Chapter Four

Tests on Alternative Improvement Techniques

4.1 Objectives of Testing

In Chapter 2 it was shown that the primary cause of failure of the stone masonry buildings of Eastern Anatolia is out-of-plane bending failure of the walls; but that numerous constructional defects contributed to their weakness in earthquakes. In Chapter Three, a number of alternative strengthening modifications have been proposed. The value of these modifications as a protection against earthquakes depends on both their cost and the level of strengthening that they provide. Only if such modifications are already widely practised can this information be obtained from field studies. The field studies performed during 1982 indicated that very few houses incorporating the principal modifications described have to date been built in the study area, and that statistics on earthquake damage would be unlikely to be able to distinguish their performance from that of unmodified traditionally constructed houses.

It was therefore concluded that only by controlled testing would it be possible to assess the effectiveness of alternative strengthening techniques. Three different approaches to testing were considered:

(1) Laboratory testing using small-scale models

(2) In-situ testing of existing structures or mock-ups in the field

(3) Laboratory testing of full scale prototypes

Laboratory model testing was considered in two separate studies in the Cambridge University Engineering Department. In the first study, a 1/25 scale model was subjected to simulated earthquake forces in a centrifuge[1] In the second study, a series of 1/10 scale models was subjected to simulated earthquake forces on a small shaking platform.[2] In neither case was it possible to represent adequately the method of construction of the traditional building, particularly at the level of detail on which earthquake performance primarily depends. It was concluded that the value of such small scale model tests was very limited.

In-situ testing in the field was also briefly considered. The difficulty is to create ground motion of sufficient amplitude and frequency to create building damage comparable with that in a real earthquake. Methods such as controlled explosions and induced ground vibrations were considered, but were found to offer poor simulation as well as being very difficult to organise.

The third approach, building full-scale mock-ups of alternative modifications was considered more feasible. By carrying out such tests in the premises of the existing laboratory of the ERD in Ankara, a proper control of the testing was possible; but the test structures themselves could be built by masons from the villages, and using materials identical to those found in the villages. The process of construction could at the same time be observed and supervised in detail.

A two-stage programme of testing was planned. In the first stage, a series of test walls was built on the ground, using both traditional and modified techniques. The methods and costs of construction were observed and filmed. The walls were then subjected to static testing under lateral loading to compare their out-of-plane bending strengths. In the second stage, a series of one-roomed test houses was built on a shaking platform, and subjected to simulated earthquake loading. In order to conduct this latter series of tests, a suitable testing device had to be designed and built. In this chapter the testing programme is described in outline, and the results presented. Fuller details have

---

1 Spence and Coburn (1982).

2 Chou and Cowdrill (1983).

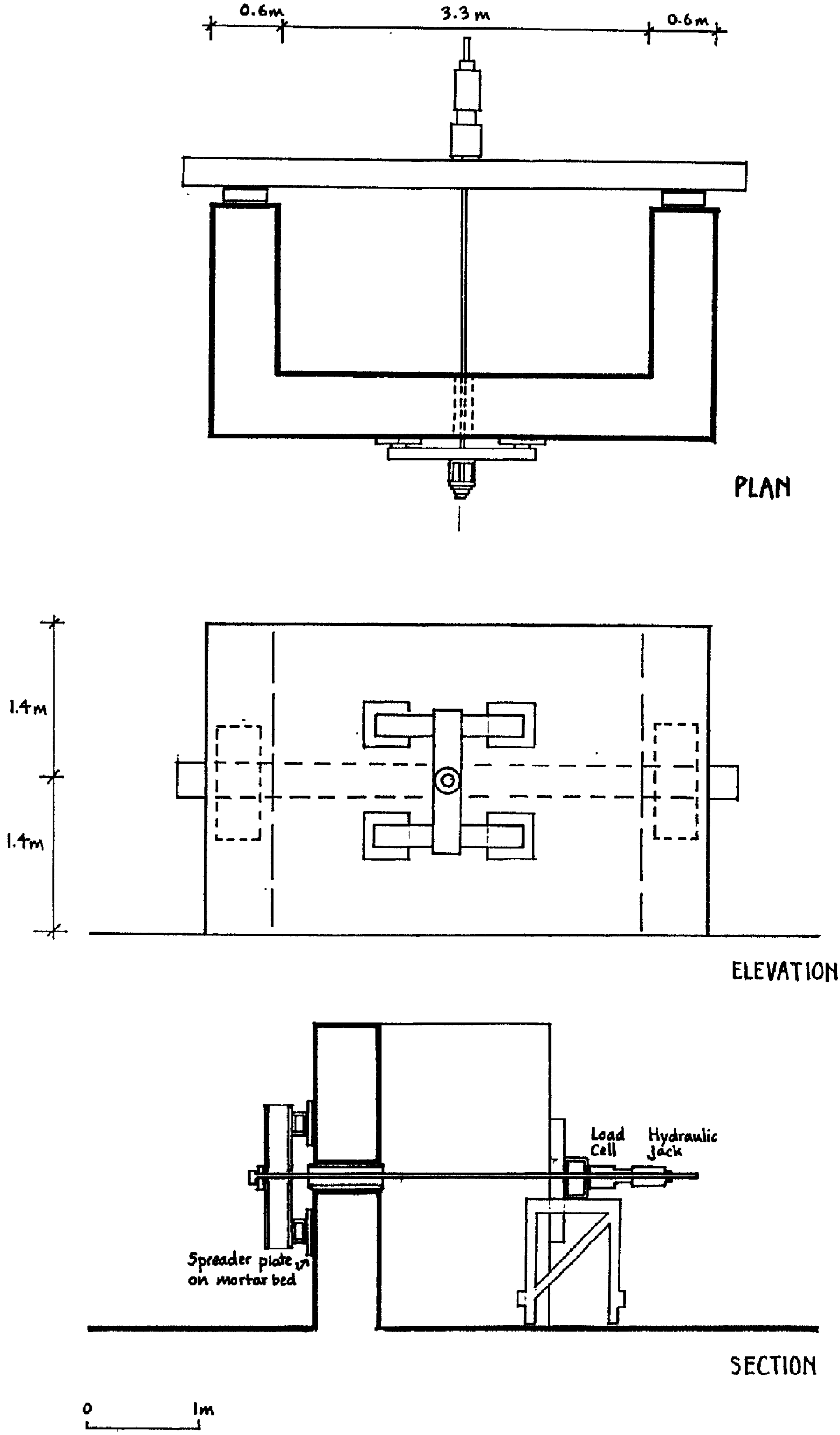

Figure 4.1 Static Tests: Loading Apparatus

been presented in laboratory reports published elsewhere.[3]

4.2 Wall Construction and Static Testing

Wall Construction.
In the surveys of traditional building stock that were carried out during the field study in Eastern Turkey, a common characteristic of many of the building forms was a structural unit of around 3.3 metres span, corresponding to the average length of timbers used as roof joists. This structural bay was taken for building portions of wall at full size. The construction experiment consisted of building a freestanding wall 4.5 metres long with two side walls 1.5 metres long and 0.6 metres thick giving the standard unsupported length of 3.3 metres. The wall height, 2.7 metres, corresponded to that of the common single storey houses surveyed. The walls carried no roof load and in that way were analagous with the non-loadbearing end wall of a rectangular room.

Four walls were built:

Wall 1. A standard, unstrengthened, random rubble wall with mud mortar as built by large numbers of villagers in Eastern Turkey today.

Wall 2. A standard random rubble wall with mud mortar, reinforced with horizontal timber hatıls; a traditional strengthening method now largely fallen out of use.

Wall 3. A random rubble wall built to Turkish Standard 2510 'Design and Construction Methods for Masonry'; using cement mortar and reinforced with horizontal reinforced concrete beams. Mainly used for school and government construction in earthquake areas.

Wall 4. Cut and dressed stonework with cement mortar, used predominantly in mosques and ornamental community buildings.

The standard stone used for most of the construction was a hard limestone, which split in planes (scale) when hammered (knapped). It was therefore used as angular pieces roughly shaped by hammer. Chiselling the stone also caused it to scale so chiselling (dressing) the pieces into square blocks was impossible. In many of the villages visited in the east of Turkey, the stone used was often a limestone, a basalt or a granite all of which tend to be similarly hard, brittle and scaley.

A slightly softer stone, a while andesite, which had been brought to the site for other building purposes, was used for the corner quoins of wall 2. This stone fragmented when chiselled and so could be shaped fairly accurately. It was used as long rectangular blocks to form an interlocking outer face at the corners and ends of the wall.

Test procedure and results
The test-rig was not designed to simulate the loading which might be caused by an earthquake, but to create a set of internal forces in the wall similar to those which would be caused by the horizontal out-of-plane component of earthquake loading. By this means it is possible:
a) to make effective comparisons between the strengths of different types of masonry which would indicate their relative performance in earthquakes, and
b) to test the validity of a theoretical approach to the estimation of the out-of-plane strength of a masonry wall.

It was important also to devise a cheap and simple test that could be replicated for a wide variety of wall types, and could be carried out without the resources of a sophisticated structural testing laboratory.

The testing apparatus designed for these tests is shown in Figure 4.1. It applies a horizontal patch load to the centre of each panel of masonry. Load and displacement are measured after each increment of loading and the apparatus is designed to accommodate the very large displacements (up to 50 cm) needed to cause failure.

3 Coburn and Spence (1984), Spence, Bayülke, Coburn and Erdik (1986), and Spence, Bayülke, Coburn and Hibbs (1987).

Load-displacement curves for the mid-wall point of load are shown in figure 4.2. Figure 4.3 shows the conditions at failure of wall W4, both at the front and at the back; the failure condition of walls W1 and W2 was similar, but wall W3 survived the maximum load without failure.

Conclusions
Given that only one wall of each type was tested, the validity of the test results numerically must be considered uncertain. Nevertheless, in a general sense, the results are useful. They indicate that:

1. Under static loading, even unreinforced random rubble masonry walls do not simply disintegrate, but deform under gradually increasing load until a pronounced failure mechanism has developed. Very considerable displacement is possible before the wall topples.

2. The failure load of an unreinforced random rubble masonry wall with mud mortar is rather low and depends primarily on its resistance to toppling, without any contribution from the interlocking of stones or internal friction.

3. The failure load of a random rubble masonry wall in mud mortar with timber reinforcements is much higher, on account of the contribution from the tensile strength of the timber hatıls This tensile strength depends on the strength of the nailed connections with the cross-members, which are therefore a crucial part of the reinforcement.

4. The failure load of a random rubble masonry wall in cement mortar with concrete ring beams is substantially higher than either of the above wall types. It is also very stiff under small levels of lateral load. The mode of failure of this type of wall has not been observed.

5. A cut stone masonry wall in cement mortar behaves in a brittle-ductile manner. After an initial load peak at small displacement due to the tensile strength of the mortar, the strength drops sharply, and at large displacements the strength is not much greater than that of an unreinforced random rubble wall, i.e. the contribution from interlocking and internal friction is not very great.

6. In the three cases of walls which were failed, a simple equilibrium theory based on the overturning strength of the wall under gravitational forces, and ignoring the tensile strength in the mortar or internal friction was able to predict the mode of failure and give a reasonable estimate of the failure load. The contribution of the strength of timber reinforcement can be allowed for in this theory. Full details of these tests and the equilibrium theory developed are presented elsewhere.[4]

4.3 Design of an Impulse Table

The table described here was built at the laboratory of the Department of Earthquake Research in Ankara in order to undertake studies of the performance of stone masonry buildings in Eastern Turkey; this requirement dictated the overall design parameters chosen. The table has an overall size of 30 m^2, and a maximum vertical load capacity of 50 tonnes, in order to enable a full-scale room-sized structural element to be tested. The dynamic operating characteristics chosen were a compromise between the need for design economy and for realistic earthquake representation. The table is designed to impart a maximum horizontal uniaxial acceleration of 2.0g to the 50 tonne load, with a maximum frequency of 5 Hz. Eastern Turkish earthquakes of which strong motion records exist commonly have peak energy in the range 4 to 8 Hz.[5]

The elemental table motion is a sinusoidal pulse of unidirectional horizontal displacement, of decaying amplitude, lasting approximately one second, figure 4.4. A series of such pulses of constant or increasing amplitude is used to simulate earthquake ground motion. In such an incremental earthquake, resonance effects of building response are inevitably lost; but there is the compensating advantage that the development of damage is easily recorded.

The spring support for the table consists of an array of standard rubber machine mountings. The required spring stiffness and displacement capacity of the rubber bearing system are derived from the maximum desired operating frequency and acceleration respectively. The desired characteristics were achieved by means of an array of 128 individual bearings, each with a shear stiffness of 260

---

[4] Coburn and Spence (1984).

[5] Ergünay, İnan, Bayülke and Koşan, (1985).

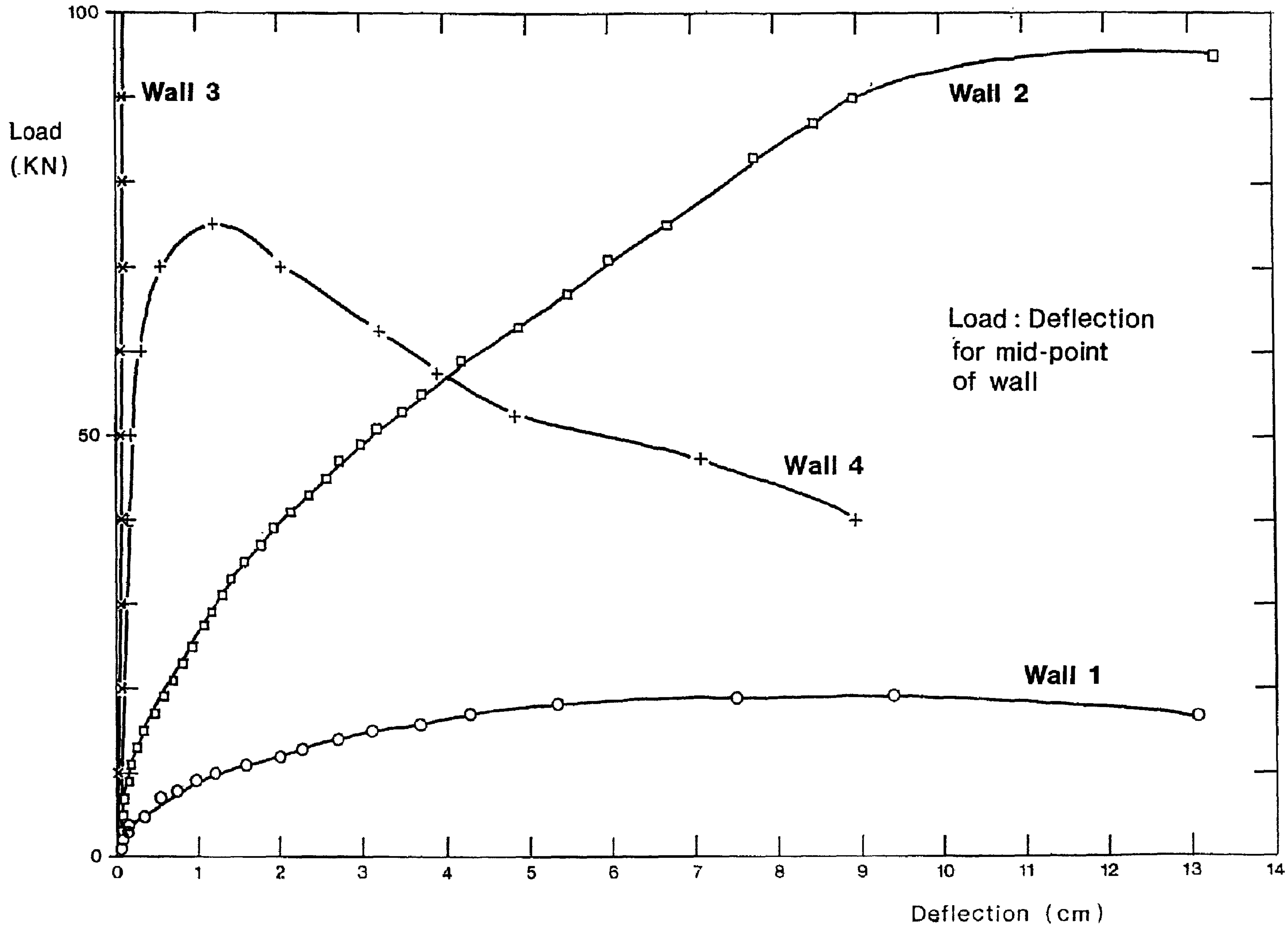

Figure 4.2 Static Tests: Load-deflection Curves for all Walls

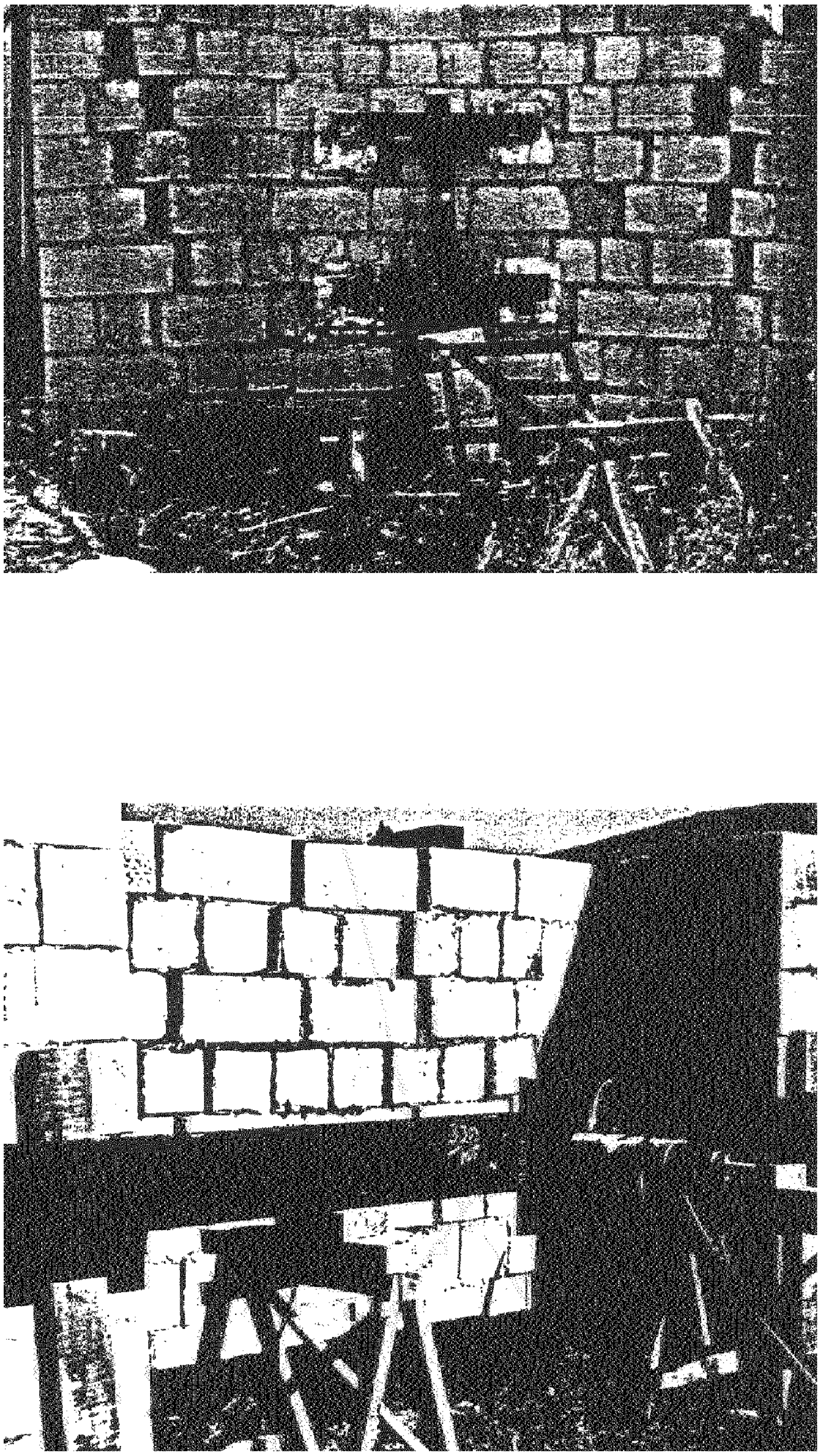

Figure 4.3 Static Tests: Condition at Failure of Wall 4

KN/m and a maximum lateral displacement capacity of 30mm.

The table is designed for a maximum of 128 bearings in any one layer, and a maximum of four layers of bearings, figure 4.5. The operating acceleration and frequency can be varied by changing the number or configuration of the bearings or the total load on the table. Figure 4.6 summarises the range of possible operating characteristics.

The trigger mechanism is designed to provide the required initial horizontal displacement to the table, storing energy in the rubber bearings, and then instantaneously releasing it. This is achieved by jacking upwards at the midpoint of a two-bar mechanism, figure 4.7. When the mechanism is horizontal, it snaps through, releasing the table instantaneously. It is then reset for the next pulse.

Figure 4.8 shows the general arrangement of the table and its foundations. The table has overall dimensions of 5.0 by 6.0 m, and a thickness of 240mm. There are four support points at which the bearing assemblies are located. For strength and simplicity of construction, the table is of composite construction, consisting of a welded steel frame, and a concrete infill. The steel frame carries the principal in-plane and out-of-plane forces arising from both static initial displacement and from subsequent oscillation, while the concrete carries the in-plane shear and out-of-plane bending forces. The table weighs 20.5 tonnes.

The bearing assemblies at the support points are designed to accomodate changes in the number of bearings in one layer or additional layers; this is achieved by jacking up one end of the table at a time, removing, reassembling and then replacing the bearing assemblies.

The foundations are massive, weighing about 250 tonnes, to provide an adequate reaction mass for the dynamic motion. The principal support points are located on upstand beams to allow access to the bearings and the hydraulic jack, and an end reaction block transmits the forces between the reaction point and the support points during initial displacement. Construction of the table took two months during June and July 1985; the total cost of the table, including bearings and trigger mechanism, was approximately $21,000.

4.4 Impulse Table Tests

In the test programme described here, three separate houses were tested. The basic layout of each of the test houses was identical; the differences were in the details of the wall construction used. Dimensions of the test houses are shown in figure 4.9. The specifications used for the three buildings are given in table 4.1.

All three test houses were constructed by stone masons using techniques of stone preparation and laying typical of those used in local villages. Each test house was built directly on the table, and construction took approximately ten days. A minimum interval of three days was left between the completion and the start of testing. Figure 4.10 shows the three test houses prior to testing.

Each test house was subjected to the same predetermined sequence of impulses until failure occurred. This loading sequence was intended to consist of three impulses at an initial (nominal) displacement of 20mm, followed by three impulses at 30mm displacement, three at 40mm displacement, and then as many impulses at 45mm displacement as were needed to cause failure, defined as total or substantial collapse of the entire building. Test 1 did not reach the maximum level of displacement. In practice there were some differences between the displacements given in each increment in the three tests, which are shown in table 4.2.

After each pulse, the trigger mechanism was reset, with the addition of packing plates if the displacement was to be increased, and a set of deformation measurements was made, and photographs taken. Each increment of loading took about 30 minutes.

During each pulse, the acceleration of the table was measured by means of two three-dimensional accelerometer arrays mounted on the table in the locations shown in figure 4.9. The accelerometers were monitored through a Sprengthner Digital Recorder (DR100) operating at a rate of 100 readings