship and building traditions (see figs. 11 and 12), yet his attitude was serene. Time was doing its work, that's all. Good practice may prevail again some time in the near future. It is hoped.

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FIGURE 12 Building a rammed earth wall today, Ait Bouguemmaz (notice joint defects).

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"To Do No Harm": Conserving, Preserving, and Maintaining Historic Adobe Structures

Steade R. Craigo

Abstract: *Earthen adobe is a simple, natural, plastic building material, which will survive many centuries if properly maintained. In seismically active regions, such as California, the southwestern United States, and other areas of the world, maintenance must include sensitive structural repairs and seismic retrofitting. Seismic hazard mitigation efforts are needed both for occupant safety and for the preservation of historic adobe resources.*

This paper provides an overview of conservation principles and the two differing definitions of the term conservation. *Explanation is provided regarding* The Secretary of the Interior's Standards for the Treatment of Historic Properties *(Weeks and Grimmer 1995), historic structure reports, and project regulatory review processes, and the paper discusses how these are applicable to best practice conservation and the seismic retrofit of historic adobe structures.*

Introduction

Adobe is one of the most natural and organic of building materials. Simple earthen structures can last centuries with appropriate maintenance and repair. Unfortunately, efforts to enhance seismic resistance can be invasive, jeopardizing the material integrity and authenticity of historic adobe structures. In seismic-prevalent regions, such as California, there is a need to retain these important historic buildings and to improve the seismic resistance of the structures (fig. 1).

The California Context

In 1769 the founding of what eventually were to number twenty-one California Missions, in what was then known as Alta California, was begun by Franciscan Father Junipero Serra, as ordered by the king of Spain. This effort was the continuation of the mission chain from the south, in Baja California, Mexico, into the present-day state of California. The primarily adobe mission structures were constructed by local Indians, and the sanctuaries were also decorated by Indians. The Alta California missions were part of a settlement pattern that included *presidios* (royal forts) and *pueblos*

FIGURE 1 Main House, Rancho Camulos, damaged during the 1994 Northridge earthquake. Photo: E. Leroy Tolles, ELT and Associates.

FIGURE 2 Petaluma Adobe State Historic Park. This large two-story adobe residence was the central feature of General Mariano G. Vallejo's rancho outside of Sonoma, California, the town that he founded. It was one of the largest and most important private ranchos in Alta California. Photo: California State Parks, © 1969.

(towns), *asistencias* (sub-missions), and, later, *ranchos* (ranches). The structures were largely constructed of adobe, although some fired brick and stone materials were also used. Following secularization of the churches by Mexico in 1834, after independence was won from Spain, the mission lands were mostly divided into private ranchos (e.g., fig. 2).

During the 1840s increasing trade and growing settlement by Americans created markets for milled lumber and fired bricks, and the California building stock began to shift away from adobe construction. The wealth and growth generated by the 1848 California Gold Rush and the resulting California statehood in 1850 influenced a major change in construction to mill- and factory-produced building materials. Although it largely disappeared, adobe construction continued to be used to a much smaller degree in the state.

In 1991 the Getty Conservation Institute compiled a list of about three hundred fifty adobe structures remaining in California, out of an estimated two thousand adobe structures constructed in the state since the late 1700s (Tolles, Kimbro, and Ginell 2002, 8). The exact number constructed is unknown. Some of the surviving adobes are in ruins or have been heavily altered and thus have suffered a diminishment of their authenticity and historic integrity. We do know from periodic mission reports to Spain and Mexico that the early mission buildings were repeatedly repaired after earthquakes.

California State Parks owns forty-two eighteenth- and nineteenth-century adobes, or about 12% of the surviving historic adobe structures in California, including three of the missions: La Purisima Mission State Historic Park, Santa Cruz Mission State Historic Park, and San Francisco Solano Mission in Sonoma State Historic Park. In 2000, eleven of these forty-two adobes were known to have been seismically retrofitted (Felton, Newland, and Kimbro 2000, 1–2). That percentage has increased in subsequent years, since several damaged adobe buildings were repaired and retrofitted after the 1989 Loma Prieta and 1994 Northridge earthquakes.

The Two Views on Conservation

Conservation is a term that carries different meanings in the United States and abroad. Internationally, in countries including Australia, China, Canada, and the United Kingdom, conservation is associated with a broad, inclusive view of historic preservation actions and is generally linked with cultural heritage values, management, planning, policy, and advocacy, as well as cultural heritage tourism.

Sir Bernard Feilden, in his book *Conservation of Historic Buildings,* defines conservation as the "action taken to prevent decay . . . that embraces all actions that prolong the life of cultural and natural heritage . . . preserving character . . . with minimal effect, reversible action, which will not prejudice future interventions" (Feilden 2003, 3). Further, Feilden writes that conservation of the built environment ranges from town planning to the preservation of a crumbling artifact—a very broad scope.

In the United Kingdom, "Conservation Areas," defined as "areas of special architectural or historic interest, the character or appearance of which it is desirable to preserve or enhance," have been established (Great Britain 1967). This definition was broadened in practice to include familiar and cherished local scenes, existing communities, and social fabric. Conservation Areas usually encompass or include listed historic buildings, but not always. Conservation Areas are similar to historic districts within the United States, which are generally described as groupings of historic buildings, structures, and resources that collectively contribute to a particular sense of time and place and historical development.

FIGURE 3 Interior of Mission San Miguel Chapel after the 2004 San Simeon earthquake. The decorative interior wall finishes were badly damaged. Photo: E. Leroy Tolles, ELT and Associates.

FIGURE 4 Adobe garden wall at Cooper-Molera State Historic Park, Monterey, California. Freestanding adobe structures are difficult to maintain and retrofit seismically.

In the United States, the term *conservation* is more narrowly defined. As the narrative at the Colonial Williamsburg Research Division Web site (no longer available) stated, "the field of architectural conservation emerged out of the historic preservation movement as a new and distinct discipline in the late 1960s." Considered a subset of the field of historic preservation, conservation is closely allied with object or art conservation, with a focus on material science and preservation theory. Architectural or material conservation is considered to be treatment of building fabric and elements, including the stone foundations, clay roof tiles, adobe walls, and earthen coatings of historic adobes (figs. 3 and 4).

The decade of the 1960s was a time of major historic preservation achievements in the United States and Europe. In 1966 the United States National Park Service established the National Register of Historic Places for listing individual historic buildings and groups of historic buildings, such as districts. Almost concurrently, a comprehensive conservation law enacted in the United Kingdom established Conservation Areas. The emphasis of both laws was on the preservation of the building or of the built environment as a whole, the sense of time and place, and the significant architectural fabric associated with its historic significance. Logically, this led to a desire to protect and preserve—as well as to restore and sometimes reconstruct when justified—historic buildings and missing architectural elements. Rather than an emphasis on historic properties valued as sites of associative and commemorative significance ("George Washington slept here"), the importance of preserving the physical historic fabric grew increasingly more important. Guidelines were developed to properly treat the building's significant architectural elements and character-defining features and eventually also to treat the environs of the historic property. These guidelines progressed into suggested scientific treatment protocols and directives designed to preserve the significant historic fabric from deterioration and damage. This approach has led to increasingly more scientifically and analytically based treatments of historic properties. As a result, conservation treatments have been developed, and preventive conservation has emerged as a widespread practice.

The American Institute for Conservation of Historic and Artistic Works (AIC) defines conservation as "the profession devoted to preservation of cultural property for the future." Cultural property is defined by AIC as "objects, collections, specimens, structures, and sites identified as having artistic, historic, scientific, religious, or social significance" (American Institute for Conservation of Historic and Artistic Works 1997).

The current trend in the United States is to move from the narrow focus of material conservation to the broader understanding of conservation as used internationally. The term *conservation* is being used in lieu of *preservation* more frequently by American professionals, but the latter term is still in common use in the United States. Concurrently, there has been a growing use of the terms *cultural heritage* and *heritage preservation* as part of a parallel trend to broaden the application and perception of historic preservation to more than historic districts and old buildings—to include historic landscapes and to encompass intangible social, cultural, and diverse ethnic heritage.

Conservation Principles

The principles below are adapted from the AIC Code of Ethics and Guidelines for Practice (American Institute for Conservation of Historic and Artistic Works 1994, 8–9):

- *Minimal intervention:* To do no more than what is required to protect and to preserve the historic resource.
- *Retreatability* (formally known as reversibility): Treatment shall be of such a nature that it will not preclude or prohibit future treatment to preserve the historic resource.
- *Historic fabric as a source of information and as a cultural resource:* Material architectural fabric and also construction methodology are significant documents of the builders and users of the historic structure.

"To Do No Harm": The Conservationist's Hippocratic Oath

> Declare the past, diagnose the present, foretell the future; practice these acts. As to [the conservation of historic adobes], make a habit of two things—to help, or at least to do no harm.
>
> Hippocrates, *Epidemics*

The above, slightly modified oath from the fourth century BC is attributed to Greek physician Hippocrates (Hippocrates 1923–88). By replacing the word *diseases* with the word *conservation,* you will see that the oath is readily applicable to the work of conservationists of historic resources. The directive "to do no harm" provides the basic foundation to guide all treatment of historic buildings, including adobe structures, and it has been philosophically incorporated into the core of the U.S. historic preservation efforts.

The U.S. national historic preservation program was established by the National Historic Preservation Act of 1966. The act requires each state to establish a state historic preservation office. These offices are responsible for the various aspects of the national program and are each administered by a state historic preservation officer, generally appointed by the governor.

The National Historic Preservation Act of 1966, as amended (United States 2002), established the following historic preservation programs and regulations:

- State and tribal historic preservation offices
- State historic resources commissions
- Historic resources inventories
- National Register of Historic Places
- Regulatory review: sections 106 and 110
- Certified Local Government Program
- Federal preservation tax incentives
- Technical assistance and education

In California the national historic preservation programs are administered by the State Office of Historic Preservation (OHP) within California State Parks. The OHP is also responsible for certain state historic preservation programs.

The California state historic preservation programs administered by the OHP include:

- State Historical Resources Commission and Public Resources codes 5024 and 5024.5 (California Code Commission, n.d., Division 5)
- California Register of Historical Resources and other state registers
- Preservation tax incentives for historic buildings
- California Environmental Quality Act (CEQA) (California Code Commission, n.d., Division 13)
- California Main Street Program
- State grants

Both the national and state preservation programs use *The Secretary of the Interior's Standards for the Treatment of Historic Properties* (Weeks and Grimmer 1995) to provide a basic framework of guidance for work on historic structures. The document was developed over several decades by the National Park Service and is firmly based upon the philosophical framework of the 1964 *International Charter for the Conservation and Restoration of Monuments and Sites* (*Venice Charter*) (International Council on Monuments and Sites and Second International Congress of Architects and Technicians of Historic Buildings 1964). The standards provide guidance for each treatment developed largely upon the principle "to do no harm."

Four treatments are defined: preservation, rehabilitation, restoration, and reconstruction. Each treatment has ten standards, with guidelines to provide further direction. The guidelines cover the areas of energy conservation and building codes, as well as cultural landscapes and archaeology.

The principles derived from *The Secretary of the Interior's Standards* include:

- "To do no harm"
- "Less is more"[1]
- Preserve historic materials
- Preserve historic character-defining features

The conservation principles above and the principles of *The Secretary of the Interior's Standards* are similar and philosophically inclusive of each other.

The last two principles are fundamental to best practices in conservation and historic preservation, as well as to regulatory compliance. Conserving/preserving historic materials means to repair rather than replace, to replace deteriorated materials in kind when repair is not possible, and to clean with the gentlest means possible. Conserving/preserving historic character requires finding a compatible use for the property; retaining distinctive features, finishes, and spaces; respecting significant changes over time; and avoiding conjectural designs.

Architectural or material conservation of historic adobes is fundamentally problematic because of the traditional use of sacrificial coatings for adobe maintenance. For example, the Bolcoff Adobe has developed a picturesque character over the decades, but the appearance clearly reveals some physical deterioration (fig. 5). The current historic appearance would be challenging, if not impossible, to retain if the building materials were properly conserved.

FIGURE 5 Don Jose Antonio Bolcoff Adobe, ca. 1840, Wilder Ranch State Park, near Santa Cruz, California. The Bolcoff Adobe's deteriorated condition is the result of deferred maintenance. Photo: California State Parks, © 1988.

Compliance with the Secretary's Standards and conservation principles requires a thorough understanding of the historic structure. Its historical significance, construction methodology and evolution, physical condition, building code issues, and potential existing or new-use impacts must be available to permit carefully considered treatment.

Historic Structure Report

The Past, Present, and Future of Historic Buildings

A Historic Structure Report (HSR) is an essential conservation tool that provides information necessary to make informed decisions regarding treatment of a historic structure (Look, Wong, and Augustus 1997; Slaton 1997). Preparation of an HSR is usually the effort of a team that includes a preservation architect and structural engineer, historian and/or architectural historian, archaeologist, and material conservator. The report can provide a brief history, construction history, architectural evaluation, existing conditions analysis, maintenance requirements, archaeology issues, proposed work recommendations, and historic documentation. These components, encompassing the past, present, and future, are very similar to the conservationist's Hippocratic oath. An HSR can also be a focused study, specific to providing developmental history, treatment and use, or record of treatment, including a ranking of character-defining features, architectural elements, and rooms, to guide new work and future planning efforts, such as a seismic retrofit.

Understanding the Building's History

The preparation and research necessary for an HSR can lead to discoveries of changes, alterations, and treatments of the buildings which may not be visible. Prior to the commencement of work, sensitive in situ removal and visual examination beneath current layers of wall covering and/or paint can provide physical evidence of early decorative treatments. Original treatment can be found beneath later plaster coatings and applied gypsum board. Historic photographs may reveal interior decorative treatment to walls or ceiling surfaces that have been covered during the intervening years. Two historic adobes within the California State Parks system are discussed below to illustrate this point.

The de la Ossa Adobe, now part of Rancho Los Encinos State Historic Park, was constructed about 1849 in the San Fernando Valley area of Los Angeles (fig. 6). The adobe was heavily damaged during the 1994 Northridge earthquake. An interior faux stone wall treatment was discovered during the planning process for the adobe's repair and seismic retrofit work (fig. 7). While inspecting the earthquake-damaged interior wall surfaces, State Archaeologist Karen Hildebrand and the project architect, Senior Architect Maria Baranowski, both of California State Parks, noticed varying colors in a deep crack that had exposed wall layers in Room 4B, the former *sala*. Conservation scientist Frank Preusser examined the room and found that the walls of the entire *sala* had been decorated in this manner during the ownership of the Garnier brothers. Conservator Molly Lambert performed the conservation work.

La Purisima Mission, a California State Historic Park, was founded in 1787, destroyed by earthquake in 1812, and subsequently rebuilt at a new site. After the secularization of the California missions in 1834, the La Purisima Mission buildings fell into ruin (fig. 8). Beginning in 1933, under the direction of the National Park Service, several of La Purisima's more significant buildings were either restored or reconstructed by the Civilian Conservation Corps (CCC) (figs. 9–10). The CCC construction photographs are examples of important documentation that was included in a historic structure report. These photographs can assist in identifying the surviving significant historic materials and the location

FIGURE 6 Exterior of de la Ossa Adobe, Los Encinos State Historic Park, Encino, Los Angeles. The adobe was damaged by the 1994 Northridge earthquake. Photo: Courtesy of Karen Hildebrand, California State Parks.

FIGURE 7 Conserved *sala* wall, de la Ossa Adobe, Los Encinos State Historic Park. This decorated surface was discovered beneath later paint layers. Photo: Courtesy of Karen Hildebrand, California State Parks.

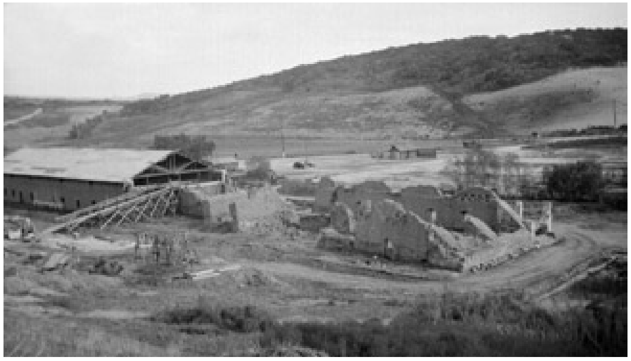

FIGURE 8 The Convento of La Purisima Mission, Lompoc, California, in 1935. The structure was in ruins prior to reconstruction by the Civilian Conservation Corps. Photo: Courtesy of California State Parks, 2007.

of 1930s structural work, so as to guide future work to avoid unnecessary loss of surviving historic building fabric and to reduce the impact of new seismic work.

The National Park Service also provides an outline for preparation of a Historic Landscape Report (HLR) (Birnbaum 1994). Similar to an HSR, the HLR guides the proper treatment of cultural and historic landscape properties. The HSR facilitates informed decisions regarding the treatment and preservation of landscapes such as the historic gardens and landscaping adjacent to historic adobe buildings. The HLR also provides direction in the conservation of archaeological resources remaining from vanished adobe structures, such as foundations, flooring, and surviving ruins. The HSR and HLR are important documents that must be prepared to help assure the proper conservation of historic properties.

Maintenance

Proper regular maintenance is critical to the preservation of all historic properties, especially adobe structures. The exterior wall and roof surfaces must be maintained and usually require periodic renewal of paint, stucco, windows, mortar joints, drainage, and roofs to protect the structure from decay. The interior must also be maintained to protect the interior fabric from wear and damage, rising damp, vandalism, moisture, and decay. A maintenance plan is a critical document that must be followed for the long-term survival and preservation of a historic building or landscape.

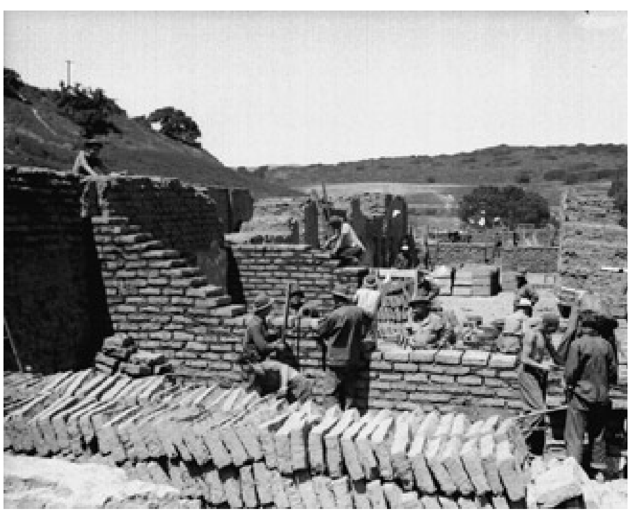

FIGURE 9 The Convento of La Purisima Mission, under reconstruction in 1935. The Civilian Conservation Corps documented building materials and structural work photographically. Photo: Courtesy of California State Parks, 2007.

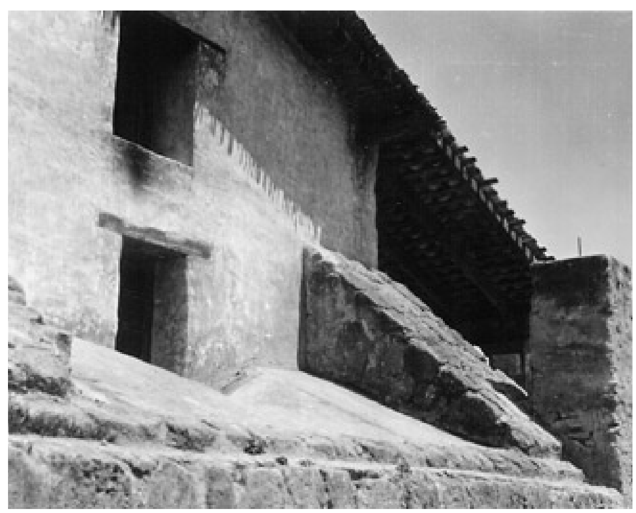

FIGURE 10 The southwest end of the Convento of La Purisima Mission, after reconstruction, ca. 1937. Note the buttresses in photos prior to and after reconstruction (compare fig. 8). Photo: Courtesy of California State Parks, 2007.

By their nature, adobes require regular cyclical maintenance, typically the renewal of exterior coatings and materials (U.S. National Park Service 1997). A weathertight roof is of primary importance. Maintenance of sacrificial exterior wall coatings is also important, as is drainage of moisture away from the bases of earthen walls to prevent rising damp and basal erosion. Historically, maintenance of adobe or earthen buildings was very low-tech, using common, easily available, inexpensive materials requiring more labor than anything else. While this may continue to be true in many parts of the world, in the United States, materials and labor are now both expensive. Because of the rising costs of adobe maintenance in the United States, there is an inevitable interest in using new treatments, coatings, and materials to reduce expenses. Such new—and in some cases untested—treatments are usually found to be detrimental to adobe structures.

Continual maintenance of adobes is less expensive in the long run and has been shown to keep structures more seismically resistant. The State of California's examination of damaged buildings after the 1994 Northridge earthquake revealed that buildings that were maintained responded better to lateral movement and had less damage than did poorly maintained buildings (Seismic Safety Commission 1995, 117–18). Furthermore, buildings that were even minimally seismically retrofitted, such as with lateral bracing and wall anchors, had less damage than those not retrofitted (Todd et al. 1994, 47). Both observations are clearly important to the survival of historic adobe buildings.

Federal, State, and Local Regulatory Processes

In California the three levels of regulatory processes applicable to work upon historic adobe structures have three aspects in common. First, they require consideration of the effect or impact of the proposed undertaking or project on the historic building. Second, each process provides for the participation of interested parties, including the general public. Interested parties and individuals can submit both oral and written comments regarding the proposed project, creating a more transparent and open process that is responsive to public input and concern. And third, *The Secretary of the Interior's Standards for the Treatment of Historic Properties* (Weeks and Grimmer 1995) is utilized to determine the appropriateness of the proposed project on the historic property.

The federal historic preservation regulations are sections 106 and 110 of the National Historic Preservation Act of 1966, as amended (United States 2002). The National Advisory Council on Historic Preservation administers the regulations at the federal level. Within the regulatory framework, each state historic preservation officer is responsible for ensuring appropriate consideration of the undertaking's effect upon historic properties.

Additionally, California has a strong state environmental regulation called the California Environmental Quality Act (CEQA). Impact to historic resources is a consideration under CEQA. The regulation requires that the lead agency, such as a city, district, municipality, or state agency, determine whether a proposed project may have a significant effect on a qualified historic resource, as defined by the regulation. If the lead agency determines that there will be a significant effect on a qualified historic resource, an environmental impact report must address the effect. The state law is enforced at the local level by municipal governments and responsible agencies and at the state level by state agencies for state projects. The California Public Resources Code permits the California Office of Historic Preservation to comment on environmental documents for both local and state projects. Additionally, Public Resources code 5024.5 requires state agencies with projects potentially impacting historic buildings, as defined, to provide the Office of Historic Preservation with the opportunity to comment formally on the work.

Furthermore, there will very likely be a review process of projects impacting historic properties by local-level historic preservation review boards or commissions. The Secretary's Standards are often utilized by local review boards to frame their comments and determinations regarding individual projects.

Given regulatory reviews, which may occur at federal, state, and local levels, historic property developers, architects, interested parties, and owners are always advised to consult early with responsible agencies. This consulting will expedite the review process, as well as provide guidance that will ensure best practice treatment of historic resources and compliance with conservation ethics. In the United States, this will also mean compliance with *The Secretary of the Interior's Standards for the Treatment of Historic Properties*.

Conclusion

Adobe structures require carefully considered treatment to preserve their surviving authentic historic fabric and historic integrity, in compliance with both the principles for conservation and the Secretary's Standards.

Succinctly, recommendations for appropriate best conservation practices for historic adobe structures, including seismic retrofit and material fabric repair, are included in the following four points:

- "Do no harm."
- Conform to the Secretary of the Interior's Standards.
- Have a full, multidisciplinary, experienced, and knowledgeable conservation/preservation project team.
- Consult early with interested parties and local, state, and federal regulatory agencies, as well as with preservation organizations and agencies.

Note

1 This aphorism was used by architect Mies van der Rohe. The phrase originated in the Robert Browning poem "Andrea del Sarto."

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PART THREE

Case Studies

Seismic Rehabilitation Study in Turkey for Existing Earthen Construction

Bilge Isik

Abstract: *Earthen architecture shows diversity according to cultural and geographical environment. Earthen masonry building and hımış (hımısh) are the main types of earthen architecture in Turkey. A hımış building is a wood-framed structure filled with earthen blocks; an earthen building is a load-bearing, solid-masonry-wall structure. Earthen masonry buildings are fragile, whereas wooden-framed structures are safe in seismic regions. Even a 150-year-old building can remain without maintenance, showing durability and stability.*

In the last century, earthen construction was produced without technological support; many people suffered from insufficient building performance. However, hımış buildings constructed by unskilled local craftsmen and without inspection withstood the 1999 Adapazarı earthquake, and the people living in these houses survived.

The structural safety of earthen buildings depends on the design principles of the building and the properties of the material used. Since 1978 research at Istanbul Technical University (ITU) has focused on upgrading material durability by mixing gypsum with soil; research has simultaneously focused on improving seismic response of the structure.

This paper discusses a survey that has been carried out on design principles for earthen masonry and hımış buildings to investigate their seismic response. Corresponding chapters of the Turkish Earthquake Code are highlighted for further rehabilitation measures.

Introduction to Earthen Construction in Turkey

Wooden frame houses and earthen buildings in Turkey are part of a heritage stretching back thousands of years and covering a wide geographical and cultural diversity. Existing structures in Turkey are mainly of earthen masonry and are of the hımış type. Solid bearing-wall buildings of earthen materials are more fragile than these wood-framed structures. Most of Turkey (92%) is in a seismic region where the safety of buildings during earthquakes is essential.

When discussing earthen structures, it is necessary to mention that earthen buildings in Turkey are rarely constructed these days, but when they are, it is often without technical support, since the necessary knowledge is not in the higher education curriculum, and industrialized construction materials have saturated the market. Education and market trends focus mainly on reinforced concrete, for which steel rebar is the predominant reinforcing material.

This study is on existing structures of hımış and bearing-wall earthen buildings in Turkey, which have been subjected to severe earthquakes over the last hundred years. It will discuss the need for rehabilitation measures.

Wood-Frame Buildings in Turkey

Traditional wood-frame buildings in Turkey vary according to the structural system, veneer, infill of the structure, and basement (Isik 2000). Locally available

FIGURE 1 Detail of a hımış building with stone infill.

FIGURE 2 Detail of a hımış building with earthen block construction.

material such as stone, wood, and brick or earthen block can be used as infill (figs. 1 and 2).

Another well-known traditional wood-frame construction is called *Baghdadi*. In this method, a wooden framework is covered with plaster siding, and laths are nailed onto the studs to create a plastering surface (fig. 3). Baghdadi buildings can also be plastered from only the inside, with wood siding on the exterior of the wall. This kind of wall does not have infill. A comfortable indoor climate is created by the thick Baghdadi plaster.

The buildings are mainly of one to three stories (fig. 4). If the first floor is constructed out of stone, it is used for utility or domestic animals but not for living (fig. 5). Wooden houses may have a basement if there is a slope to the land so that the basement level can be constructed half above ground, allowing windows for lighting.

Design of Hımış (Wood-Frame) Building in Turkey

A Turkish wood-frame house is characteristically symmetrical in plan. Rooms (four in most cases) are situated around a central space, called the *sofa*. The sofa

FIGURE 3 Wall constructed using the Baghdadi method: wood frame with lath and plaster siding.

FIGURE 4 Hımış structure in Tarakli, Turkey.

FIGURE 5 Hımış structure in Ayas, Turkey. The ground floor is constructed of stone and used for utility or domestic animals rather than for human habitation.

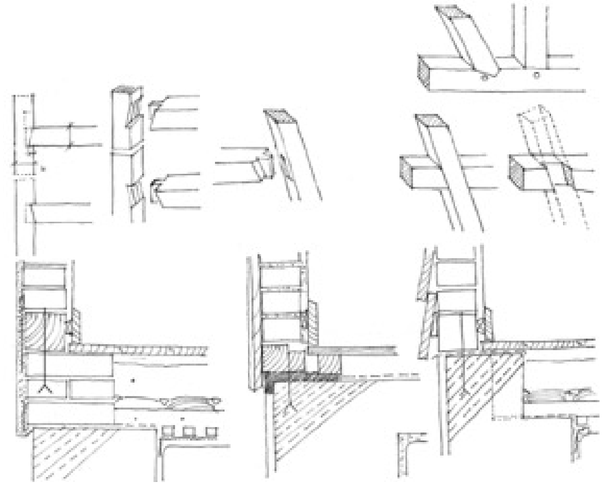

FIGURE 6 Joints in hımış framework.

is emphasized from the outside with a bay window or a balcony. The *sofa* can be an enclosed space or it can be open-air, depending upon the climate of the region. Each room is designed to be multifunctional, with utility spaces between two rooms. This enables living in the daytime and sleeping at night. Utility spaces are furnished with a small bathroom and with storage for the bed, linens, and wardrobe.

Structure of the Hımış Building

The wall of a hımış building consists of a load-bearing wooden skeleton or framework, infill material, and a veneer on both external surfaces. Historically, buildings have used varying dimensions of timber according to particular needs.

The design of the framework involves a header (head beam) and footer (foot beam), which carry vertical wooden posts at 1.5 m (59 in.) intervals (fig. 6). Between these load-bearing main posts and running every 60 cm (about 23 in.), there are intermediate vertical posts and intermediate horizontal beams for holding the infill and veneer. The cross section of the main post is about 12 × 12 cm (4.7 × 4.7 in.) to 15 × 15 cm (5.9 × 5.9 in.); the intermediate post ranges from 6 x 12 cm (2.3 × 4.7 in.) to 6 × 15 cm (2.3 × 5.9 in.). Diagonal braces connect the foot beam to the main post (fig. 7), creating a triangle for the wall's stability. These diagonal braces (fig. 8), which have the same cross-sectional dimension as the main post, carry the horizontal load. It is therefore important that a diagonal brace be one continuous piece of wood and not divided where it intersects intermediate posts or beams.

Floor framework carries the loads of the building from wall to wall. In this case each beam has a larger dimension than the posts. Beams are placed equidistant from one another at 40–60 cm (15.6–23.4 in.) intervals. Head beams on the wall studs carry all floor beams. The floor framework is rigid and acts as a horizontal diaphragm. In the last century, standardization has led to limited lumber dimensions for ease of production and delivery.

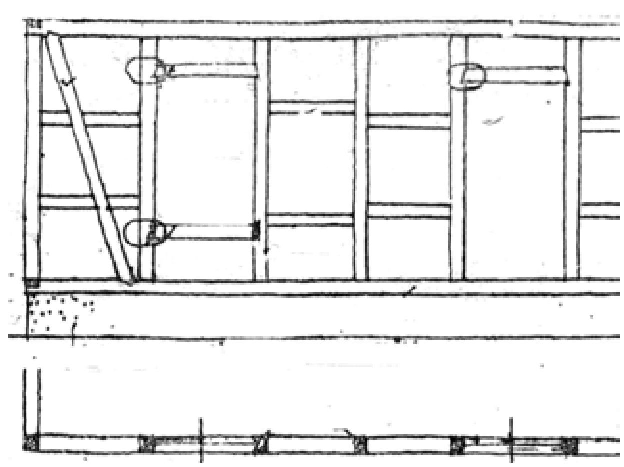

FIGURE 7 Hımış frame wall construction.

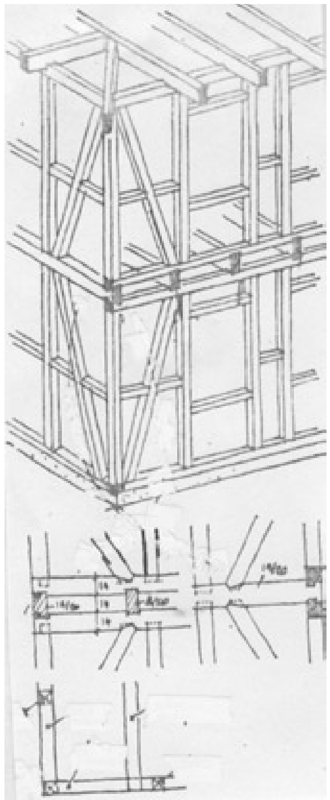

FIGURE 8 Two-story hımış structure with wooden framing.

FIGURE 9 Earthen bearing-wall system in Cyprus.

FIGURE 10 Earthen bearing-wall system in Güre, on the Aegean coast of Turkey.

The walls of the upper floors are situated on top of and are carried by the walls of the lower floor (fig. 7). Floors of the whole building, whether located on the ground or between the stories, function together as diaphragms. Along with the walls, they establish the stability of the whole building. Roofs are pitched and covered with ceramic tiles.

Earthen Bearing-Wall Buildings

Earthen bearing-wall buildings are constructed of unburned bricks. Mainly they have one or two stories. In some parts of the country, buildings do not have wooden structures, even though wood is available. Climate, workmanship, and safety are among the criteria determining whether buildings are wood framed or made with a solid earthen bearing-wall system (figs. 9 and 10).

Design of Earthen Bearing-Wall Buildings

Earthen bearing-wall buildings are smaller than hımış buildings. The number and dimensions of the rooms are smaller. Wooden beams for flooring and roofing are shorter. If the building is two-storied, there are generally two rooms on each level. Most earthen bearing-wall buildings do not have basements. They are constructed directly on the ground with stone foundations, and window openings are small.

Structure of Earthen Bearing-Wall Buildings

Door and window openings are structurally framed with wooden lintels. Floor or roof levels have wooden bond beams. A flat roof is often used on earthen bearing-wall buildings (fig. 11). Skilled labor is needed to construct

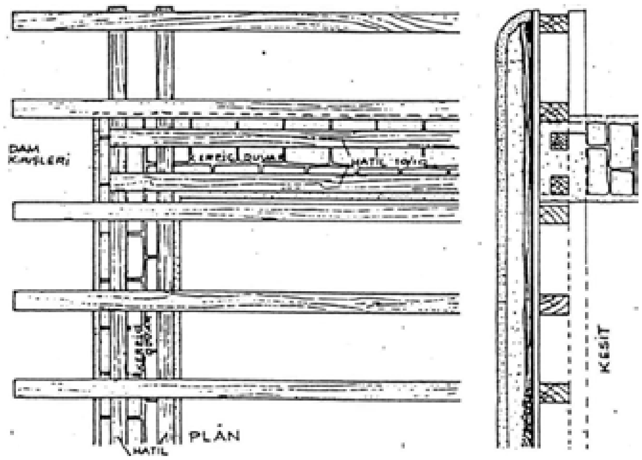

FIGURE 11 Wooden construction with a flat roof and tie beam.

the floors, which function as diaphragms, holding the structure together while allowing the horizontal forces to move loose and round joists. If a flat roof has a thick earth layer for heat insulation, loose elements of the structure move with more acceleration, resulting in fatal collapse. On the other hand, heavy loading on rigid diaphragm flooring contributes to the stability of the bearing-wall building.

Seismic Response of Earthen Buildings

Existing earthen buildings are heritage from the last century. Good examples in city centers have been demolished to make way for new structures of reinforced concrete. Existing buildings remain in small towns and at the edges of cities. Only a small number of earthen buildings are being conserved. There is a need for research and education on design and construction of earthen buildings.

Seismic Response of Traditional and New Wood-Frame Building

Wood as a structural material is easy to carry and handle during construction, and it offers advantages in earthquake performance. It is lightweight; thus, earthquake acceleration generates less energy than with other structural materials. The material and structures are flexible and are consequently able to absorb and dissipate seismic energy.

New design principles for wood-frame buildings depend on industrialized wooden products. Posts are available in standardized lumber sizes and are smaller; therefore, they can be spaced more closely. If necessary, larger dimensions of posts or beams can be obtained by multiple connections. Shear walls are designed using panel products such as plywood or oriented strand board, instead of diagonal braces between vertical posts. This kind of design is also called platform framing. Floor framing is anchored to the foundation with bolts. Shear walls underpin the roof.

Traditional wood-frame construction in Turkey uses lumber of specific dimensions. The size of the main post varies according to the loads and function. Braces carry the lateral forces from header to footer and are of the same size as the main post. There is no reason, except visual image, for holding onto this traditional method. It would be logical to use standardized lumber for new buildings.

There are remarkable differences in the foundations of these two building systems. As opposed to the more modern platform framing technique, traditional wood-frame buildings have foundations of natural round stones with a soil mortar, and the rigid building structure sits loosely on this foundation (fig. 12).

According to the Turkish Earthquake Code, the design of traditional wall framing has been

FIGURE 12 Traditional wood-frame building with stone foundation.

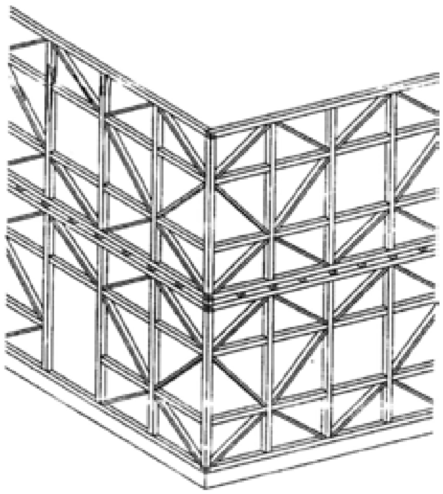

FIGURE 13 Diagonal bracing, as illustrated in the Turkish Earthquake Code.

revised in the following ways (T.C. Bayindirlik ve Iskan Bakanligi 1996):

1. posts with maximum 1.5 m (59 in.) spacing
2. base beams to be placed under the posts
3. head beams to be placed above the posts
4. horizontal intermediate beams connecting posts along the wall together with base and head beams, to form rectangular areas in the wall
5. diagonal braces converting rectangular areas into triangular areas (fig. 13)

These measures seem to be unnecessary, and they generate design problems for wall openings such as windows and doors. Traditional buildings throughout history have shown a secure response to earthquake forces, although they are simple in design. Traditional existing hımış buildings seem to have sufficient protection against earthquakes.

Lack of durability and maintenance can be mentioned as seismic vulnerabilities. Exposing hımış buildings to external conditions can cause plaster to spall, earthen infill blocks to deteriorate, and wooden parts to decay (fig. 2). Maintenance is therefore essential and can sustain the designed earthquake response properties of the hımış building type.

Seismic Response of Earthen Bearing-Wall Buildings

Structural reliability is gained through material and design properties. Unlike wood used for frame structures, earthen material is low in strength and has markedly lower tensile strength than other structural materials.

Research into Earthen Construction Systems at Istanbul Technical University

Global research on earthen material has focused mainly on improving its strength. In the last century, earth was mainly stabilized with cement. This practice conflicts with an ecological environment and healthy living. Cement is an energy consuming material, and the physical characteristics of the earth and cement composite are similar to those of concrete—far from the properties of natural earthen materials.

On the other hand, natural earthen materials produced by skilled craftsmen have the required compressive strength, unless exposed to moisture. In order to address adobe's vulnerability to moisture, further research into the effects of added materials other than cement on water-resistant stabilization is needed.

Basic Materials Research

To reduce the water vulnerability of earthen building material, the Architectural Faculty of Istanbul Technical University instituted Project MAG 505 (Kafesçioðlu et. al 1980), sponsored by TÜBİTAK (Scientific and Technology Research Council of Turkey). This was the first study in which earth was mixed with gypsum in different percentages. The mixture hardens in a few minutes and becomes load bearing, while traditional adobe must be stored and dried in the sun for several days before it is taken into the next construction stage. Also, the shrinkage decreases to 1% of the length, and appearance becomes smoother. Compressive strength of the dried (for 15 days) gypsum-stabilized earthen material is 2–4 N/mm^2 (290.1–580.2 psi). Flexural strength of this material is 1.1 N/mm^2 (159.5 psi), which is higher than that of bricks made of earth alone. The most remarkable result is that the new composite is more water resistant, due to chemical reaction between gypsum and clay. Deterioration due to rain is not notable.

Earthquake Rehabilitation Measures on Bearing-Wall Buildings

Better water resistance will increase the material's strength in case of water penetration and humid conditions, but earthen material is still of low strength when compared with the other types of structural materials that engineers are accustomed to working with. Consequently, constructing with low-strength material is confusing, and a lack of clear understanding can lead to failure.

Obviously this peculiarity of earthen material must be considered in building design. The theory of energy dissipation as a seismic measure in civil engineering can be adapted to earthen bearing-wall structures. This means that the structural design will not depend solely on the degree to which earthen bearing walls resist the severe lateral forces of an earthquake; in addition, the design will introduce some energy absorbing and dissipating features. Figure 14 shows a low-strength structure in Bam, Iran, destroyed by the 2003 earthquake. The building, which was compressed between the resisting cross wall and vertical seismic force, collapsed.

Studies at Istanbul Technical University showed that this energy-dissipating feature is ripe for further design and applications. A rammed earth wall sample was produced with horizontal joints. Biaxial loading, representing gravity and lateral earthquake forces, was applied. The wall sample cracked horizontally at the joints created by the geomesh layer, while the wall parts between the joints were undamaged (fig. 15). Geomesh was applied horizontally, in 17 cm (6.6 in.) intervals during the ramming process, and it is expected to take the tensile forces in the wall during an earthquake. Tests showed that the mesh layer performed as a friction surface.

Conclusion

Earthen material has been used up until the last century in bearing-wall systems and as infill within a wooden framework in the hımış construction system. In the seismic arena, wood-framework response is ductile under lateral forces, whereas bearing earthen walls are more rigid. Although earthen material is energy dissipating to some extent, earthen walls cannot resist lateral forces of earthquakes.

Existing measures for improving lateral resistance of bearing-wall systems include cross walls constructed perpendicular to the forces to impede movement of the building. The problem with this attempted solution is that the cross walls can remain without deformation, while the earthen building body itself can be destroyed by the action of earthquake forces and the reaction of the cross walls.

Studies at Istanbul Technical University showed that it is logical to design the bearing-wall systems to

FIGURE 14 Building collapse due to earthquake, Bam, Iran, 2003. Photo: © Randolph Langenbach, 2004.

FIGURE 15 Horizontal joints in an earthen wall sample and their response to lateral forces.

be energy dissipating. Working joints can function as energy absorbers and dissipate lateral forces.

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Observing and Applying Ancient Repair Techniques to Pisé and Adobe in Seismic Regions of Central Asia and Trans-Himalaya

John Hurd

Abstract: *The opportunity in the loess clay belt of China, Trans-Himalaya, and Central Asia to examine both highly sophisticated aseismic building techniques and repair techniques taps into a transmission of empirically learned skills and techniques going back at least three thousand years. In pisé construction across the Tibetan Plateau, in the southern Himalayan foothills, and in central and northern China, as well as in adobe construction across Central Asia, in the Turkic countries, and in the northern ranges of the Altai Mountains, the use and tradition of aseismic features and techniques survived until recently.*

Because these techniques come from a regional cultural understanding, they are often overlooked in favor of modern cement concrete solutions, or in favor of retrofitting using modern materials and techniques that are adequate but not necessarily suitable for the region. The ancient techniques known as "soft stitching" or "laminated stitches" and "dry packing" demonstrate their effectiveness by their survival over long periods in generally high seismic activity regions.

The author has examined repairs in Afghanistan, Pakistan, the former Soviet Central Asian Republics, the Himalayan chain states, and the Tarim Basin of western China. For the last fifteen years, he has been applying the observed techniques in cob structures in Britain, Central Asian archaeological structures, and Himalayan ruined structures, and he has built emergency shelters and used horizontal lifts of vegetable "mattresses" in the Badakshan 1998 earthquake in northern Afghanistan. The author had the opportunity to inspect these structures following another quake in 2000; the performance of the structures was adequate, and most occupants survived. In structures from the fourth century of the Christian era until the present day, there is demonstrated in the majority of traditional structures the use of aseismic ring beams of many types, incorporating timber, brick, and vegetation including woven sackcloth and straw. Repairs, including stitching repairs, show use of the same protocol of repeated ring beam lifts of different materials. Important techniques of dry packing still exist in the region today, and these long-practiced answers to seismic cracking and weakness deserve to be investigated, studied, and tested.

Background

During a "year out" after university in 1970, the author visited the northern Afghan, southern Pamirs city of Faizabad. During this visit he spent some days watching repairs being made to a bell-shaped circular watchtower to the west of the city. His interest was aroused, since the academic degree that he had just completed was in geology and focused on clays; the tower was built from adobe blocks. The tower had several almost-vertical cracks caused during an earth tremor the previous year, and it was these cracks that were under repair. Four circular chases had been cut into the tower, two internally and two externally. In the author's memory—he carried no camera—the chases were each about 0.5 m wide (about 20 in.) and perhaps 40 cm (about 16 in.) deep, cut into the wall, which was approximately 1 m (39 in.) thick. Only one chase was filled during his visit, an external chase that circled the central part of the whole tower. This chase was filled with flat adobe "bats," or thick tiles, and in between each lift were placed coarse

FIGURE 1 Stainless steel and adobe soft-stitch repairs to a corner and central end wall in Leicestershire, UK. The structure is an example of cob construction.

hemp fiber mats that were woven in situ, which made a complete circular "ring beam" around the tower. The author has no idea what happened to the other three chases formed, but he assumes that they were dealt with in a similar way. The masons made no explanation of what they were doing, but the activity remains vivid in the mind of the author.

In 1994 the author, by then a conservator of historic buildings, often of mud, cob, wattle and daub, or clay on posts, attended a conference on earthen architecture in Plymouth, UK. The Devon Historic Buildings Trust produced a leaflet (Keefe 1993) on repair techniques to cracks in cob that recognized a technique of forming chases filled with adobes, but replacing hemp matting with stainless steel expanded metal lath.

In 1995 the author repaired cracks in a cob barn in Leicestershire, England, using the same techniques (fig. 1). This barn was converted to a residence, and from completion of the work to the present day, the repairs have remained strong, and no further movement has occurred.

The Horizontal Ring Beam Principle

In 1998 a severe earthquake occurred to the west of the region of Faizabad in Afghanistan. The author—because of his familiarity with the region and because no infrastructure existed to access the earthquake area because of the proximity of the Taliban/Northern Alliance front line—took charge of rehabilitation on behalf of the Irish charity Concern Worldwide, which, together with the group Shelter Now International, was required to build emergency shelters for the homeless inhabitants prior to the onset of winter. A tower repaired in 1970 was at that time severely damaged by warfare, and no trace of the upper repaired area survived. In the village of Shar-I Berzerg, only one old house remained standing; it featured wooden horizontal ring beams in its construction. Survival houses were built using recovered material from collapsed homes and featuring plaited hemp and bamboo ring beams. These structures were replicated by local masons, and thousands were completed across the region. The survival houses withstood several strong tremors and a second earthquake in the region in 2000. The author experienced a strong series of aftershocks while in a survival house; it shook and moved, but no large cracks developed.

All documentation held by the author, including his camera and film, were confiscated when the region was overrun by the Talibs. All foreign workers were forced to exit the region. The Swedish Committee for Afghanistan retains some photographs and drawings of the emergency shelters. The author himself escaped by hitchhiking a ride on a United Nations helicopter to Islamabad in Pakistan, but he remained in touch with courageous representatives of the Afghan NGO Pamir Reconstruction Bureau, who continued the work and eventually walked out of the dangerous region to safety on the Pakistan side of the Hindu Kush.

Across the region from Armenia in the Caucasus, Central Asia, and Trans-Himalaya, the author has noted structures, both archaeological and standing, which demonstrate the "ring beam" principle. Some observers believe that the use of vegetable, wooden, turf, and even brick lifts in adobe and pisé construction is simply decorative. The author believes that these methods represent a sophisticated and ancient understanding of aseismic construction.

A selection of photographs and archaeological drawings demonstrate their use from the fourth to the nineteenth centuries (figs. 2–12). Locations include Turkey, where Ottoman stonework has continual and frequent lifts of brick tiles, and Armenia, where pisé structures have regular lifts of wood, bark, and other vegetation and also bear evidence of adobe repairs using the same protocols.

FIGURE 2 Pisé mosque with horizontal bark and twig ring beams in regular lifts, Armenia.

FIGURE 3 Mihrab of a pisé mosque in Armenia.

FIGURE 4 Pisé lifts in a mosque in Armenia. The visible lifts include organic fiber.

FIGURE 5 Bark ring beam, Armenia.

FIGURE 6 Timber-edged stitch with additional twig reinforcement, Armenia.

FIGURE 7 Pisé city wall, Yerevan, Armenia. There are regular horizontal lifts, each lift line concealing a turf lift.

FIGURE 8 City wall with pisé lifts, Armenia. A horizontal turf lift is revealed behind plasters on the lower wall.

FIGURE 9 City wall with horizontal turf ring beam, Armenia.

FIGURE 10 Byzantine city wall with brick ring beams, Istanbul.

FIGURE 11 Nineteenth-century wall with large-block ring beam, Kyrgyzia. These modern remnants show horizontal changes in block size.

FIGURE 12 Adobe wall with horizontal vegetation mattresses in Yazd, Iran. The tower, which has been rebuilt, has no ring beam.

In the Silk Road cities of Central Asia, the theme continues right across China (figs. 13–16).

Ancient Repair Techniques

At a recent summer school in Ladakh, on the western Tibetan plateau, the author had the opportunity to demonstrate repairs to pisé structures from the fifteenth century. Here the pisé was constructed in short lifts of approximately 15 cm (about 6 in.) each. In every third lift, a vegetable "mattress" was rammed into the construction (fig. 17). Here cracks were stitched in the same techniques examined by the author across Asia, and

FIGURE 13 Pisé on stone plinth castle and monastery at Basgo, Ladakh.

FIGURE 14 Pisé lookout tower in Ladakh, exhibiting typical tremor damage.

FIGURE 15 Stone and pisé castle in Ladakh.

FIGURE 16 Detail of pisé, Ladakh.

FIGURE 17 Detail of horizontal vegetation (*yagtsa*) mattresses at every third lift. Ladakh.

these were confirmed to have been described in cultural records both by Ladakhi monks and by Bhutanese builders. A Himalayan architect kindly made a drawing of the techniques used (fig. 18).

A chase is cut to almost half the thickness of the wall (fig. 19); this chase has deep returning ends in the form of a staple. The chase is continually wetted down with water during the construction process to eliminate suction leading to hairline cracks around the repair (fig. 20). The chase is then filled with alternative lifts of wet vegetation or woven matting with lifts of adobe bricks (figs. 21 and 22). The top course of the stitch some 10–15 cm (about 4–6 in.) deep is then wetted down and strongly but carefully mallet-dry-packed with loose material identical to that of the blocks (fig. 23). The dry packing presses down the whole stitch into a dense and strongly rammed fill (fig. 24). Alternate lifts are achieved at approximately 0.5 m (about 20 in.) intervals, internally and externally. The stitches are of varying length to allow for stitching of subsidiary cracks and to prevent the formation of new cleavage planes that may develop from stitches of regular length (figs. 25 and 26).

Internal walls can be reinforced with triangular stitches where cracks occur.

The author has added to the range of stitch forms by the invention of a capping stitch that uses the same techniques as other soft stitches but which is shaped like a butterfly and is rammed directly from above. This “butterfly cap stitch” can then be covered by a domed shelter coat, generally used by the author to protect historic structures from weathering erosion and decay. While the soft stitches need little maintenance, situated as they are within the body of the wall and having

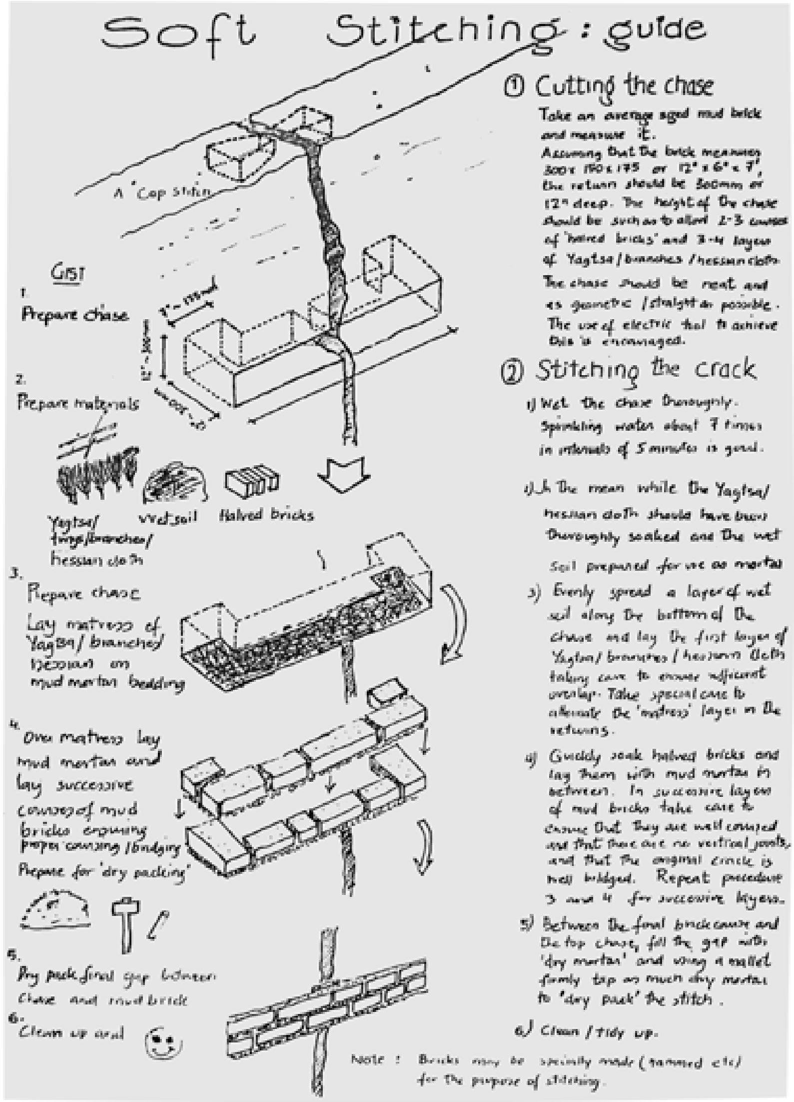

FIGURE 18 Soft-stitch crack repair techniques, including butterfly cap stitch. Drawing: Karma Gelay.

FIGURE 19 Cutting the chase in the shape of a staple.

FIGURE 20 Wetting the finished chase.

FIGURE 21 Filling the chase with *yagtsa* and hemp matting.

FIGURE 22 Filling the chase.

FIGURE 23 Dry packing the top of the stitch.

FIGURE 24 A completed stitch.

FIGURE 25 A second stitch above the preceding one.

FIGURE 26 Conservation work in progress, Ladakh.

a compaction similar to the rest of the surrounding masonry, the topmost shelter coat requires regular maintenance.

This soft stitching technique is recognized throughout the seismic regions of Asia and is empirically understood by masons, who frequently describe examples in their own regions. Many subsidiary craft skills were described to the author, including admixtures to the earth used for block repairs.

The author has not had the opportunity to test or to examine technically the engineering performance of the techniques, but he has observed the use of still-functional soft stitches on structures that have survived many earth tremors and quakes over the centuries. This paper is therefore a show-and-tell description of historical techniques observed and used by the author over the last twelve years. He has never read nor has he encountered any other description of the techniques that he has observed and applied.

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Research on the Upgrade of Traditional Seismic Retrofits for Ancient Buddhist Temples in the Region of Spiti and Kinnaur in the Western Himalayas

Sandeep Sikka and Charu Chaudhry

Abstract: *The ancient Buddhist temples in the western Himalayas have evolved spontaneously in response to the region's extreme climatic conditions and limited material resources at hand. These structures have been improved through a constant process of trial and error over the years by the local craftsmen/builders to withstand the seismic vibrations and other natural calamities. Unfortunately, these earthen buildings lie in one of the most vulnerable seismic zones, Zone IV (Bureau of Indian Standards 1993), and have experienced some damage in the past. Buildings in the region today are susceptible to damage from the annual precipitation of 7.8–15.6 in. (200–400 mm). One of the main areas of research on the conservation of these historic earthen buildings, conducted as part of a scholarship from the Museum of Archaeology and Anthropology, University of Cambridge, UK, was to design and develop reinforcements for the structural components of the earthen buildings to mitigate the effects of seismic vibration. The research involved detailed documentation and analysis of existing seismic retrofits installed in the buildings, followed by a detailed assessment of the structural risk, through (1) precise documentation of the structural members and their deflection after earthquakes, (2) understanding of the stress on the walls through study models of the historic roofs, and (3) strength analysis of the existing historic adobe bricks. The study also addresses the climatic change the region is experiencing along with an expected increase in moisture levels, which has led to a considerable decrease in the ability of adobe structures to resist any tectonic movements.*

Conservation of this living heritage raises two important conflicting issues. On the one hand, the preservation of the ancient architecture and its features in their original form is of utmost importance as a document of history. On the other hand, this living heritage poses a serious threat to the safety of the inhabitants during an earthquake. Retrofitting could alter and interfere with the historic fabric and poses a serious threat to the resources' authenticity. The practical design and development of seismic retrofits for such ancient existing earthen buildings in the region should consider the potential hazards to life, the present condition of the structures, and materials and their behavior before another earthquake. This paper puts forward the results of the study and describes the condition of historic earthen structures in the region after the earthquake of 1975. It describes the traditional seismic retrofits existing in the structures and explains possible techniques and materials for the development of new seismic retrofits to strengthen the structures and material components before another earthquake.

Introduction

The Spiti and Kinnaur region in the northern Indian state of Himachal Pradesh has some remarkable ancient adobe Buddhist temples. Constructed between the tenth and fifteenth centuries, these temples preserve some of the earliest artistic heritage of Tibetan Buddhism in the form of mural paintings, polychrome clay sculptures, and decorative wooden ceiling members (Luczanits 2004). Over a period of five hundred years, this arid region in the western Himalayas has witnessed the gradual development of Buddhist temple architecture, from simple, single-story buildings constructed on relatively flat land to a complex maze of multi-story fortresses on the mountaintop. Standing on a highly seismic zone and having survived for several centuries, the buildings are undoubtedly living evidence of highly engineered structures.

Study of the materials and the structural configuration of these historic earthen buildings provides information about the evolution and function of each structural module, as well as the traditional seismic retrofit methods that developed gradually and were installed to counter the movements of various components during an earthquake (Sikka 2002b). Constant intervention and experimentation by local craftsmen after each past earthquake furthered understanding of the general behavior of the earthen structures in the region, their inherent construction defects, the effectiveness of locally designed and engineered traditional seismic retrofits, and possible methods and materials for the further reinforcement of these ancient structures.

FIGURE 1 The tapered load-bearing adobe walls at a temple in Spiti provide extra stability to the building during seismic vibration.

Buddhist Temple Architecture: Materials and Method of Construction

Buddhist temple design is characterized by rectangular spaces with carefully designed structural members, the result of years of trial and error under extreme climatic conditions and natural disasters. The walls of these historic structures are made of adobe—large, sun-dried mud brick laid in mud mortar. The foundations of rubble-stone masonry generally rest on stable solid ground.[1] The thickness of the walls varies from 2.5 ft. (0.76 m) to about 5 ft. (1.52 m) in some of the early-period structures. A survey of some twenty-five ancient earthen structures in the region reveals that the ratio of the height (h) of the single story to the thickness of the adobe walls (t) lies within the recommended safe limits of $t > h/8$ of the modern Indian seismic code for earthen structures (Bureau of Indian Standards 1993). Vertical measurements show that sometimes the outer faces of the walls are slanted, so that the wall thickness is wider at the base and gradually tapers to the top, providing extra stability to these tall and flexible structures (fig. 1).

Because of the cold climatic conditions most of the year, the openings in the walls of the temples are kept to a minimum. They contribute to less than 5% of the total wall surface of the rectangular space and are generally located at the center of the wall. The only source of light and ventilation is generally a low and narrow entrance doorway. In addition to the wooden doorframes, punctures in the walls are reinforced with thick vertical and horizontal wooden members connected to one another by flexible joints. There is a series of wooden lintels laid next to one another along the thickness of the walls; they are anchored deep and extend into the masonry on both sides of the openings like additional horizontal tie members.

The roofs of the temples are flat; because of the lack of rainfall in the region, a flat roof with little provision for drain-off is a practical design. Roofs are made of mud laid in various layers and compacted. About 7 in. (0.17 m) of compacted mud rest on 2 in. thick (0.05 m) rectangular wooden panels or a mesh of willow twigs, with a layer of local shrubs or birch bark sandwiched between the two for waterproofing. These are in turn supported by wooden rafters and beams, which are supported directly on load-bearing mud walls and wooden columns.

This historically well-engineered building typology is today susceptible to innumerable natural threats and human interventions. Fluctuation in the climate in the past few years and frequently occurring earthquakes, the two major natural agents of decay, have put these water-soluble and brittle structures under serious threat.

The Western Himalayan Region and Its Tectonic Evolution

The tribal area of the western Himalayan region, which covers most of the area of the Spiti and Kinnaur districts of the state of Himachal Pradesh, stands on the relatively young and highly unstable Himalayan Mountains. Following the collision of the Indian subcontinent with the Eurasia plate about forty-five to fifty million years ago, the uplift caused by the collision resulted in the development of the Himalayan mountain range at the intersection of two tectonic plates (Bagati and Thakur 1993). Further studies on the geology of the western Himalayan region of Lahaul Spiti and Kinnaur in India propose that the tectonic plate of the Indian subcontinent is still moving slowly toward southern Tibet (Bilham et al. 1998; Bilham, Gaur, and Molnar 2001). This results in an increase in the height of the young Himalayan Mountains every year. It is thus understood that the area is geologically active and structurally unstable. This is the reason the region experiences the inevitable and frequent earthquakes and landslides.

Earthquakes in Spiti and Kinnaur

Areas of the Spiti and Kinnaur districts have experienced some significantly strong earthquakes in the last few decades. They lie in the highly seismic Zone IV, identified by the Indian Standard IS 1893:1984 (Bureau of Indian Standards 1993).[2] A major earthquake (Richter magnitude 6.0) struck the region of Lahaul Spiti on the morning of June 17, 1955, causing enormous damage to the villages in the Spiti Valley (www.himvikas.org/jan2004/seis.htm). The most powerful earthquake that struck the region was on the afternoon of January 19, 1975; it killed sixty people in the most sparsely populated region of India. This earthquake registered 6.2 on the Richter scale, with an aftershock of magnitude 5.8. It caused serious damage to several villages in the Lahaul Spiti and Kinnaur districts and even caused some structural damage to the buildings in Ladakh, which is adjacent to the district bordering Himachal Pradesh. Besides such major earthquakes, the region experiences periodic minor tremors. The earthquakes in the past have been fatal for the people of the region and have caused acute damage to the landscape and the cultural heritage. Improving understanding of the earthquake resistance provided by both traditional and modern retrofits, as well as the economic costs of incorporating them in future conservation work, can reduce the seismic risk to people and buildings.

History of Seismic Retrofits in the Region

Historically, earthen buildings in Spiti and Kinnaur were reinforced with additional structural supports to guard against damage from frequently occurring earthquakes. The load-bearing walls of the temples are reinforced with a wooden framework of horizontal wall ties (fig. 2), with a cross section of 6–8 in. by 4–5 in. (approx. 0.15–0.20 m and 0.10–0.12 m), forming a series of ring beams around the building. These are installed externally and flush with the surface of the wall. The ring beams tie the entire structure together, with each beam running at a distance of approximately 3 ft. 3 in. to 6 ft. 6 in. (1.50 to 2.00 m) from the other. These horizontal wall ties are then joined to each other at the corners with wooden vertical ties.[3] This arrangement prevents any outward

FIGURE 2 The external surface of the temple at Nako village in upper Kinnaur (AD 1025) is fitted with horizontal wooden wall ties and supporting rubble stone buttresses at the corners.

movement during seismic vibration, and these wall ties, along with the wooden lintels, disrupt structural cracks that could otherwise extend the full height of the wall, eventually causing total collapse. The strength of the adobe and the mortar joints varies all along the wall; the result is differential loading. The horizontal ties therefore redistribute the load evenly throughout the wall.

In some buildings, in addition to these horizontal members, the load-bearing walls are reinforced at the corners with symmetrically placed buttresses to avoid shear or separation of load-bearing walls at the corners. These buttresses are either made of adobe blocks or rubble stones stacked one over the other against the corners of the building, forming a pyramid (fig. 2). The buttress rests on a solid stone masonry foundation placed or built at the corners as additional support. As buttresses are not integrated into the masonry walls, it is possible they were added later to support the masonry at the corners. There are no vertical tie members.

Aftermath of the Earthquakes

Most of the villages and the vernacular buildings in the region of Spiti and Kinnaur were badly damaged during the last earthquake in 1975, as were the historic Buddhist temples in the region. Structural documentation of the internal and external surfaces of some of the historic structures, conducted by manual measurement of the profiles of the walls in a grid of 1 ft. 7 in. × 1 ft. 7. in. (0.50 x 0.50 m), revealed that there is tremendous outward movement and deformation in the upper portion of the load-bearing walls (Sikka 2002b). The displacement of masonry is not uniform, and neither is the curvature and bulging in the walls. Vertically induced oscillations during the earthquake caused a sudden increase in the roof load, which, when applied against the compressive strength of the adobe walls, resulted in an out-of-plane movement. The upper courses of the adobe wall along the intersection with the roof moved outward as a response to the additional load.

As mentioned above, earthquakes are not the only threat to the historic adobe structures. The historic Buddhist structures, which were originally designed for an arid climate, are now facing problems caused by the increased precipitation and regular rainfall of the last few years. The rain has washed away the clay from the compacted mud on the flat roofs, causing large-scale seepage into the interiors. As an immediate response, the local residents applied additional layers of mud and clay in an attempt to waterproof the roofs. Applications of extra layers of mud have considerably increased the load on the existing structures (Sikka 2002a). This led to further lateral outward horizontal movement in the load-bearing walls, resulting in the separation of joints at the corners of the building which caused vertical cracks in the masonry. Consequently, the lateral movement of masonry resulted in the loss of structural integrity, which further resulted in uneven distribution of concentrated loads on the structural members resting on the load-bearing walls. The structures normally react to additional loads through sagging of the wooden beams and rafters supporting the roof (fig. 3). This makes the structure more vulnerable to future seismic vibrations.

The earthen buildings in the region, although designed very carefully to resist seismic vibrations, have certain inherent construction defects. Ceilings, with their structural wooden beams and rafters, rest directly on the load-bearing mud walls without any wall plates. Point loads exerted by the structural members resting on the walls in the absence of wall plates have created enormous stress on adobe walls, especially during the vertical oscillation of an earthquake. This has resulted in major and minor structural cracks below the ceiling level where the brackets support the wooden beams and rafters on the walls. These points have now become inlet points for the ingress of water into the interiors; they are now a regular feature in many temples in the region of Spiti and Upper Kinnaur.

Not only has water entering the structures caused enormous damage to the murals inside the Buddhist temples, it has also washed away mud from underneath the brackets supporting the beams. Consequently, the brackets have shifted from their original position and have caused further movement in the wooden structural members, leading to loss of structural integrity. The horizontal and vertical wall ties in most of the historic structures are either missing or are discontinuous; therefore, the ties may fail to provide any kind of protection to the structure during an earthquake.

The temples that were built like multi-story forts were affected the most. Earthquake vibrations induced in the integrated structural system of multi-story temples, such as the Dhangker Temple in the Spiti Valley, resulted in serious detachment of the entire building

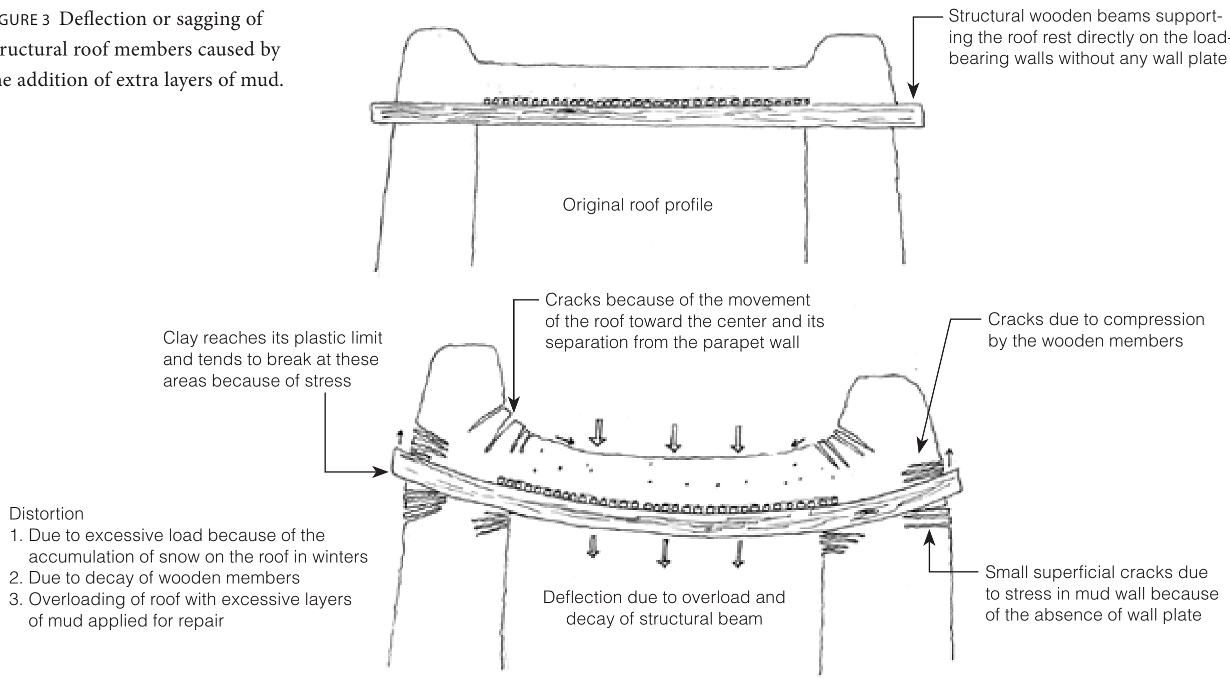

FIGURE 3 Deflection or sagging of structural roof members caused by the addition of extra layers of mud.

from the point of anchorage into the adjacent hill, causing major vertical structural cracks (fig. 4).[4] The process has left the building extremely vulnerable and defenseless to withstand any further seismic vibration.

Last but not least are the environmental humidity and the induced humidity into the structure caused by ingress of water through cracks left by previous earthquakes. This causes expansion and shrinkage of finer particles, which in turn causes fine cracks in the masonry and plaster. Water gets into these cracks and causes further deterioration every season. Not only has rainwater washed out the mortar from the masonry, it has also reduced the compressive strength of the historic adobe blocks by washing away the finer particles. High-velocity winds in the western Himalayan region have abraded the external surface, displacing finer particles from the external render and causing voids in the plaster (Warren 1999, 88). The impact of rain, particularly in the presence of high-velocity winds, has been fatal to these water-dispersible earthen structures. In addition, air movement extracts water from the structures by evaporation that has not only changed the surface volume but has also made the surface brittle. Excessive moisture swells the binder (clay), and sudden evaporation leaves voids between the platelets of the clay. Loss of moisture due to excessive evaporation leaves very little adhesion between the binder and the aggregates; hence the walls are vulnerable to erosion (Warren 1999, 95).

FIGURE 4 Vertical structural crack between buildings and the adjacent hill. This significant detachment was caused by the earthquake.

The wooden structural members (rafters, beams, and wooden ceiling panels) are also affected by the induced humidity and water ingress. The damaged wooden structural members removed during the process of investigation and conservation revealed that high moisture content had rendered the wood susceptible to insect attack. The roofing members—ceiling panels and wooden rafters and beams—were partially or completely eaten, so that structural elements are rendered hollow. The lack of ventilation inside the temples has caused dry and wet rot in the wooden members, diminishing their structural strength and increasing their vulnerability to insect attack.

First Response to Earthquake Disaster

Immediately after the earthquake of 1975, local residents responded to the building movements and structural cracks by instituting remedial measures. The corners of the buildings, which had separated because of the outward movement of the walls, were either filled and stitched with rubble stone masonry, or they were supported externally with piled stone (fig. 5). The piles of stones acted as buttresses to halt further outward movement of the load-bearing walls. The rubble stone buttresses at the temples in the upper Kinnaur village of Nako are not coherent, and since they were added later, they are not interwoven with the load-bearing walls. During an earthquake, the rubble stone buttresses vibrate independently of the main structure because of the lack of cohesive bonding between the buttresses and the walls. This might eventually cause damage to the outer surface of the adobe wall. Furthermore, the buttresses at the Nako Temple trap moisture inside the gap between the walls and the rubble stone buttresses. The moisture eventually seeps into the walls, causing several other problems of masonry deterioration. Similar adobe or dressed stone buttresses were added to the historic earthen structures at Tabo in Spiti.[5]

FIGURE 5 Piles of rubble stones stacked against the corners of the buildings after an earthquake in Nako village, to serve as buttresses to halt further outward movement of the masonry walls.

After the earthquake, the sagging structural beams inside the temples were immediately supported with additional props. In some villages, entire roofs were demolished, and new wooden beams were laid that again rested directly on the wall, without wall plates. Instead of repairs with adobe block, the damaged masonry at the intersection of the roof and the walls has been repaired with rubble stone masonry and mud mortar. Stone blocks, although more resistant to moisture, are unable to provide a compatible bond with the historic material. At the same time, the excessive weight of the stone masonry laid over the low-compressive-strength adobe blocks to support sections of the heavy roof has resulted in large-scale detachment and bulging of the masonry walls. Repairs conducted after the earthquake could not do much to heal the heritage buildings, as their structure is now far more vulnerable to another earthquake than previously.

Response to the structural disintegration has to be planned and should be carefully designed, especially when the buildings are being used every day. The historic Buddhist temples have weathered over a period of nine hundred years and have lost some of their original strength. To develop retrofits for such buildings to increase their ability to resist future seismic waves, it is essential to evaluate the current condition of each structural component to be supported and strengthened.

Research Results for the Upgrade of Adobe Structures Against Earthquakes

The research results discussed in this section address each building component and possible new installations that contend with the inherent defects responsible, both directly and indirectly, for the overall stability of the buildings and their behavior during a seismic vibration. Each component that could be used for conservation or repair work has been upgraded and tested separately in the field with similar material and environmental conditions.

Study Models for Designing Roof Load

The static load of the roof increases several times due to earthquake acceleration forces. These loads should be kept to the minimum possible and should be distributed evenly over the load-bearing walls. It was therefore decided to calculate the current actual roof load to further assess the total strain on the load-bearing walls.

To ascertain the roof load exerted on the adobe blocks, a model of the historic roof was constructed using materials obtained locally and following traditional construction methods. The model was assembled inside a cardboard box 1 ft. 3 in. × 11 in. × 11 in. (0.39 × 0.29 × 0.26 m) and weighed with a spring balance. The roof assembly was composed of a layer of willow twigs about 2 in. (0.05 m) in diameter laid at the bottom, with a layer of local shrubs laid in a perpendicular direction on the top to form a denser mesh. These layers of shrubs were then covered with a layer of wet mud approximately 1.5 in. thick (0.04 m), followed by a 6 in. (0.15 m) layer of dry compacted mud. The final layer was covered with a thin layer (1 in., 0.03 m) of local clay. The layer of clay was then covered with a 1.5 in. (0.04 m) mud slurry. The total height of the mud roof model was 10.25 in. (0.26 m), which is approximately equal to the height of the historic as well as the vernacular roofs existing in the region.

The roof was weighed after it was completely dry and was found to be 56.3 lb./ft.^{2} (25.6 kg/ft^2 or 273.9 kg/m^2). The weight of the rafters and beams supporting the mud must be added to calculate the cumulative weight. The well-seasoned wooden rafters of local cedar were weighed and measured. The weight of a cylindrical rafter of an average diameter of 4.3–5.9 in. (0.11–0.15 m) and length of 20.7–22.0 ft. (6.3–6.7 m) is 57.2–77.0 lb. (26–35 kg). It is therefore estimated that the average roof load in this region lies between 61.4 and 71.7 lb./ft.^{2} (300–350 kg/m^2). The roof load thus calculated was studied against the compressive strength of the existing mud blocks to assess whether the structure could take the load of the roof (Sikka 2002b). The dry compressive strength of three samples of the historic adobe blocks taken from the Chango Temple, tested at the laboratory in New Delhi, were found to be between 85.3 and 120.8 psi, or 12,283 and 17,395 lb./ ft.^{2} (6.0–8.5 kg/cm^2, or 60,000–85,000 kg/m^2).

Although the historic adobe blocks are able to withstand far more than the load of the historic roof, it is still crucial to make the roof lighter to prevent sagging of structural wood, as well as to promote resistance to earthquakes. Although reducing the thickness of the compacted mud roof may reduce the weight of the roof, it may not be able to withstand the increased precipitation in the region during the last few years. A 1.5 in. (0.04 m) layer of stabilized soil (1:3 parts lime:local sieved [0.7 in., or 0.018 m] soil) was applied to 4 in. (0.1 m) of compacted mud roof, and the model of the roof was weighed (Sikka 2003). The load of the roof thus configured weighed about 20.5–30.7 lb./ft.^{2} (100–150 kg/m^2).

Design of Wall Plates

Interventions made to the building should be minimal in order to preserve as much of the original fabric as possible. Because of the intrusion of moisture, the brackets supporting the beams and the rafters have been lowered from the original position, a change that affects not only the structural integration but also the surrounding wall paintings. To retain the original well-crafted and painted wooden brackets as well as the painted murals around them, the wall plates have to be designed and placed on the walls appropriately. The wall plates can be inserted along the level of the brackets (fig. 6) but slightly higher, so that the load from the beam or the rafter is not directly transferred onto the bracket supporting the structural members (Khosla 2004). The wooden frame with 6 × 3 in. (0.15 × 0.08 m) rectangular sections can be inserted in the wall after the careful removal of top courses of adobe up to the bracket level. As all the brackets are at different levels, it is essential to support the structural member up to the lowest level. This omits the point load and evenly distributes the load generated by the wooden beams (structural members) onto the mud wall. Some of the historic earth structures in the region are fitted with antiseismic retrofits in the form of horizontal continuous wooden ties. Wall plates inserted at the top of the walls, below the ceiling, function as wall-roof connections. These members can then be joined with the rest of the tie beams at the corners of

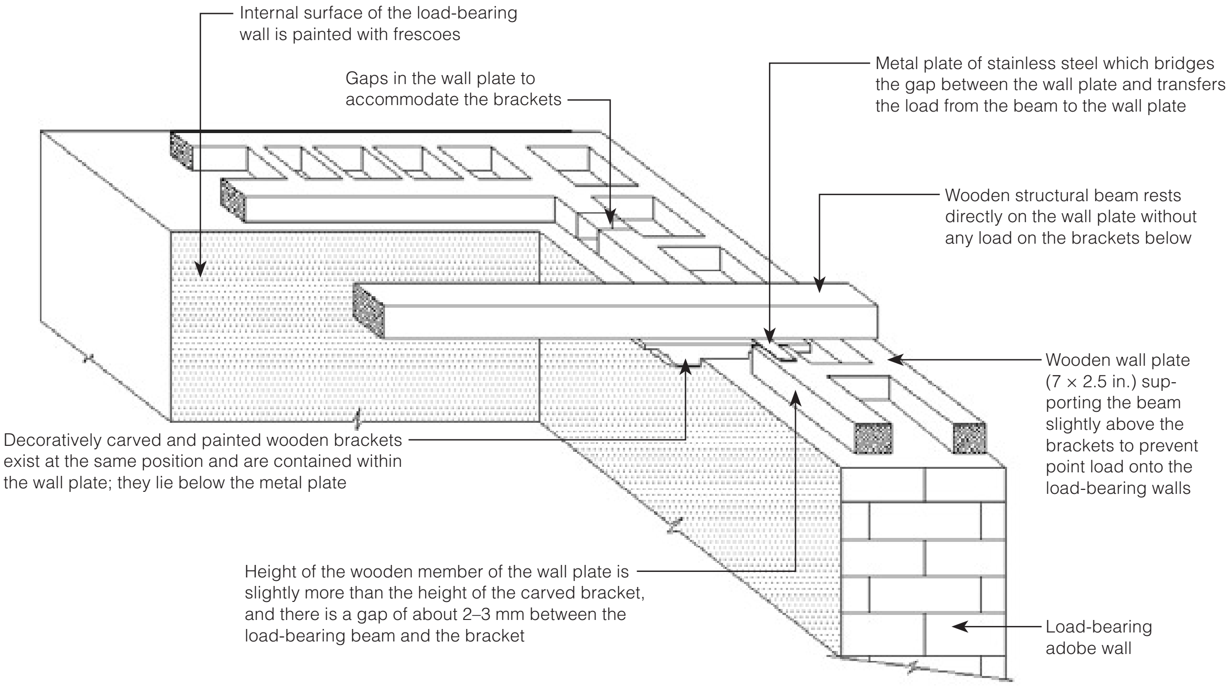

FIGURE 6 Wall plate detail for the historic earthen structures in the Spiti and Kinnaur Valley of the western Himalayan region, giving detail and sizes of the wooden wall plates running all along the length of the load-bearing adobe walls, to evenly distribute the roof load carried by the rafters and beams. Originally drawn by C. Chaudhry; taken from the phase 3 report of the Nako Preservation Project (Khosla 2004), with some modifications.

the building and with the vertical members, to form a frame for the entire building.

Insertion of wall plates in the Buddhist temples in the region could well be an interdisciplinary task, as the mural paintings need to be stabilized and shored before and after the insertion of wall plates. The painted plaster with murals inside these buildings comes almost to the ceiling level. The brackets supporting the wall plates are supporting the beams below the ceiling level. Insertion of wall plates at the bracket level may cause damage to the wall paintings, especially at the point of interface between the brackets, wall paintings, and ceiling. An alternative solution may be to increase the height of the wall and rest the members slightly above the original ceiling level, leaving an unpainted band, thus securing the overall structure and the paintings.

Design of Wall Ties

The walls of the historic earth structures can be strengthened against earthquakes using horizontal wooden wall ties in the form of ring beams inserted at a vertical distance of about 5 ft. (1.5 m) around the entire building. Existing wall ties can be carefully joined together with new members to form a continuous ring. These horizontal ties are then connected at the corners of the building with flexible vertical wooden members tied together with wooden pegs (fig. 7).[6] The lengths of the walls of the Buddhist temples are sometimes enormous and unsupported. Absence of any external intermediate vertical ties reduces their ability to resist horizontal earthquake forces, resulting in out-of-plane flexural cracks. Installation of vertical tie members at standard intervals (on the external face of the wall) depending on the height and the thickness of the walls is crucial.[7] The vertical tie members should be fixed, both at the base and at the roof level, to resist in-plane sliding, rotation, and out-of-plane movement, depending on the direction of the oscillations. These ties are installed inside the adobe walls under exterior and interior plasters. They run horizontally all around the perimeter of the building. The horizontal wall ties are laid after every 6.67 ft. (2.00 m) of mud brick wall courses and

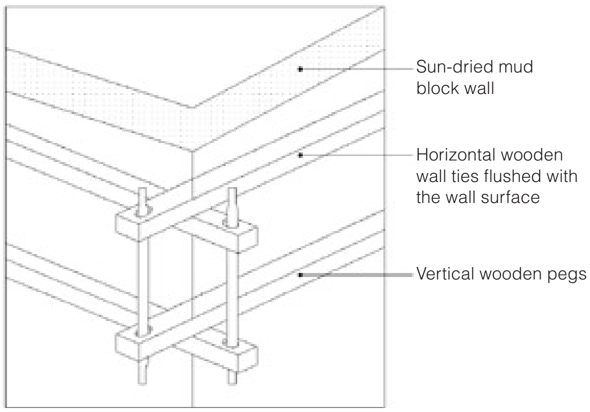

FIGURE 7 Seismic retrofit with horizontal and vertical wall ties, which are fixed like a protective framework, along the external facade of the earthen structures.

are anchored into the masonry with vertical pegs. The horizontal ties are connected to the orthogonal or vertical ties at the corners (see figs. 2 and 7 for connection of vertical with horizontal, as has been done for decades).

Natural Fiber Jackets

Deformations and bulges in adobe structures in the western Himalayan region, the result of prior resistance to seismic vibrations, are the vulnerable areas most likely to collapse in the event of future horizontal force. Close observation of the Buddhist temples in the region of Spiti and Kinnaur revealed flexural cracks and strains developed from compression that are visible both in the mud blocks and in the mortar, mostly near the ceiling level. Unreinforced adobe walls are brittle, with low tensile and flexural strengths, and therefore cannot withstand horizontal forces.

There is an urgent need to reinforce damaged and unreinforced masonry with protective composite laminates or jackets, in order to impart additional tensile and mechanical strength in an effort to avoid the loss of valuable polychrome artwork and wall paintings. Recent work shows that the jacketing of masonry applied on the external surfaces improves the masonry's ability to resist high-energy impact and increases the ability of the structure to further deform, delaying out-of-plane spalling (El-Dakhakhni et al. 2006). Recent publications and examples from research on reinforced concrete structures (RCC) also explain the effectiveness of external composite jackets in providing lateral reinforcement against earthquake forces (Kazemi and Morshed 2005; Perera 2006). Some work has been published on the use of steel and wire-mesh jackets on adobe walls, which have shown good results during an earthquake (Blondet, Garcia, and Brezev 2003).

The properties and efficiency of jute as a material have been explored recently. Published results regarding jute's use in reinforcing cement concrete and natural soil drains show that these natural, organic, polymer fibers have high tensile strength, as well as good thermal and mechanical resistance (Aziz, Paramasivam, and Lee 1981; Mansur and Aziz 1982; Lee et al. 1994). Its interface with other materials as composites increases its mechanical strength and resistance to environmental aging (Gassan 2002; Doan, Gao, and Mäder 2006; de Albuquerque et al. 2000). Jute and coir have been used in traditional building construction. More recently, many industries are using these environmentally friendly natural fibers as a reinforcement material for doors, wall panels, and partitions. Introduction of jute textile as a superficial wrapping, covering the walls underneath the protective external plaster, not only may help in improving adherence of new plaster to historic earthen surfaces in the western Himalayan region, but may also provide increased resistance to earthquakes. Its performance on ancient adobe structures as a seismic retrofit and its ability to provide additional tensile strength against seismic vibrations are yet to be tested. This solution could present some problems if it is used for historically significant decorated surfaces.

Additional Diagonal Bracing and Buttresses

Documentation of the sizes of cracks on the external and internal surfaces of the Buddhist structures revealed that additional diagonal bracing, preferably 1 × 1 ft. (0.30 × 0.30 m) with a cross section of 4 × 2 in. (0.10 × 0.05 m), can be used to support the few upper courses of the load-bearing exterior walls, to increase the structural torsion strength.

The piles of rubble stone masonry used for supporting the load-bearing walls at the corners of several buildings in the region must be reinforced or reconstructed with stabilized adobe blocks on a stable foundation. Careful detailing of masonry joints between the buttresses and the load-bearing walls will not only make a coherent structure but at the same time prevent intrusion of water into the historic masonry during the rainy season. The buttresses have to be plastered eventually with lime-stabilized local soil (1:3 lime:sieved local soil) to keep them dry (Bilham, Gaur, and Molnar 2001).

As weathering agents do not act alone on a building, the action of one may render the materials more susceptible to the subsequent action of another. Efficiency and performance of all the installation and seismic retrofits discussed in this paper depend on many other factors—including wind velocity, snow load, site drainage and bearing capacity of the soil after the recent change in climate, condition of the wall and its present moisture content, changing humidity and temperature in the interiors and outside the building, distance of the building from the epicenter, intensity and direction of seismic waves, and, most important, how the building is used and maintained in the future.

Conclusion

Conservation of this living heritage raises two important conflicting issues. On the one hand, the preservation of the ancient architecture and its features in their original form is of the utmost importance as a document of history. On the other hand, this living heritage poses a serious threat to the safety of the inhabitants during an earthquake. To a certain extent, retrofitting would alter and interfere with the historic fabric and poses a serious threat to its authenticity. The practical design and development of seismic retrofits for such ancient existing earthen buildings in the region should keep in mind the potential hazards to life safety, the present condition of the structure, and its materials and their behavior.

Acknowledgments

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Notes

1 Test pits dug next to the outer wall to the bottom of the foundation of the buildings revealed that the foundations generally rest on either rocky outcrops or solid ground.

2 The seismic zones in India are divided into five zones, V to I, with respect to the magnitude of the earthquakes on a decreasing scale: Zone I (no risk), Zone II (low risk), Zone III (moderate risk), Zone IV (high risk), and Zone V (very high risk).

3 The vertical ties are present in some, while missing in others.

4 Because of its precarious condition, the temple of Dhangker in Spiti was included on the 2005 World Monuments Fund Watch List of the hundred most endangered sites in the world.

5 The buttresses were plastered externally with mud, and there is no photographic record or published literature that tells us about the core material.

6 The vertical pegs are not rigidly fixed into the horizontal ties. This will provide leverage during a seismic vibration and at the same time prevent outward movement of the horizontal ties. Horizontal ties may or may not move together in the event of an earthquake, depending upon the type of seismic waves under the building wall. They are tied together with the help of vertical pegs, so that one prevents the other from dislodging from the wall and becoming independent.

7 Determination of standard distance for the installation of vertical tie members may depend on several factors, including the thickness and height of the walls, the strength of the masonry and the mortar, the intensity of the earthquake, and the closeness to the epicenter.

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Seismic Vulnerability and Conservation Strategies for Lalitpur Minor Heritage

Dina D'Ayala

Abstract: *Nepal lies in a region of the world with one of the greatest seismic risks. Nepal's Kathmandu Valley is home to a very high concentration of unique architectural heritage in the three capitals of Kathmandu, Lalitpur, and Bhaktapur, where buildings dating to the thirteenth century form a consistent portion of the urban fabric. While studies on the seismic vulnerability of other elements of the built environment have been undertaken, especially on primary services such as schools and hospitals, very modest technical research on the seismic vulnerability of the historic architecture of this region and on suitable retrofitting techniques has been undertaken. The procedure described below follows a methodology developed for vulnerability assessment of historic urban city centers in Europe. The work entails:*

- *identification of the most common structural typologies, in terms of layout, structure, and agglomeration*
- *a census of existing traditional seismic-resilient features*
- *selection of a sample of buildings of each typology in a particular area of the urban center*

For the selected sample the following is carried out:

- *development of a tailored survey form*
- *street surveys aimed at identifying geometric and structural features*
- *analysis of the data based on plasticity theory and collapse mechanism to assess seismic vulnerability*
- *definition of damage scenarios*

On the basis of the results obtained, recommendations for repair and strengthening form the conclusions of this paper.

Introduction

This paper presents the seismic vulnerability analysis and possible strengthening strategies of local traditional houses in the city of Lalitpur, Kathmandu Valley, Nepal. This work was carried out within the ASIA-URBS NPL-3-05 Development Project sponsored by the European Aid and coordinated by UMEDP Lalitpur and the Chester City Council, UK (Urban Management and Economic Diversification Project 2004). Since the 1988 Nepal-India border earthquake, the awareness of seismic risk has grown greatly in Nepal. Risk analysis and earthquake scenarios have been produced for Nepal generally and for the Kathmandu Valley specifically (National Society for Earthquake Technology Nepal [NSET-Nepal]) (GeoHazards International 1999). This has been followed by studies on the seismic vulnerability of important public buildings, such as hospitals (Guragain, Pandey, and Shrestha 2004) and schools (Bothara et al. 2002; Bothara, Guragain, and Dixt 2002), and by the compilation of a building inventory of the Kathmandu Valley for seismic vulnerability purposes (Ohsumi et al. 2002).

Given the very high profile of the architectural heritage present in the Kathmandu Valley, and given the region's susceptibility to highly destructive earthquakes, great concern is voiced on the effects of a destructive earthquake on monumental buildings. Studies on specific typologies (Ranjitkar 2000; Shakya 2000) are producing an increasingly lively debate on the best ways to strengthen these buildings without affecting their architectural and historical value (Yeomans and Michelmore 2000). The preservation of entire urban blocks of traditional buildings, albeit of lesser individual value, is a problem of different scale and magnitude in terms both of developing awareness and of devising effective and sustainable policies. To date, no detailed analysis of the seismic vulnerability of ordinary residential historic buildings has been conducted in the Kathmandu Valley.

Traditional urban housing in developing countries has been substantially eroded in the past twenty years, owing to supposedly better and safer housing conditions offered by new typologies, such as reinforced concrete, infill-frame apartment blocks. Vernacular historic buildings fall prey not only to socioeconomic advancement but also to the lack of specifically developed analytical models that professionals can reliably use to evaluate the actual safety of these buildings with respect to seismic hazard. This phenomenon is common to many countries worldwide, notwithstanding the evidence of time—i.e., the fact that a vernacular building type might have survived many destructive events in the past.

In earthquake prone countries, vernacular architecture has typically evolved over centuries, with recurring construction details that testify to the viability of practices hundreds of years old that directly respond to the seismic hazard of these regions. These features, which enable ordinary buildings to withstand seismic shaking, were developed and modified through centuries of direct experience and observation of damage. Specifically, in regions of medium seismicity, the following features will typically be found: corner returns and quoins, connection with party walls, regular masonry fabric (stone or brickwork), floor and wall ties, and alternate orientation of floor structures. In regions of higher seismicity, the above features will be accompanied by others, such as timber ring beams, monolithic lintels and stone frames around openings, and framing and bracing of masonry with timber post and struts. Not all of these details were consciously developed to satisfy the demands earthquakes pose to structures, but it is likely that the observation of performance during shaking of buildings with and without certain features, and the recurrence of satisfactory seismic behavior, have resulted in a sort of natural selection.

In the present paper, a sample of buildings built with sun-dried and fired brickwork in part of the historic city center of Lalitpur is analyzed in detail. Specifically, the paper discusses the construction features that qualify these buildings' seismic behavior and presents the results of a limit state statistical vulnerability analysis, which provides a measure of the efficacy of the construction features highlighted above. The procedure adopted, Failure Mechanisms Identification and Vulnerability Evaluation (FaMIVE), follows a methodology developed for vulnerability assessment of historic urban city centers in Europe (D'Ayala et al. 1997; D'Ayala 1999; D'Ayala and Speranza 2002; D'Ayala and Speranza 2003). It is used to identify collapse mechanisms corresponding to specific construction features and to quantify the collapse load factor for each mechanism, so as to determine the level of shaking that will trigger a given behavior. Each building is given a vulnerability measure, and on the basis of the statistical distribution of it within the sample, fragility curves and damage scenarios are developed. From the results obtained, repair and strengthening recommendations are given, and they form the conclusions of this paper. The study was carried out within an EU-funded rehabilitation project (Urban Management and Economic Diversification Project 2004).

In order to understand the seismicity of Nepal, it is essential to study the earthquake sequence of the Himalayan region. A search on a catalogue of significant earthquakes from 1063 to 1984 (U.S. Geological Survey 2009) compiled from Indian sources (Tandon and Srivastava 1974; Chandra 1977; Rao and Rao 1984; Srivastava and Ramachandran 1985) reveals that at least 100 earthquakes with magnitudes between 5 and 8.25 on the Richter scale occurred in the period between 1816 and 1984 (excluding the 1980 earthquake) with epicenters within the borders of the Nepalese territory, while as many as 350 would have been felt in Nepal from neighboring regions during the same period. This count does not include the more recent earthquakes of 1980 and 1988 or the Gujarat earthquake of 2001. The records prior to 1816 are much more scattered; in the period between 1255 and 1816, there are records of only seven earthquakes at intervals of approximately 150 years, all highly destructive.

From a worldwide seismic hazard study that ran from 1992 to 1999, the Global Seismic Hazard Assessment Program (GSHAP), it is expected that peak ground accelerations as high as 3.2–4.8 m/sec.^{2} (10.50 × 15.75 ft./sec.^{2}) can have 10% probability of being exceeded in the next fifty years for the whole territory of Nepal (International Lithosphere Program 1999). These values equate to an earthquake with a maximum peak ground acceleration of 0.3–0.5 g. The eastern cluster of earthquake epicenters is located within the proximity of the Kathmandu Valley, which is also the most populous area of Nepal. Good accounts of the effects of at least two earthquakes are available for this area, the great Bihar-Nepal earthquake of 1934 (Brett 1935; Rana 1985) and the 1988 Udaypur Gahri earthquake. Extensive studies have been carried out on these two earthquakes, which often represent the basis for the development of future seismic scenarios for the Kathmandu Valley.

Choice and Description of the Sample

The sample chosen for the analysis is made up of the houses clustered around the Chyasal Square in Lalitpur, one of the three royal cities of the Kathmandu Valley. The neighborhood is particularly interesting for its layout and its mixture of buildings of different periods, with very different levels of maintenance, from the fifteenth century onward. A significant number of original buildings have been replaced by five-story, concrete-frame structures. This is very worrisome, not only in terms of the loss of original fabric but also in terms of the associated seismic risk that these buildings pose, given the very poor construction quality revealed during the visual survey. This, associated with their substantially greater height and their small footprint, identifies them as a very vulnerable type.

The traditional *newari* house is usually of a rectangular-shaped plan of about 6 m (19.7 ft.) in depth with facades of various widths, but most commonly between 4 m to 8 m (13.1 to 26.2 ft.) (Korn 1976; Guragain, Pandey, and Shrestha 2004). The organization of the house is vertical, over three stories, with a spine wall running the full height, creating front and back rooms. At the upper story, the spine wall is sometimes replaced by a timber-frame system, called *dalan*, so as to create a larger continuous space. The staircase is usually a single flight to one side of the plan. The bathroom, where present, is found at ground floor level, while the kitchen is on the top floor, usually directly under the roof as a fire prevention measure. Units are arranged in long rows or arrays around squares and common courtyards. The construction of each unit is usually independent, so that the facades are not continuous over party walls, but each unit forms a separate cell. However, the brickwork of the facade and the party wall are continuous and connected around the corner, providing a good connection between facades and sidewalls. The inherent seismic resilience of this construction type is proven by the high rate of survival from historic earthquakes, such as the great Bihar-Nepal earthquake of 1934 and the more recent 1988 Udaypur Gahri earthquake (Pandey and Molnar 1988).

Because of this system of construction, the growth of the block is not necessarily homogeneous, and adjacent plots are built at different times. Hence, each house is structurally independent, although there is virtually no gap between adjacent buildings. Until recently, due to the continuity of style and building practice, both layout and size of openings and level of floors were fairly homogeneous throughout. However, due to inheritance laws and customs, very often the property is split in equal parts among the male children, and the house is divided vertically by the introduction of party walls and new sets of stairs. In these cases the orthogonal wall is not usually connected to the facade and might run through the middle of a row of openings. Most interestingly, it appears that the two portions of the house are then further altered at different times and in different ways, according to the needs and wealth of the occupants, creating differences in floor levels, with substantial consequences for the structural behavior and hence the seismic vulnerability of the original unit.

Another typology of the same period is the *math*, or Hindu priest's house, also organized around a courtyard but with a different arrangement of spaces on different sides of the courtyard in relation to the owner's occupation. Normally the *math* is fully integrated into a terrace of houses along a street and may only be recognized by its superior wood carving and more extravagant decoration (Korn 1976).

Within the sample, there is also a minority of isolated buildings built during the late nineteenth and early twentieth centuries in the neoclassical *Rana* style. These buildings have the typology of Italianate palaces; they

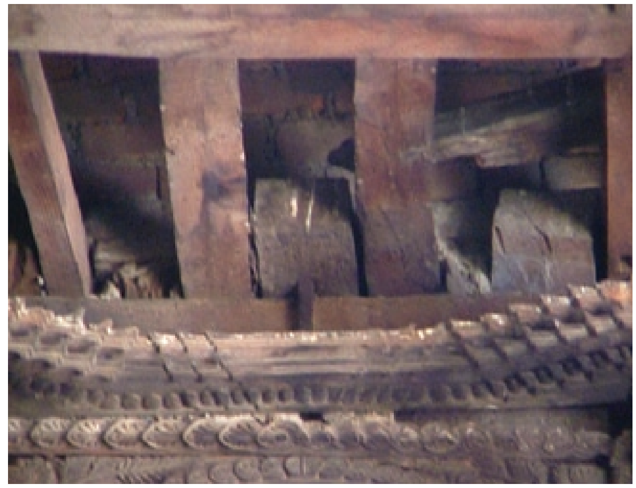


FIGURE 1 Timber pegs, or *chokus*.

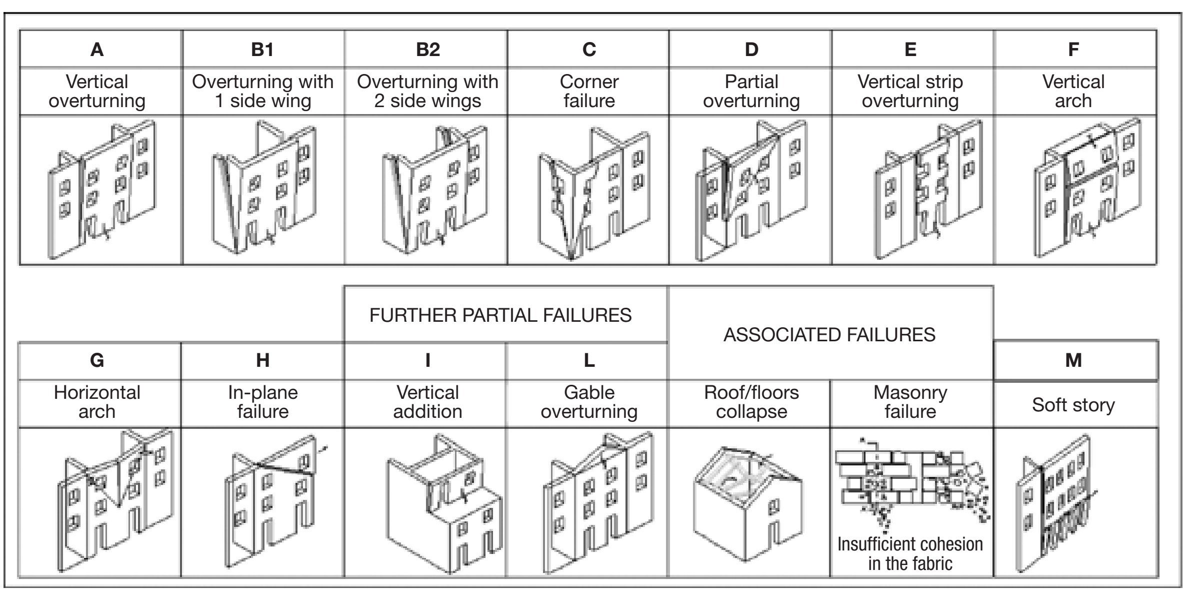
are usually of three stories with higher floor-to-ceiling heights and larger window openings. They do not have timber frames with long lintels around the openings, but they maintain the substantial roof overhangs and pegged floor construction.

Timber Pegs and Timber Bands

A rather common feature of Nepalese traditional construction is the insertion of pegs, called *chokus*, to restrain floor joists from sliding over walls. Two vertical pegs are usually inserted through a joist on each side of the wall. Typically this will occur every two or three joists (fig. 1). From an external visual inspection, the *chokus* are easily identified at roof level, due to the presence of the overhang; however, they are also present at intermediate stories on joists passing over the internal wall. For the intermediate stories, the common practice is for the joists to be anchored with pegs on the internal face of the external wall and in between the two masonry leaves or wythes. This practice is very effective in preventing relative sliding of the floor structure on the walls in the presence of lateral forces and hence creates a box effect, while at the same time, given the flexibility of the pegs and their position, it does not prevent other movements associated with temperature and other environmental effects. The presence of the pegs is also effective in limiting any substantial out-of-plane movement of the external walls due to uneven settlements.

The presence of *chokus* at roof level means that the fundamental mechanism of the facade moves from free overturning (fig. 2, types A to E) to an arch effect (type F), in which the top of the wall is prevented from moving out of plane. From the histogram in figure 3, it can be noted that the majority of buildings with pegs at roof level (63%) have collapse load factors (the value of

FIGURE 2 Facade mechanisms of failure.



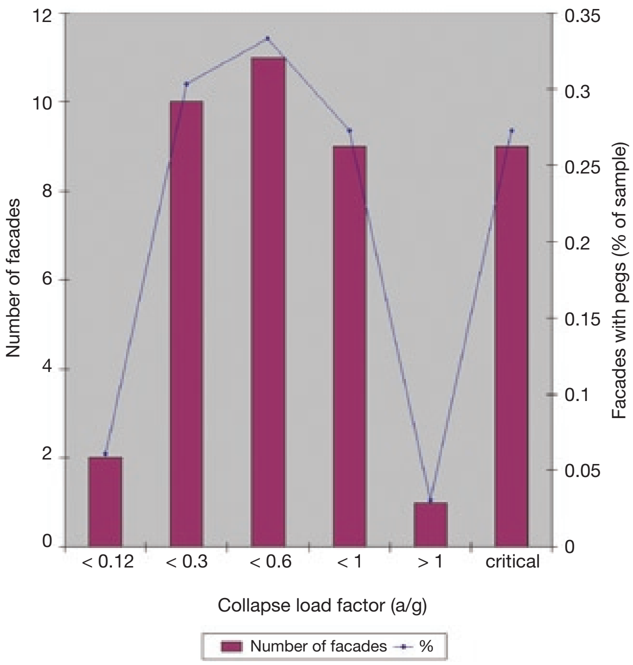


FIGURE 3 Occurrence of timber pegs (*chokus*) in the sample, and associated collapse load factor.

horizontal acceleration, *a*, as a proportion of *g*, which can be considered as the ultimate capacity of the facade) greater than 0.3 and that this mechanism is critical—i.e., it determines the vulnerability of the building, only for the 27% of the set with pegs, where the load factor is less than 0.3 (equivalent to 8.5% of the entire sample).

Most commonly, the pegs butt on a timber wall plate running along the width of the facade on which the joists sit. In most cases the timber wall plate is positioned directly above the level of the window frames, spanning the openings, and it runs the entire width of the facade (fig. 4).

While the best traditional practice uses wall plates on both masonry leaves or wythes of the facade, which are connected by transversal struts dovetailed into them, as can be seen in some of the oldest and better-built examples, nowadays the common practice is to use only one wall plate spanning over the internal masonry leaf or wythe of the wall.

From a structural point of view, the double wall plate is effective in redistributing the vertical loads more evenly across the wall; furthermore, in the original arrangement, it has the dual function of tying together the two masonry leaves or wythes of the wall and, in the presence of lateral load, preventing shear cracks in the masonry from running from one floor to the next.

A similar function is played by the timber bands included in the masonry at the mid-height of the wall within the masonry piers (fig. 4). Their presence is most effective when they run the entire length of the facade and continue around the corner, so as to form an effective ring beam that ties the orthogonal walls together. They are rather uncommon in the sample studied.

The Dalan

Among the many striking timber construction details of traditional buildings in the Kathmandu Valley, the *dalan* is certainly the most obvious and interesting in structural terms. The *dalan* is a timber frame made of twin wooden columns surmounted by a capital on which sits a double beam. The two adjacent timber frames are usually connected only at the level of the beam. The *dalan* is most commonly found at the ground floor of the main facade of buildings in which the front room is used as a shop or workshop. It is also common in upper stories as an internal structure in place of the spine wall. The columns usually have a square cross section of about 100 × 100 mm (3.9 × 3.9 in.) at the minimum and 150 × 150 mm (5.9 × 5.9 in.) at the maximum, and they are pinned to the ground 100–150 mm (3.9–5.9 in.) apart. The capital and the beam are also connected to the column by timber

FIGURE 4 Structure with timber bands at mid-height of the wall.

FIGURE 5 *Dalan* structure at the ground floor of a courtyard.

pins, and the joists of the floor above sit directly on the beam, connected to this in some cases by timber pegs. Therefore, the first-floor joists directly support the facade of the upper stories. The *dalan* usually spans the width of the building, with only small masonry piers of about 200 mm (7.8 in.) in width restraining it laterally and connecting it to the rest of the masonry structure.

In seismic terms, the *dalan* construction can be compared to a modern, concrete, soft-story structure and its associated failure mechanism, as all connections are simply pinned. The only lateral restraint, when present, is provided by the shear strength of the masonry piers at the edge of the facade. Figure 5 shows a well-preserved example of *dalan* in the internal courtyard of a *math* house, while figure 6 is an example of *dalan* walled in at a later stage to create more residential accommodation on the ground floor.

FIGURE 6 Original *dalan* structure walled in at a later time. Note that there is no external masonry pier adjacent to the external *dalan* column.

The histogram in figure 7 shows the range of collapse load factor associated with the *dalan*-type structure. None is greater than the reference design acceleration 0.32 g, provided by the Nepalese Seismic Code (Nepal 1995a; 1995c). In fact, the majority has a collapse load factor smaller than 0.15, and in 63% of facades with a *dalan*, the soft-story mechanism with lateral overturning becomes the critical element—i.e., the one yielding the highest value of vulnerability for the facade.

Opening Size and Window Frames

Window openings vary in size depending on the period of construction. Older buildings have generally smaller square windows with lintels extending well into the surrounding masonry. These are usually built with a double

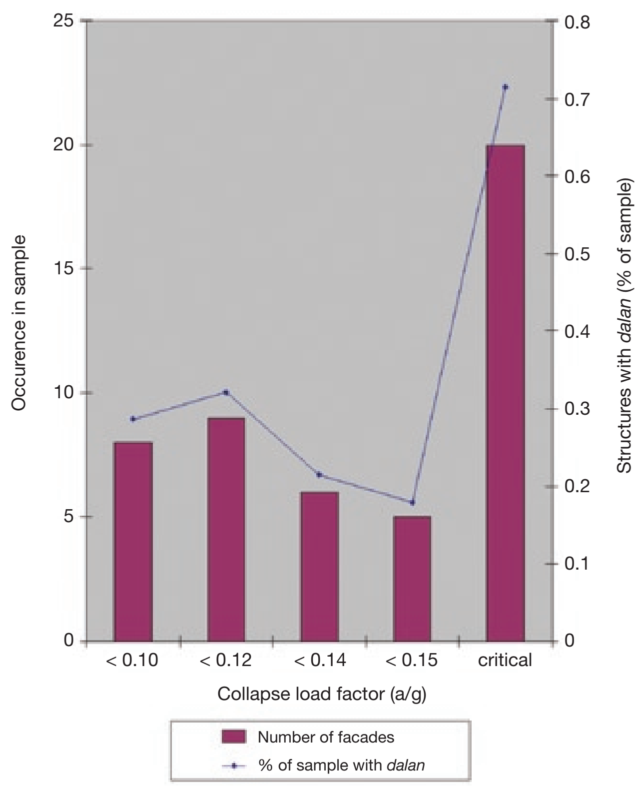


FIGURE 7 Statistical analysis of data characterizing the *dalan*.

FIGURE 8 Malla period (ca. 1200–1767) window.

FIGURE 9 *San Jhya* window.

frame, one within the external masonry leaf or wythe and a slightly larger one within the internal masonry leaf or wythe. The two frames are connected by timber elements embedded in the masonry (fig. 8). The size of the windows within a story may vary, depending on the use of the room.

A feature of older buildings is the *San Jhya* window (fig. 9), a richly decorated window that spans most of the facade at the third-story level with seating framed within it. Later buildings have more homogeneous openings; they are usually taller and narrower, of about 800 mm (31.2 in.) in width and extending almost from floor to floor (fig. 10). In this typology, spandrels above windows are very narrow. In more recent construction or alterations, the concept of the *San Jhya* has been extended to each floor, so that there is very little masonry left on the front facade of the house.

In more modern construction, window lintels are made of flat brickwork arches and, in a minority of cases, by stone frames (fig. 11). Traditionally the openings are

FIGURE 10 Full-height windows based on the *San Jhya* model.

FIGURE 11 Full-height windows with flat brickwork arches.

FIGURE 12 Full-width windows that do not leave sufficient width for the lateral pier.

placed at a fair distance from the facade's edges, leaving sufficient width for the lateral pier, constant throughout the full height of the building. This means that the pier can develop good structural behavior with substantial in-plane shear stiffness and, in turn, effective connection with lateral walls.

The increasing alteration of the openings, due to population overcrowding and internal subdivision of units, has led, as mentioned above, to a reduction in the width of lateral piers (fig. 12). The lateral capacity of the facade is hence reduced to the piers' flexural capacity, which is modest because of the poor tensile strength of the masonry.

Seismic Vulnerability Evaluation

In the present analysis, the seismic vulnerability (V) of each facade is evaluated based on the following formula (D'Ayala 1999):

$$V = \frac{d_i \, d_e}{ESC}$$

where *ESC,* the collapse load factor, is a function of the slenderness, the connection with other walls and floor structures, and the friction coefficient; d_e and d_i are two factors that are functions of the extension of the facade and floor structures involved in the collapse and the catastrophic character of the collapse, respectively.

Depending on the value of the product, four classes of seismic vulnerability are defined: low $V < 3.5$; medium, $3.5 < V < 7$; high, $7 < V < 15$; extreme, $V > 15$. These classes have proven to have good correlation with damage levels (D'Ayala 1999) for Modified Mercalli macroseismic intensity level (MMI) of VIII. This intensity level is especially significant, as it is defined as the level at which at least a quarter of masonry houses are seriously damaged. Although few collapse, many become uninhabitable.

According to the above division into four categories, a sample of 100 facades in Lalitpur was surveyed by a group of ten local architects and engineers supervised by the author over a period of two weeks in November 2002; 11.6% of the facades showed extreme vulnerability; 26.7%, high vulnerability; 50%, medium vulnerability; and 11.6%, low vulnerability (Urban Management and Economic Diversification Project 2004). This distribution also correlates well with the vulnerability classes A to C1 associated with the EMS '98 scale (Grünthal 1998; D'Ayala and Speranza 2002) and with their expected damage for macroseismic intensity (MMI) of VIII.

Although the boundaries between classes are represented by a deterministic value and hence by a line in figure 13, the transition between classes should be smooth, so that facades with values close to the boundary can be considered as belonging to either class. This is especially significant for facades with low-medium vulnerability.

In figure 14 the distribution of each mechanism in vulnerability classes is shown. The most common mechanism is the soft-story or *dalan*, mechanism type M, followed by the overturning of the facade, mechanism types D and A, and the collapse of the upper spandrel, mechanism type G (see fig. 2 for illustrations of facade mechanisms of failure). Figure 13 gives the same results plotted against the slenderness of the facade. The slenderness here is calculated as the ratio between the height of the facade and its average effective thickness. *Effective thickness* is defined as the geometric thickness reduced by a factor ranging between 0.05 and 0.15, depending on the level of maintenance of the facade and accounting for loss of mortar or brick due to decay.

The *dalan* mechanism is associated with the class of extreme vulnerability. This is because the mechanism is triggered by low levels of lateral acceleration, typically lower than 0.1 g; triggering of the mechanism leads to

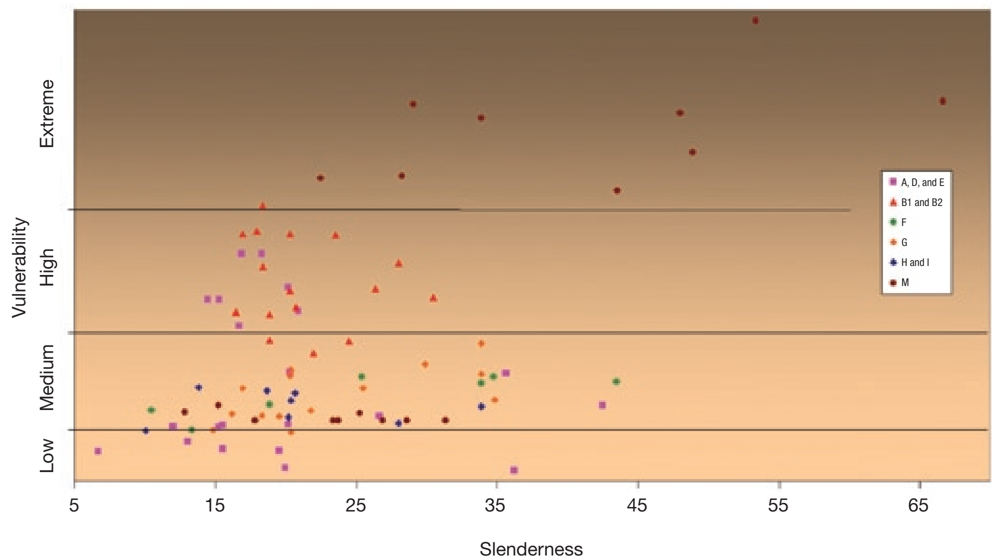


FIGURE 13 Distribution of failure mechanisms by slenderness and vulnerability classes (see fig. 2 for mechanisms of failure).

total collapse. However, the facades with highest vulnerability, those associated with high slenderness ratios, are affected by mechanism type D (overturning of the facade). These are very thin walled buildings of five stories, of which the lower two are original masonry and the upper three are concrete frame and infill masonry. These proved to be the most dangerous types of buildings in the sample.

Besides these, there is only one other case of extreme vulnerability. It is associated with a building with pegs but with a very poor level of maintenance, which fails by mechanism type F. These buildings show an average collapse load factor of 0.08 g, and they are likely to be damaged by an earthquake of MMI = VII.

Facades with high vulnerability are affected by either the *dalan* mechanism or by overturning of the facade, mechanisms A and D, when this is poorly connected to party walls. These are typically facades that do not have visible pegs at the roof level and hence do not benefit from the restraining action exerted by the horizontal structural elements. These buildings, with an average load factor of 0.11 g, are slightly more resilient than the previous class and will receive serious damage in an earthquake of intensity MMI = VIII.

The medium vulnerability class comprises facades affected by all types of mechanisms except the *dalan*. The most common type, however, is B2, overturning with party walls, occurring when there is good connection between the facade and the walls normal to it. As seen in figures 7 and 14, this mechanism has a weak correlation with slenderness, and the vulnerability range for this type is rather narrow. To this class also belong the majority of arch effect mechanisms, type F; this also provides the lower bound of vulnerability for facades with pegs. When facades have both pegs and good connections with party walls, the out-of-plane mechanism requires rather high accelerations to be triggered, and the in-plane mechanism takes place instead. This is the case of the facades failing with mechanism type H. It has been assumed that the diagonal cracks will run the

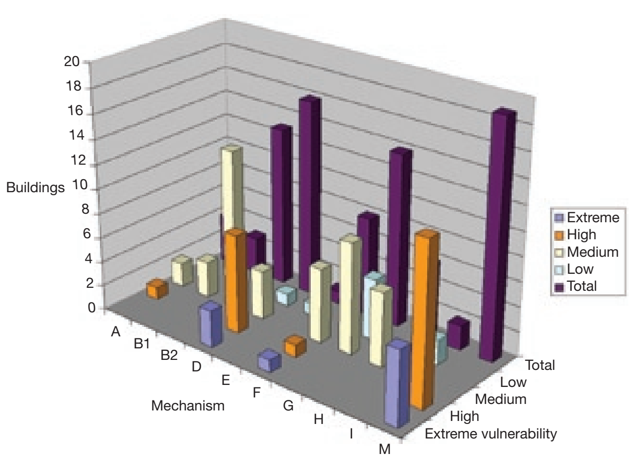


FIGURE 14 Distribution of failure mechanisms by class of vulnerability (see fig. 2 for mechanisms of failure).

whole height of the facade. In reality, where timber wall plates (wooden beams or tick planks rested upon and/or connected by pegs to the top of the wall on which the joists or rafters rest) are in place, the cracks will be confined to each story height, but this has been disregarded here in favor of safety. The average collapse load factor for these buildings is 0.30 g. They will survive an earthquake of intensity MMI = VIII with minor damage, and they will be seriously damaged by an event of intensity MMI = IX.

Also common within this class is mechanism type G—i.e., the failure of the upper spandrel of the facade between the lintels of the last row of windows and the roof structure. This mechanism is relatively common in this sample, as this strip of masonry is usually very narrow (it is made of few brick courses) and hence prone to collapse. Although the average collapse load factor is rather low, at 0.085 g, the extent of the facade involved is modest—hence the medium level of vulnerability.

Finally, to the class of low vulnerability belong otherwise well-built facades that are characterized by partial failure, either of the upper stories of the facade (types D or A), of the vertical addition (type I), or of the spandrel above the last row of windows (type G). This is particularly common in this sample.

To summarize, the vulnerability analysis shows that buildings with *dalan*-type construction are extremely vulnerable unless there is appropriate masonry restraint to the sides of the *dalan* structure. The construction details of the *dalan* need to be studied more closely, in order to identify better constraint conditions that might possibly reduce the vulnerability calculated so far. Given the present assumptions, all these facades need to be strengthened.

Facades with poor lateral connections and no visible presence of pegs are highly vulnerable, as the full height of the wall is prone to overturning and involves the floor structure in the collapse. In order to prevent this, as the reinstatement of the connection with the party walls is a difficult intervention, the recommended action would be to reintroduce *chokus* at the roof and possibly also at the floor levels. This would consistently reduce the vulnerability from high to medium. Facades with basically good construction standards—such as good maintenance of the masonry accompanied by connection to party walls and the presence of *chokus* at the roof level—all show medium-low levels of vulnerability and collapse load factors in the range of 0.20 to 0.50 g, depending on slenderness ratios and maintenance. Facades with localized defects, such as narrow upper spandrels, or connections only on one side or only at the lower levels, fail by mechanism types G and D respectively, with a collapse load factor of about 0.10 g, and they show the value of vulnerability in the upper range of the medium class. The behavior of these facades can partially improve if in the analysis, the effect of wall plates is accounted for. If these prove to be insufficient, then reinstatement of the *chokus* and connection at the sides by means of ties might provide the solution.

Finally, the analysis shows that well-built buildings will have high collapse load factors associated with global collapse mechanisms, typically in the range of 0.40–0.60 g, while the partial failure of upper spandrels or vertical later additions would occur for acceleration typically in the range of 0.20–0.30 g. These, however, would not create major damage or threat to life.

It is worth noting that all assumptions made have been made in favor of safety, while taking into account the level of reliability of the available data. In reality, some of the buildings in the sample might turn out to be more resilient than shown here.

Damage Scenarios

Damage scenarios for the houses surveyed used the same data as used in the design of new structures in the Kathmandu Valley. To properly quantify the type and extent of damage and hence best identify the strengthening strategies that would be most effective in reducing such damage, the following steps were taken with reference to the sample of houses surveyed.

First, fragility curves have been developed for the four most recurring structural typologies identified: facades with *dalan*, facades with *chokus* or with connections to lateral walls, facades with both *dalan* and *chokus*, and facades without *chokus* or connection to lateral walls. Second, it has been assumed that the extent of damage can be described and quantified by the six-level scale defined in Grünthal (1998). Third, the number of buildings in the sample in each damage state for a given level of design acceleration has been calculated.

In figure 15 the curves of cumulative distribution for each damage state are plotted against different levels of expected ground acceleration. These curves show the

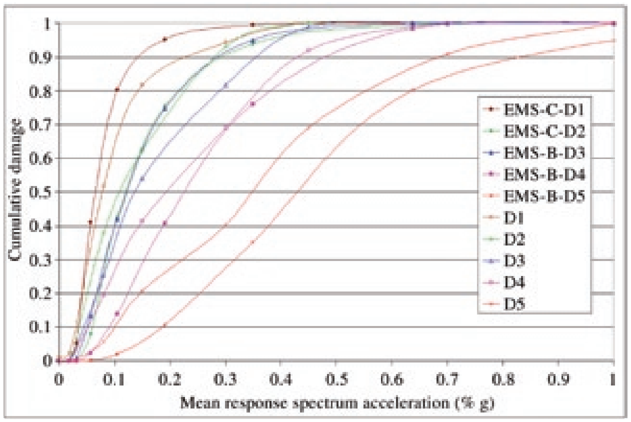


FIGURE 15 Cumulative damage distributions for values of peak ground acceleration (EMS = European macroseismic scale; D = damage level).

percentage of the building sample that would reach or overcome a given state of damage. Of particular interest to the present discussion are the values of the curves in correspondence to 0.32 g (seismic acceleration), corresponding to the level of design acceleration for new masonry structures prescribed in section 2.4 of the Nepalese code (Nepal 1995b). In correspondence to this value, figure 15 shows that more than 90% of the buildings will be damaged, and 80% will have at least slight damage, as characterized by table 1. About 62% of the buildings will suffer heavy damage, level D3; 35% will experience partial collapse, D4; and 11% will collapse, D5 (Grünthal 1998).

Most important, if the level of peak ground acceleration expected for a 50-year-return period is considered (i.e., g = 0.4, as explained earlier), it emerges from the diagram that at least 20% of the buildings will collapse, 45% will undergo partial collapse, up to 70% will be seriously damaged, and only 5% will survive unscathed.

These results highlight the urgency of adequate implementation of strengthening intervention, in order to substantially reduce the expected damage discussed above.

Conclusion

The following points emerge from the study of the seismic vulnerability of historic buildings in the Kathmandu Valley:

- From historic and seismological evidence, a destructive event can be expected with a return period of approximately 80 to 100 years in the Kathmandu Valley. According to the Global Seismic Hazard Assessment Program (GSHAP) study, an event with a return period of 50 years would be characterized by peak ground acceleration in the range of 0.40–0.48 g.
- Earthquake scenarios developed by other authors, taking as reference an event with characteristics similar to those of the 1934 earthquake, show that seismic vulnerability at an urban level has increased in the last 50 years, and that the death toll and destruction

Table 1 Definition of damage levels (after Grünthal 1998)

Damage level	Mean damage ratio	Damage type	Description of physical extent
D0	0.00	Undamaged	No visible
D1	0.05	Slight damage	Hairline cracks
D2	0.20	Moderate damage	Cracks 5–20 mm
D3	0.50	Heavy damage	Cracks > 20 mm or heavy damage to structural walls
D4	0.90	Partial collapse	Collapse of individual wall or individual roof support
D5	1.00	Collapse	More than one wall collapsed or more than half of roof

in the Kathmandu Valley would possibly be greater than those recorded for the 1934 event.

- A number of national documents prepared by the National Society for Earthquake Technology Nepal (NSET-Nepal), but not yet ratified as law, are available for the design of new masonry buildings in earthquake prone areas. These documents assume a maximum value of 0.32 g as design acceleration for masonry buildings, lower than the one suggested by the GSHAP study. These documents are also rather useful, as they indicate with diagrams and sketches correct construction and repair techniques and details. However, they often do not take into account conservation principles, such as minimal disturbance to the original fabric, in-kind repairs, and use of the same materials.
- The analysis of the architectonic and construction typologies in Kathmandu has identified three types of building agglomerations: buildings in straight arrays or rows, buildings around courtyards, and some hybrids. Two principal masonry typologies have been identified, one built with sun-dried bricks and one with fired traditional bricks (*dachi aapa*). The traditional floor plan typology is made up of closely spaced timber joists covered by floorboards with mud and, in a minority of cases, tiles. Alterations of the original floor structure are usually weakly reinforced flat concrete slabs.
- Numerous traditional details have been identified, most importantly the *dalan* structure, with crucial structural properties. Other structural features that make the traditional buildings seismically resilient have been identified as the *chokus* restraining the roof and floor structure within the masonry, the timber wall plates and timber bands, and the traditional design of the timber lintels. Among the features that constitute seismic weaknesses, it is important to point out that the jetty (the projecting part of a building) is present in 20% of the sample, and that 86% of the roofs have overhanging portions. Of this 86%, 33% are concrete slabs either flat or sloping.
- In order to quantify the seismic vulnerability of the sample, the Failure Mechanisms Identification and Vulnerability Evaluation (FaMIVE) procedure has been applied to all surveyed facades, identifying for each the crucial collapse mechanisms and its associated collapse load factor and vulnerability class. The overturning of the facade, either complete or partial, has an occurrence of 23% in the sample; followed by the *dalan* mechanism, with an occurrence of 22%; the overturning of the facade with side walls in 18.5%; the failure of the upper spandrel with 16%; followed by 9% of failure by arch effect; and a minority of in-plane failure. According to the classification of seismic vulnerability in four categories, in Lalitpur's sample, 11.6% of the facades show extreme vulnerability, 26.7% high vulnerability, 50.0% medium vulnerability, and 11.6% low vulnerability.
- Fragility curves for the sample have been developed in order to forecast levels and types of damage associated with different levels of seismic input. The results show that, in the event of a seismic input of the same level as the one considered by the Nepalese code of practice, 90% of the buildings will be damaged, and 80% will exhibit damage level of at least D2. About 62% of the buildings will suffer heavy damage, level D3; 35% will undergo partial collapse, D4; and 11% will collapse, D5. However, if the level of peak ground acceleration expected for a 50-year-return period is considered (i.e., g = 0.4), it can be seen that at least 20% of the buildings will collapse, 45% will undergo partial collapse, up to 70% will be seriously damaged, and only 5% will survive without damage.

The results summarized above show the necessity for a strategic policy of strengthening that will safeguard the historic character of the buildings of the Chyasal District, while at the same time improving their seismic performance. While the design specifics for each building should be defined upon the results of more detailed analyses, based on a thorough survey of each case, some general guidelines can be drafted based upon the results obtained:

- Buildings with *dalan*, especially in the case of those with small lateral masonry piers, need to be strengthened to prevent the soft-story mechanism. This occurs at very low values of the collapse load factor, and it is catastrophic, as it leads to complete collapse of the walls and floor structure above. The strengthening strategy needs to prevent the lateral overturning of the *dalan* columns. It was not possible during the visit to inspect the connection between the columns and the ground floor structure and foundation; this probably needs to be strengthened to prevent rotation. The connection with the timber beams supporting the masonry above would probably need to be strengthened, too. This can be achieved in both cases by introducing flitch plates within the existing timber structure. An alternative to this, if there is sufficient lateral space, is to build lateral masonry piers and connect these to the upper masonry.
- For buildings for which facade overturning will take place, it is first necessary to inspect more accurately the level of connection with party or other internal perpendicular walls. If this is lacking, it should be checked to see whether the horizontal structures provide sufficient restraint. If this is also lacking and the floor structures are of timber, then *chokus* should be introduced or reinstated, both externally at roof level and internally at lower levels. This will reduce the vulnerability level from high to medium and low.
- The analysis also proves that the level of maintenance is a crucial factor to the performance of these buildings. Hence, repointing and replacement of decayed masonry should be the first treatment for all buildings in the sample. In order to reduce the decay of the masonry, it is essential to use lime or mud mortar, avoiding the use of stabilizers or cements that can cause chemical attack on the sun-dried bricks. It is also advisable to restore and maintain in all buildings the overhang of the roof, as this shelters the masonry from direct rainfall.
- The most vulnerable buildings have proven to be those with additions of two or three stories in concrete above a slender masonry structure. Although this phenomenon is relatively limited in Chyasal, it is becoming increasingly common in many of the historical districts of the Kathmandu Valley. Short of demolishing these buildings, it is very difficult to envision effective ways of improving their seismic performance. As it is unlikely that demolition (which might also cause damage to adjacent buildings) will be pursued, more should be done to prevent the proliferation of these additions.
- The upper spandrels, between the lintels of the last row of windows and the roof structure, have proven to be vulnerable to low-level acceleration. Although this failure is localized, it can cause collapse of the roof and death in the street below; hence it should be prevented. The spandrels' behavior can be improved by consistent introduction of wall plates and timber bands at this level, and by connecting them vertically to the row of lintels below.

The application of the FaMIVE method to a sample of buildings in Lalitpur presented in this paper represents the first attempt to consistently apply the concept of seismic vulnerability assessment, as developed in literature for existing engineered buildings, to the historic architectural heritage of the Kathmandu Valley. By a thorough analysis of the construction details, buildings have been classified in typologies and vulnerability classes. For each typology, a fragility curve was created, relating to the type of collapse mechanism that develops. This type of analysis, which directly relates construction to seismic vulnerability, also allows the identification of the most suitable strengthening strategies to reduce the seismic risk of this urban agglomerate while preserving and enhancing the use of local historic characteristics.

The study has also emphasized that the greatest threat to the preservation of the historic environment and the greatest seismic risk are posed by uncontrolled urban development, as characterized by refurbishments of internal layouts and additions of stories. These alterations are carried out with materials extraneous to the building tradition—reinforced concrete, lightweight fired bricks, and corrugated steel sheets—and techniques that are borrowed from the modern construction industry without the necessary engineering knowledge and quality control.

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Seismic Retrofitting Guidelines for the Conservation of Doctrinal Chapels on the Oyón Highlands in Peru

Patricia Navarro Grau, Julio Vargas Neumann, and Maribel Beas

Abstract: *In the province of Oyón, Department of Lima, at altitudes between 2500 and 4100 m (8202–13,451 ft.) above sea level, there exist more than forty doctrinal chapels built in the seventeenth century. The Catholic Church built the chapels in order to eliminate persistent pagan idolatries throughout one hundred years of Spanish presence in Peru. The architectural expression of these chapels corresponds to a mestizo-vernacular version of the late Renaissance of the sixteenth century in the central Andes. This paper describes the characteristics of a structural typology that represents this group of chapels and develops recommendations for seismic retrofitting and intervention on historic buildings. Two retrofit designs that drew upon guidelines developed by the Getty Seismic Adobe Project (GSAP) are presented.*

Introduction

The doctrinal chapels of Oyón are located northeast of the city of Lima in the central highlands of Peru, between 2000 and 4000 m (6562–13,123 ft.) above sea level. Under the direction of Spanish missionaries, forty chapels were built by indigenous craftsmen at the end of the sixteenth and beginning of the seventeenth centuries. This area was well known in colonial times because the inhabitants faced violent actions by the Spaniards as they tried to eliminate persistent pagan idolatries. These actions were immediately followed by the construction of chapels, in order to indoctrinate the Indians through paintings, imagery, and exuberant religious icons. Most of these chapels were built over sacred Incan areas or buildings called *huacas*.

Peru is located close to the border of the Nazca and South America plates, which slide and collide during an earthquake. At the border of these plates is the Oceanic Fosse, located approximately 150 km (93 miles) from the coastline (under the Pacific Ocean). The greater sources of seismic risk are the superficial offshore earthquakes of the interplate (at a depth of 0–50 km, or 0–31 miles) and the intermediate-depth earthquakes (50–70 km, or 31–43 miles) along the coast and under the continent. There are deeper earthquakes, but they cause less damage on the surface. The seismic activity instrumentally registered during the last one hundred years reveals an almost total absence of earthquakes in Oyón.

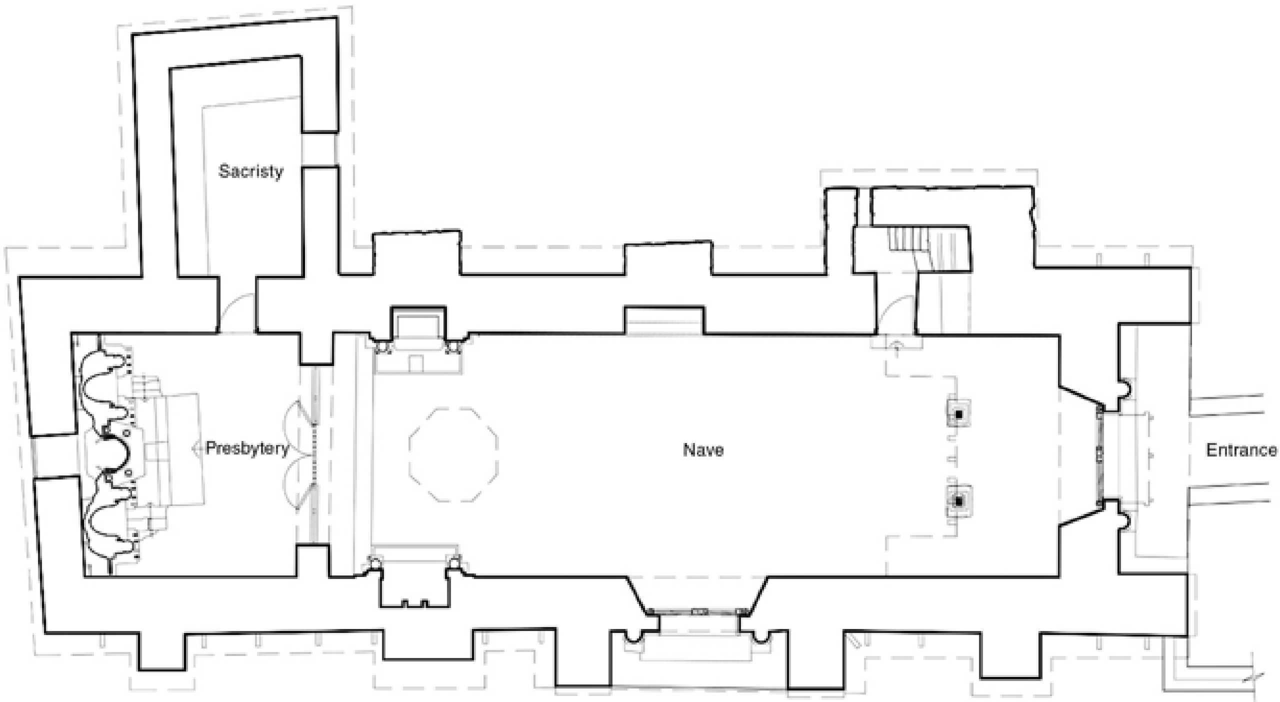
Typical Structural System

The typical configuration of a chapel consists of a single nave ending at the presbytery and an adjacent sacristy. These areas form an L-shaped floor plan, which is irregular and asymmetrical (fig. 1). The walls are generally constructed of adobe or mixed masonry. The roof is a lightweight wooden truss structure.

Existing Materials

Soil Foundation

The soil behavior and the soil-structure interaction are satisfactory. We have not found evidence of differential displacement or wall cracks related to the soil stability.



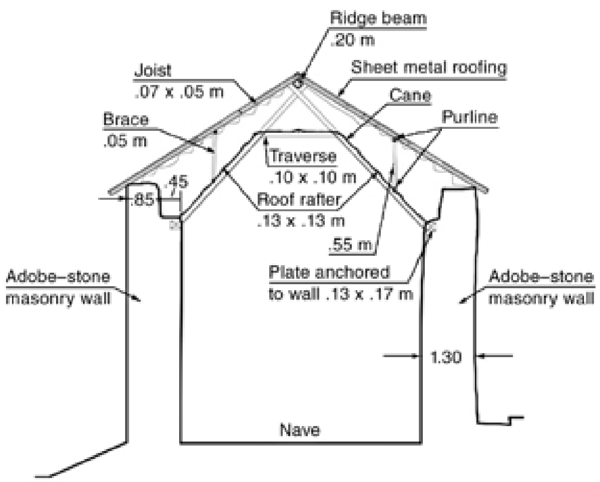


FIGURE 1 Plan and cross section of the Rapaz Missionary Chapel.

Foundation

The chapel's foundation is made of stone masonry with earthen mortar that reaches a depth of about 1.50 m (about 5 ft.). The condition of the material is satisfactory. However, this foundation is permeable and allows moisture to permeate the walls and stucco.

Adobe Masonry

The soil masonry has been proven in the laboratory to be of poor quality and of low resistance to large seismic forces. Some areas are of mixed masonry: layers of adobe alternating with stone layers.

Buttresses

All of the chapels of Oyón have buttresses that are either individual elements or buttresses that run along the longitudinal walls. In any case, these elements are properly tied into to the walls, but they still show large separation cracks. The buttresses are made of stone or adobe masonry.

Wood and Mats

The interior wooden roof trusses are the pair and knot type, spaced 80 cm (31.2 in.) from each other and laid over a bond beam on top of the walls (fig. 2). Secondary wooden rafters resting over the upper wall beams support exterior metallic corrugated sheets. The condition of these wooden elements is structurally acceptable because of the dryness and the altitude of the Andes. Areas exposed directly to the weather and moist walls have mold and show deterioration.

Secondary Nonstructural Elements

The nonstructural or secondary elements of the chapels are very important because they represent the artistic

FIGURE 2 Interior of the Rapaz Missionary Chapel showing roof construction. Photo: Daniel Giannoni.

value and cultural heritage that make these chapels outstanding examples of colonial art history.

Triumphal, or Main, Arch

This element, built in wattle and daub, regionally known as *quincha,* separates the presbytery from the long nave. It is plastered with lime and is polychromed on all surfaces.

Chorus

This is a wooden structure built at the end of the nave, supported on wooden columns and corbels, with polychrome and valuable carved elements.

Retablos

Major and lateral retablos of molded gypsum on a wooden structure are tied to the walls. The artistic mestizo-vernacular expression is unique because of the elements and decoration, which includes telamons, angels, and cherubs.

Signs of Structural Damage

Wall Cracks

Moderate cracks are found along the chapel walls. This cracking is old but cumulative. Cracks are due to seismic activity and weather in the highlands, which, at an altitude of 4000 m (13,123 ft.) above sea level, are very cold and dry, with a heavy rainy season. Cracks are located at wall intersections, wall joints, buttresses, and the corners of openings (door and windows). If new seismic activity should occur, the cracks will increase in number and size, forming wall segments separated by cracks. These segments will move independently, shifting until partial or total collapse of walls and roofs occurs.

Wall Moisture

There are three sources of wall moisture in the chapels. The first one is rainwater that falls directly onto the roof. When the roof leaks, water reaches the walls. This moisture is concentrated in the upper sections of the walls and can be detected by the deterioration of mural painting and stucco in these areas (fig. 3). The second source is the effect of rainwater and wind falling laterally on the exterior walls, degrading the plaster and leaving the adobe support exposed to weather conditions. The third is rainwater moisture that rises through capillarity, causing significant damage on lower sections of the wall, especially inside the chapels where decorated finishes are found.

Community Interventions

This damage is inadvertently caused by the good intentions and goodwill of the community people, who,

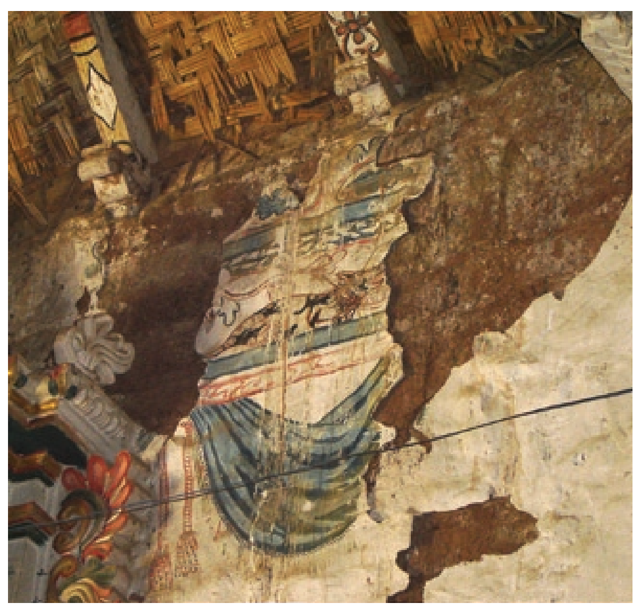


FIGURE 3 Moisture damage at the top of the walls, Rapaz Missionary Chapel.

FIGURE 4 Incompatible materials used at Huacho sin Pescado Missionary Chapel.

through ignorance of adobe techniques and the incompatibility of materials, spend money trying to protect their chapels by using cement plaster on exterior and interior walls to prevent humidity penetration. New concrete elements (columns, towers, interior plaster) tied to the adobe walls have also been built (fig. 4).

Structural Dynamic Conditions

Dynamic conditions refer to the structural performance of the chapels under seismic activity.

Floor Plan Configuration

The L-shaped floor plans of the chapels are asymmetrical and irregular. In the event of seismic activity, the walls tend to vibrate independently, and the stresses are concentrated at the wall joints because of the lack of flexure and shear deformation compatibility. These vertical lines at the wall intersections or at the wall and buttress junctions are the cause of major cracks evident in the chapels.

Slenderness of Walls

The slenderness, or wall height-to-thickness ratio, of the lateral walls is different from that of the frontal and rear walls. The slenderness of the main chapel walls and the sacristy are similar and are approximately 5 for the lateral walls and 7.5 for the walls with gables. This slenderness ratio is satisfactory for walls with appropriate transverse wall connections, but not for walls that are loose or behave independently as a result of cracking.

Materials of Low Quality and Resistance

The materials used in earthen construction are heavy, weak, and fragile. Because of their weight, earth materials undergo greater inertial forces, originated by the acceleration of earthquakes. Since the material is weak, the cracks appear at low stress levels during minor earthquakes. Since the material is brittle, the wall cracks abruptly, with no warning, leaving no time for inhabitants to escape before structural collapse.

Stability Versus Strength Criteria

The traditional systems for repairing historic monuments have emphasized increasing strength and delaying the time it takes for the wall to crack. Nevertheless, severe earthquakes will always cause cracks in adobe walls, separation between them, and eventual collapse. To confront this situation, the GSAP project proposed the design of retrofit measures that control the displacement of walls damaged by earthquakes and prevent collapse (fig. 5). These new criteria point toward the

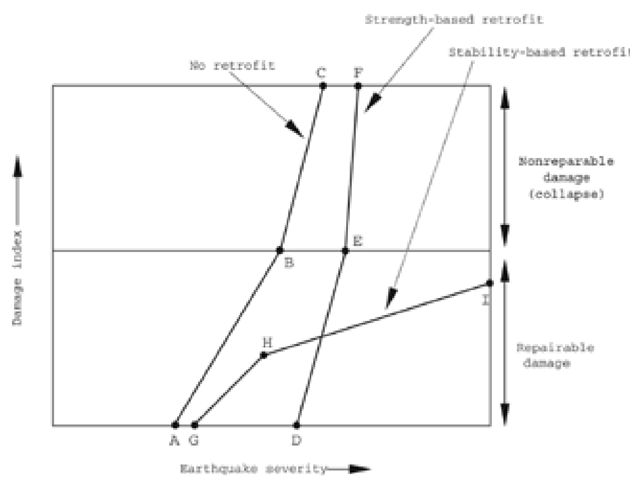


FIGURE 5 Damage-progression index versus earthquake severity for unretrofitted structures (ABC) and for strength-based (DEF) and stability-based (GHI) retrofitted structures. In the stability-based retrofit, cracks and displacement are controlled; this prevents collapse and allows for future repair. (From Tolles, Kimbro, and Ginell 2002, 45, fig. 4.1.)

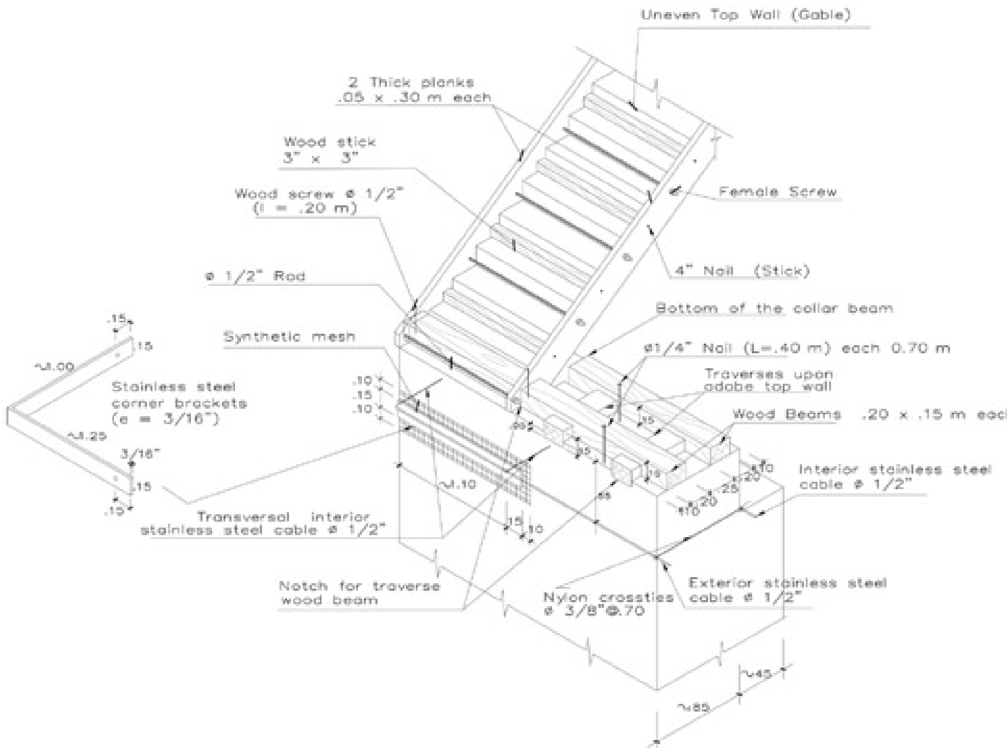


FIGURE 6 Design of a minimal intervention reinforcement for the Church of Rapaz (detail of upper corner).

preservation of human life and cultural heritage. The way to achieve this is to apply a series of redundant reinforcements to the structure, providing alternate paths for the distribution of earthquake-induced forces. These reinforcements are meant to confine the walls, preventing greater displacements of damaged walls. This design ensures the continuity and connection of cracked elements (Tolles, Kimbro, and Ginell 2002).

Structural Intervention Proposal

The doctrinal chapels of Oyón described herein show damage levels that within GSAP are classified as moderate (Tolles, Kimbro, and Ginell 2002, 54), meaning that cracks are shown at all expected locations of the building, but there are no major or permanent deformations and no unstable segments. According to this statement, the goals of interventions are:

- Minimize the impact on the historic integrity and actual appearance of the building.
- Allow for possible future removal of the intervention without permanent effect.
- Locate work in areas of less patrimonial or legacy value (respect for the mural paintings).
- Diminish damage due to low or moderate earthquakes; avoid or decrease structural damage and avoid total collapse in the event of large earthquakes.

On behalf of these goals, the following structural interventions have been decided upon (fig. 6).

Upper Wooden Bond Beam

There is a perimeter bond made of wooden elements on the main sacristy. On the side walls of the chapel, reinforcement consists of two parallel longitudinal pieces well connected by transverse wooden elements. Vertical constraint between the bond beam and the walls is achieved through 6 mm (0.23 in.) diameter and 0.60 m (23.6 in.) long forged iron nails. The main purpose of the beam is to achieve anchoring and continuity between the masonry walls and the wooden roof system, which rests on the side walls and the pair and knot trusses. Simultaneously, the pair and knot trusses rest on the longitudinal beam embedded in the wall, which transfers vertical and horizontal loads. It is expected that this continuity delivers the upper lateral restriction that the long side walls require during a seismic event that tends to overturn the walls. During the inelastic

FIGURE 7 Typical vertical seismic cracks at the wall intersections, Church of Rapaz.

phase when walls have cracked, the bond beam system also provides lateral restriction, keeping the wall segments formed by the major cracks together.

Stainless Steel Upper Cables

The installation of perimeter horizontal cables on the interior and exterior surfaces of the walls produces a structural global confinement that is very effective in controlling crack development during earthquakes. The cables should be of stainless rather than galvanized steel, in order to prevent alkaline reactions when they are in contact with lime products, commonly used in stuccos and paintings. Cables will be concealed between the two roof structures previously described.

Crack Repair at the Corners of the Walls

Because of the importance of the mural paintings along the interior and exterior walls (fig. 7), it is extremely difficult to repair these cracks without intrusion. The traditional solution for large crack repair was to demolish parts of the wall and rebuild them, producing an effective structural integration and recovering monolithic behavior. There are alternative repair methods yet to be developed, and studied in seismic tests in order to verify their effectiveness, such as sealing by injection of an adhesive grout. Use of grout injections that produce rigid material, such as cement mortars, is not recommended because they create stiffness discontinuities in the masonry. These discontinuities would concentrate stresses and cause new cracks during an earthquake. The current state of the art of the injection technique does not guarantee the real structural integration of the walls that is needed to recover a monolithic behavior.

Series of Nonstructural Emergency Works

We have designed very simple tasks or work that can be done in order to prevent damage (fig. 8) and to help the chapels to survive even before structural work is performed:

- Avoid rainwater penetration by replacing the deteriorated metal roof sheets and enlarging or oversizing the washers at the points of attachment.
- Avoid rainwater penetration by installing new glass in the chapel skylight.
- Construction of a perimeter stone walkway that slopes away from the monument to prevent groundwater accumulation along the foundation walls (fig. 8).
- Conservation and consolidation of the retablos, wooden elements, main arches, and pulpits.

FIGURE 8 A perimeter stone walkway diverts water from the foundation walls.

Fieldwork

During 2005, Patrimonio Perú received a grant from the World Monuments Fund to develop the Identification and Emergency Works of Nine Doctrinal Chapels Project. This project gave the authors the opportunity to execute some of the proposals described above and, most important, to instruct the local people about the benefits and the techniques of earthen construction, which they have already forgotten.

San Pedro de Navan

The San Pedro de Navan Chapel was one of the most damaged and unstable chapels in the area. We designed several restoration details (fig. 9), relying on engineer Julio Vargas Neumann's expertise. Additionally, with the help of the community's people, we were able to work on the following tasks relating to crack repair:

- Cracks in the sacristy and baptistery walls were repaired in the traditional way, since there was a lack of evidence of mural painting on the interior and exterior surfaces (fig. 10).
- We performed field tests to check strength and microcracking in the adobe, in an effort to optimize the seismic strength of the adobe masonry (Vargas N. et al. 1984; 1986, 257).

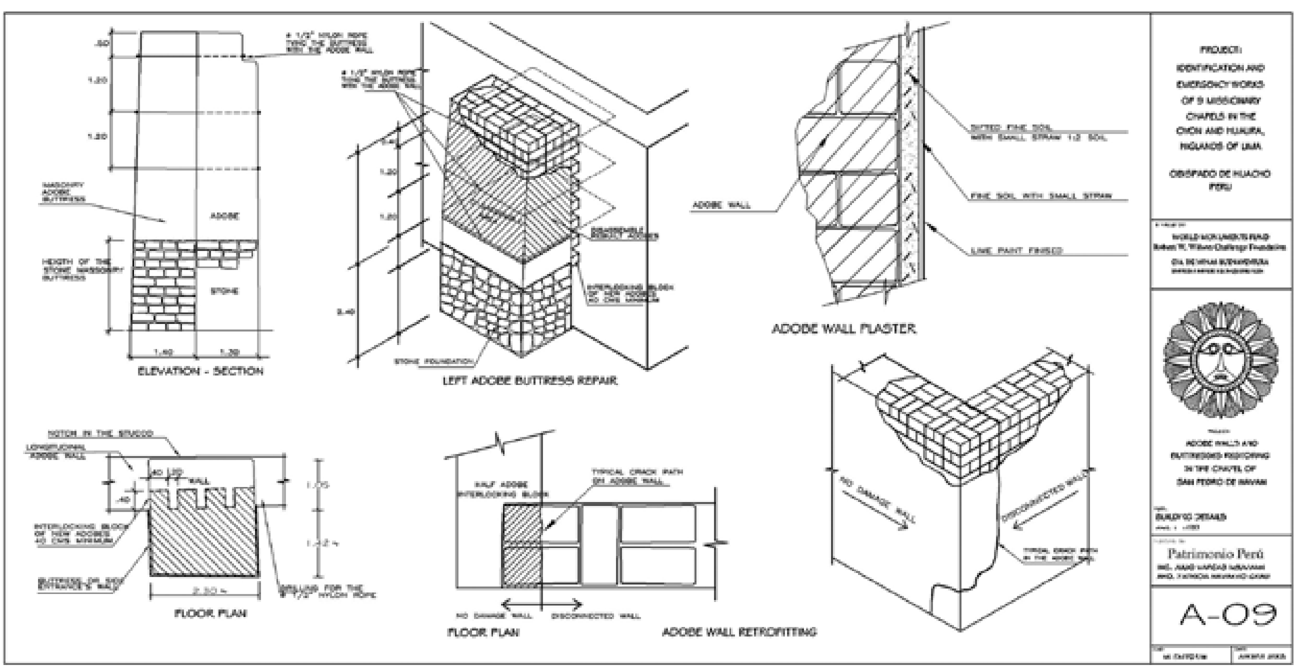


FIGURE 9 Retrofit design details, San Pedro de Navan.

FIGURE 10 Damaged corner in the process of being demolished and rebuilt, San Pedro de Navan.

FIGURE 11 Bell tower in the process of being repaired, San Bartolomé de Curay.

San Bartolomé de Curay

The community of Curay asked Patrimonio Perú to restore the bell tower with funding by a locally operated mining company (fig. 11). We designed our working schedule in order to carry out the emergency works and the bell tower restoration at the same time. We had the hand labor of people in the community, and therefore we were able to instruct them on the earthen construction techniques and the field tests. Restoration of the bell tower included:

- removal of cement mortars
- filling of eroded areas with adobes and mud plaster
- application of mud and lime plasters
- application of color according to evidence found on the intrados of the high windows

Conclusion

Because of similarities between Peruvian monumental earthen buildings and the California earthen building prototypes used in developing the GSAP guidelines, the guidelines can be applied to many buildings in Peru and throughout the Latin American region. The Spanish- and Moorish-influenced design of these monumental structures has deep roots that reach back to the Spanish colonization of the Americas. We have drawn upon the GSAP principles in designing the work described in this paper. These are the first steps toward the application of GSAP techniques to buildings in Latin America.

Acknowledgments

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Doctrinal Chapels of Oyón” (2004–5); and Compañia de Minas Buenaventura y Empresa Minera Los Quenuales, for the matching grant “Identification and Emergency Work of Nine Doctrinal Chapels of Oyón.”

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PART FOUR

Getty Seismic Adobe Project Implementation

Application of Stability-Based Retrofit Measures on Some Historic and Older Adobe Buildings in California

Frederick A. Webster

Abstract: *Recent earthquakes in California, including the San Simeon earthquake of 2003, have resulted in losses and serious damage to California's earliest and most culturally significant buildings, its historic and older adobes. As destructive as these earthquakes were, they have provided opportunities for engineers concerned with historic preservation to study the types of damage that occur to soft (unburned earth) masonry buildings as a result of significant ground shaking.*

In addition to a damage survey of historic adobes following the Northridge earthquake of 1994, testing of adobe structural models on the shake tables at the University of California, Berkeley (UC Berkeley), and Stanford University were carried out in the 1980s by the National Science Foundation and in the 1990s through the Getty Seismic Adobe Project (GSAP), to duplicate many of the types of damage observed in the field and to determine the efficacy of various stability-based methods of retrofitting unreinforced adobe buildings. These stability-based methods limit relative displacement between elements of the structure and use gravity as a restoring force. Stability-based retrofitting is seen to be less invasive to the historic fabric than is strength-based retrofitting, and it is sensitive to both life-safety performance and the requirements of historic preservation.

As California state law and local building code ordinances have been enacted in recent years to address the problem of strengthening of unreinforced masonry (URM) buildings, the application of stability-based retrofit measures to historic and older adobes has been gaining acceptance by both historic preservationists and building officials. This paper briefly discusses the development of stability-based retrofit measures, as developed by GSAP, and presents four examples of recent rehabilitated historic and older adobes in California with seismic retrofits based on these concepts.

Introduction

California's historic and older adobe buildings pay a heavy toll during large earthquakes. Events, such as the Loma Prieta (1989) and Northridge (1994) earthquakes, as well as the more recent San Simeon (2003) earthquake, were no exceptions. In fact, the Northridge earthquake resulted in the greatest loss to California historic and older adobes since the 1925 Santa Barbara earthquake.

Starting in the 1970s, interest in the preservation and rehabilitation of California's historic and older adobes yielded various attempts to use structural engineering concepts to design seismic retrofit measures appropriate for historic adobes. Prior to the development of codified regulations in the form of the California Historical Building Code (CHBC) (California Building Standards Commission 1998), first printed in 1979 and made mandatory in 1985, guidance for seismic retrofitting was frequently sought from the Uniform Building Code (UBC) (International Conference of Building Officials 1979). Since adobe is not recognized in the UBC as having the potential for seismic load resistance, basing retrofit design on the UBC resulted in rather heavy-handed interventions, such as independent steel or reinforced concrete structures designed to carry roof, ceiling, and floor loads. These independent structures, in the form of added shear walls or structural frames, were overly disruptive to the historic fabric and removed the stabilizing gravity loads from the tops of the historic adobe walls. Introduction of the CHBC had a dramatic effect on the philosophy of seismic rehabilitation of historic adobes. It is a code that sets safety standards while recognizing the unique qualities and importance of historic structures, and it explicitly recognizes the inherent strength of extant adobe walls that have withstood the test of time. The CHBC allows:

1. engineering judgment in the evaluation of strength and performance based on historical evidence;
2. use of maximum height-to-thickness ratios for one- and two-story structures, in lieu of a more complete out-of-plane wall analysis;
3. a maximum shear stress of 4 psi (0.28 kg/cm^2).

However, the early versions of the CHBC (1979–90) also required a reinforced concrete bond beam at the top, interconnection of all walls, and a minimum depth of 6 in. (15 cm) and width of 8 in. (20 cm). This limited choice and definition of a bond beam, as well as limits on height-to-thickness ratios, spurred further research, testing, and field surveys in the mid 1980s and throughout the 1990s. Sponsored first by the National Science Foundation (1980s) and later by the Getty Conservation Institute (1990s), much of this research involved review of previous testing efforts in Mexico (Meli, Hernandez, and Padilla 1980) and Peru (Vargas N. et al. 1984), as well as review of previous efforts at developing seismic retrofit measures for historic adobes in California (Thiel et al. 1991).

Shake table testing of adobe model structures has been carried out at both UC Berkeley's Richmond Field Station (Scawthorn and Becker 1986) and Stanford University's John Blume Center in the 1980s (Tolles and Krawinkler 1990), with additional shake table testing during the 1990s at Stanford University and at the Institute of Earthquake Engineering and Seismology in Skopje, Republic of Macedonia (Tolles et al. 2000).

Field studies of the condition and performance of historic and older adobes have been an ongoing activity since 1987, subsequent to the Whittier Narrows earthquake, continuing in 1989 following the Loma Prieta earthquake, and through to the present. A significant Getty Conservation Institute reconnaissance survey effort was carried out in 1994 following the Northridge earthquake (Tolles et al. 1996). Recently, damage to historic adobe structures due to the San Simeon earthquake of 2003 was investigated by the author.

Although adobe structures are often vulnerable to earthquake shaking, it has been observed that some adobes have performed well during past earthquakes and that specific types of damage can be expected to occur during earthquakes. Shake table testing has shown that with the introduction of simple stability-based retrofit measures, these structures can perform well during large earthquakes.

Observed Seismic Performance of Adobe

Estimates of Modified Mercalli intensity (MMI) and peak ground acceleration (PGA) at each of twenty historic and nine older adobe sites included in the Northridge earthquake survey (Tolles et al. 1996) were determined based on volume 1 of the Earthquake Engineering Research Institute (EERI) reconnaissance report (Hall 1995), California Strong Motion Instrumentation Program (CSMIP) station data (Shakal et al. 1994), and the preliminary report of the Earthquake Engineering Research Center (EERC) at UC Berkeley (Stewart 1994).

To correlate damage with intensity, damage state definitions were adopted from EERI and modified specifically for historic and older adobes. Damage state definitions were developed by EERI for the purpose of comparing relative damage levels in unreinforced brick masonry buildings. Table 1 lists damage states A through E along with their descriptions. The table also includes commentary on these damage states relative to the specific behavior of historic and older adobe buildings. Overall seismic performance of each adobe was rated during the survey.

Figure 1 is a plot of damage versus peak ground acceleration for unreinforced, well-maintained historic and older adobes (sixteen out of the twenty-nine surveyed). These buildings had insignificant preexisting conditions; thus, adobes were excluded that had unrepaired or poorly repaired preexisting crack damage, severe water intrusion damage, or previous retrofits or upgrades. Figure 1 also includes a linear least-squares relationship ("best estimate") of damage as a function of PGA, which serves as a baseline for judging the performance of adobes that either suffered from or were enhanced by preexisting conditions. Even though con-

Table 1 Earthquake Engineering Research Institute (EERI) standardized damage states

Damage state	EERI description	Commentary on damage to historic and older adobes
A (0)[1] None	No damage, but contents could be shifted. Only incidental hazard.	No damage or evidence of new cracking.
B (1) Slight	Minor damage to nonstructural elements. Building may be temporarily closed but could probably be reopened after minor cleanup in less than 1 week. Only incidental hazard.	Preexisting cracks have opened slightly. New hairline cracks may have begun to develop at the corners of doors and windows or at the intersection of perpendicular walls.
C (2) Moderate	Primarily nonstructural damage; there also could be minor but nonthreatening structural damage. Building probably closed 2 to 12 weeks.[2]	Cracking damage throughout the building. Cracks at the expected locations, and slippage between framing and walls. Offsets at cracks are small. None of the wall sections are unstable.
D (3) Extensive	Extensive structural and nonstructural damage. Long-term closure could be expected due either to amount of repair work or uncertainty on feasibility of repair. Localized, life-threatening situations would be common.	Extensive crack damage throughout the building. Crack offsets are large in many areas. Cracked wall sections are unstable; vertical support for the floor and roof framing is hazardous.
E (4) Complete	Complete collapse or damage that is not economically repairable. Life-threatening situations in every building of this category.	Very extensive damage. Collapse or partial collapse of much of the structure. Repair of the building requires reconstruction of many of the walls.

[1] An arbitrary numerical is included for the purpose of plotting damage state data versus ground shaking intensity.

[2] Times are difficult to assign because they are dependent on many factors, including building size.

siderable scatter is evident, some trends are reasonably clear. It appears that PGA in the range of 0.1–0.2 g is needed to initiate damage in the well-maintained adobe buildings. At this level of shaking, cracks will begin to form at door and window openings and at the intersections of perpendicular walls. At a PGA of about 0.4 g, the damage is moderate to extensive and includes more general crack damage throughout the structure.

Figure 2 is a plot of damage level versus PGA for those adobes with the preexisting conditions (thirteen

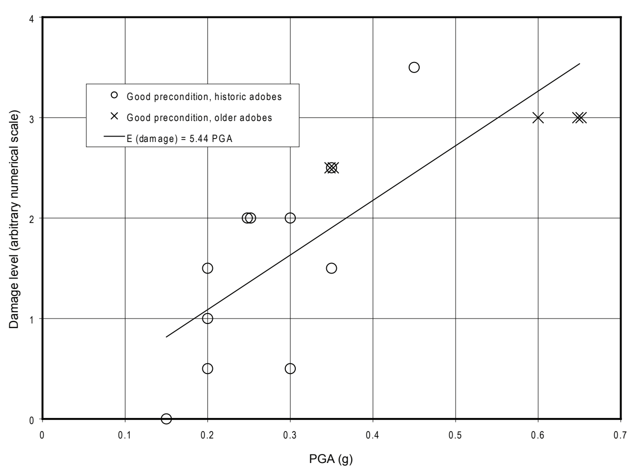

FIGURE 1 Northridge earthquake damage versus peak ground acceleration (PGA) for historic and older unreinforced and well-maintained adobes.

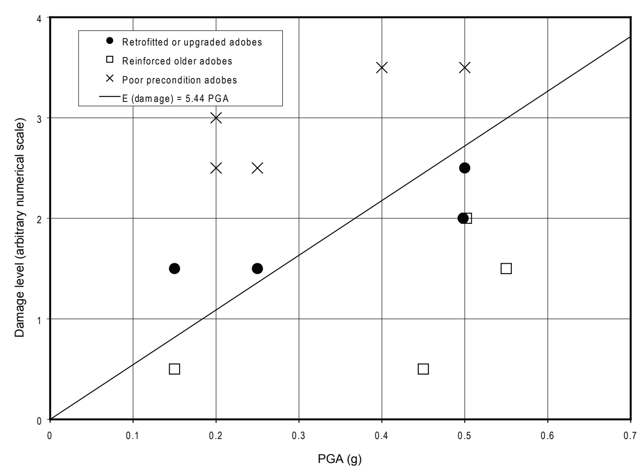

FIGURE 2 Northridge earthquake damage versus peak ground acceleration (PGA) for adobes other than unreinforced and well maintained.

out of twenty-nine surveyed). Relative to the "best estimate" for unreinforced, well-maintained adobes, preexisting conditions have definite effects on the resulting damage states. Obviously, adobes in a poor precondition state fared worse than those that were well maintained. Even at moderately intense ground shaking (0.1–0.2 g), poorly maintained adobes suffered substantial damage. Another trend observed is that reinforced older adobe buildings show greater resistance to damage than the unreinforced, well-maintained adobes at all levels of ground shaking.

Not so obvious, because of the sparse data, is the effect that seismic retrofits or upgrades have on the performance of historic and older adobes. A preliminary conclusion from figure 2 might be that the value of seismic retrofitting or upgrading is not realized until relatively high levels of ground shaking (i.e., above 0.3 g). At lower levels of ground shaking, the retrofit measures do not appear to affect performance. At these lower levels, the retrofitted buildings behave much the same as unreinforced, well-maintained adobes. Similar performance has been observed during shake table tests (Tolles et al. 1993; 2000).

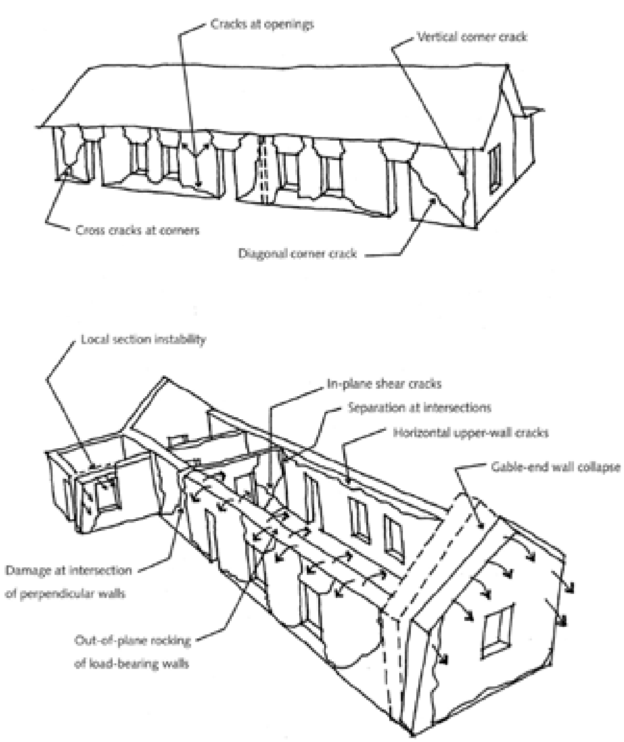

FIGURE 3 Types of damage observed in historic and older adobe buildings. (Reproduced from Tolles et al. 1996, 20.)

Damage Typologies

Designation of standardized damage states is useful in seismic risk studies or for insurance purposes. However, designing effective stability-based retrofits requires knowledge of specific types of damage. Based on field reconnaissance surveys, the types of damage observed that influence the overall seismic performance of historic and older adobe buildings are shown in figure 3.

Out-of-Plane Flexural Damage

Out-of-plane damage is initiated as vertical cracks that form at the intersection of perpendicular walls. These cracks extend downward or diagonally to the base and run horizontally along the base between transverse walls. During an earthquake, walls rock out of plane, rotating about the horizontal crack at the base. As a consequence of out-of-plane wall motion, longitudinal walls pull away from the transverse walls. In many cases there is no physical connection at the intersection of longitudinal and transverse walls, because the walls were constructed by simple abutment.

Gable-wall collapse is a special case of out-of-plane flexural damage. Gable walls are taller than longitudinal walls and usually are not well supported laterally. Unless anchored to the roof diaphragm, they can slip out from underneath roof framing.

Mid-height horizontal cracking is another special case of out-of-plane flexural damage, and it affects long, tall, and slender walls. Crack damage from this type of out-of-plane movement may not be serious in and of itself, but it signifies the potential for much greater and more serious damage—i.e., buckling of the wall and collapse of the roof.

Slippage of the top plate and/or displacement of the top courses of adobe blocks are other results of the out-of-plane movement of longitudinal walls. Very limited friction is generated by the dead weight of the roof bearing on the wall, and because of the friable nature of the top of the walls, slippage may occur.

Finally, vertical cracks on two perpendicular wall faces at a building corner caused by rocking of one or both walls results in a freestanding wall column at this location that is quite vulnerable to overturning and collapse.

In-Plane Diagonal and X-Cracking

Diagonal and x-cracks result from shear forces in the plane of the wall. These cracks are generally not serious unless the relative displacement across the cracks is large. These cracks represent a lessening of in-plane lateral stiffness, but unless a segment of wall on one side of the crack is in danger of losing purchase on the adjacent segment, such as at or near a corner, the gravity load path remains intact. Diagonal cracks also occur at the stress concentrations at the corners of doorways and windows and result from PGA levels as low as 0.1–0.2 g.

Moisture Damage

Although not the result of earthquake ground shaking, moisture in adobe walls does affect the seismic performance of the walls. This includes excessive spalling of plaster and adobe as the wall rocks out of plane, instability caused by basal erosion that removes material at the base of the wall, and/or reduced wall strength from repeated wet-dry cycles or rising damp. If the base of the wall is wet during ground shaking, a through-wall slip plane may develop along which the upper portion of the wall can slip, collapse, and overturn.

Stability-Based Retrofits

Field observations of damage to historic and older adobes and shake table testing of various stability-based retrofit measures clearly suggest that these structures can perform well during large earthquakes. The principal goals of a stability-based retrofit system (see table 2) are to:

1. provide structural continuity by interconnecting all walls with a bond beam or continuity hardware at the top of the walls;
2. prevent out-of-plane overturning of walls with horizontal straps (including bond beam or continuity hardware) and/or vertical straps or center core rods interconnected with the bond beam or top-of-wall continuity hardware; full or partial diaphragms with top-of-wall anchorage are also included;
3. contain the wall material by limiting the relative displacement across cracks or potential cracks of adjacent wall elements. Relative displacement of adjacent elements may be limited either by local ties between elements or by applied surface mesh with through-wall ties.

Table 2 Stability-based retrofitting goals and measures for some recently retrofitted historic and older adobes

Stability-based system goal	Stability-based measure
Structural continuity at floor and roof/ceiling	Existing bond beam interconnecting all walls
	Top-of-wall continuity hardware (straps, cables), through-wall tied
	Miscellaneous continuity hardware (connecting discontinuous existing bond beam elements)
Out-of-plane overturning stability	Top-of-wall pins (steel or fiberglass)
	Vertical center core rods (steel or fiberglass)
	Diaphragm (partial or full)
	Top-of-wall anchorage
	Through-wall floor anchorage
Containment of wall material	Horizontal and/or vertical straps or cables, through-wall tied
	Horizontal and/or vertical center core rods
	Surface mesh, through-wall tied
	Top-of-wall continuity hardware, through-wall tied, in conjunction with top-of-wall pins

Stability-based measures do not stiffen the structure in any significant way. In fact, they do not come into play until old cracks reopen and the structure has developed some new cracks and has moved enough to engage the stabilizing elements. These measures, however, provide reduction in the response of the building in at least two ways: (1) by increasing the structural damping due to friction hysteresis across the cracks; and (2) by lowering the response frequency due to wall rocking.

A short list of some historic and older adobes in California for which stability-based measures have been designed and utilized is presented in table 3, which includes the stability-based elements used to achieve the three stability-based system goals. The following is a discussion of three of these examples.

Table 3 Specific stability-based system measures to promote structural continuity, restrain overturning, and contain wall material for some California historic and older adobes (N = new; E = existing)

Structure name/location	Structural continuity	Overturning restraint	Wall material containment
Shafter Courthouse (1992) Shafter, CA	(N) misc. continuity hardware to connect (E) discontinuous bond beam elements	(N) top-of-wall fiberglass vertical pins; (E) bond beam and diaphragm	(E) and (N) wire stucco mesh; (N) through-wall ties with oversize washers
Lydecker Adobe (1992) Aptos, CA	(N) misc. continuity hardware to connect (E) discontinuous bond beam elements	(N) steel diagonal top-of-wall pins; (E) bond beam and diaphragm	(E) horizontal rebar
O'Hara Adobe (1994) Los Angeles	(N) steel top-of-wall and misc. continuity hardware	(N) top-of-wall vertical pins; (N) top-of-wall anchorage; (E) diaphragm	(E) and (N) wire mesh containment; (N) through-wall ties and oversize washers
Salvador Vallejo Adobe (1998) Sonoma, CA	(N) steel-strap top-of-wall continuity hardware	(N) center core vertical rods; (N) top-of-wall and through-wall floor anchorage	(N) wire mesh on select walls; (N) through-wall ties and oversize washers on walls with wire mesh
Leese-Fitch Adobe (1998) Sonoma, CA	(E) bond beam; (N) top-of-wall continuity hardware	(N) center core vertical rods; (N) top-of-wall and through-wall floor anchorage	(N) wire mesh on select walls; (N) ties from center core rods to wire mesh
Mission San Miguel (2005) San Miguel, CA	(N) top-of-wall continuity hardware	(N) steel top-of-wall vertical pins; (N) diaphragm; (N) top-of-wall anchorage	(N) top-of-wall strap, through-wall-tied and vertical top-of-wall pins

Shafter Courthouse

Background

The Shafter Courthouse, in Shafter, California, is an adobe structure built in 1940 by the Works Progress Administration (WPA). It is typical of many that were constructed in California during the 1930s and 1940s by the WPA. It is a well-built, one-story building with an L-shaped plan. The building was given to the City of Shafter by the County of Kern in 1992, and the city council decided to rehabilitate it for use as the new city hall.

A significant test of the seismic capability of this structure occurred during two earthquakes in 1952. Shafter is located in a seismically active area, heavily influenced by the proximity of the San Andreas, Garlock, and White Wolf faults. Movement along the White Wolf was responsible for the damaging Kern County and Bakersfield earthquakes of 1952. The July 21, 1952, Kern County earthquake had a Richter magnitude of 7.7 and caused major damage to structures in towns southeast of Bakersfield. The August 22 aftershock had a Richter magnitude of 5.8 and caused major damage in Bakersfield, particularly to buildings already weakened by the earlier quake. As a result of either or both of these earthquakes, slight damage occurred to the courthouse adobe, primarily at the bond beam level.

Building Description

Wall thickness of the courthouse adobe ranged from 12 in. to 24 in. (30–60 cm). Wall heights varied from 10 ft. to 12 ft. (3.0–3.7 m), with gable walls extending up another 3 ft. (0.9 m). Height-to-thickness ratios varied from approximately 5 to 7, a relatively stable configuration. A series of adobe piers on the inner face of the L formed an enclosed corridor. All wall surfaces except the interior corridor were rendered with stucco over galvanized-wire stucco lath.

The building was constructed with reinforced concrete bond beams, from 5 in. to 8 in. (13–20 cm) deep and as wide as the wall thickness. Bond beams at different elevations were discontinuous at wall intersections. The roof framing and top plate were bolted to the bond beam.

Stability-Based Retrofit Measures

Although the courthouse adobe was not seriously damaged in the 1952 earthquakes, there were clear signs of distress at the discontinuities of the bond beam. Three stability-based retrofit measures were utilized in this project:

1. continuity hardware at discontinuous bond beams
2. bond beam anchorage to walls
3. use of wire mesh as containment

Miscellaneous continuity hardware in the form of steel straps and brackets was used to tie the bond beams together at various levels. Tube steel posts were used to anchor discontinuous bond beam elements to continuous bond beam elements at a different elevation, as well as to the foundation (fig. 4).

The concrete bond beams were anchored to the tops of the walls, including the gable walls, by drilled-in 1 in. (2.5 cm) diameter fiberglass rods that penetrate through the concrete and into the top courses of adobe block to a depth ranging from 2 ft. to 3 ft. (0.6–0.9 m). These rods were grouted in place with a fly-ash/soil mixture, which had been used on other historic adobe retrofit projects in California (Roselund 1990).

Since most of the wall surfaces were already rendered with wire mesh and stucco (with no signs of adobe deterioration), it was decided to cover the remaining surfaces with wire mesh and stucco, and to through-tie all new and existing stucco mesh with all-thread rods and oversize washers. This system acts as a containment of the adobe; it does not permit blocks or pieces that crack to fall out of the wall during ground shaking, thereby assuring a continued load path.

The rehabilitated courthouse adobe was dedicated as the new Shafter City Hall in August 1992.

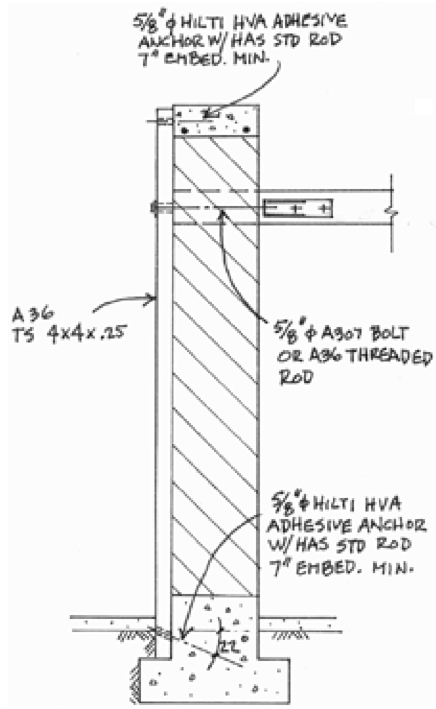

FIGURE 4 Typical wall section of the Shafter Courthouse, showing continuity hardware interconnecting discontinuous bond beams.

O'Hara Adobe

Background

The unreinforced O'Hara Adobe was built in the Toluca Lake area of Los Angeles just after the Long Beach earthquake of 1933. The main adobe structure is 34 × 80 × 13.5 ft. (10.4 × 24.4 × 4.1 m) high, and it has gable walls that extend to 18 ft. (5.5 m) tall. This great room features an adobe tower structure 11 × 12 × 18 ft. (3.4 × 3.7 × 5.5 m) tall. The original plan was to construct a much higher tower, so that the entire building would mimic mission-style architecture. However, following the Long Beach earthquake, building authorities would not allow such a tall adobe tower, so it was constructed to match the height of the roof ridge.

The walls were measured at 3 ft. (0.9 m) thick, and they had what was thought to be a relatively stable height-to-thickness ratio. Yet, during initial retrofit activity, it was discovered that the walls of the great room were actually two single-wythe, 12 in. (30 cm) thick walls with a 12 in. (30 cm) air gap between them. Thus, the height-to-thickness ratio was actually 13.5, not 4.5, as originally assumed. A height-to-thickness ratio of 13.5 is a relatively unstable wall configuration unless the wall is confined. It was decided, then, to fill the 12 in. (30 cm) gap with a urethane-type foam (3 lb./ft.^{3}, or 48 kg/m^3) to within 2 ft. (61 cm) of the bottom of the existing bond beam. The remaining 24 in. (60 cm) were filled with super-lightweight concrete (70 lb./ft.^{3}, or 1121 kg/m^3).

The building has two adjacent low-rise wings with walls 12 in. (30 cm) thick and 8–10 ft. (2.4–3.0 m) tall. The exterior walls of the wings were capped with a concrete bond beam, while the interior cross walls, although closely spaced, had no bond beam.

Northridge Earthquake Damage

The O'Hara Adobe responded to the 1994 Northridge earthquake with typical adobe crack damage, but with little serious effect. The major damage was concentrated in the great room, where evidence of out-of-plane rocking of the massive north and south longitudinal walls was observed. Out-of-plane rocking damage was also observed in the east gable wall. Some of the observed crack damage appeared to consist of a reopening of preexisting cracks from earlier earthquakes.

New damage included a classic short-column, diagonal-shear crack that opened up in one of the adobe tower legs above the level where it abuts the cross wall of the mezzanine floor. One of four adobe chimneys was also damaged when the concrete cap slid and pushed off one side of the chimney above the roofline; the other three chimneys performed well during the earthquake. The building's two adjacent low-rise wings sustained no damage from the earthquake. These wings appeared to be quite stable, with several cross walls relatively closely spaced.

Stability-Based Retrofit Measures

The stability-based seismic retrofit design for the O'Hara Adobe was based on the nonprescriptive requirements of the CHBC, in particular the Alternative Structural Regulations section of the code. Structural upgrading was intended to encourage harmonic rocking response between parallel walls. The simple stabilization techniques that were utilized focused on structural continuity at the tops of the walls by providing steel straps to interconnect the intermittent concrete bond beam (a departure from the code-required installation of a continuous reinforced concrete bond beam), anchoring the walls to the existing roof structure with all-thread through-wall bolts, and connecting the existing plywood diaphragm sheathing to the fiberglass top-of-wall pins.

To completely confine the inner and outer wythes of the great room walls, stucco netting was added to the interior wall surfaces and through-tied with threaded rods and oversize washers to the existing stucco netting on the exterior surface, thereby providing a complete containment of the wall mass. The gable walls were also stabilized by anchoring them to the roof structure with fiberglass rods. Design of the fiberglass rods was based on a 0.8 W_p lateral force on the gable wall (W_p being the weight of the gable-wall section above the bond beam), and the design also took into account the stabilizing effect of the weight of the gable wall.

The tower leg that had suffered a short-column shear failure was stabilized by cutting a 4 in. (10 cm) gap in the mezzanine floor supporting wall where it abuts the tower leg, thus allowing the leg to rock freely, as did the other three legs. Steel plates to anchor through-bolts were added to this leg, to assure that it would respond in a rocking mode during future events.

The damaged chimney was dismantled and reconstructed with wood-frame and stucco construction starting at the bond beam level. Fiberglass rods were installed in the other chimneys to pin the concrete caps and ensure against sliding. The chimneys were also wrapped with stucco netting above the roofline for added confinement and stability.

In the two adjacent wings, where the cross walls had no bond beams, steel strap continuity hardware was installed to ensure that these walls were interconnected to the longitudinal wall bond beams, as well as to ensure that the exterior walls were positively supported by the cross walls.

All cracks were repaired with low-pressure mud grout injection.

Salvador Vallejo Adobe

Background

The Salvador Vallejo Adobe is a designated historical building in the city of Sonoma, California. The city's seismic upgrade ordinance of 1990 required the adobe to be evaluated and retrofitted in 1995 in accordance with the CHBC. Initially constructed of adobe in 1843 and then nearly twice as long as it is today, the building has been altered numerous times for commercial purposes and also because of earthquake damage sustained during the 1906 San Francisco earthquake. Wall thickness of the adobe ranges from 24 in. to 36 in. (60–90 cm) at the first-floor level and from 12 in. to 24 in. (30–60 cm) at the second-floor level. From first floor to second floor, the structure is 13.5 ft. (4.1 m) in height and another 11 ft. (3.4 m) to the second-floor ceiling, with height-to-thickness ratios ranging from 4.5 at the first-floor level

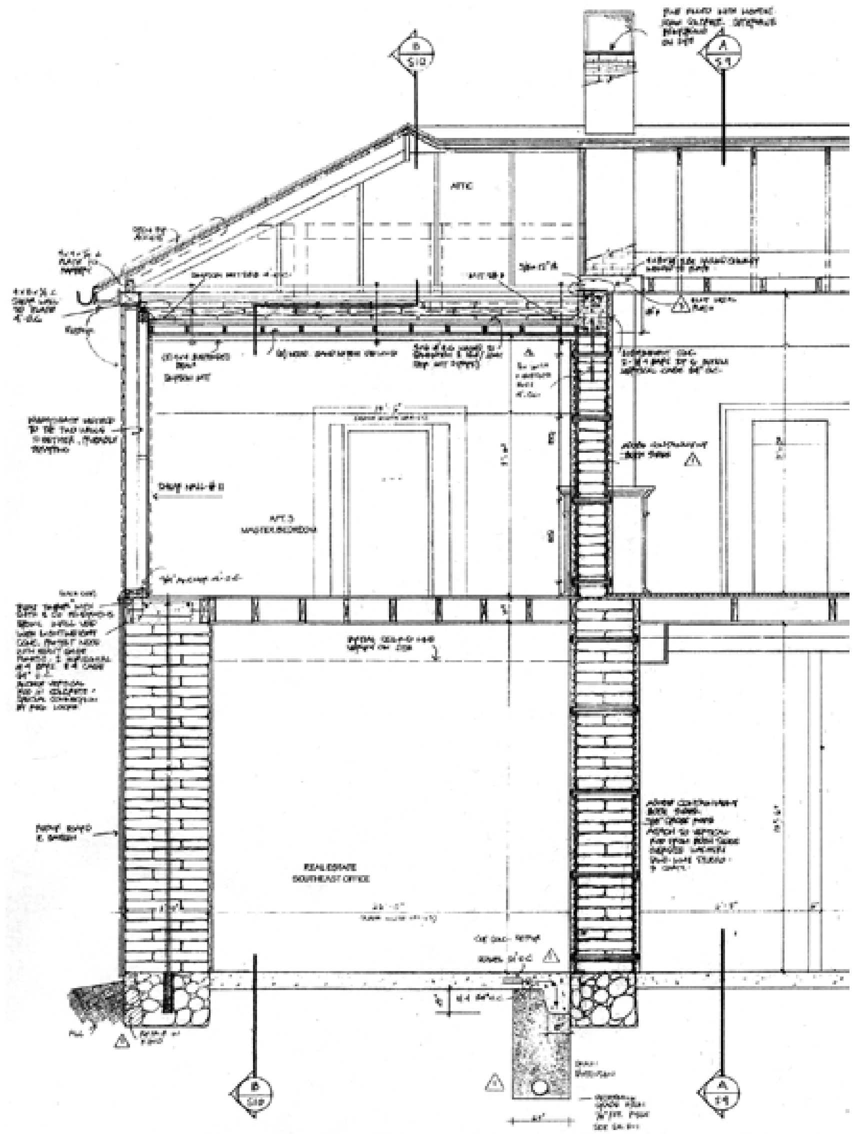

FIGURE 5 Salvador Vallejo Adobe wall and floor section, showing retrofit measures.

to 11 at one second-floor cross wall. A two-story wood-frame structure was added to the west side of the structure in about 1875. The second-floor adobe south wall was replaced with a wood-frame wall, probably as a result of damage caused by the 1906 earthquake.

Building Condition

No foundation settlement problems were observed. However, water damage to the adobe was evident, particularly at the bases of the west and east walls, where spalled adobe and bulging plaster could be seen. The south wall had serious coving damage resulting from rising damp. A few areas at the tops of the walls also showed erosion damage from roof leaks.

Crack damage could not be observed through the various wall coverings of plaster, shiplap boards, and board-and-batten siding on the exterior, and lime plaster and drywall on the interior. Many of the adobe blocks along the tops of the walls at the roof level are eroded, displaced, or loose, and the sill plate was not bearing on all of them.

Seismic Upgrade Measures

The initial concept for seismically stabilizing the Salvador Vallejo Adobe was to pin the tops of the walls with fiberglass rods and to anchor the tops of the walls to the horizontal bracing provided by the roof and second-floor framing. However, in the end, the walls were center-cored with steel rods extending the full height of the walls and epoxy-grouted into the footings (fig. 5). This change was based on the contractor's experience with center core techniques and his ability to convince the owner of the building to install the more expensive center cores.

The design also provided wall steel-strap continuity at the second floor and a lightweight reinforced concrete bond beam at the roof level. Roof and floor diaphragms were upgraded where necessary. Other seismic upgrade measures for the Salvador Vallejo Adobe included:

- French drains to mitigate surface water runoff
- welded wire mesh containment on the surface of the first- and second-floor adobe cross wall, with through-wall threaded rods and oversize washers

- reconstructed wood-frame shear walls in wood-frame addition
- bracing for the two chimney remnants that protrude above the roof, with steel straps and struts anchored to the roof structure

Mission San Miguel Gift Shop and Museum

Background

Mission San Miguel, the sixteenth mission in the chain of Spanish missions along the El Camino Real in Alta California, was founded on July 25, 1797, by Friar Fermín Lasuén. In 1816 the foundation stones for the existing church were laid; the building was ready for roofing in 1818. In addition to the church and sacristy structures that were completed in 1821, the present-day mission is made up of several large, single-story adobe buildings that form the quadrangle. Some date to the 1800s, while others were reconstructed in the 1930s and 1940s on original foundations and incorporated original wall material where it still existed.

The walls of the church are 156 ft. long, 30 ft. high, and 5.5 ft. thick (47.5 × 9.1 × 1.7 m). Walls of the quadrangle buildings, of which the gift shop and museum make up the southeast corner, are 10–15 ft. high and 2.0–3.5 ft. thick (3.0–4.5 m high and 0.6–1.1 m thick). A concrete bond beam was placed on the top of the long walls of the older buildings during a reroofing effort in the 1940s. The buildings that were reconstructed in the 1930s and 1940s have concrete bond beams at the top of both longitudinal and cross walls.

Earthquake Response

Since the completion of the church and sacristy in 1821, the mission has been subjected to frequently occurring earthquake tremors—the San Andreas Fault being quite close by. In 1857 a Richter magnitude 7.6 earthquake struck very close to the mission site, its epicenter just south of Parkfield, a distance of about 17 miles (27 km). The fault rupture was approximately 180 miles (290 km) in length, and it uplifted the area up to 30 ft. (9 m). Although scant information is available on the resultant damage to the mission structures, photographs taken at later dates (1882 through ca. 1900) indicate earthquake-type damage to portions of the church. Since 1857 the mission site has been subjected to numerous tremors of various levels of intensity, including aftershocks from the 1906 San Francisco earthquake that were centered near the mission.

In December 2003 the mission was damaged by the Richter magnitude 6.5 San Simeon earthquake. Immediately following the earthquake, the church and sacristy buildings were red-tagged by the County of San Luis Obispo.[1] A few of the buildings forming the quadrangle were yellow-tagged, while the remainder were green-tagged. However, in November of that year, the entire mission was shut down by the County of San Luis Obispo for noncompliance with the county's unreinforced masonry (URM) hazard mitigation ordinance, which requires that all URM buildings within the county be subjected to a structural analysis upon service of an order and within specified time limits. If a building is found not to comply with the ordinance's minimum earthquake standards, the owner is required to either demolish the building or structurally alter it to conform to the minimum standards. Time limits for developing conforming structural repair plans were not met, and a "Notice to Vacate" placard was placed on the mission. Therefore, in addition to seismic repairs, all the adobe buildings that form the mission (except for a novitiate built in the 1960s) are required to be seismically upgraded in accordance with the county URM ordinance in order to be permitted to reopen.

Stability-Based Upgrade Measures

Because of the critical issue of raising funds to accomplish the seismic upgrade, the repair and retrofitting efforts were split into phases that could be completed as funds became available. The phase 1 effort included the southeast corner of the quadrangle, which encompasses the gift shop and a portion of the museum. Since the friars of the mission rely on the proceeds from the gift shop and museum, it was the first area to be addressed, and the phase 1 buildings were reopened after completion in November 2005.

Stability-based measures utilized in the phase 1 retrofit effort included adding structural continuity at the ceiling level, supporting out-of-plane overturning stability, and containing wall material. Thin stainless steel straps that were through-wall-tied with stainless steel all-thread rods to a continuous ledger beam on the inside surface were installed at the tops of the walls, to serve as continuity hardware. The through-wall ties also acted as top-of-wall anchorage in conjunction with

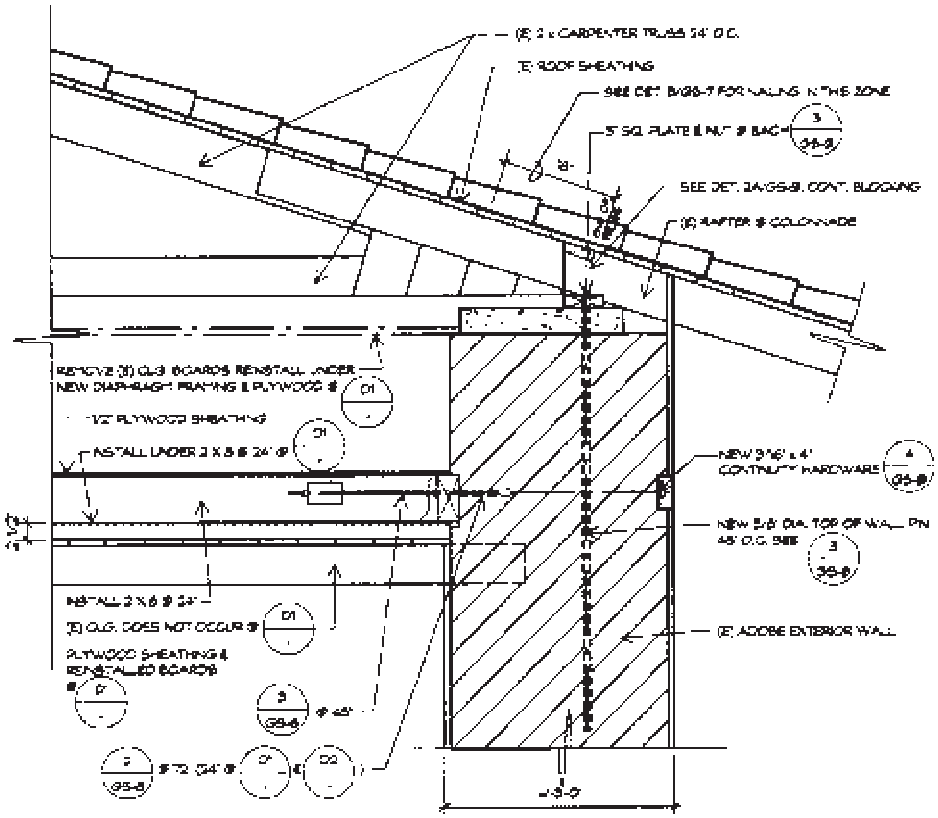

FIGURE 6 Typical top-of-wall stability-based measures for the Mission San Miguel gift shop.

a plywood diaphragm installed at or just above the ceiling level. Overturning stability was accomplished with stainless steel top-of-wall pins 3–4 ft. (0.9–1.2 m) on center, in conjunction with the top-of-wall continuity strap and diaphragm. Containment of the friable top courses of the wall was provided by the top-of-wall pins in conjunction with the top-of-wall continuity hardware, through-wall-tied to the ledger beam and diaphragm on the interior. Figure 6 shows a typical top-of-wall section and the stability-based measures utilized in this first phase of the Mission San Miguel seismic upgrade.

Conclusion

The information obtained during field studies of the seismic behavior and performance of historic and older adobes following an earthquake event is invaluable to the development of appropriate, cost-effective, and minimally intrusive stability-based retrofit measures (see Tolles et al. 1996 for more complete details on historic adobes). Categorization of the types of damage allows an evaluation of the causes and criticality of such damage types, so that effective retrofit measures may be developed and implemented. Indeed, this information, in conjunction with the shake table test results (Scawthorn and Becker 1986; Tolles et al. 2000), has been the basis for the design of appropriate seismic retrofit measures that ensure life safety while protecting historic fabric and cultural value.

The challenge of improving the structural and life-safety performance of historic and older adobes in future earthquakes, while saving historic fabric and cultural value in the process, is a great one. The key is to understand how these buildings perform and to direct minimal intervention and stability-based mitigation efforts to the specific needs and structural behaviors. We can, in fact, improve the performance of historic and older adobe buildings without significantly compromising their historic fabric or the architectural heritage embodied in these important resources.

Note

1 Following an earthquake, counties and cities in the affected area perform rapid safety evaluations of buildings in their jurisdiction, posting every building reviewed as either "Inspected" (i.e., apparently safe) or "Unsafe." Buildings posted Unsafe require repair or demolition, and they must be closed until such time as the appropriate repairs are complete. Buildings are posted as Unsafe with a red tag, as Inspected with a green tag, and as Limited Entry with a yellow tag. "Limited Entry" means simply that the building is off limits to unauthorized personnel, and further engineering evaluation needs to be performed before a red or green tag can be posted.

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Seismic Retrofit Applications of Getty Seismic Adobe Project Technology to Historic Adobe Buildings

E. Leroy Tolles

Abstract: *This paper summarizes a range of seismic retrofit strategies that have been designed by the author for historic adobe buildings in California. The range of the fundamental retrofits includes the principal retrofit strategies covered in the Getty Seismic Adobe Project (GSAP) research program conducted in the 1990s by the Getty Conservation Institute (GCI). The range of buildings includes a single-story building with thick adobe walls, several two-story adobe buildings with thick adobe walls, a single-story adobe building with thin adobe walls built around 1920, and the ruins of an adobe house. The five buildings covered in this paper demonstrate a broad range of seismic retrofit applications.*

The only building with particularly thin walls was the small adobe at Rancho Camulos. The small adobe was built around 1920 and has an architectural style that is unlike the typical nineteenth-century adobe, which is represented by the other buildings covered in this paper. The remainder of the historic adobe buildings primarily had walls that are 1.6 ft. (0.5 m) to nearly 3.3 ft. (1 m) thick.

The range of retrofit measures included anchorage at the floor levels and at the roof. Vertical center core rods were used both in existing adobe walls and in adobe walls that were reconstructed. Vertical straps and horizontal cables and rods were used to stabilize more severely damaged adobe walls.

The final project is the stabilization of the single-story ruins of the Las Cruces Adobe. The stabilization measures are primarily composed of a lightweight steel frame used to prevent the overturning of the adobe walls, which are largely freestanding. Viscous dampers were used to reduce the size of the steel members in the exterior steel.

Introduction

The nature of a seismic retrofit system for a historic adobe building will depend on the goals of the project, financial flexibility, and the characteristics of the specific historic adobe building. The five projects presented in this paper provide a brief overview of the range of retrofit options that are available and that were tested during the research phase of GSAP. This multiyear project, conducted by the GCI, sought to develop structurally effective, minimally invasive seismic retrofitting strategies for historic adobe buildings. This project and its outcomes were described in three publications (Tolles et al. 1996 and 2000; Tolles, Kimbro, and Ginell 2002).

The first mode of failure common to adobe buildings is the overturning of the walls. Therefore, the first step in each of these retrofits was to attach the adobe walls at the roof and floor levels. The details of these connections are very important because the forces that may be imparted to these connections may be large. Therefore, the durability of these connections is critical.

The second step is the addition of vertical straps or center cores to individual walls, which can add significant stability to adobe walls. The walls may suffer significant cracking, but the restraint provided by the straps or center cores can prevent progressive types of failure. In addition, straps or center cores can prevent the out-of-plane failure of thinner walls between the support points at the floors and/or between the floor and the roof. Finally, center cores can increase the strength and ductility of a wall in both the out-of-plane and the in-plane directions.

Castro-Breen Adobe

The Castro Breen Adobe is currently owned and administered by the California Department of Parks and Recreation as part of the San Juan State Historic Monument. Mission San Juan Bautista is included in this complex. The adobe was commissioned by General Jose Maria Castro and was constructed between 1840 and 1841. The ownership of the house was passed to the Breen family in 1848: hence the name, the Castro-Breen Adobe.

The Castro-Breen Adobe is a two-story adobe building with multiple rooms and interior adobe walls on each floor. The exterior walls on the first floor are 33 in. (0.84 m) thick. The interior walls and the exterior walls on the second floor are 22 in. (0.56 m) thick, except for the gable-end walls, which are 33 in. (0.84 m) thick to the roofline. The walls are 10 ft. (3.05 m) high from the ground to the second-floor level and approximately 9 ft. (2.74 m) high from the second floor to the tops of the walls. Therefore, the slenderness (S_L = height-to-thickness ratio) of the first- and second-floor walls is only 3.6 and 4.9, respectively.

Retrofit of the Castro-Breen Adobe

The principal restraint for the retrofit system on the Castro-Breen Adobe was the tile roof, which is fragile and historically significant. Therefore, it was desirable to remove only portions of the roof as necessary for the installation of center core rods. Fortunately, the ceiling above the second-floor level is approximately 1.5 ft. (0.46 m) below the tops of the adobe walls. A partial plywood diaphragm was constructed to provide out-of-plane restraint at the tops of the adobe walls.

Anchorage was also supplied along the lengths of the long walls at the second-floor level. Anchors were placed below the floor level, and because the walls step nearly 1 ft. (0.30 m) at the second-floor level, it was possible to hide these anchors from view. The gable-end wall is the portion of the building most susceptible to significant earthquake damage (fig. 1). The original north gable-end wall is now enclosed by a wood-framed addition and does not pose the same problem as the south wall, because of the restraints provided by the floor, ceiling, and roof system.

The south gable-end wall was anchored to the roofline, and the tile roof was removed from the edge of the roof back 10 ft. (3.05 m) to provide room to reinforce the roof sheathing with plywood. Center core rods were placed in the wall from the tops of the walls to the ground level and anchored with a nonshrink, standard, commercially available nonmetallic cementitious grout. The average height of the wall was nearly 21 ft. (6.40 m), and the full height of the wall has a slenderness ratio of less than 7.6. A partial plywood diaphragm was installed and anchored to a horizontal steel rod. The locations of the elements of the retrofit system are shown in figure 2.

Casa de la Torre

Casa de la Torre was built in 1852 as a one-and-one-half-story building. Originally the adobe consisted of three rooms and an entrance hall. The building was modified in the early 1900s by removal of most of the second-floor framing and the addition of a large window in the north gable-end wall, as shown in figure 3. The adobe walls are 24 in. (0.61 m) thick and approximately 14 ft. (4.27 m) high to the plate line on the long east and west walls. The slenderness ratio is approximately 7 for the long walls and 8 for the gable-end walls. There are wood-framed rooms on both the south and west sides of the building. The kitchen at the northwest corner of the building was constructed with 12 in. (0.30 m) thick adobe walls, and the height of the walls ranged from 9 ft. (2.7 m) down to 7 ft. (2.13 m).

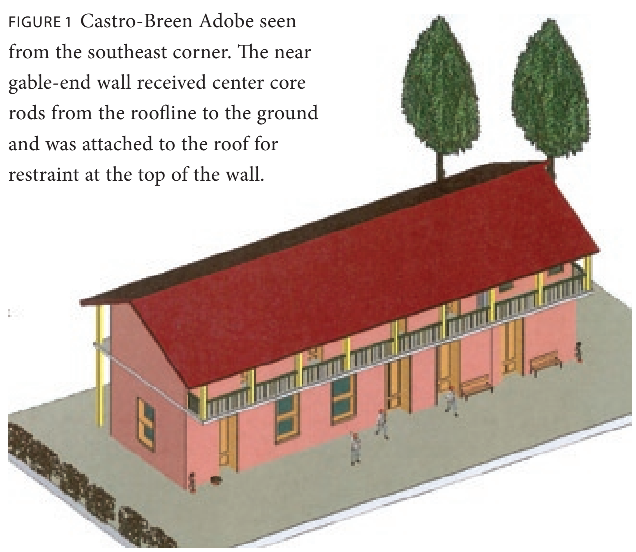

FIGURE 1 Castro-Breen Adobe seen from the southeast corner. The near gable-end wall received center core rods from the roofline to the ground and was attached to the roof for restraint at the top of the wall.

FIGURE 2 Selective use of full-height center core rods shown at the near gable-end wall, in the retrofit of the Castro-Breen Adobe. A partial plywood diaphragm with a horizontal exterior rod is used just below the roofline.

Retrofit of Casa de la Torre

All adobe walls were anchored to the roof system at the tops of the walls. No additional measures were used for the south and west walls of the large central room, since these walls are braced by a lower roof and by perpendicular walls.

Center core rods were installed into the north and east walls and anchored to the plywood roof diaphragm. The 0.75 in. (0.02 m) diameter center core rods were epoxy-coated steel reinforcing bars that were threaded on one end. The rods were placed in 2 in. (0.05 m) diameter holes that were then filled with a nonshrink cementitious grout. The north gable-end wall was already susceptible to collapse during strong ground motions, and the addition of the large window in this wall made it even more susceptible to severe damage or collapse. The long east wall is attached to the porch roof, but there are no transverse walls to provide additional lateral support. Therefore, full-height center core rods were also installed in the east wall.

Small Adobe at Rancho Camulos

The Small Adobe at Rancho Camulos was built in 1920, more recently than the other adobe buildings described in this paper. It is a one-story building with relatively thin adobe walls and many large openings throughout (fig. 4).

The roof is flat, and there is an interior open courtyard in the center of the building. The courtyard is formed by a wood-framed addition in the south-central area of the building which was added at some uncertain date prior to 1950.

FIGURE 3 Casa de la Torre seen from the northeast corner. The retrofit included anchoring the adobe walls into the upper roof on all sides of the building; center core rods were also placed in the two walls that are visible.

FIGURE 4 Small Adobe at Rancho Camulos seen from the northwest corner. The walls were retrofitted with center core rods throughout, because of the thinness of the walls and the large number of openings.

FIGURES 5A AND 5B Small Adobe at Rancho Camulos, which suffered severe damage during the 1994 Northridge earthquake. Shown here are the collapsed walls at the southwest corner (a) and the southeast corner (b) of the building.

(a)

(b)

Retrofit of the Small Adobe at Rancho Camulos

The Small Adobe suffered severe damage from the 1994 Northridge earthquake. The south walls of two of the rooms along the south side of the building collapsed (figs. 5a and 5b). Other walls were damaged so severely that they needed reconstruction.

The principal stability issues for this building are relatively thin walls ($S_L = 8$) and numerous doors and windows. To compensate for these features, which are not typical of historic adobe construction, center core rods were placed in all the walls. The rods are 0.75 in. (0.02 m) in diameter and were placed in 2 in. (0.05 m) diameter holes that were then filled with a nonshrink cementitious grout. The center core rods are placed at a maximum of 6 ft. (1.8 m) on center. In addition to the center core rods, there are anchors that connect the center core to the roof at 2 ft. (0.61 m) intervals.

Main Residence at Rancho Camulos

The Main Residence at Rancho Camulos was constructed starting in the 1840s and was completed in the 1860s. The original house has two stories. Its exterior walls are 24 in. (0.6 m) thick, and interior walls are 12 in. (0.3 m) thick. The first-floor story height is 11 ft. (3.35 m). The additions to the main residence are all one story and extend to the west and north of the main residence. As in the original construction, the exterior walls of the additions are 24 in. (0.6 m) in thickness, and the interior walls are 12 in. (0.3 m) thick. The walls in the additions are 9.5 ft. (2.9 m) in height. The building is shown in figure 6.

The Main Residence at Rancho Camulos also suffered serious damage during the 1994 Northridge earthquake, as shown in figures 7a and 7b. There was crack damage to many of the walls throughout the building. The walls of the bedrooms in the southeast and southwest corners of the building collapsed and required reconstruction.

FIGURE 6 The Main Residence at Rancho Camulos seen from the southeast corner.

(a)

(b)

FIGURES 7A AND 7B Severe earthquake damage to the Main Residence at Rancho Camulos. Collapse of the walls occurred in the southwest corner (a) and the southeast corner (b) of the building.

Retrofit of the Main Residence at Rancho Camulos

The retrofit of the Main Residence used a variety of retrofit techniques. The roof system was tied into the roof diaphragm with top-of-wall anchors attached with epoxy grout. The more severely damaged walls were reinforced with vertical straps on both sides of the walls. A perimeter cabling system was used throughout the building to provide longitudinal continuity to the walls. The second-floor framing was attached to the cabling system through the adobe walls. Finally, the walls that had collapsed were reconstructed with new adobe bricks and were reinforced with horizontal ladder ties in the mortar joint of every fourth course, and with vertical center core rods in 2 in. (0.05 m) diameter holes spaced at approximately 26 in. (0.66 m) on center; they were anchored in epoxy grout.

Las Cruces Adobe

The Las Cruces Adobe was constructed in the 1840s and has been unoccupied since the early 1900s. A shelter was built over the site in the 1970s to protect the fragile ruins (fig. 8). The walls (fig. 9) and roof framing are both in poor condition. What remains of the building is original and indicative of the type of construction characteristic of that period.

Stabilization of the Las Cruces Adobe

The goal of the retrofit was to have as minimal an impact as possible on the original building but to stabilize the ruins so that the public could access the building safely. To achieve these goals, a lightweight steel frame was designed to provide overturning stability to the walls. To increase the effectiveness of the steel frame, viscous dampers were added which allowed the steel frame to be even lighter than the original design. By increasing the damping of

FIGURE 8 Las Cruces Adobe covered by a wood shelter that protects the ruins. Photo: Gail Ostergren.

FIGURE 9 An interior wall of Las Cruces Adobe, representative of the general condition of the building. Photo: Gail Ostergren.

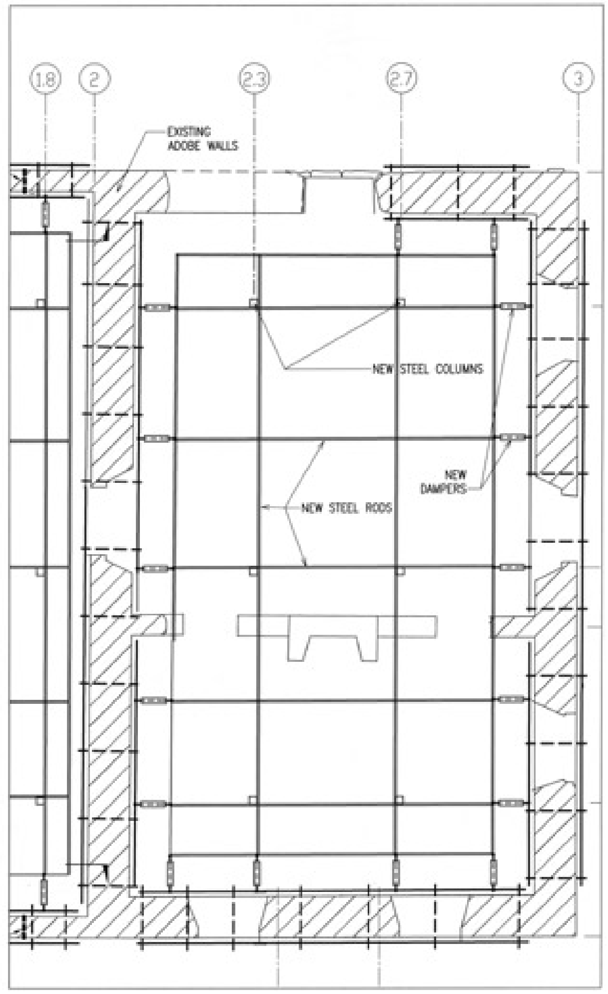

FIGURE 10 Lightweight steel frame designed for Las Cruces Adobe.

the support frame, the viscous dampers would also significantly decrease the displacements that might occur in larger seismic events. The framing system is shown in figure 10. The steel columns were 3.5 in. (0.089 m) square tubes, and the horizontal steel rods were 1 × 2 in. (0.025 × 0.05 m).

The GSAP began with a vision to develop innovative methods for retrofitting historic adobe buildings. The vision was based upon the author's doctoral research work at Stanford University (Tolles 1989). This early research demonstrated that minor interventions could have a major impact on the seismic stability of thick-walled adobe buildings.

The examples of the successful design and implementation of the methods developed during the multiyear GSAP research effort presented in this paper demonstrate that these methodologies can be effectively implemented in the field. Building officials throughout California have approved these techniques. The governing building code for historic adobe buildings in the state of California is the California Historical Building Code, which has reduced seismic force levels and recognizes the use of identified "archaic" materials such as adobe.

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Evolving Methodology in Seismic Retrofit: Stabilizing the Las Flores Adobe

John M. Barrow, Douglas Porter, Stephen Farneth, and E. Leroy Tolles

Abstract: *The Las Flores Adobe National Historic Landmark, constructed in 1868, has been seismically and structurally stabilized over a three-year period. Located in an active seismic area of Southern California, the complex of buildings represents one of a few authentic nineteenth-century, two-story adobes combining the Hacienda and Monterey styles, which are unique to Hispanic traditions of California. The buildings are constructed of adobe brick and are surfaced with a combination of earthen- and lime-based finishes.*

Since the 1970s, the unused buildings had fallen into disrepair. A phased stabilization project to save the landmark started in 2000, with participation of a multidisciplinary team. The team applied its collective expertise in architecture, engineering, and conservation to develop a design that satisfied life-safety and fabric preservation agendas.

The California Historical Building Code (CHBC) was applied to allow for alternative performance-based solutions (California Building Standards Commission 1998a). Stability-based retrofit design for this project was developed out of the Getty Seismic Adobe Project (GSAP) research program supported by the Getty Conservation Institute during the 1990s. The ranch house, or main house, and carriage house were stabilized in 2002–4 with retrofit designs that took advantage of the energy dissipation characteristics of thick adobe walls in the postelastic phase. These minimally invasive systems, using rods, steel strapping, grouted pins, and plywood sheer panels, served to improve structural continuity, prevent overturning of walls, and minimize loss of historic fabric by limiting displacement. Work on the carriage house incorporated the use of earthen grouts in the installation of center core rods. Earthen grouts are readily available, compatible with historic adobe, and reversible. A training component was integrated into the construction program.

Introduction

The Las Flores Adobe, a National Historic Landmark, is located between two major fault lines aligned with the coast of Southern California; the area is in seismic zone 4, an area in which there is a 1 in 10 chance that an earthquake with an active peak acceleration level of 0.4 g (4/10 the acceleration of gravity) will occur within the next 50 years. It is in one of the most active tectonic fault zones in the world. The building has survived hundreds of seismic events, including a major earthquake (Richter scale = 6.8) associated with the nearby San Jacinto fault zone, on April 21, 1918.

Built in 1868, the Las Flores complex includes a ranch house, or main house, consisting of a formal two-story Monterey block and a long, low Hacienda block with rooms opening onto a *portal*, or porch (figs. 1a and 1b). There is also an attached carriage house. The Las Flores site is one of a few authentic nineteenth-century adobe ranch houses combining the Hacienda and the Monterey styles, which are unique to California. The United States Marine Corps, the National Park Service (NPS), the Graduate Program in Historic Preservation at the University of Vermont, and private sector architectural and engineering professionals have partnered in the planning, design, and stabilization of the building complex.

(a)

(b)

FIGURES 1A AND 1B The Las Flores complex with a ranch house (main house) consisting of the two-story Monterey block (a) and the Hacienda block, which is the long, low section connecting the Monterey block on the left and the carriage house on the right (b).

Seismic stabilization has focused on implementing techniques advanced by the Getty Seismic Adobe Project (Tolles, Kimbro, and Ginell 2002; Tolles et al. 2000 and 1996), which impart stability to adobe walls while preserving the historic fabric and structural system. The interventions also comply with performance standards for structural design, as outlined in the California Code for Building Conservation (International Conference of Building Officials 1998), particularly with respect to the lateral design of unreinforced masonry buildings, and the California Historical Building Code.

Prior to GSAP research, retrofit technology routinely applied in similar cases involved the installation of invasive concrete post-and-beam and bond beam assemblies requiring major demolition of historic fabric. At Las Flores, the team has installed minimally invasive systems utilizing rods, steel strapping, grouted pins, and plywood sheer panels. The carriage house work required that a preexisting concrete bond beam be incorporated into the retrofit design and presented the team with an opportunity to use earthen grouts as a more compatible material substitute for the epoxies used on the main house project. This case study represents one of several in which this type of technology has recently been implemented in the field.

Historical Background

The Las Flores Adobe Ranch House is a 557 m^2 (5995 sq. ft.) two-story adobe building and once was part of an over 52,600 hectare (129,922 acre) ranch. It was taken over by the federal government in 1941 for use as a U.S. Marine military training base during World War II. It continues to be under the jurisdiction of the Marine Corps.

The site is representative of settlement patterns throughout most of California history, contained in one compact and largely undisturbed microenvironment. Archaeological and historical records at the Las Flores site indicate nearly two thousand years of occupation by Native Americans. In the eighteenth century, the first European colonization of California followed the spread of Franciscan missions throughout the region, and in 1798, the pueblo of Las Flores was established under the jurisdiction of nearby Mission San Luis Rey.

In 1834 the mission system was secularized after Mexico gained its independence from Spain, and Las Flores was made a free pueblo. Borders with the United States were opened under Mexican control and trade practices were liberalized, followed by the proliferation of *rancho* culture. Las Flores was purchased from the natives in 1844, and it became part of the larger Santa Margarita Ranch. California came under U.S. sovereignty in 1848. In 1868 Juan Forster, the property owner, constructed the adobe house at the Las Flores site as a wedding gift for his son. Following the collapse of the *rancho* economy in the 1860s, the Forster family fortune went into a slow decline, and Las Flores was sold in 1882 to pay family debts. A San Franciscan named James Flood bought the property and hired Richard O'Neill to manage it. O'Neill leased the Las Flores adobe and

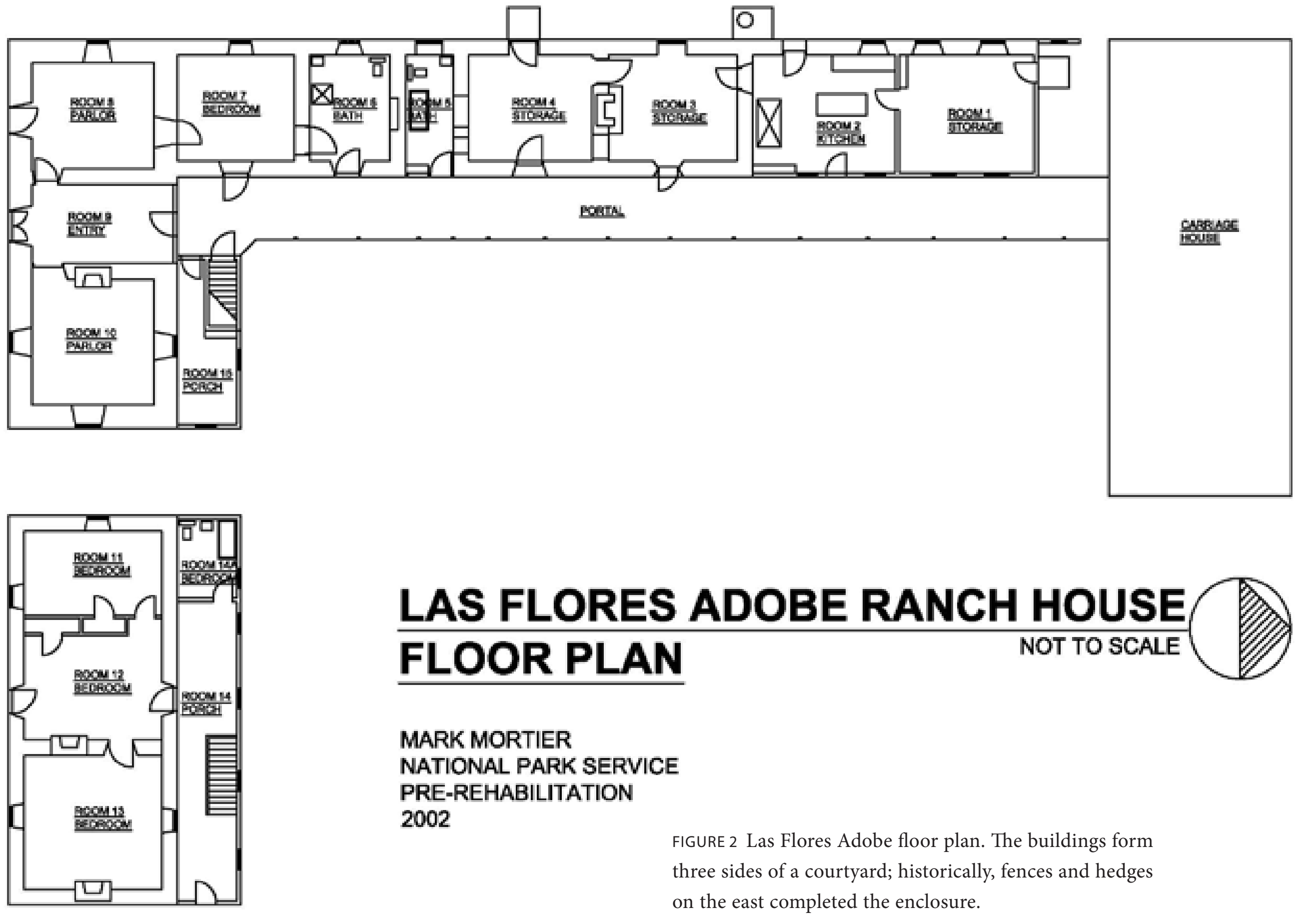


FIGURE 2 Las Flores Adobe floor plan. The buildings form three sides of a courtyard; historically, fences and hedges on the east completed the enclosure.

1500 acres (608 hectares) to the Magee family in 1888, and the site was farmed by this family for more than five decades before being acquired by the Marine Corps. The military presence has not negatively impacted the site, and in contrast to the surrounding communities, where development has obliterated all historical context of the landscape, this property is unique.

The main house embodies the joining of Hacienda and Monterey architectural styles. The house is fronted by an elegant two-story Monterey block, with a full-length two-story porch facing the Pacific Ocean (fig. 2). The ground level of the house is built of 61 cm (23.8 in.) thick adobe walls; wall thickness at the second level is reduced to 46 cm (17.9 in.). Adjoining the Monterey block on the north side of the main house, the utilitarian Hacienda section is a long, one-story wing, one room deep in plan, with doors opening onto a covered *portal* (porch). At the south end, the *portal* connects to a large hallway running through the center of the Monterey block. The Hacienda block terminates at the carriage house, which is parallel to the Monterey block, so that the complex forms a large U around a central courtyard (which is a feature of the Hacienda style). Over the years many changes have been made to the buildings to accommodate new occupants and uses. Historic images indicate that a major construction campaign was undertaken between 1917 and 1919, at about the time of a major earthquake in the region. This campaign included replacement of the roof frame and covering, construction of porches on all four sides of the Monterey block, and the introduction of new doors, windows, and woodwork.

By 1968 the main house and surrounding buildings were in an advanced state of disrepair. Public intervention saved the house and carriage house and had them placed on the National Register of Historic Places and proclaimed a National Historic Landmark, which is the highest designation for a historic property in the United States. In 1999 the Marine Corps initiated a program to stabilize the house using a multidisciplinary team.

The team included a Marine Corps archaeologist, the base museum specialist, an NPS architectural conservator, an NPS historical architect, a consulting historical architect, and an engineer specializing in seismic and adobe preservation. At certain key points in planning, the California State Historic Preservation Officer was included in the review of program and design, since ultimately the state has jurisdiction over the historic designation. In 2002 the NPS invited the Graduate Program in Historic Preservation at the University of Vermont to participate in the program.

Project Planning, Design, and Implementation

Project goals were to reverse deterioration, ensure seismic and structural stabilization, accomplish limited restoration related to stabilization, and plan for future rehabilitation. Project work began with a condition survey focused on the main house in 2000–2001, which led directly to stabilization work in 2002 and 2003. During the 2003 season, a similar survey was conducted on the carriage house, which led to stabilization in 2004. The adobe walls of the main house were found to be generally sound, although they did have some localized cracking. The two-story porch on the Monterey block had been removed in the 1980s for safety reasons, leaving exterior adobe walls exposed to the weather and resulting in losses of large sections of lime plaster. The second-story roof frame did not meet code, and shingles throughout the entire complex were at the end of their useful life.

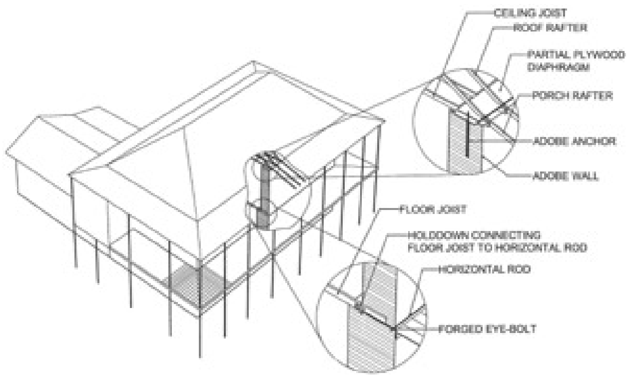
Roof frames on all of the buildings lacked substantial connections to the adobe walls. Floor frames on the ground level were set on grade and completely deteriorated. Second-floor joists were set into pockets in adobe walls without connection. Evidence of termite and fungal deterioration of wood was seen throughout the structure. Windows and doors were severely damaged or missing entirely as the result of vandalism, fungal decay, and termite infestation. Clear sheets of Plexiglas were placed over openings to secure the structure, reducing ventilation throughout the buildings. Infestations by burrowing animals and bees went unchecked, and beehives entirely filled many of the stud bays of the north porch enclosure. The electrical service, not improved after many decades of use, was a fire hazard.

The carriage house was extensively renovated in 1974. Installation of a concrete bond beam resulted in changes in the elevation and construction of the roof, and in consequent loss of the connection between the carriage house and the main house. A large section of the south wall of the carriage house, near the juncture of the two buildings, had collapsed from water damage and was repaired with a large concrete infill. The building was plastered inside and out with hard, cement-based stucco. At the same time, a concrete skirt, or partial retaining wall, was poured around the base of the exterior walls.

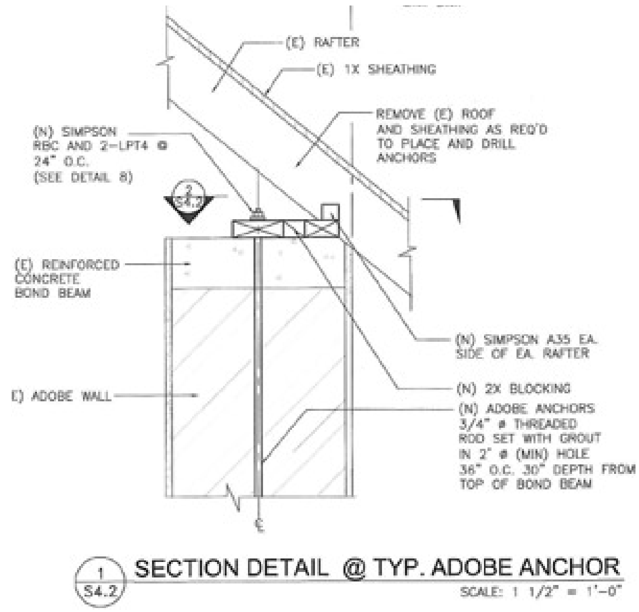
The team established design criteria and performance expectations. The main objective of the stabilization was to preserve the National Landmark values inherent in the architecture. Restoration would be limited to elements essential to meet preservation and stabilization goals. Because many construction details were hidden from view, a flexible design process was adopted, so that treatments might evolve in response to hidden conditions. The team elected to interpret a relatively long period of the buildings' history in order to preserve fabric from time periods associated with *rancho* culture and agricultural use of the property. The government's 1941 takeover of Las Flores changed the use of the property and removed maintenance incentives and proprietary interests from the occupants. These conditions resulted in very expedient and negative alterations, and so mark the end of the period of significance.

The building code applicable to this project is the CHBC. This code applies to all designated historic properties in California and serves as an amending document to the regular code, the California Building Code (CBC) (California Building Standards Commission 1998b). The CHBC is a performance-based code intended to achieve the life-safety objectives of the CBC while allowing greater flexibility in the methods for achieving those objectives. In this way, it encourages the preservation of historic materials and features of the historic property. For the Las Flores project, the CHBC was applied to egress issues, as well as to vertical and horizontal loadings of structural elements.

With respect to the seismic retrofit, the team selected a minimal intervention among a range of options, balancing the life-safety requirement against the preservation objective to impact the integrity of the structural components in the smallest way possible.

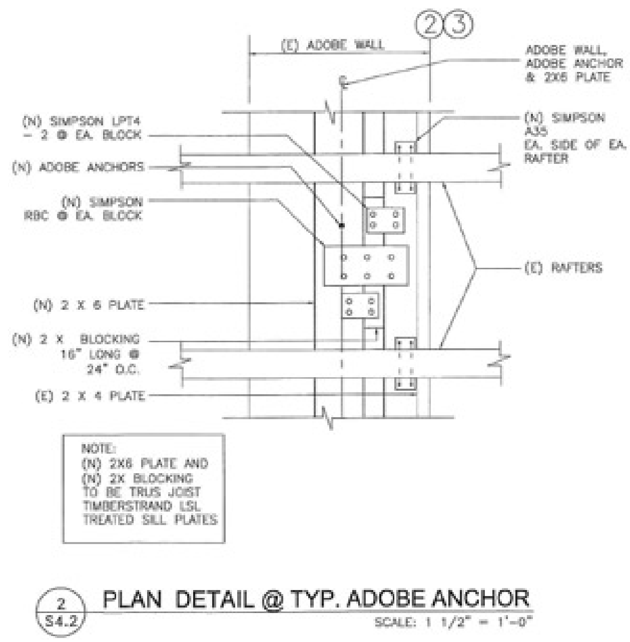


(a)



(b)

FIGURES 3A–C Las Flores Adobe retrofit design drawings. The design focused on providing lateral restraint at the tops of walls and at the level of the second-floor frame in the Monterey block (a); a section detail (b) and a plan detail (c) of the adobe anchoring system give specifics of the installation.



(c)

The seismic objective was to ensure the life safety of building occupants by preventing collapse, while recognizing that repairable damage to the building will occur. Since the adobe walls have slenderness (height-to-thickness) ratios of ≤ 5, they would require minimal lateral restraint in order to prevent overturning.

The initial design plan and second-stage carriage house design were worked out on site by the team. These on-site design meetings focused primarily on seismic retrofit concepts, coupled with architectural considerations and related stabilization issues. The GSAP guidelines were followed for development of the retrofit designs. The scope of work included replacement of the roof covering, allowing for access to the adobe walls from the top. To achieve lateral restraint of the walls, threaded rods (76 cm [29.6 in.] long x 1.90 cm [0.74 in.] diameter) would be grouted into the adobe walls at approximately 80 cm (31.2 in.) intervals on center (see figs. 3a–c). These interventions as planned could be installed without changing the visual aspects of existing walls and roof timber. Stainless steel containing molybdenum for increased corrosion resistance to chlorides and sulfides was prescribed for all rods, nuts, and straps. All nails and other fastenings would be stainless steel or galvanized.

During the 2002 and 2003 seasons, the work was focused on the main house. The seismic retrofit consisted of the addition of a wooden bond beam to the tops of the adobe walls, installation of center core pins, attachment of the roof frames to the tops of walls, and installation of a steel band around the exterior of the Monterey block at the level of the second-floor frame.

Where the tops of the adobe walls were uneven and out of level, the walls were capped with soil cement (portland cement and local soil mixed at a ratio of approximately 1 part by volume of portland to 5–7 parts

(a)

(b)

FIGURES 4A AND 4B Seismic retrofits for the main house. The tops of the walls were leveled by the addition of a pisé course. The wooden bond beam incorporated a stainless steel strap and was fastened to the walls with center core pins and to the roof frame with commercially available metal clips (a). At the level of the second-floor frame of the Monterey block, a stainless steel belt was installed below exterior plaster and tied back to floor joists with eyebolts and Simpson anchor brackets (b).

by volume of soil) applied in pisé technique. The plywood bond beam functions as a partial diaphragm and consists of two overlapping layers of pressure-treated material glued and nailed to provide longitudinal strength. Once the bond beam was placed, holes 2.5 cm (1.0 in.) in diameter were drilled down through the plywood and adobe wall center to a design depth of 80+ cm (31.2+ in.) to receive the threaded rod. Center core pins were set into the adobe with an epoxy grout. The epoxy selected for the project was a proprietary high-viscosity epoxy that is designed for stabilizing anchors into unreinforced masonry. Stainless steel strapping was nailed at 10 cm (3.9 in.) intervals on center along the top of the plywood, and each rafter was fastened to the plywood with a nailed-on Simpson L tie. Every other bolt was torque-tested to at least 27 kg (59.4 lb.) force to ensure good binding and grab in the section. This process proceeded linearly around the building until the interconnected strapping system was complete on both the one- and two-story sections (fig. 4a).

The steel belt installed at the second-floor level attaches the floor system to the perimeter walls (fig. 4b). A 1.5 cm (0.59 in.) channel was cut though the existing lime plaster and adobe on the walls just above the level of the second-floor porch deck. Simpson HD 5A brackets were fastened to every other interior joist behind the interior surface of the wall. A threaded eyebolt was fastened into the Simpson tie with the eye set in the channel on the exterior. A 1.27 cm (0.50 in.) threaded rod was inserted into the eye, wrapped around the entire house, and fastened at the four corners to an L flange. On the east and west end walls in line with parallel running joists, stabilization required longer rod connectors drilled through two perpendicular joists, bolted with nuts and washers, and extending through the adobe wall to tie to the belting rod. This type of belted anchorage will improve the ductility of the wall construction. The anchoring capacity no longer depends on the strength of the adobe; rather, it serves to restrain the adobe wall. It is highly unlikely that the rod could be pulled through the wall, and only localized crushing of the adobe is anticipated in a seismic event.

The decision was made to reconstruct the lost two-story porch to better protect adobe walls, integrate seismic interventions, and recover the lost architecture. The porch, which completely surrounds the Monterey section, was based on photographic documentation that included detail adequate for producing construction drawings. The structural design of the porch incorporated through-wall fastening to interior floor joists, new upgraded foot-

(a) (b)

FIGURES 5A AND 5B Grout techniques for the Las Flores Adobe retrofit. For the carriage house, techniques were developed for installing an earthen grout (a). In contrast, the crew had used a resin-based grout for installing center core pins in the main house retrofit (b). The earthen grout is more compatible with the historic wall materials and provides greater possibilities for reversibility of the treatment.

ings, custom column stands, and wind uplift retention. By installing the connecting elements in the substrate, the installation was designed to be mostly hidden.

During the 2004 construction season at Las Flores, the team implemented the seismic and structural stabilization of the carriage house; the work represented a departure from several aspects of the main house intervention. Work accomplished during the 2002 campaign was evaluated, and changes in tool use, method, and materials were adopted. The scope of work included installation of a seismic retrofit system, replacement of concrete wall infill with adobe materials, reinstatement of the connection between the main house and carriage house, conservation of earthen and lime plasters and finishes, and replacement of the roof covering.

Since a concrete bond beam had been added to the carriage house without documentation during a 1970s repair project, the retrofit needed to guarantee attachment of the bond beam to the walls, and of the roof frame to the bond beam. Removal of this bond beam was determined to be potentially damaging to historic fabric, as well as unnecessary. The large section of concrete infill was determined to be detrimental to the structure, since material and structural continuity was broken and traditional lime plasters would not adhere well to the surface. Adobe repairs were done to replace the concrete infill; new work was integrated into the wall by stepping and lacing new adobe into the old.

Since building performance during an earthquake is dependent, to a large extent, on the integrity of adobe walls at the base, a protocol was developed for evaluating adobe condition in the lowest courses. A grid was laid out on interior walls approximately 30 cm (about 12 in.) above the finished floor. Holes were drilled at each grid point; resistance of the adobe to the drill was evaluated by the drill operator, and wall materials removed from the holes were evaluated with respect to moisture content. When a void was encountered, plaster was removed, and the wall was evaluated for repair.

Essentially following that of the main house intervention, the seismic retrofit system for the carriage house consisted of a series of center core anchors that pass vertically through a continuous wooden plate, through the bond beam, and 76 cm (30 in.) into the adobe wall below. A series of holes was bored with light coring bits; holes were bored through the concrete bond beam and into the adobe with nonvibratory drilling equipment.

Rods were fixed into place with a soil-cement grout (fig. 5a). This choice marked a major change in design and represents a desire for greater compatibility and reversibility of treatments. In contrast, the main house retrofit system relied on a resin-based grout with strength characteristics in excess of design criteria (fig. 5b). Earthen grouts are compatible with historic adobe materials and offer greater potential for reversibility and thus were chosen for the carriage house. The grout mix selected was similar to one developed by Nels Roselund (1990) and tested for use in the repair of the historic Pio Pico Adobe in Whittier, California; it consisted of adobe soil, sand, a small amount of portland cement, and a grout additive (Sika GroutAid) to minimize shrinking during curing.

Initial testing of the grout included qualitative evaluation of material samples with respect to shrinkage, hardness, abrasion resistance, and permeability, and injection of the grout into test panels that then could be visually evaluated to assess crack-filling properties. Subsequent testing by Krakower and Associates included tension and shear tests of anchors installed with the grout, in order to determine bond strength of the grout to anchors and to adobe wall materials. At Las Flores, sample cylinders were made for conducting compression strength tests in the lab. Tests were conducted on a Tinius Olson machine operating in the lowest range (0–272 kg, 0–600 lb.). Results, including load versus deflection and shear angle, were consistent from sample to sample.

Low-tech methods were developed for placing the grout. An adobe test wall was constructed, and placement methods were developed and practiced until placement could be effected smoothly and consistently. A placement device was made by attaching a grout bag to a length of PVC pipe (fig. 5a). The pipe allowed placement to begin at the bottom of the holes, preventing voids due to trapped air bubbles. The method for placement involved filling the device with grout and twisting off the bag to prevent loss of material when transferring the grout to the hole to be filled. Holes were minimally prewetted with a 1:1 mix of denatured alcohol and water to retard absorption of the mix water by the wall materials. Workers placed grout in a prewetted hole by squeezing the grout bag and simultaneously withdrawing the pipe from the hole. Once the hole was filled to within 5 cm (2 in.) of the top, the threaded anchor was inserted. After setting, samples were exposed and visually evaluated with respect to shrinkage and voids. Sample anchors, set in grouted center cores in the test wall, were torque-tested to 88 Nm (65 ft.-lb.) to ensure that connections could be adequately tightened without failure of the grout.

A new wooden plate was installed around the building perimeter on top of the concrete bond beam, fastened to the threaded anchors with stainless steel nuts and washers. With commercially available L clips, the existing roof structure was fastened to the new wooden plate, effectively tying the roof structure to the walls. Because the west wall of the carriage house is of wood-frame construction, a bond beam had never been installed along this wall. To tie the north and south bond beams together, the crew installed a steel tie rod in the west wall cavity; the rod is fastened to commercially available clips anchored to the ends of the bond beam at the northwest and southwest corners of the building. Interior plaster was removed, and this wall was resheathed in plywood to improve its performance in shear.

Conclusion

Key to the success of the Las Flores project—measured by limited alteration to the historic character-defining features of the house—was the multidisciplinary planning and design process, and the flexibility built into the construction phase by the use of architectural and engineering services throughout. This practice prevented the break in linkage among disciplines that often occurs in large construction campaigns. Management participation ensured that project goals and resource allocation stayed viable throughout. Bringing the University of Vermont into the process offered capacity building, training, and research opportunities. The design solutions represent a minimal and efficient treatment approach that achieved the basic goals while simultaneously accomplishing resource preservation agendas (fig. 6).

FIGURE 6 The Las Flores Adobe after completion of structural and seismic stabilization of the buildings. By the close of the 2004 construction season, porches on the Monterey block were reinstated, building envelopes were secured against weather, and interior rehabilitation was started.

Documentation and maintenance by site stewards will ensure that, in the future, a post-seismic event review occurs that will fully evaluate the levels of efficacy achieved. The true test will occur during and after a future earthquake, and results cannot be presupposed or fully anticipated until that time.

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Summary of Discussions

Mary Hardy and Claudia Cancino

Almost two decades ago, the Getty Conservation Institute (GCI) began researching and developing methods to provide seismic stabilization for historically and culturally significant buildings located in seismic regions. The Getty Seismic Adobe Project (GSAP) investigated less-invasive, stability-based alternatives to existing strength-based retrofitting methods. After studying historic adobe buildings, analyzing recent earthquake damage, and developing and testing new retrofitting techniques, GSAP devised ways to provide seismic protection while preserving the authenticity of historic adobe structures in California.

The methods and techniques proposed by GSAP can be adapted for use in communities with limited resources around the world. Several years after the GSAP guidelines were published and disseminated, the GCI hosted the Getty Seismic Adobe Project 2006 Colloquium, which gathered a multidisciplinary group of professionals working on the seismic retrofitting of earthen structures, both within California and outside of the United States, to discuss the applicability of these guidelines and techniques in a variety of contexts. The first two days of discussions focused on previous experiences with stability-based, earthquake-resistant design, appropriate testing methods, and building codes and standards specific to earthen architecture, along with case studies from around the globe.

The third and final day of the colloquium was designed to promote discussion among all of the participants, with the aim of jointly creating a list of recommendations for moving the field of conservation of earthen buildings in seismic regions forward. To facilitate discussion, a series of roundtables was organized around four topics: (1) the California State Historic Building Code, specifically the shift from a strength-based to a stability-based design approach for the seismic retrofit of earthen structures; (2) national building codes for earthen architecture; (3) future research and testing; and (4) information dissemination and training. Following each roundtable, the topic was opened to the entire group for discussion.

During the concluding session, four rapporteurs synthesized the day's discussions and presented a draft list of recommendations that emerged from those exchanges. The main points are summarized below.

1. Shift from a Strength-Based to a Stability-Based Design Approach for the Seismic Retrofit of Earthen Structures

The GSAP guidelines represent a shift away from mainstream, strength-based methods of retrofitting earthen structures, which add independent structural systems of steel or reinforced concrete to historic buildings and can result in the removal of substantial amounts of historic material in order to accommodate new structural elements. In a manner similar to techniques of some vernacular construction traditions, the methods recommended in the GSAP guidelines allow buildings to move and crack in an earthquake, thereby dissipating energy. However, GSAP's stability-based retrofits also help prevent collapse by adding flexible, interactive structural elements that provide overall structural continuity and keep walls from overturning by minimizing the relative displacement of cracked wall sections. While this principle was agreed upon by colloquium participants, the following points were suggested in order to advance the concept:

- A shift in design approach from strength-based to stability-based design will require the reeducation of many site managers, engineering and design professionals, architectural conservators, and policy makers, as well as building occupants and the general public. This education will require an active program for disseminating alternative design criteria and their supporting test results, in a manner and language that are thoroughly understood by diverse audiences.
- While it was agreed that materials in retrofit projects must be *durable*, *readily available*, and *compatible* with the original materials, and that interventions should be *minimally invasive* and *reversible,* if possible, it was apparent that the definitions of these terms were not consistent among the various disciplines. Efforts must be made to standardize the understanding and use of these terms across disciplines.
- The design of a structural retrofit project should include a methodology for evaluating the project over time and should be carried out by a multidisciplinary group of professionals.

2. National Building Codes

National building codes, norms, or standards, if well conceived and rigorously enforced, result in safer buildings and consequently protect public safety and save lives during earthquakes. Codes legitimize construction materials and methods that are included in a code, and they essentially outlaw methods and materials excluded from a code. It is therefore important that earthen construction, as well as intervention methods for historic earthen structures, be introduced into every national building code. In order to develop earthen building codes in different countries, the colloquium participants suggested the following:

- Model guidelines and standards should be crafted to serve as references for governments developing their own building codes for earthen structures.
- Model guidelines and standards should be based on sound engineering principles and draw upon the best existing codes, guidelines, and standards to formulate their content. Guidelines and standards should allow for revision over time, based on any new understandings gained from earthquakes and testing programs.
- Codes must address the care and sensitivity to character-defining features required when existing historic buildings are retrofitted. This is generally different from requirements for new construction or for the retrofitting of nonhistoric vernacular buildings.
- Complementary building codes, standards, guidelines, and manuals addressing the conservation of historic earthen sites in seismic regions should be designed to target different audiences (i.e., professionals, builders, and the general public). If this is not possible, illustrations should be included in the code itself to make the content accessible to users with different levels of technical understanding.
- Slenderness ratios specified in existing codes should be standardized in relation to local seismic zones, to allow real comparison among codes and case studies.
- While addressing the structural components of earthen buildings, codes should consider the masonry, mortar, and plaster as one complete wall assembly. Tests such as those recently carried out at PUCP have shown that earth- or lime-based plasters dramatically improve the strength of earthen walls and control cracking during earthquakes while protecting walls from direct contact with water.
- Codes should consider the local and regional cultural contexts and settlement patterns, and the resulting building traditions. A national code may well need to address several very different regional patterns, construction techniques, and building cultures.
- Codes for earthen architecture borrow heavily from codes for stone masonry, brick, and con-

crete. It is important to study the possibility that aspects of codes for earthen architecture, especially in reference to historic resources, could influence the codes for other building materials as well.

3. Future Research and Testing

Colloquium participants agreed on the need to develop scientific data on historic earthen sites, including material behavior and seismic information, then use that data when designing retrofitting plans. Documentation collected should include historic structure reports, tests, and, most of all, statistical data on ground and structure behaviors. The following list addresses potential engineering and conservation research topics and testing methods or programs that could be useful in advancing engineering and conservation knowledge pertaining to seismic issues in earthen sites.

- Expand the types of models used in future shake table testing. Data from shake table tests thus far are based on newly constructed models of simple, one-room adobe structures with relatively lightweight roof systems. Include in future testing more complex floor plans; construction techniques for earthen buildings other than adobe/mud brick, such as rammed earth (pisé) and wattle and daub (also called *bahareque* or *quincha*); structures with massive roofs (i.e., domed or vaulted structures); and historic material.
- Carry out evaluations of traditional construction in seismic zones by multidisciplinary teams. While such teamwork is challenging because of the difference in professional languages and attitudes, multidisciplinary input is essential to a full understanding of these complex cultural resources.
- Identify retrofit methods and materials most appropriate for a particular region. Available materials, financial resources, and technical skill levels vary significantly throughout the earthquake-prone regions of the world where earthen architecture is common.
- Explore the potential for virtual earthquake testing through computer modeling. Computer modeling could answer some of the shortcomings of costly shake table tests and could test more complex building configurations under multidirectional impulses. This type of testing tool should consider variations in existing site conditions, such as types and conditions of the soil, masonry moisture content, and existing structural cracks, among others.
- Expand our understanding of field conditions. Laboratory test data have been derived from samples made of clean, homogeneous material, while in fact, material properties of existing earthen structures are generally different because of such factors as the presence of salts, moisture, and biological infestation.
- Carry out research and testing of building components, construction details, and material assemblies to answer fundamental questions and provide necessary data for computational models.
- Explore the feasibility of base isolation and other energy dissipation techniques for the retrofitting of historic earthen architecture.
- Investigate methods of structural crack repair—stitching, grouting, and rebuilding—to identify the appropriate application and materials for each method.
- Identify materials that are compatible with earthen construction and that can be used for grouting, crack repair, and structural retrofitting. In particular, investigate soil-based grouts that could replace epoxies now commonly used. Carry out tests on injectability and penetration behavior.
- Define performance expectations for earthen building materials under dynamic conditions. Current performance standards are for static loads only.

4. Dissemination and Training

Research programs, such as GSAP, have identified appropriate methods for strengthening earthen buildings against earthquake damage. The pressing challenges are to disseminate this information throughout the diverse, earthquake-prone regions of the world and to train and support those who will implement these retrofit methods before the next major earthquake occurs. The following is a list of recommendations to facilitate better dissemination of the GSAP guidelines, as well as their adaptation to different cultural contexts:

- Use the Internet, which is becoming an accessible tool for growing numbers in the earthen architecture community. Publications on specialized topics, such as these proceedings, are often of limited print runs and tend to be expensive and difficult to obtain, particularly in the developing world. The Internet should be exploited to facilitate and encourage regional and international communication networks, as well as publications.
- Keep local building officials apprised of advances in research pertaining to earthen architecture.
- Support face-to-face exchange of information—a mode of communication that remains important. Seminars, colloquia, and international conferences should continue to be organized and supported.
- Develop an array of educational materials targeted to specific audiences. These audiences will range from academics and national policy makers to rural community members. Each audience will have its own needs, expectations, and limitations. Illustrations enhance understanding and can be a means of bridging between technical and nontechnical audiences.
- Establish a centralized database and Web site where interested parties can find appropriate methodologies, case studies illustrating best practices, model codes, and information on traditional knowledge and building techniques in active earthquake regions. This database could include an atlas of significant earthen buildings and prototypes, as well as a network of professionals working in the field.
- Integrate the engineering of earthen architecture into the curricula of existing academic programs in schools of engineering, architecture, architectural history, conservation, and allied fields, encouraging an interdisciplinary approach to teaching this subject at the university level.
- Strengthen links between professional activities and academic work by engaging schools of engineering, architecture, conservation, and construction in retrofitting projects.
- Address the challenge of persuading policy makers of the viability of reinforced earthen architecture in seismic zones.
- Capture and disseminate the intangible and oral traditions associated with earthen architecture in seismic regions. Include local people with traditional knowledge in this process.
- Encourage the two-way exchange of knowledge between traditional builders and professional "experts."

The GSAP colloquium provided the opportunity for a creative, multidisciplinary group of professionals working on the conservation of earthen sites in seismic regions to meet and discuss ideas and challenges and to collectively identify steps to advance the field. The ideas expressed in this summary of discussions will serve as the basis for designing the GCI's future work in this area. It is hoped that the colloquium discussions will also encourage other institutions, organizations, and practitioners to continue working to improve the preservation of earthen heritage sites in seismic areas throughout the world.

Appendix A: Steering Committees

GSAP 2006 Colloquium Steering Committee

John M. ("Jake") Barrow, Senior Exhibit Specialist, National Park Service, Intermountain Regional Office

Claudia Cancino, Project Specialist, Getty Conservation Institute

Anthony Crosby, Historical Architect; Chair, US/ICOMOS Scientific Committee for the Study and Conservation of Earthen Architecture; GSAP Advisory Committee Member

Stephen Farneth, Architect, Architectural Resources Group

William S. Ginell, Retired Senior Scientist, Getty Conservation Institute; former GSAP Project Director

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E. Leroy Tolles, Structural Engineer, ELT and Associates; GSAP Principal Investigator

Julio Vargas Neumann, Structural Engineer; GSAP Advisory Committee Member

Frederick A. Webster, Civil Engineer, Fred Webster Associates; GSAP Research Team Member

New Concepts in Seismic Strengthening of Historic Adobe Structures Steering Committee

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Appendix B: Colloquium Participants

Titles and affiliations of participants are given as of the time of the colloquium

First Name	Last Name	Title	Affiliation	Country
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Contributors

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Maribel Beas is a preservation architect and architectural conservator in private practice. She is a registered architect and received a master's degree in historic preservation from the University of Pennsylvania. A former chair of the US/ICOMOS Earthen Architecture Committee, she has a particular interest in the conservation of earthen historic structures and their decorated surfaces. As founder of the nonprofit organization Patrimonio Perú, Beas has secured several conservation grants for historic earthen structures in Peru.

Marcial Blondet is professor of civil engineering and dean of the graduate school at the Pontificia Universidad Católica del Perú (PUCP), where he graduated in 1973 as a civil engineer. He obtained a master of engineering (1979) and a PhD (1981) at the University of California, Berkeley (UCB). From 1992 to 1999 he worked as principal development engineer at UCB's Department of Civil Engineering, where he was in charge of all of the department's laboratories and conducted several full-scale tests of structural components.

Claudia Cancino is an associate project specialist in the Field Projects department of the GCI, where she works on the Earthen Architecture Initiative. She graduated in architecture and urban planning from Universidad Ricardo Palma in Lima, Peru, and earned a certificate in conservation at ICCROM in 1995. Cancino practiced preservation architecture and was on the faculty at the Universidad Peruana de Ciencias Aplicadas (UPC) in Lima (1996–99), teaching restoration of monuments and earthen building techniques. She earned a master of science in historic preservation (2001) and an advanced certificate in conservation (2002) from the University of Pennsylvania.

Charu Chaudhry is a conservation architect. She received an MS in historic preservation from the University of Pennsylvania and a BArch in India. Her master's thesis focused on the use of grouts as a structural repair technique for earthen buildings in seismic areas. She was a US/ICOMOS intern (2002 and 2005) and a Charles Wallace Conservation Fellow (2004). Chaudhry has worked on several field and research projects related to the documentation, risk assessment, and conservation of cultural heritage in India, Great Britain, and the United States.

Steade R. Craigo is a senior restoration architect with the California State Parks Office of Historic Preservation. He received his bachelor of architecture degree from

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Dina D'Ayala is a senior lecturer in structures in the Department of Architecture and Civil Engineering of the University of Bath, UK. She obtained a first degree and a doctorate in structural engineering from the Faculty of Engineering, Università degli Studi di Roma, La Sapienza. She has eighteen years of research experience and has written more than sixty international publications on the seismic behavior of historic monuments and on the seismic vulnerability of historic masonry buildings.

Dominic M. Dowling earned his doctorate at the University of Technology, Sydney. His dissertation, entitled "Seismic Strengthening of Adobe-Mudbrick Houses," involved extensive experimental testing coupled with field research and application, mostly in El Salvador. Dowling is currently engaged in disaster risk reduction initiatives in the Middle East, Asia, and Latin America.

Stephen Farneth is a founding principal of Architectural Resources Group and has thirty years of experience in the field of architecture and planning. His background includes training in architectural conservation from ICCROM. An expert in the design and rehabilitation of architecturally significant buildings and sites, he is currently vice chairman of US/ICOMOS.

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Mary Hardy is a senior project specialist for the Field Projects department of the GCI, where she manages the Earthen Architecture Initiative. She managed the institute's El Salvador Earthquake Relief Project (2001–2) and is a member of the international scientific advisory board for the development of building codes for traditional materials in Morocco. She holds an MArch from the University of California, Berkeley, and an MS in historic preservation from Columbia University. She also pursued postgraduate training in architecture and urban design for historic cities at the International Laboratory of Architecture and Urban Design (ILAUD) in Italy.

John Hurd received his BSc in the geology of clays, earned a two-year conservation science diploma at the University of Lincoln in the UK, and an "objects" higher national diploma, which included placements in the department of sculpture conservation at the Victoria and Albert Museum. In 2000 he started Hurd Conservation International. Hurd is a senior conservation consultant to the UNESCO World Heritage Centre. He chaired the ICOMOS-UK National Earth Committee from 1994 to 2000 and was elected president of the ICOMOS International Scientific Committee on Earthen Architectural Heritage in 2006.

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Hugh Morris is a senior lecturer at the University of Auckland, with primary responsibility for teaching introductory design, timber engineering, and the legal, ethical, and environmental aspects of professional engineering. Morris has conducted research on the seismic and durability performance of earth buildings and has served on committees developing the New Zealand earth building code. His current work is on a soil-cement fiber composite for low-cost earth buildings with specific appeal for the indigenous Maori people.

Patricia Navarro Grau is the principal of Patrimonio Perú, a nonprofit association dedicated to historic preservation in Peru, which she cofounded in 1999. She earned her bachelor of architecture degree at the Rhode Island School of Design and received a master's degree from the School of Architecture at the Universidad Politécnica de Madrid. She entered private practice in 1992 and has been involved in a number of public and private preservation projects in Lima.

Gail Ostergren is a research associate with the GCI, where she works with publications and provides research support to a variety of projects, including the Earthen Architecture Initiative, the Getty Seismic Adobe Project, and the Los Angeles Historic Resource Survey project. In 2005 she earned a PhD in history from the University of California, Los Angeles, where she specialized in urban, architectural, and cultural history.

Stefania Pandolfo is associate professor of anthropology at the University of California, Berkeley. Educated in Italy and the United States, she has lived an extended part of her life in Morocco. Her work has centered on cultural hermeneutics, memory, and the study of forms of knowledge and techniques—from poetry to building to traditional medicine, with a focus on Arab civilization and Islam and the question of postcoloniality. Her current research and publications include participation in the Morocco seismic project, and a study of transnational migration, youth, and the return of theological vocabularies.

Douglas Porter holds a master of science in historic preservation from the University of Vermont (UVT). From 2002–6, he was on the research faculty and served as the field study coordinator of the UVT graduate program in historic preservation. Between 2002 and 2007, he was the field services representative for the Preservation Trust of Vermont and the National Trust for Historic Preservation, offering technical assistance to Vermont communities working on historic preservation projects. Porter's background is in the building trades, including preservation work as a general contractor, and with the Architectural Conservation Projects Program of the National Park Service, Santa Fe Regional Office.

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E. Leroy Tolles is principal of ELT and Associates and holds a PhD in engineering from Stanford University. He conducts structural engineering research, analysis, investigation, and design of new and existing buildings, primarily with regard to their seismic performance. Currently the majority of his work consists of designing the retrofits for historic adobe buildings. He has also performed extensive investigations of earthquake-damaged wood-framed and earthen buildings. Tolles was principal investigator for the Getty Seismic Adobe Project.

Daniel Torrealva is principal professor of engineering at Pontificia Universidad Católica del Perú (PUCP), where he has taught since 1975. He graduated as a civil engineer from PUCP in 1972. He received his master's degree from the University of California, Los Angeles, in 1980. He is currently a member of the ICOMOS International Scientific Committee on Earthen Architectural Heritage and a member of ICOMOS-Perú. Since 1980 he has been involved in experimental research on the seismic resistance of adobe and brick masonry buildings. He is also member of the national committee for the elaboration of several adobe, seismic, and masonry building codes.

Julio Vargas Neumann is principal professor in the engineering department at Pontificia Universidad Católica del Perú. He started teaching in 1963 and became chief of the engineering department in 1969. Since 1970 he has led a research team for the study of earthen construction in seismic areas. He was professor of earthquake engineering and dynamics of structures. In 1985 he became vice minister of housing and was in charge of the dissemination and implementation of various programs supporting earthen construction across Peru.

Frederick A. Webster has a PhD in structural engineering from Stanford University and has researched, tested, designed, and developed building code standards for earthen construction since 1981. He participated in research and development of seismic upgrade techniques for existing earthen structures sponsored by the National Science Foundation during the 1980s and was a member of the Getty Seismic Adobe Project research team. He has designed seismic retrofits and upgrades for several historic and older adobe buildings in California.

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