ABSTRACT

ASEISMIC DESIGN OF ADOBE HOUSING

by

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Submitted to the Department of Civil Engineering on September 8, 1980
in partial fulfillment of the requirements for the degree of
Master of Science

A literature search was carried out to find what research has been
done on improving the seismic behavior of adobe housing, and the ade-
quacy of several building codes. It was found that little research
has been done on this topic, and that most of it is of little practical
significance. The building codes reviewed were found to be inadequate.

Ideas based on simple mechanical principles are suggested to im-
prove inexpensively the overall seismic behavior of adobe houses with
special emphasis on the prevention of roof collapse. Tables are pro-
vided to facilitate their implementation, and examples are given to
illustrate their use.

Recommendations are made, based on the present state of knowledge,
and suggestions are made for necessary future research.

Thesis Supervisor: H. Max Irvine
Title: Edgerton Associate Professor of Civil Engineering
Biographical Note

Pierre Hernando Montauban was born and raised in Lima, Peru. He attended M.I.T. from September 1976 to June 1980 and was awarded a B.S. degree in Civil Engineering. As an undergraduate he became a member of the M.I.T. Student Chapter of ASCE and of the honorary societies of Chi Epsilon and Tau Beta Pi. In his senior year he was awarded the Howe-Walker Student Award by ASCE for his services to the M.I.T. Student Chapter and the Richard Lee Russell award by M.I.T. for academic achievement. He is coauthor of the paper entitled "On Hot Air Balloons," to be published by the International Journal of Applied Mechanics.
Acknowledgements

I wish to express my gratitude to my advisor, Professor H. Max Irvine, for his continuous help and encouragement; to Mrs. Malinofsky, for her patient job in typing the manuscript; and to the several people who contributed by sending relevant material. Only with the help of all these people was this thesis made possible.
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<td></td>
</tr>
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<td>l</td>
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<td></td>
</tr>
<tr>
<td>l_r</td>
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\( \lambda_w \) length of each bracing wire

\( M \) bending moment

\( M_{A_f} \) moment of area \( A_f \) about 0 (see Fig. 22b)

\( M_{B_f} \) moment of area \( B_f \) about 0 (see Fig. 22b)

\( M_u \) ultimate moment-resisting capacity of wood section

\( n \) number of wires required

\( P \) concentrated load, axial load

\( P_{\text{max}} \) maximum transverse concentrated load which can be equilibrated by the self-weight of the wall (see Fig. 20)

\( P_u \) ultimate axial compression strength of wood section

\( r_b \) radius of wall bond beam

\( r_p \) radius of wood posts

\( T \) total tension to be taken by wire bracing

\( t \) wall thickness

\( V \) shearing force

\( v \) shear stress

\( v_f \) shear stress at fracture

\( W \) weight of the roof

\( W_i \) internal work

\( W_e \) external work

\( w_r \) weight of roof per unit area

\( z \) height of the wall over the point being considered

\( \alpha \) angle to the horizontal at which cracks appear in adobe walls (see Fig. 3)

\( \beta \) angle between bracing wires and horizontal in the plane of the roof (see Fig. 3)

\( \gamma \) total shear deformation
\( \gamma_a \) specific weight of adobe
\( \Delta, \delta \) total deflection
\( \delta_c \) unit deformation of compressed diagonal
\( \delta_f \) deflection due to flexure
\( \delta_s \) deflection due to shear
\( \delta_t \) unit deformation of tensed diagonal
\( \Delta \delta \) increment in deflection due to openings in wall
\( \theta, \phi \) angles used to describe rotations when applying virtual work principles (see Fig. 21)
\( \lambda \) ratio of wall height to wall length (= h/k)
\( \nu \) Poisson's ratio
\( \sigma_c \) compression strength (kg/cm\(^2\))
\( \sigma_t \) tensile strength (kg/cm\(^2\))
\( \sigma_{tf} \) tensile strength in flexure tests (kg/cm\(^2\))
\( \tau \) shear strength (kg/cm\(^2\))
INTRODUCTION

Soil in the form of adobe bricks has for centuries been one of the most common building materials in several parts of the world. Even now, its economy and ease of construction make adobe housing irreplaceable in the rural areas of less developed countries (LDC's). Unfortunately, due to social, cultural and economic factors, even in the seismic regions of LDC's, adobe houses are built with little or no engineering input. As a result of this, the collapse of unreinforced masonry and adobe housing accounts for about 90% of the more than one million earthquake-related casualties of this century [1].

It is clear that modern construction methods will remain unviable in LDC's for the next few decades. In the meantime it is of utmost importance that research should be done on improving the seismic behaviour of adobe housing. This research should be oriented towards devising simple solutions which utilize cheap local materials and can be implemented by the unskilled local laborer on a self-help basis. In this way construction costs will remain low, and the local people will in time require some more meaningful construction skills.

The behaviour of adobe structures is complex, erratic and hard to model, and so it doesn't lend itself to the rigors of mathematical or computer dynamic analysis. This area of research is also plagued with social, cultural, economic, financial and technological problems and has failed so far to significantly stimulate the interest of the academic communities in both LDC's and in the more developed nations.

Despite the complexity of adobe construction, three common factors can be distinguished in the failure of these structures:
1) Heavy roofing system with no capacity for deformation
2) Brittle behavior of the walls
and 3) Poor connections between walls and roof.

This thesis is aimed at suggesting solutions to these three general problems.

The subject matter of this thesis is organized into 4 chapters.

Chapter I contains a summary of some of the research already done in the field of adobe construction, with discussions regarding their relevance and significance.

Chapter II provides information in tabular form, on the specifications provided by different codes on adobe construction. These code provisions are also discussed.

Chapter III is devoted to new suggestions developed by the author on how to improve the seismic behavior of adobe housing. Particular emphasis is placed on prevention of roof collapse. The shear resistance of walls under moderate earthquakes is also examined.

Chapter IV establishes the conclusions obtained and gives recommendations for future research.
CHAPTER I - CHARACTERISTICS OF ADOBE HOUSES

Adobe bricks are widely used for construction in LDC's. Sizes, composition, and methods of manufacture vary from place to place.

Bricks are molded from a clay-based mixture with a high enough water content to produce a plastic or workable consistency that allows the material to be formed in simple molds. Optimum combinations of sand, silt and clay have to be developed on the bases of local materials, and no hard and fast rules respecting combinations can be given.† Experience and the "feel" of the mixture are the criteria. Traditionally the adobe bricks are reinforced with straw or other fibers, but opinions differ as to their value.

The mixture of mud and straw is usually placed by hand in simple wood single- or multi-cavity molds. Shortly afterwards the mold and brick are placed on a suitable bed and the mold is removed, leaving the brick to dry in the sun. Once dried, the bricks are laid up in the wall in much the usual fashion, with horizontal courses and with vertical joints in successive courses staggered or "broken" to avoid long continuous vertical joints. Mortars are commonly of construction similar to that of the bricks. The roof is usually supported solely by the walls.

Low shrinkage is one of the chief advantages of adobe brick, since the shrinkage occurs while the bricks are drying and before they are built into the wall. Single bricks dry rather quickly, usually in about two weeks, so there is no long drying period. Another advantage is the uniform size of the bricks, which permits them to be laid accurately [2].

† One way of determining the adequacy of a given soil for making adobe bricks is by making a rod with little water and one finger thick and measure the length of it that can be cantilevered before the rod breaks. For a good soil this length should be between 5 and 15 cms. [16].
A major disadvantage of adobe construction is the large amount of labor it entails, since the heavy material has to be handled twice. First the mud is dug, mixed, formed into bricks and dried. Then the bricks must be rehandled and transported to the building site. The need for protecting the bricks from rain during the drying and hardening period is another disadvantage. Once the bricks are dry, damage due to rain is not so important. For the finished house the adobe walls are protected from the rain by overhanging roofs and whitewashing.

Adobe has two major structural disadvantages: it is a brittle material, and it has a low strength-to-weight ratio. Adobe houses are thus very heavy, and during an earthquake they experience strong inertial forces. This is further aggravated by the presence of heavy roofs which are needed for thermal insulation—most adobe construction is done in areas where extreme cold or hot temperatures are common. Once the material strength of adobe is exceeded (usually in plain or diagonal\(^\dagger\) tension) brittle fracture occurs, and the collapse of the walls and roof follows almost instantaneously. Usually there is little structural integrity, and the roof is not designed to carry any lateral loads. Roof collapse is the main cause of human injuries.

Doors and windows are usually placed with little regard to structural soundness. Quite frequently they are not properly framed, and this results in stress concentrations at the corners. Failure of window or door lintels are common reasons for roof collapse. A further mechanism of failure is by overturning of the walls transversal to the seismic motion. This occurs because of the lack of proper ties between the

\(^\dagger\)Diagonal tension occurs at 45° to the direction of principal shear and is caused by it. For adobe houses fracture occurs in stepped fashion along the joints which are the weakest section of the wall.
walls at the corners. Connection between perpendicular walls is usually limited to the interlocking of bricks and the adherence of mortars.

Material Properties. An experimental program was carried out at the Pontificia Universidad Católica del Perú to determine the material properties of adobe [3,4,5,6,7]. Their results are listed below.

1. **Compression Tests** [3,4] (8 cm. cubes with drying periods of one month and one year).

<table>
<thead>
<tr>
<th>Drying time</th>
<th>Number of samples</th>
<th>Average $\sigma_c$</th>
<th>Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 month</td>
<td>6</td>
<td>14.8 kg/cm$^2$</td>
<td>1.4 kg/cm$^2$</td>
</tr>
<tr>
<td>1 year</td>
<td>6</td>
<td>14.4 kg/cm$^2$</td>
<td>2.0 kg/cm$^2$</td>
</tr>
</tbody>
</table>

Brittle failure was observed in all specimens. After a drying period of one month the length of this period ceases to be a significant variable.

2. **Tension Tests** [3,4] (Splitting test cylinders of 15 cms. diameter and 30 cms. long).

<table>
<thead>
<tr>
<th>Average of samples</th>
<th>Average $\sigma_f$</th>
<th>Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>1.12 kg/cm$^2$ (about 8% of $\sigma_c$)</td>
<td>0.24 kg/cm$^2$</td>
</tr>
</tbody>
</table>

3. **Flexure Tests** [3,4] (Adobe bricks of dimensions 20 x 40 x 8 cms and 30 x 60 x 8 cms).

<table>
<thead>
<tr>
<th>Drying time</th>
<th>Number of samples</th>
<th>Average $\sigma_{tf}$</th>
<th>Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 month</td>
<td>5</td>
<td>3.4 kg/cm$^2$</td>
<td>0.5 kg/cm$^2$</td>
</tr>
<tr>
<td>1 year</td>
<td>5</td>
<td>4.4 kg/cm$^2$</td>
<td>0.7 kg/cm$^2$</td>
</tr>
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</table>
Bricks dried for one year resisted a load up to 20% larger than when dried for only a month. The results of flexural tests can be used only on a comparative basis since, due to the arch effect of the uncracked section, $\sigma_{tf}$ is considerably larger than the real tensile strength $\sigma_t$.

4. Axial Compression Tests on Adobe Piles [3,4]. (Piles made with eight adobe bricks of dimensions 20 x 40 x 8 cms. and formed by mortar. Pile height 80 cms).

a) Load perpendicular to principal joints (see Fig. 1a).

<table>
<thead>
<tr>
<th>Number of samples</th>
<th>Average $\sigma_c$</th>
<th>Standard deviation</th>
<th>Average $E$</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>8.32 kg/cm$^2$</td>
<td>0.71 kg/cm$^2$</td>
<td>1,000 kg/cm$^2$</td>
</tr>
</tbody>
</table>

b) Load parallel to principal joints (see Fig. 1b).

<table>
<thead>
<tr>
<th>Number of samples</th>
<th>Average $\sigma_c$</th>
<th>Standard deviation</th>
<th>Average $E$</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>5.11 kg/cm$^2$</td>
<td>0.55 kg/cm$^2$</td>
<td>2,400 kg/cm$^2$</td>
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</table>

c) Load parallel to principal parts (see Fig. 1c).

<table>
<thead>
<tr>
<th>Number of samples</th>
<th>Average $\sigma_c$</th>
<th>Standard deviation</th>
<th>Average $E$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>4.70 kg/cm$^2$</td>
<td>0.75 kg/cm$^2$</td>
<td>1,500 kg/cm$^2$</td>
</tr>
</tbody>
</table>

Young's modulus was defined as the secant modulus at half the failure load. These tests clearly demonstrate that adobe walls are not isotropic. However, for practical purposes Vargas
FIG. 1. Configuration of Axial Compression Tests on Adobe Piles Joined by Mortar. (a) Load Perpendicular to Principal Joints, (b) and (c) Two Variations on Tests with Load Parallel to Principal Joints.
Neumann [3,4] on the basis of finite element solutions, suggests using only one modulus of elasticity equal to 1,700 kg/cm². It is also clear that the compressive strength of adobe walls is considerably lower than that of individual bricks. This is because of the weaker mortar and the fact that the weakest brick determines the strength of the pile.

5. Diagonal Compression Tests [3,4]. Wall sections of 60 x 60 x 20 cms. made of adobe bricks of dimensions 20 x 40 x 8 cms. (see Fig. 2).

<table>
<thead>
<tr>
<th>Number of samples</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average ( v_f )</td>
<td>0.27 kg/cm²</td>
</tr>
<tr>
<td>Standard deviation on ( v_f )</td>
<td>0.03 kg/cm²</td>
</tr>
<tr>
<td>Average ( G )</td>
<td>378 kg/cm²</td>
</tr>
<tr>
<td>Standard deviation on ( G )</td>
<td>319 kg/cm²</td>
</tr>
</tbody>
</table>

where
\( v = \text{shear force/diagonal area} \)
\( v_f = \text{shear stress at fracture} \)
\( \gamma = \text{shear deformation} = |\delta_c| + |\delta_t| \)
\( \delta_c = \text{unit deformation of compressed diagonal} \)
\( \delta_t = \text{unit deformation of tensed diagonal} \)
\( G = \frac{v}{\gamma} \) taken when \( v = \frac{v_f}{2} \).

These tests are hard to perform and give very dispersed data as shown by the coefficient of variation in the values of \( G \), which is almost 1. The practicality of these tests is thus highly questionable.
FIG. 1. Configuration of Diagonal Compression Tests.
6. **Shear Tests on Walls [3,4].** (Walls of 2.4 x 2.4 and 2.4 x 4 m with thicknesses of 20 and 30 cms).

Shear stresses were induced in the wall by a lateral load in plane with the wall applied by a hydraulic jack. (A similar loading arrangement is shown in Fig. 7). The results of 21 samples show a good fit to the formula:

\[ \tau = 0.09 + 0.55\sigma \]  (1.1)

An initial elastic stage was observed where as predicted by elastic theory the effect of axial compression on the value of the wall stiffness is negligible. See Appendix I and references [3,4] for further details.

7. **Tests on Flexural Behavior of Walls [3,4,5].** (Walls of dimensions 2.4 x 4 m and thickness of 20 and 30 cms).

The walls were flexed by loads perpendicular to the plane of the wall applied by a hydraulic jack. Two stages were identified in the flexural behavior of the wall.

a) **Elastic Stage.** In this stage the principles of elastic theory can be applied and the wall can be assumed to be fixed at the base. This stage ends when cracking occurs. In the experiments this happened at the load values given below:

<table>
<thead>
<tr>
<th>Number of samples</th>
<th>1.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average ( \sigma_{tf} )</td>
<td>1.42 kg/cm(^2)</td>
</tr>
<tr>
<td>Standard deviation</td>
<td>0.31 kg/cm(^2)</td>
</tr>
<tr>
<td>Recommended E for elastic analysis</td>
<td>1,700 kg/cm(^2)</td>
</tr>
</tbody>
</table>

It was noticed that once more, in flexure the strength of individual bricks is substantially higher than that of an entire wall. In fact, mortar joints are probably incapable of taking any significant amount of tension and the values given above are really a consequence of ignoring the compressive self-weight load of the wall.
b) **Plastic Stage.** In the tests performed in vertical walls an "apparent ductility" higher than 4 was observed in all cases. This apparent ductility, defined as the ratio of the deformations at collapse load and fracture load, is not a material property, but is due to the stability of the cracked wall from its own weight. Thus, at a constant load, these gravitational forces maintain the equilibrium of the system at deformations of over 400% of the elastic deformation. The understanding of the real nature of this phenomenon permits the correct utilization of yield line theory and virtual work principles to calculate the collapse load for a wall in flexure. The principal considerations in using this method are:

I. The moment acting along the fracture lines is equal to \( \gamma_a b^2 z/2 \), where \( \gamma_a \) is the specific weight of adobe, \( b \) the thickness of the wall, and \( z \) is the height of wall over the point being considered.

II. Restoring moments will be present only in the horizontal sections of inclined (stepped) fracture lines, which is where the gravitational forces act. No restoring moment develops in vertical fracture lines.

III. The moment acting in an inclined fracture line is equal to \( M \cos^2 \alpha \), where \( \alpha \) is the angle the fracture line makes with the horizontal and \( M \) is the average moment mentioned in I.

When figuring out the pattern in which fracture lines are likely to occur, it is useful to remember that fracture usually occurs in a stepped fashion. Thus in a wall with bricks of dimensions of 20 x 40 x 8 cms fracture lines will tend to form at an angle of 27° with the horizontal, while in walls made with bricks of 30 x 60 x 8 cms this angle will be 19° (see Fig. 3).

\[ \gamma_a \] It is important to note that during load reversal the load-deformation diagram will just retrace itself and there will not be any area enclosed by the hysteresis loop; i.e., the wall does not have the capacity to dissipate energy.
1. Fracture line in homogeneous materials.
2. Overall slope of fracture line in adobe walls. The value of angle $\alpha$ depends on the size of the bricks and is given below for the two most common brick dimensions.

<table>
<thead>
<tr>
<th>Brick Dimensions</th>
<th>$\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 x 40 x 8 cms</td>
<td>27°</td>
</tr>
<tr>
<td>30 x 60 x 8 cms</td>
<td>19°</td>
</tr>
</tbody>
</table>

FIG. 3. Typical Angles for Fracture Lines in Adobe Walls.
An example of the application of this method to a simple case is shown in Appendix II. Applying this method to the pattern of fracture lines observed in the tests, with small idealizations, the simple empirical formulas shown in Fig. 4 were deduced. These formulas can be of great practical importance. For preliminary design the following formula is suggested:

\[ C = \frac{n t}{h} \]

where \( C \) is the seismic coefficient of rupture (horizontal acceleration in terms of \( g \)), \( n \) the number of supported sides in the wall, and \( h \) the height of the wall.

In vertical tests the increase in ductility caused by adding cane reinforcement is masked by the "apparent ductility" mentioned above. However, from the results of horizontal tests the increase of ductility due to the cane reinforcement is estimated to be on the order of 50 to 100%.

8. **Tests on Modules [4,6]** (Rooms 4 x 4 m and 2.4 m high. Wall thickness of 20 cms).

Static tests were performed on 21 modules simulating horizontal loads by tilting the modules so that the gravitational forces would have a component in a direction parallel to the structure's floor [4,6].

For the purpose of simplifying the analysis of these modules, it was found convenient to identify, based on the fracture pattern, its constitutive pieces, which could then be treated separately. From the observations of several tests, five distinct pieces or prototypes were identified, and these are shown in Fig. 5.

In the first four prototypes, flexural behavior predominates, and the transverse walls do not contribute to the shear resistance of the structure. For prototype V, where the walls are less slender and, despite openings, better structural integrity is provided, shear behavior predominates. Here the transverse walls, especially when
C = \frac{7t}{2h} \text{ for } \frac{2h}{3} \leq \ell \leq \frac{5h}{3} \quad (1.2)

(a) Walls supported by the floor and two transverse walls at the edges.

C = \frac{9t\ell}{h(3h-\ell)} \text{ for } h \leq \ell \leq 2h \quad (1.3)

C = \frac{4t}{h} \text{ for } \ell \leq h

(b) Walls supported as in (a) plus by a rigid wall bond beam at the top.

C = \frac{9th}{\ell(3h-\ell)} \text{ for } \ell \leq h \quad (1.4)

(c) Walls supported by only the floor.

C = \frac{t}{h} \quad (1.5)

FIG. 4. Seismic Coefficients of Rupture for Walls of Different Geometries. (All walls of thickness t).
FIG. 5. Crack Patterns in Different Adobe Prototypes.
they support significant axial compression concurrently, contribute to the shear resistance of the unit. Suggested criteria to determine the effective shear-resisting area of adobe modules are shown in Fig. 6. The use of these criteria is consistent with the formula \( \tau = 0.09 + 0.55\sigma \) given before.

These tests show the necessity of using some kind of reinforcement to guarantee structural integrity and to provide the structure with a minimum ductility. Horizontal cane reinforcement does not improve the shear resistance of the longitudinal walls, but prevents the collapse of transverse walls and retards the formation of vertical shear cracks.

9. Tests on Mortars [7]. To find the influence of the composition of the mortar on the lateral strength of the wall, tests were performed on 14 walls of dimensions 2.4 x 4.0 cm and thicknesses of 20 and 30 cms. These tests consisted in applying a horizontal load at two-thirds of the wall height by means of an hydraulic jack. The load is distributed by a wood block 80 cms in length. These walls were placed in a concrete base with no other lateral restraint than the one provided by friction and the adherence of a layer of mortar. Fig. 7 shows the configuration of the tests as well as the typical fracture pattern observed. The results are shown in Table 1.

The improvements in the lateral strength of the wall caused by superior mortars is evident from looking at Table 1. It is interesting to note that with the cement-sand mortars, it is usually the weaker mortar (smaller ratio of cement to sand) that performs better. This may be due to the absorption by the adobe of the necessary water for the setting of the cement, a problem which may be aggravated in richer mortars. Another possible explanation is the reduced shrinkage in low-cement-content mortars. Thus, the strength of the mortar seems to be a bad indicator of its performance as a binding material.

Very similar results were obtained from cement-gypsum-sand and cement-
(a) Walls not properly joined above the openings to exhibit structural integrity (Prototypes I through IV).

(b) Walls with or without openings but properly reinforced to insure structural integrity (Prototype V).

(c) Cases in which the compressed transverse wall contains substantial openings or longitudinal walls ending in T or L shape.

FIG. 6. Areas to be Considered as Shear Resistant (shown shaded in above plan views).
FIG. 7. Configuration of Tests on Mortars. (Also showing typical fracture patterns).
Table 1 - Influence of Mortar Composition on the Lateral Strength of Walls [7].

1. **Cement and sand mortars**

<table>
<thead>
<tr>
<th>Test</th>
<th>Shear Stress at rupture (kg/cm²)</th>
<th>Deformation at rupture (cm)</th>
<th>Mortar Proportions</th>
<th>Fracture Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.33</td>
<td>0.10</td>
<td>1:3</td>
<td>1 *</td>
</tr>
<tr>
<td>2</td>
<td>0.43</td>
<td>0.16</td>
<td>1:3</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>0.48</td>
<td>0.49</td>
<td>1:4</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>0.38</td>
<td>0.22</td>
<td>1:4</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>0.54</td>
<td>0.26</td>
<td>1:5</td>
<td>1 and then 2</td>
</tr>
<tr>
<td>6</td>
<td>0.61</td>
<td>0.25</td>
<td>1:5</td>
<td>1 and then 2</td>
</tr>
</tbody>
</table>

2. **Cement, gypsum and sand mortars**

<table>
<thead>
<tr>
<th>Test</th>
<th>Shear Stress at rupture (kg/cm²)</th>
<th>Deformation at rupture (cm)</th>
<th>Mortar Proportions</th>
<th>Fracture Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>0.45</td>
<td>0.11</td>
<td>2:1:5</td>
<td>1 and then 3</td>
</tr>
<tr>
<td>8</td>
<td>0.59</td>
<td>0.26</td>
<td>2:1:5</td>
<td>1</td>
</tr>
<tr>
<td>9</td>
<td>0.55</td>
<td>0.09</td>
<td>1:2:10</td>
<td>1 *</td>
</tr>
<tr>
<td>10</td>
<td>0.47</td>
<td>0.13</td>
<td>1:2:10</td>
<td>1 and then 2</td>
</tr>
</tbody>
</table>

3. **Cement, lime and sand mortars**

<table>
<thead>
<tr>
<th>Test</th>
<th>Shear Stress at rupture (kg/cm²)</th>
<th>Deformation at rupture (cm)</th>
<th>Mortar Proportions</th>
<th>Fracture Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>0.59</td>
<td>0.27</td>
<td>2:1:10</td>
<td>2 and then 1</td>
</tr>
<tr>
<td>12</td>
<td>0.43</td>
<td>0.10</td>
<td>1:2:10</td>
<td>1</td>
</tr>
<tr>
<td>13</td>
<td>0.51</td>
<td>0.13</td>
<td>1:2:10</td>
<td>2 and then 1</td>
</tr>
</tbody>
</table>

4. **Gypsum and mud mortars**

<table>
<thead>
<tr>
<th>Test</th>
<th>Shear Stress at rupture (kg/cm²)</th>
<th>Deformation at rupture (cm)</th>
<th>Mortar Proportions</th>
<th>Fracture Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>0.25</td>
<td>0.03</td>
<td>1:4</td>
<td>1 *</td>
</tr>
</tbody>
</table>

*Some broken bricks.*
lime-sand mortars, with the mortars richer in cement performing only marginally better.

The addition of small quantities of gypsum to the mortar (1:4 gypsum-mud ratio) increased the lateral resistance of the wall by up to 25%.

An important advantage of using superior mortars is the sharp increase in ductility they can provide to the wall, which allows for the possibility of combining them with ductile materials to obtain a better overall monolithic behavior.

Despite the clear influence of the mortar's composition in the behavior of adobe structures, it is not clear whether this area of research may prove to be of practical importance in the rural areas where the composition of the mortar is dictated by the materials locally available. However, applications might be found in urban areas where the required additives are available. With this last consideration in mind, further research in this area might pay off.

This chapter reviewed the results of several tests performed to determine the material properties of adobe. In the next chapter a summary and discussion of some building codes will be presented.
CHAPTER II - CODE PROVISIONS ON ADOBE CONSTRUCTION

As mentioned before, adobe houses are usually built on a self-help basis by people with little or no knowledge of engineering principles. Building codes are therefore needed to guide these people in the construction of adequate housing. To serve their purpose, however, these codes should be made keeping in mind the low level of education of their future users. They should therefore be simple, clear, but reasonably comprehensive.

The purpose of this chapter is to examine the adequacy of the building codes of some LDC's. The building codes of Chile [8], China [9], Mexico [10] and Peru [11, 12] were chosen for this study because of their availability and, more important, because in all these four countries adobe construction is widespread in rural areas and the occurrence of severe earthquakes is common.

Surprisingly, it was found that the Mexican building code does not have any provisions on adobe construction. In the Chilean building code there is only one provision regarding adobe housing, which limits the height of adobe houses to one story or 3 m. In both these countries adobe construction is "not-recommended." On the other hand, the Chinese and Peruvian codes are more realistic and acknowledge the large amount of construction with adobe going on in both countries, and the consequent need to regulate it. The specifications of these two codes are summarized in Table 2.

Looking at this table, we can notice that the Chinese and Peruvian building codes complement each other to a significant extent and agree reasonably well in the provisions they share in common. This suggests
the possibility of producing a better, more comprehensive international
code on adobe construction by combining the provisions of several national
(or regional) codes.

Comparing the test results of Chapter 1 with the provisions of the
Peruvian code, the latter were found to be very conservative. For in-
stance, the factor of safety in the allowable compressive stress for
adobe walls could be as high as 5 (8.32/1.6). The formula for allowable
shear stress given in the code is also very conservative, and it prac-
tically neglects the beneficial effect of compressive stresses in the
shear strength of an adobe wall.

In both tabulated codes it was found that in general the provisions
concerning dimensional restrictions or material properties are specific,
while provisions dealing with the overall structural system tend to be
vague and therefore of little guidance to the user. There is an urgent
need for research in structural solutions which can be easily described
in detail by codes. Some suggestions for these will be given in the
next chapter.
Table 2 - Summary of the Specifications on Adobe Construction Found in the Chinese and Peruvian Building Codes.

<table>
<thead>
<tr>
<th>Specification on:</th>
<th>Chinese Code†</th>
<th>Peruvian Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing Soil:</td>
<td>Bearing walls should be designed to give uniform subsidence.</td>
<td>- The bearing strength of the soil should not be less than: 2.0 kg/cm² in zone 1 (the most seismically active, see reference [11]) unless the adobe is stabilized.†† 1.0 kg/cm² in other zones. Construction can be done in soils with bearing strengths between 0.5 and 1.0 kg/cm² provided that the seismic coefficient used in design is by a factor: [ \gamma = 2.0 - \alpha_b ], where ( \alpha_b ) is the bearing strength of the soil in kg/cm².</td>
</tr>
<tr>
<td>Material Properties:</td>
<td>- It is recommended that the dry density of lime-mud wall should not be less than 1,600 kg/m³.</td>
<td>- For design purposes the density of adobe should be taken as 1,600 kg/m³.</td>
</tr>
<tr>
<td></td>
<td>- The compressive strength of lime-mud walls shall not be less than 15 kg/cm³.</td>
<td>- In the absence of tests, the allowable stress values to be used in design of adobe walls are: Axial compression: 1.6 kg/cm². Compression in contact: 2.0 kg/cm². Shear: [ v = 0.054 + 0.035 \sigma ] where: ( v ) = allowable shear stress and ( \sigma ) = compressive stress, both in kg/cm².</td>
</tr>
<tr>
<td></td>
<td>- It is not recommended to use different wall materials in the same house.</td>
<td></td>
</tr>
<tr>
<td>Specification on:</td>
<td>Chinese Code&lt;sup&gt;+&lt;/sup&gt;</td>
<td>Peruvian Code</td>
</tr>
<tr>
<td>------------------</td>
<td>--------------------------</td>
<td>---------------</td>
</tr>
<tr>
<td>Maximum Building Height:</td>
<td>- 6m for design intensity of MM VII or less (adobe construction is not recommended for higher design intensities).</td>
<td>- Tensile strength of mortar: 0 kg/cm&lt;sup&gt;2&lt;/sup&gt;.</td>
</tr>
<tr>
<td>Overall Shape and Structural Restrictions:</td>
<td>- Split levels and sudden elevation changes should be avoided</td>
<td>- Tensile strength of cane: 250 kg/cm&lt;sup&gt;2&lt;/sup&gt;.</td>
</tr>
<tr>
<td></td>
<td>- Each bay should be provided with transverse walls.</td>
<td>- 3m from floor to wall bond beam</td>
</tr>
<tr>
<td></td>
<td>- Same story levels should be maintained in a house unit.</td>
<td>- 4m from floor to roof top.</td>
</tr>
<tr>
<td>Restrictions on Dimension and Aspect Ratios:</td>
<td></td>
<td>- Only 1-story construction is allowed.</td>
</tr>
<tr>
<td>a) Individual Adobe:</td>
<td>none</td>
<td>none</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- The ratio of length to height should be more than:</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 for normal adobe and 3 for stabilized adobe.</td>
</tr>
<tr>
<td>b) Walls:</td>
<td>- The minimum wall thickness shall be 25 cms.</td>
<td>- The minimum wall thickness shall be the larger of:</td>
</tr>
<tr>
<td></td>
<td>- Each story shall have a wall bond beam which shall be carried through all transverse walls. The sectional height of such a beam shall be:</td>
<td>1/8 of the wall height between horizontal bracing elements, or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1/12 of the distance between vertical bracing elements.</td>
</tr>
<tr>
<td>Specifications on:</td>
<td>Chinese Code†</td>
<td>Peruvian Code</td>
</tr>
<tr>
<td>------------------</td>
<td>---------------</td>
<td>---------------</td>
</tr>
<tr>
<td>RC, 8-12 cms.</td>
<td>- The distance between a free end of a wall and its nearest vertical bracing element should not exceed 0.4 times the height of the wall between horizontal bracing elements.</td>
<td></td>
</tr>
<tr>
<td>Reinforced brick, 4-6 courses.</td>
<td>- The vertical side of an unframed door or window can be considered as a free end.</td>
<td></td>
</tr>
<tr>
<td>Wood, 6-10 cms.</td>
<td>- If the roof is designed to transmit horizontal forces it should be adequately rigid and it should provide structural integrity and transmit the gravity and horizontal loads.</td>
<td></td>
</tr>
<tr>
<td><strong>Roofing:</strong></td>
<td>- When the roof is not designed to transmit horizontal forces the functions described above should be performed by wall bond beams.</td>
<td></td>
</tr>
<tr>
<td>Wood pads shall be provided for purlin supports.</td>
<td>- Unless the wall is adequately reinforced it should be braced by vertical elements like walls and/or columns and by horizontal members such as a rigid roof or a wall bond beam.</td>
<td></td>
</tr>
<tr>
<td><strong>Bracing:</strong></td>
<td>- The length of a bracing wall should not be less than 3/4 of its height unless it has vertical reinforcement properly anchored to the foundations.</td>
<td></td>
</tr>
<tr>
<td>Each story shall have a wall beam which shall be carried through all transverse walls. The required sectional heights of these beams were listed above.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Specification on:  Chinese Code†  Peruvian Code

Reinforcement Connections and Other Construction Details:

- Interior and Exterior walls shall be built simultaneously and be well bonded.
- At the corners of exterior walls and at the intersections of interior and exterior wall, tying material (i.e.: bamboo bars, wooden rods, reeds, etc.) shall be provided at 30-cm intervals along the wall height. These materials shall extend into the wall a length of 1m or more.
- Any material with reasonably stable properties and with behavior compatible with the adobe can be used as reinforcement.
- Sun-dried strips of ripe hollow cane can be used as reinforcement provided that necessary precautions are taken to avoid the detrimental effects of humidity.
- Lap joints and anchors should take the total tension to which the reinforcement is subjected.
- The mortar surrounding the reinforcement should be the same as that used between the adobe bricks.

† The specifications in the Chinese building code refer specifically to lime-mud bearing walls; that is, one in which lime is added to a wall built of mud, or a wall of adobe bricks with a lime mortar.

†† According to the Peruvian building code the adobe should be considered stabilized when it contains additives whose objective is to improve the resistance of the adobe to humidity.

††† If tests can be performed, the allowable stresses to be used in the design of adobe houses should be taken as:

- Axial Compression: 1/5 of the average compression strength of at least three piles made by adobes joined by mortar and with a height-to-thickness ratio of 2.
- Compression in contact: 1.25 times the allowable stress in axial compression.
- Shear: $\gamma = 0.45 \tau$

(footnote continued on next page)
where  \( \tau = u + f\sigma \) = shear strength \((\text{kg/cm}^2)\)
\( u = \) cohesion intercept \((\text{kg/cm}^2)\)
\( f = \) coefficient of apparent friction
\( \sigma = \) compressive stress \((\text{kg/cm}^2)\)

The parameters \( u \) and \( f \) are to be determined from shear tests as shown in the figure below.

At least six tests should be performed, three with \( \sigma = 0 \) and three with \( \sigma = 0.5 \) to \( 1 \, \text{kg/cm}^2 \). The values of \( u \) and \( f \) will then be determined so as to best fit the data.

- Allowable tensile stress of cane: \( 1/4 \) of the average value obtained from at least three tensile tests.
CHAPTER III

IMPROVING THE SEISMIC RESISTANCE OF ADOBE HOUSING

The importance of improving the seismic resistance of adobe housing was established in the Introduction. Unfortunately, little of the research presently being done concentrates on the two most important problems, which are: 1) How to prevent collapse of the roof, and 2) How to provide the adobe house with some ductility and structural integrity. Quite a lot of time, effort and money have been spent trying to determine and improve the material behavior of adobe, which is an area of research with little room for significant (and easy to implement) new solutions. A better approach to the problem is to adopt structural solutions, where objectives are easier to identify and can be dealt with individually to obtain the best solution given the economic and other constraints that must be applied.

Following Razani's recommendations \cite{1,13} the overall objectives in the design of adobe housing should be that: 1) No structural damage should occur under any earthquake having a local intensity less or equal to MM VII. (For a description of the Modified Mercalli scale see Appendix III), and 2) No roof collapse should occur under any earthquake having a local intensity less or equal to MM IX.

When designing for an earthquake of given intensity, it is convenient, for the sake of generality, to use the maximum response acceleration when calculating the inertial forces. This eliminates the need for producing an adequate response spectrum and for determining the period of the structure. The maximum response acceleration can be obtained with
the necessary accuracy from the simple formula: [1,13]:

\[ a_r = 0.015 \times 10^{0.31}, \]  

(3.1)

where \( a_r \) is in m/sec\(^2\) and I in MM scale. Thus the maximum response accelerations for earthquakes of intensities VII and IX will be 1.9 and 7.5 m/sec\(^2\) respectively (i.e., 19% g and 75% g).

**Roofing System**

To prevent collapse of the roof during a seismic event, two things need to be accomplished: 1) The roof should be provided with some in-plane stiffness; and 2) the adobe house should be provided with an auxiliary structural system capable of holding the roof loads after the walls have become too damaged to do so.

Adobe houses are usually built with peaked roofs supported by wood trusses spanning in the short direction. The roofs thus tend to be strong in this direction, but very weak in the long direction. A solution to this problem is to use diagonal wire bracing as shown in Fig. 8. To compute the required tension these wires must provide, the roof can be analyzed as having a single degree of freedom in the horizontal direction—an in-plane shearing motion along the roof. Lumping the mass of the roof at its center of gravity and taking moments about one of the lower corners of the roof, the required tension on each set of wires is found to be approximately

\[ T = \frac{c}{2} \frac{W}{\ell_{r}} \sec \beta \]

(3.2)

where \( c = a_r/g, W \) is the total weight of the roof, \( w_r \) the weight of the roof per unit actual area and \( \ell_{r}, d_{r} \) and \( \beta \) as shown in Fig. 8.
FIG. 8. Roof Wire Bracing System.
From geometry

\[ d_r = \frac{b_r}{b} \sqrt{(b/2)^2 + h_r^2} = b_r \sqrt{(1/2)^2 + (h_r/b)^2} \]  

and

\[ \sec \beta = \frac{1}{k} \sqrt{\alpha^2 + (b/2)^2 + h_r^2} = \frac{1}{k} \sqrt{1 + (b/2)^2 + (h_r/\alpha)^2} \]  

Clearly the ratio \( d_r/b_r \) and the angle \( \beta \) vary from roof to roof. However, it would be convenient to have single values of \( d_r/b_r \) and \( \sec \beta \) which could be conservatively applied to all cases. This can be accomplished by calculating these values for the least advantageous geometry that is likely to occur and then, in the absence of more careful analysis, use them in all designs. A particularly disadvantageous geometry occurs when the ratio \( h_r:b:\alpha \) is equal to 1/4:1:1, for this geometry

\[ d_r = \frac{\sqrt{5}}{4} b_r \]  

and

\[ \sec \beta = \frac{\sqrt{21}}{4} \]  

Substituting these values, and the maximum response acceleration proposed by Razani [1,13], \( c = 0.75 \), into Eq. (3.3) yields

\[ T \approx 0.25 w_r A_r \]  

where \( A_r = b_r \ell_r \) is equal to the area, in square meters covered by the roof. For practical purposes it is helpful to present this information in terms of the number of wires of a given size required to supply this tension. Perhaps the most suitable wire to be used for this purpose is galvanized #8 steel wire (0.415 cms. diameter). Given that the yield load of one of these wires is approximately 500 kg, then the number of
wires required, \( n \), is equal to

\[
n = \frac{0.25}{500} w_r A_r = \frac{w_r A_r}{2,000}.
\]

A typical value for \( w_r \) in Peru would be 200 kg/m\(^2\). Substituting this value in the previous equation gives

\[
n = 0.1 A_r.
\]

Values of \( n \) obtained using this formula for different values of \( A_r \) are given in Table 3. The wires can be anchored to the wood frame described below and tightened by twisting each set of wires with a rod. A minimum of two wires is thus required in each face of the roof in each direction. Details of the wire bracing are shown in Fig. 9.

As mentioned before, after providing the roof with the necessary strength, there is still the need to provide the house with an auxiliary structural system capable of taking the roof load after wall collapse. This can consist of wooden frames enclosing the walls as shown in Fig. 10. Besides its ultimate load-carrying function, these wood frames will have a confining action which can considerably increase the strength and ductility of the adobe walls. For the purpose of illustration Fig. 11 shows the results of confining the adobe walls with a bamboo frame obtained by Koerner et al. at Drexel University [14].

If the wood columns are properly embedded into the ground, say a distance of 1 m, they can be assumed to be fixed at this end and pin-jointed at the top. The required section of the columns can then be found from the formula
FIG. 9. Details of Wire Bracing. (Only wires in one direction are shown in each face).
FIG. 10. Isometric View of Wall Wood Frame.
FIG. 11. Load vs. Deflection Curves of Bamboo Frame, Adobe Wall and Adobe Wall with Bamboo Frame. Taken from Koerner et al. [14].
Table 3
Suggested Values for Use in the Design of Roof Systems
Including their Auxiliary Support Structures

<table>
<thead>
<tr>
<th>(b_r) (m)</th>
<th>(l_r) (m)</th>
<th>(A_r) (m²)</th>
<th>(n)</th>
<th>(r_p) (cms)</th>
<th>(r_b) (cms)</th>
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<td>2</td>
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<td>13.4</td>
<td>8.4</td>
</tr>
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</table>
where \( P \) and \( M \) are the tributary vertical load and bending moment in each column respectively and \( P_u \) and \( M_u \) the column's ultimate axial load and moment-resisting capacity respectively. By inspection

\[
\frac{P}{P_u} + \frac{M}{M_u} \leq 1
\]

By inspection

\[
P = \frac{W}{4}
\]

and

\[
M = \frac{cWh}{4}
\]

Considering round posts which are probably the most readily obtainable in rural areas

\[
P_u = A_p f_u
\]

and

\[
M_u = A_p f_u \frac{r_p}{4}
\]

where \( A_p \) is the cross-sectional area of the post, \( r_p \) its radius and \( f_u \) the ultimate strength of the wood in flexure. Substituting the above relations in Eq. (3.8) and after some manipulation, the following equation is obtained:

\[
1 + \frac{4ch}{r_p} \leq \frac{4 A_p f_u}{W}
\]

Typically, \( \frac{4ch}{r_p} >> 1 \)

and therefore the left-hand side of the equation can be modified to yield
Calculations of stability and shear resistance show that the value of \( r_p \) given above controls the design.

The wall bond beam can be analyzed by assuming the top connections to be pin-jointed. This beam will thus be subjected to a maximum moment

\[
M = \frac{W}{4} \times \frac{8}{8} = \frac{W \phi}{16} \approx \frac{W \phi}{32},
\]
equating this value with the ultimate bending strength of the beam,

\[
M_u = \frac{\pi r_b^3 f_u}{4},
\]
the following expression for \( r_b \) is obtained,

\[
r_b \geq \sqrt{\frac{3 W \phi \sigma}{8 \pi f_u}}. \tag{3.10}
\]

Using \( c = 0.75 \) and the typical values

\[
W = \frac{\sqrt{5}}{2} \times 200 \times A_r = 224 A_r, \]
\[
h = 2.4 \text{ m},
\]
and \( f_u = 3 \times 10^2 \text{ kg/cm}^2 \),
formulas (3.9) and (3.10) become respectively:

\[
r_p \geq 3.5 \sqrt{A_r}, \tag{3.11}
\]
and
\[
r_b \geq 1.4 \sqrt[3]{A_r} \sigma_r, \tag{3.12}
\]
where $A_r$ is in square meters, $l_r$ in meters and $r_p$ and $r_b$ in centimeters. The same value of $r_b$ can be used for the beams spanning in the short direction and for the beam running in the long direction at the vertex of the roof. For long walls, say over 5 m, it will be better if an extra post is placed at the middle of the wall. For walls containing this third post formulas (3.11) and (3.12) become:

$$r_p \geq 3.1 \sqrt[3]{A_r}$$  \hspace{1cm} (3.13)

and

$$r_b > 0.9 \sqrt[3]{A_r} l_r$$  \hspace{1cm} (3.14)

However, the radius of the corner posts is controlled by the requirements of the wall having the least number of posts. Also, the beam along the vertex of the roof has to be always designed according to formula (3.12).

Values of $r_p$ and $r_b$ obtained using the above formulas for different values of $l_r$, $A_r$ and, for the longer walls, number of posts are given in Table 3. These values are rather small despite their conservatism, which demonstrates the economy of this structural solution.

Connections are a critical aspect of this framing system, since the wall bond beams must remain connected to the columns to perform its load-carrying function. Suggestions for these and other connections are shown in Fig. 12.

Walls

In adobe houses walls are needed to provide lateral stiffness, and as such they should be regarded as structural components and should be
FIG. 12. Details of Frame Connections
able to remain undamaged after the occurrence of an earthquake of intensity MM VII or less [1,13]. Because of the brittle behavior of adobe, this demand is equivalent to requiring the walls to perform elastically under the specified load. This condition is typically met in adobe houses with the possible exception of critical sections like section A-A in Fig. 13. It is therefore important, in the design of adobe houses, to check the elastic shear resistance of these sections. In sections like B-B calculations show that it is impossible to prevent diagonal tension cracks, but the confinement provided by the door and wall frames should prevent further damage.

Collapse of the walls transverse to the seismic motion can be prevented by the use of diagonal wood rafters tying perpendicular walls at the top corner and the addition of cane ties along the height of the wall. This reinforcement could go around the wood columns, and if extended along the entire length of the wall will improve its seismic behavior. The Chinese Building Code [9] recommends the use of ties at 30-cm intervals along the entire wall height and embedded at least 1 m into each wall. Cane strips can also be used to improve the link between the walls and their respective wall bond beams.

Commonly in adobe houses all openings are placed in a single wall, making it weaker than its parallel counterpart. However, because of the geometry of the problem both walls are required to take almost equal lateral loads, and this can lead to a torsional problem in the initial elastic stage. This problem is somewhat alleviated by the fact that the walls with openings are lighter, and thus subjected to smaller inertial forces. Once the walls start to fail, only the roof and its
FIG. 13. Critical Sections in Adobe Walls. (Wood framing not shown)
auxiliary support system are left, symmetry is restored, and the torsional problems tend to disappear. An analysis of the elastic behavior of the house will first require an estimation of the weakening effect of openings in a wall. One way of doing so, using elastic theory, is given in Appendix IV. However, this method ignores the influence of door and window frames and stress concentrations on the stiffness of the wall which could be of significant importance. To take these factors into account will require a detailed finite element formulation which would be outside the scope of this thesis and contradictory to the design philosophy here adopted.

To illustrate the use of the suggested procedures given in this chapter, an example of their application in the design of an adobe house is given below.

Example Design of Adobe House

Dimensions: \( l = 6 \text{ m} \)
\( b = 4 \text{ m} \)
\( h = 2.4 \text{ m} \)
\( t = 30 \text{ cms} \)
\( x_r = 7 \text{ m} \)
\( b_r = 5 \text{ m} \)
\( h_r = 1 \text{ m} \).

These dimensions are also shown in Fig. 14.

a) **Roof System**

From Table 3:
\[ n = 4 \]

The length of each diagonal wire can be easily calculated by looking at Fig. 8.
\[ l_w = \sqrt{b^2 + \chi^2 + h_r^2} \]

\[ = 7.3 \text{ m} \]

Actually, each wire will need to be somewhat longer, say by 30 cms, so that it can be properly attached to the wood frame as shown in Fig. 9. Considering both faces and both directions, the total length of galvanized #8 wire required to brace the roof is

\[ 2 \times 2 \times 4 \times 7.6 = 122 \text{ m} \]

b) Wall Framing (with an intermediate post in the long direction).

From Table 3, for the four corner posts:

\[ r_p = 11.5 \text{ cms} \]

and for the two intermediate posts in the middle of the long walls,

\[ r_p = 10.2 \text{ cms} \]

Also,

\[ r_b = 8.8 \text{ cms} \]

for the two beams spanning in the short direction and for the beam along the roof's vertex. Finally, for the two beams spanning in the long direction,

\[ r_b = 5.7 \text{ cms} \]

Therefore, assuming the columns to be embedded 1 m into the ground, the wall framing system will require the following logs:

- 4 23 cms x 3.4 m (diameter x length)
- 2 20.4 cms x 3.4 m " "
- 2 11.4 cms x 7 m " "
- 1 17.6 cms x 7 m " "
- 2 17.6 cms x 4 m " "

plus the smaller inclined timber required to hold the roofing material.
Cane Reinforcement

Following the recommendation of the Chinese Building Code [9], each corner will require seven strips of cane, about 2.4 m long. Identical cane ties can be used to connect both halves of the long walls around the intermediate posts. Cane strips, spaced one brick apart, should also be used to improve the connection between the adobe wall and its bond beam. Details of tie placement are shown in Fig. 15. The entire house will thus require:

\[
\begin{align*}
42 & \quad 2.4 \text{ m } \\
46 & \quad 1.0 \text{ m }
\end{align*}
\] cane strips.

\[c\] Shear Check in Critical Sections. Fig. 16 shows the front wall of the house. The shear force in critical section A-A is given by

\[V = \frac{cw}{2} + c \gamma_a t [(10 \times 1.4) - (2.8 \times 0.8)]\]

where the first term arises from the inertia of the roof and the second from the inertia of the walls. In deriving this equation, it was assumed that the longitudinal walls, half each, take the whole shear force caused by the inertial forces of the section of the house above A-A.

From formulas (3.2) and (3.3)

\[W = 200 \times 2 \times 5 \sqrt{(1/2)^2 + (1/a)^2} \times 7 \approx 7,800 \text{ kg.}\]

Following Razani's recommendation [1,13], \(c\) can be taken to be equal to 0.2 for the walls and therefore, using \(\gamma_a = 1500 \text{ kg/m}^3\) [9,12] the shear force becomes:

\[V = 0.2 \left[\frac{7,800}{2} + (1,600 \times 0.30 \times 11.8)\right]
\[= 1,900 \text{ kg.}\]
FIG. 15. Details of Cane Ties.
FIG. 16. Front Wall of Example Adobe House.
The area of critical section is given by:

\[ A_c = 0.30 (6 - 2.8) = 0.96 \, m^2 \, . \]

The shear stress in this section is thus

\[ \tau = \frac{V}{A_c} = \frac{1,900}{0.96 \times 10^{-4}} = 0.20 \, kg/cm^2 \, . \]

Assuming that the walls, due to their greater stiffness (as compared with the wood frame) take all the compressive load of the roof, the axial load on the front wall, at section A-A can be approximated by:

\[ P = \frac{w}{4} + \gamma_a t [(6 \times 1.4) - (2.8 \times 0.8)] \]

\[ = 4,900 \, kg \, , \]

which results in a compressive stress

\[ \sigma = \frac{P}{A_c} = \frac{4,900}{0.96 \times 10^{-4}} = 0.51 \, kg/cm^2 \, . \]

Applying Eq. (1.1), the shear strength of the adobe at section A-A is given by:

\[ \tau = 0.09 + 0.55 \sigma \]

\[ = 0.37 \, kg/cm^2 \geq 0.21 \, kg/cm^2 \, , \]

and therefore the shear resistance of critical section A-A is adequate.
CHAPTER IV - CONCLUSIONS

In searching the literature it was found that very little research of any significance has been done on improving the seismic resistance of adobe housing. Most of the research done so far concentrates on the material properties of the adobe rather than on preventing the types of catastrophic failure that result in human injuries. As has been repeatedly pointed out [3,13], economic constraints do not permit, particularly in Less Developed Countries, the building of damage-proof housing. Therefore, in utilizing the available resources, the first priority should be placed on protecting human lives. Doing so requires, first of all, that the house be furnished with a more reliable roofing system: one that doesn't depend on the integrity of the walls. This liberates the designer from dealing with the uncertainties in the mechanical properties of the adobe. After the roof is taken care of, very little more needs to be modified in the traditional methods of construction. The use of cane reinforcement is recommended, especially to tie perpendicular walls at the corners.

As in most technical problems, the question of how to improve the seismic behavior of adobe housing has some obvious answers and these are mentioned time and again by investigators in this field. For example, some of these are: 1) houses should be made as square as possible, 2) roofs should be light and rigid in their own plane, and 3) wall openings such as windows and doors should be kept at a minimum [4,15]. Unfortunately these solutions often overlook economic, cultural, esthetic, environmental and other constraints which should be taken into account.
The specifications on adobe construction given in codes were often found to be inadequate and occasionally unrealistic. For example, in Chile and Mexico, which are seismically active countries and where adobe construction is widespread in rural areas, this building material is not recommended, and the codes do not have any significant specification of direct relevance. In China and Peru there is a more practical attitude towards adobe construction, and this is reflected in their building codes. Even so, apart from some geometrical ratios and material property values, most of the specifications found in these codes are of a very vague and qualitative nature. Clearly, statements such as "the roof should be adequately rigid" or "walls should be designed by the method of allowable stresses" are of very little use to the laymen who, as mentioned before, are the ones responsible for most of the adobe construction. There is thus a need for a simple, comprehensive and more specific set of rules which can be followed by unskilled people. Data could be conveniently given in table form as in Table 3. A good attempt at making a simple code (yet still somewhat too qualitative) is the booklet, "Construyendo con Adobe" (Building with Adobe) published in Peru [16].

With the present state of knowledge, it is clear that a well designed adobe house should have:

1) a roof bracing system;
2) an auxiliary roof support system;
3) door and window frames;
4) a wall bond beam (usually part of the roof support system); and
5) some sort of wall reinforcement to increase their ductility and to tie perpendicular walls at the corners.
Chapter III discussed the importance of providing adobe houses with these requirements and a way of doing so which allows for retrofitting. Clearly, a lot more research is needed to determine more and better ways of accomplishing the above. Research is also needed on alternate, lighter roofing systems and on the effect of framed and unframed wall openings in the performance of walls. All research, however, should have in mind the constraints present in LDC's for their results to have any practical significance.

As mentioned before, some damage is to be expected in adobe houses after the occurrence of moderate to strong earthquakes. This may represent a substantial financial loss to the house owner, and to alleviate this problem Razani [1,13] proposes a national insurance policy where home owners could insure their property against earthquake-caused damage. With this policy an optimum level of design can be accomplished, avoiding the unnecessary expenditure of scarce resources in "over-designed" construction. Besides creating an insurance fund, a small fraction of the money collected can be used to finance necessary research and to run short courses to teach people some basic construction skills. This latter effort should be a primary goal of state and local governments in countries where adobe construction is widespread.
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APPENDIX I

(a) Calculation of in-plane lateral wall stiffnesses

(b) Explanation of "effective wall" concept.
(a) **Stiffness calculations**

For the initial linear-elastic stage of deformation the lateral stiffness of walls can be calculated for different loading conditions using elastic theory. Calculations for three different loadings are presented below.

**Case 1.** Wall subjected to a bending moment and a concentrated load acting at the top (see Fig. 17a).

From elastic theory:

\[
\delta_f = \frac{P h^3}{3EI} + \frac{M h^2}{2EI}, \quad (A.1)
\]

and

\[
\delta_s = \frac{1.2 P h}{GA}, \quad (A.2)
\]

where \( \delta_f \) and \( \delta_s \) are the deformations at the top due to flexural and shear deformation respectively. By superposition the total deformation at the top is:

\[
\Delta = \delta_f + \delta_s, \quad (A.3)
\]

and defining:

\[
K = \frac{P}{\Delta}
\]

then,

\[
K = \frac{P}{\delta_f + \delta_s}. \quad (A.4)
\]

Substituting the relations

\[
e = \frac{M}{P},
\]

\[
I = \frac{t h^3}{12},
\]

\[
\lambda = \frac{h}{\lambda},
\]

\[
G = \frac{E}{2(1+\nu)}
\]
(a) Moment and concentrated load at the top.

(b) Uniformly distributed load.

(c) Uniformly increasing distributed load.

FIG. 17. Walls Under Different Loadings.
and $A = t\ell$

in Eqs. (A.1) and (A.2) yields

$$\delta_f = \frac{p^2}{Et\ell} (4h + 6e), \quad (A.5)$$

and

$$\delta_s = \frac{2.4 Ph (1+\nu)}{Et\ell}, \quad (A.6)$$

which can be substituted in Eq. (A.4) to produce the desired result,

$$K = \frac{Et\ell}{2\lambda[2\lambda^2d + 1.2d (1+\nu) + 3e\lambda]} \quad (A.7)$$

A similar procedure can be used to obtain the results in the next two cases and therefore only the important results are listed below.

Case 2. Wall subjected to uniformly distributed load (see Fig. 17b).

$$\delta_f = \frac{wh^4}{8EI} = \frac{3w}{2Et}, \quad (A.8)$$

$$\delta_s = \frac{1.2 wh^2}{2GA} = \frac{1.2 wh (1+\nu)}{Et\ell}, \quad (A.9)$$

$$K = \frac{Et}{1.5\lambda^3 + 1.2\lambda (1+\nu)} \quad (A.10)$$

where $K$ is here defined as:

$$K = \frac{wh}{\Delta}.$$

Case 3. Wall subjected to uniformly increasing distributed load (see Fig. 17c).

$$\delta_f = \frac{11 wh^4}{120 EI} = \frac{11 wh^2}{10Et}, \quad (A.11)$$
\[ \delta_s = \frac{1.2 \, \text{wh}}{6GA} = \frac{1.2 \, \text{wh} \lambda (1+\nu)}{3Et} \quad \text{(A.12)} \]

\[ K = \frac{Et}{2.2\lambda^3 + 0.8\lambda (1+\nu)} \quad \text{(A.13)} \]

where now \( K \) is defined as

\[ K = \frac{wh}{2\Delta} \]

(b) "Effective wall" concept

Fig. 18a shows a wall loaded by a concentrated lateral load \( P \) acting at two-thirds the height of the wall. To analyze the behavior of this wall properly it is necessary to isolate the bottom part in the free-body diagram shown in Fig. 18b. On its own this section of the wall is easy to deal with analytically, and the results can be matched against experimental observations.

Using \( \lambda = h/2\lambda \) and \( \epsilon = h/6 \), the results of part (a), Case 1 of this appendix can thus be used to model the elastic stage behavior of the "effective wall." Modeling the wall in this way has resulted in good correlations with experimental data [3,4].

Also, the free-body analysis of the lower part makes it easy to determine the relation between maximum shear resistance and compressive load.
(a) Wall loaded by a concentrated load P.

(b) Free body diagram of lower half.

FIG. 18. "Effective Wall" Concept.
Appendix II

(a) Explanation of "apparent ductility."

(b) Example of the use of Virtual Work Principles and Yield Line Theory to determine the seismic coefficient of rupture of an adobe wall.
(a) **Explanation of "Apparent Ductility".**

Consider a wall supported only at its base and subjected to a concentrated, transverse load $P$ acting at the top as shown in Fig. 19. As the load increases a point will be reached when cracks will occur in the tensile side but, provided that no crushing occurs at corner $0$, the self-weight of the wall will preserve equilibrium for:

$$p < \frac{\gamma_a t^2 \ell}{2h},$$

or, alternatively,

$$M = Ph < \frac{\gamma_a t^2 \ell h}{2}.$$

Once $P$ reaches its maximum value ($= \gamma_a t^2 \ell/2$) the wall will rotate rigidly about corner $0$ until the line of action of the gravitational force $W$ falls to the right of $0$ ($\delta < t$) and then collapse will occur. A force-deformation diagram for the loading of this wall is shown in Fig. 20. In an idealized version this diagram can be approximated as bilinear, showing two states, an initial linear-elastic stage followed by a post-fracture stage (after the entire cross-section of the wall is cracked), where large deformations occur at constant load. Actually, nonlinearities will start occurring soon after the adobe is subjected to some tension. This will occur when the vertical force resultant lies outside the middle third of the wall. That is:

$$\frac{M}{W} = \frac{p}{\gamma_a t \ell} > \frac{t}{6},$$

or

$$P > \frac{\gamma_a t^2 \ell}{6} = \frac{P_{\text{max}}}{3}.$$
FIG. 19. Forces Acting on an Adobe Wall.

\[ W = \gamma_a t l h \]

FIG. 20. Force-Deformation Diagram for Adobe Wall Loaded Perpendicularly to its Plane.

\[ P_{\text{max}} = \frac{\gamma_a t^2 l}{2} \]

idealized

"actual"

elastic deformation deformation at collapse
After $P$ exceeds this value the wall will become progressively less stiff and eventually the force-deformation diagram will asymptotically approach the line $P = \gamma_a t^2h/2$. Both idealized and "actual" curves are shown in Fig. 20.

This capacity of the wall to maintain a restoring moment after fracture occurs and to sustain large deformations is what has been termed "apparent ductility" [3,4,5].

(b) Example of the determination of the seismic coefficient of rupture of a wall.

Consider the wall rigidly fixed at its four sides shown in Fig. 21. From geometry, for a unit deformation ($\delta=1$) at the center of the wall

$$\theta = \frac{\sqrt{2}}{h} \quad \text{and} \quad \phi = \frac{2}{h}.$$ 

Using the principles stated in part 7 of Chapter I, the internal work due to this deformation is equal to:

$$W_i = (2 \sqrt{2} h \times M_\theta \cos^2 \alpha \times 2\theta) + \left(\frac{h}{2} \times M_\phi \times 2\phi\right) + \left(\frac{3}{2} h \times 2M_\phi \times \phi\right),$$

where $M_\theta = M_\phi = \frac{\gamma_a t^2h}{4}$, which is the average restoring moment over the height of the wall. ($2 \sqrt{2} h$ is the total length of diagonal cracks and $2M_\phi$ is the restoring moment at the base of the wall). Substituting $\alpha = 45^\circ$ and the values of $\theta$ and $\phi$ given above yields:

$$W_i = 3\gamma_a t^2h.$$
FIG. 21. Diagram of Fissured Adobe Wall.
The external work undergone by the wall is:

\[ W_e = C \gamma a t [(2 \times \frac{1}{3} \times h \times \frac{h}{2}) + (\frac{1}{2} \times h \times \frac{h}{2})] \]

\[ = \frac{7}{12} C \gamma a t h^2 \]

and by the principle of virtual work (\(W_i = W_e\)) an expression can be obtained for \(C\):

\[ C = \frac{36 t}{7 h} \]

The Peruvian Building Code (see table 2 in Chapter II), recommends

\[ \frac{t}{h} \geq \frac{1}{8} \]

which results in

\[ C \geq \frac{9}{14} = 0.64. \]

For comparison, using formula (1.3a) in Fig. 4,

\[ C = \frac{27 t}{7 h} \]

\[ = 0.48 \text{ for } t/h = 1/8, \]

which is a conservative result. Alternatively, using the formula proposed for preliminary design,

\[ C = \frac{nt}{h} = 4 \times \frac{1}{8} = 0.50 \]

gives also a conservative result very similar to the one obtained from the more complex formula. All the values of \(C\) obtained above are considerably higher than 0.2, the minimum value proposed by Razani [1,13], and therefore the wall shown in Fig. 21 can be considered inadequate.
Appendix III

Modified Mercalli Intensity Scale
MODIFIED MERCALLI INTENSITY SCALE [17]

I. Not felt except by a very few under exceptionally favourable circumstances.

II. Felt by persons at rest, on upper floors, or favourably placed.

III. Felt indoors; hanging objects swing; vibration similar to passing of light trucks; duration may be estimated; may not be recognized as an earthquake.

IV. Hanging objects swing; vibration similar to passing of heavy trucks, or sensation of a jolt similar to a heavy ball striking the walls; standing motor cars rock; windows, dishes, and doors rattle; glasses clink and crockery clashes; in the upper range of IV wooden walls and frames creak.

V. Felt outdoors; direction may be estimated; sleepers wakened, liquids disturbed, some spilled; small unstable objects displaced or upset; doors swing, close, or open; shutters and pictures move; pendulum clocks stop, start, or change rate.

VI. Felt by all; many frightened and run outdoors; walking unsteady; windows, dishes and glassware broken; knick-knacks, books, etc. fall from shelves and pictures from walls; furniture moved or overturned; weak plaster and masonry D* cracked; small bells ring (church or school); trees and bushes shaken (visibly, or heard to rustle).

VII. Difficult to stand; noticed by drivers of motor cars; hanging objects quiver; furniture broken; damage to masonry D, including cracks; weak chimneys broken at roof line; fall of plaster, loose bricks, stones, tiles, cornices (also unbraced parapets and architectural ornaments); some cracks in masonry C*; waves on ponds; water turbid with mud; small slides and caving in along sand or gravel banks; large bells ring; concrete irrigation ditches damaged.
VIII. Steering of motor cars affected; damage to masonry C or partial collapse; some damage to masonry B*; none to masonry A*; fall of stucco and some masonry walls; twisting and fall of chimneys, factory stacks, monuments, towers and elevated tanks; frame houses moved on foundations if not bolted down; loose panel walls thrown out; decayed piling broken off; branches broken from trees; changes in flow or temperature of springs and wells; cracks in wet ground and on steep slopes.

IX. General panic; masonry D destroyed; masonry C heavily damaged; sometimes with complete collapse; masonry B seriously damaged; general damage to foundations; frame structures if not bolted shifted off foundations; frames racked; serious damage to reservoirs; underground pipes broken; conspicuous cracks in ground; in alluviated areas sand and mud ejected, earthquake fountains and sand craters appear.

X. Most masonry and frame structures destroyed with their foundations; some well-built wooden structures and bridges destroyed; serious damage to dams, dikes and embankments; large landslides; water thrown on banks of canals, rivers, lakes, etc; sand and mud shifted horizontally on beaches and flat land; rails bend slightly.

XI. Rails bent greatly; underground pipelines completely out of service.

XII. Damage nearly total; large rock masses displaced; lines of sight and level distorted; objects thrown into the air.

*Masonry A, B, C and D as used in MM scale above.

Masonry A. Good workmanship, mortar, and design; reinforced, especially laterally, and bound together by using steel, concrete, etc; designed to resist lateral forces.

Masonry B. Good workmanship and mortar; reinforced, but not designed in detail to resist lateral forces.
Masonry C. Ordinary workmanship and mortar; no extreme weaknesses like failing to tie in at corners, but neither reinforced nor designed against horizontal forces.

Masonry D. Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally.
Appendix IV

Use of Elastic Theory to Calculate the Decrease in Stiffness of an Adobe Wall Due to Openings.
Consider the wall and the simple loading shown in Fig. 22a. The deflection at the top of the wall is given by the addition of two terms: a flexural term which can be obtained from the second moment-area theorem, and a shear term given by the area under the 1.2P/GA curve. Diagrams of the M/EI and 1.2P/GA curves along the height of this wall are given in Fig. 22b.

Let:

- \( MA_f \) be the moment of area of \( A_f \) about point 0,
- \( MB_f \) be the moment of area of \( B_f \) about point 0,
- \( A_f, B_f, A_s \) and \( B_s \) be the areas shown in Fig. 22b.

Using the moment-area theorems, in the absence of any openings in the wall the top deflection \( \delta \) is equal to:

\[
\delta = MB_f + B_s, \quad (A.14)
\]

while the additional top deflection, say \( \Delta \delta \) due to the openings is equal to:

\[
\Delta \delta = MA_f + A_s. \quad (A.15)
\]

Defining \( K \), the stiffness of the wall, as \( P \) over the top deflection of the wall, then for the solid wall

\[
K = \frac{P}{\delta},
\]

and for the wall with openings,

\[
K_R = \frac{P}{\delta + \Delta \delta}.
\]

The reduction of stiffness caused by the opening \( K \) is then equal to:
FIG. 22. Reduced Stiffness of Walls with Openings. (a) Diagram of Wall and Loading, and (b) Corresponding M/EI and 1.2P/GA Diagrams.
Example Calculation:

Neglecting the effect of the wood frames for the wall shown in Fig. 16, the decrease in stiffness can be calculated to be:

\[ \begin{align*}
A_s &= 8.57 \times 10^{-6} \text{ P cms}, \quad (\text{P in kg}) \\
B_s &= 2.29 \times 10^{-5} \text{ P cms}, \\
M_{A_f} &= 3.81 \times 10^{-6} \text{ P cms}, \\
M_{B_f} &= 5.02 \times 10^{-6} \text{ P cms}.
\end{align*} \]

Therefore, using formulas (A.14), (A.15) and (A.16) yields:

\[ \begin{align*}
\delta &= 2.79 \times 10^{-5} \text{ P cms} \\
\Delta \delta &= 1.24 \times 10^{-5} \text{ P cms}
\end{align*} \]

and

\[ \Delta K \approx 0.30 \text{ K}. \]

For comparison, neglecting the flexural components of deformation, the decrease in stiffness can be calculated to be:

\[ \Delta K \approx 0.27 \text{ K}, \]

which is a very approximate solution to the "exact" one obtained previously. The accuracy of this simplified procedure increases with increasing wall length-to-height ratio and with decreasing area of openings.

Another approach, naive and with no theoretical bases, is to calculate the decrease in wall stiffness as:
\[ \Delta K = \frac{\text{Area of Openings}}{\text{Area of Solid Wall}} \times K \]

or, substituting the appropriate values,

\[ \Delta K = 0.16 K \]

which is not a very accurate result. The above ratio of areas is therefore not a good way to determine the decrease in stiffness of walls.