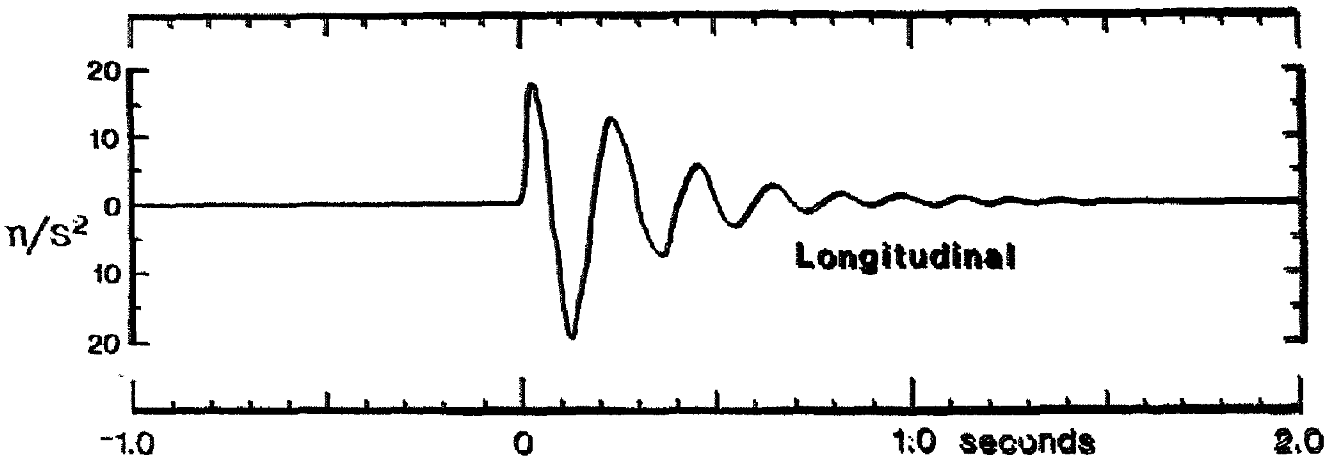

Figure 4.4 **Impulse Table: Longitudinal Acceleration of a Single Pulse**

Figure 4.5 Impulse Table: Bearing Assembly

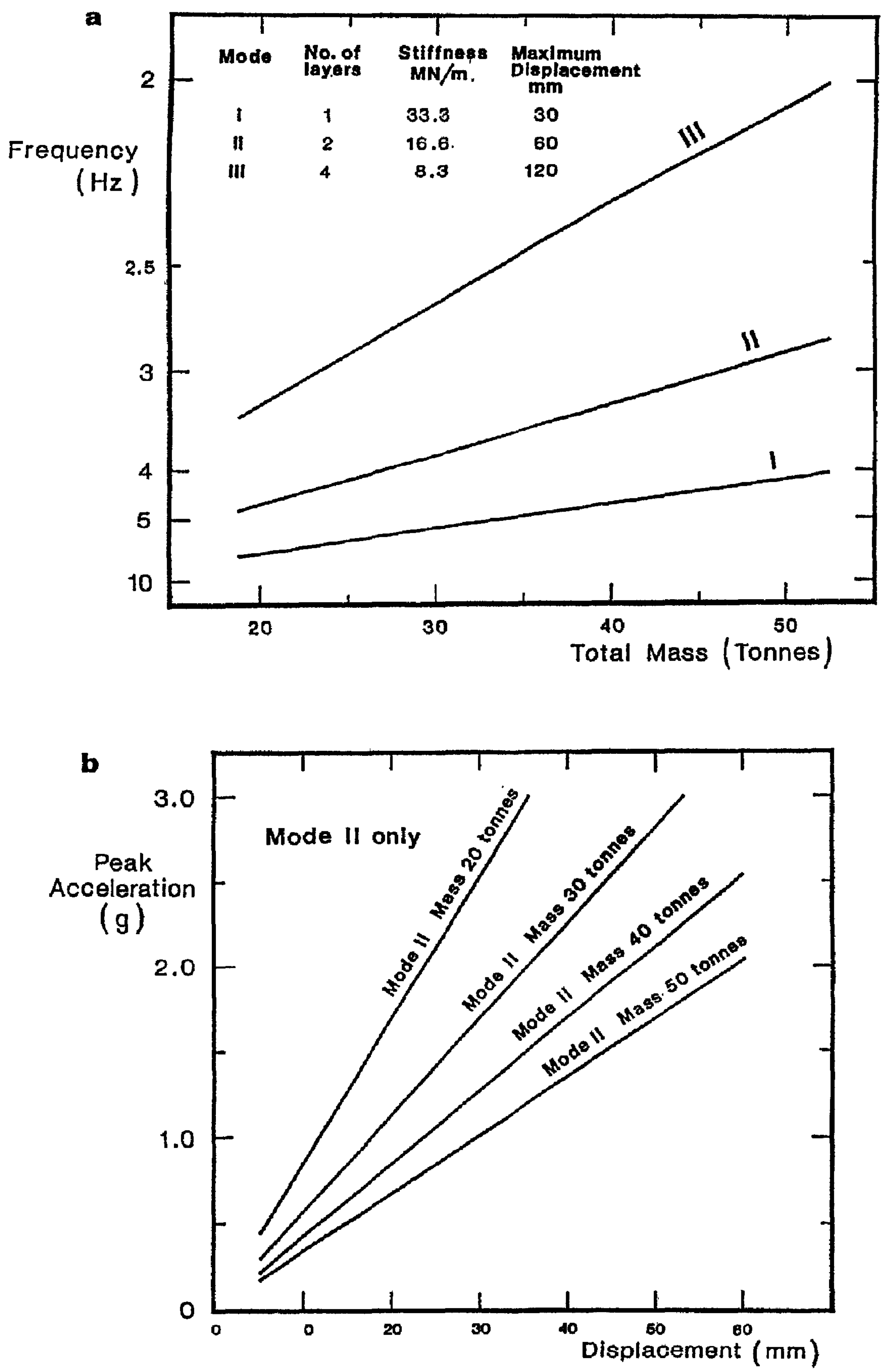

Figure 4.6 Impulse Table: Operating Characteristics

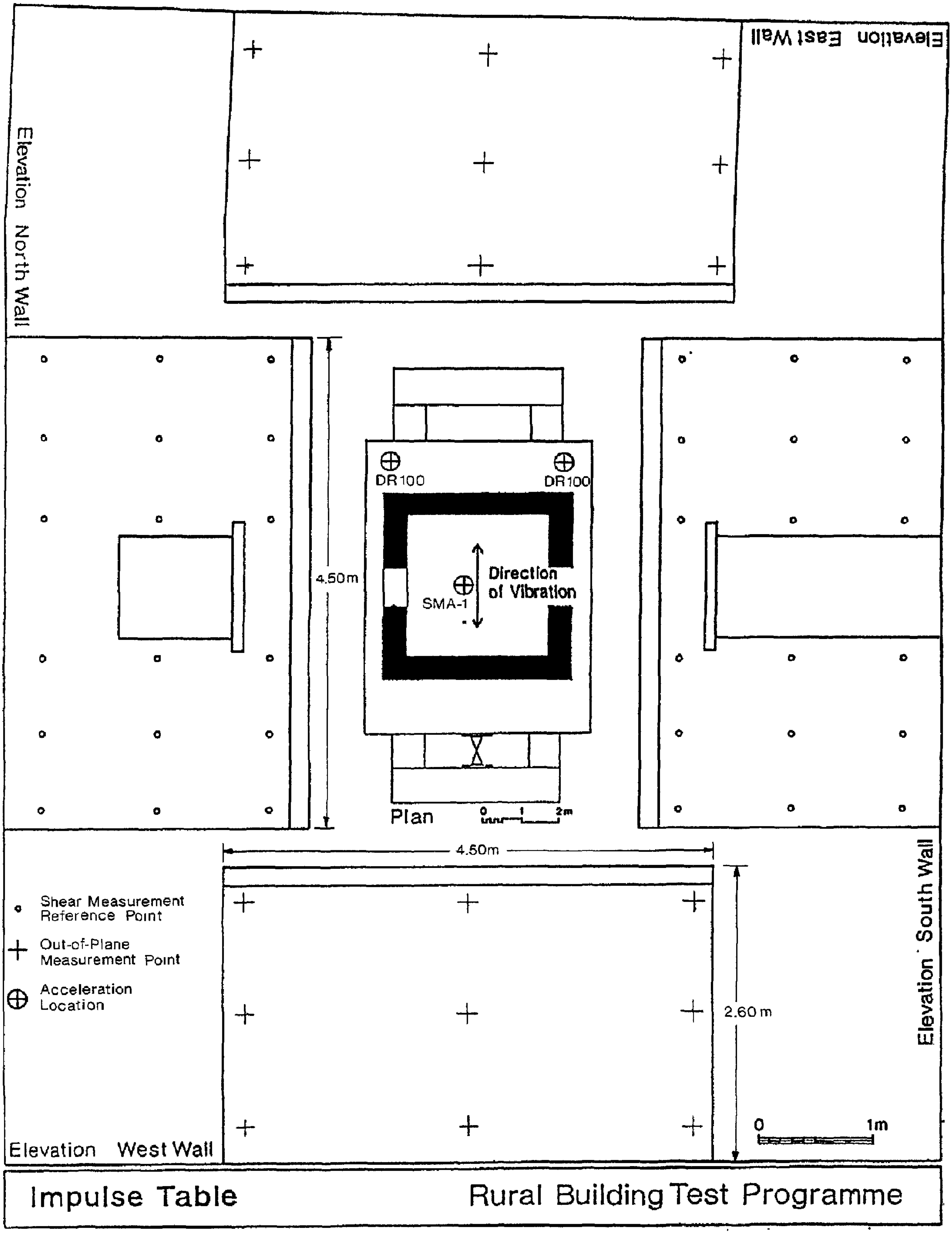

Figure 4.9 Impulse Table Tests: Test Structures and Instrumentation

CONSTRUCTION SPECIFICATIONS OF STRUCTURES TESTED ON THE IMPULSE TABLE

Structure No. 1: Unstrengthened Traditional Construction (L0)
Walls: Limestone rubble masonry walls laid in clayey mud mortar, constructed without through-stones or dog-toothing. Limited use of dressed stone corner quoins. No horizontal strengthening used. Vertical height of unsupported stonework: 2.6 metres.
Lintols: 4 no. 10 x 10cm timbers with 15cm bearings over each opening.
Roof: 10 x 10cm joists laid at 0.6m intervals, bearing onto single 15 x 3cm unfinished wall plate, not nailed. Bearing length 10cm, total span 3.3 metres. Joists covered by 20 x 2cm planking not nailed to joists. 20cm of soil compacted onto planking.

Structure No. 2: Triple Concrete Hatıls (Strengthening Level L5)
Walls: Limestone rubble masonry laid in clayey mud mortar, with regular dog-toothing (approximately once every metre in each course). Dressed stone corner quoins.
Concrete hatıls: 3 hatıls cast at heights of 0.9m (cill level), 2.1m (lintol level) and 2.6m (eaves level). Each concrete hatıl 10cm deep across full thickness of wall (60cm); concrete mix 1:5, cement:aggregate.
Reinforcement: 2 x ϕ12 bars with 1 metre overlap at corners.
Curing time before testing: 9 days.
Lintols: Precast concrete lintols, bearing length 20cm.
Roof: 5 x 10cm joists laid at 0.6m intervals, bearing onto wall plate cast into eaves level concrete ringbeam. Each joist nailed diagonally into wall plate. Bearing length 10cm. Total span 3.3 metres. Joists covered by 20 x 1cm planking, nailed to joists. 20cm of soil compacted onto planking.

Structure No. 3: Triple Hatıls of Timber (Strengthening Level L3)
Walls: Limestone rubble masonry laid in clayey mud mortar, with regular dog-toothing (approximately once every metre in each course). Dressed stone corner quoins.
Timber hatıls: Three hatıls, placed at heights of 0.9m (cill level), 2.1m (lintol level) and 2.6m (eaves level). Each hatıl: 2 no. 10 x 5cm rails on either face of wall, with 10 x 5cm cross pieces at 1 metre c/c. All joints nailed (5 nails). Rail splice joints: half-cut lap joint, nailed (5 nails).
Lintols: 4 no. 10 x 10cm timbers with 15cm bearings over each opening.
Roof: 5 x 10cm joists laid at 0.6m intervals, bearing onto top hatıl. Each joist nailed diagonally into hatıl. Bearing length 10cm, total span 3.3m. Joists covered by 20 x 1cm planking, nailed occasionally to joists. 20cm of soil compacted onto planking.

Table 4.1
Construction specifications for structures tested on the Impulse Table.

per second. After each pulse, in-plane strain measurements were made on the outside face of each wall. Measurements were made between nails embedded in plaster attached to the face of the stones, using an ordinary steel measuring tape. Preliminary trials indicated that such readings could be accurately repeated to within +/-1mm.

After each pulse, out-of-plane displacements were measured on the two walls perpendicular to the direction of motion. Readings were made using a theodolite established away from the table sighted on rulers attached to the face of the wall by quick-setting plaster. In a few cases the rulers were dislodged as the accelerations became large. Otherwise the theodolite readings were accurate to within about +/-2mm.

The photographic record consisted of a video recording of each pulse, colour and monochrome photographs of the general condition of each wall after each pulse, and a more detailed study of the failure condition of each wall.

4.5 Analysis and Comparison of Test Results

The table motion corresponded closely to that predicted by the design calculations. The peak longitudinal acceleration was closely proportional to the peak displacement, figure 4.11 . The motion was highly damped, with a second accleration peak of around 40 to 50% of the first; thus only abut two complete cycles were of significant amplitude. Transverse horizontal accelerations did not exceed 5% of the longitudinal accelerations. Vertical accelerations were also small, with peak values not exceeding 10% of the longitudinal values, but with a much higher predominant frequency. A typical set of accelerometer readings is shown in figure 4.12. The peak acceleration in each pulse for each of the three tests is given in table 4.2.

Structure No. 1. Unstrengthened Traditional Stone Masonry L0

Pulse No.	Max Displ.	Max Acceln.	Observations	Damage State
1	18mm	0.65g	No visible cracks	0
2	18mm	0.65g	Loosening of mortar	1
3	18mm	0.75g	Large vertical crack in west end of north wall	1
4	28mm	1.00g	Crack enlarges to >10mm	2
5	28mm	1.10g	Outer skin collapse of east wall	3
6	28mm	1.00g	Large bulge on north wall, slight bowing of east wall	3
7	38mm	1.55g	West wall collapse -overturning failure. Test terminated.	4

Table 4.2(a) Impulse Table Test Results - I. Unstrengthened Traditional.

Structure No. 2. Triple Concrete Hatıl Strengthening L5

Pulse No.	Max Displ.	Max Acceln.	Observations	Damage State
1	18mm	NA	No visible damage	0
2	18mm	0.55g	No significant damage	0
3	18mm	0.55g	Mortar loosened, minor cracks in N & S walls	1
4	28mm	0.55g	Cracks and bulging, N & S walls	1
5	28mm	0.90g	Large deformations in N & S walls, cracks in ringbeam	1
6	28mm	0.85g	Further loosening of corners, and larger deformations	1
7	38mm	0.90g	Major cracking of in-plane walls, particularly N wall, W end	2
8	38mm	1.60g	Internal collapse of masonry,	2
9	38mm	1.60g	Unstable corner of N wall	2
10	48mm	1.50g	Further deterioration of N wall	3
11	48mm	1.60g	Stone loss from N & S wall	3
12	48mm	1.50g	Extensive stone loss, N & S walls Test terminated.	3

Table 4.2(b) Impulse Table Test Results - II. Triple Concrete Hatıls.

The performance of the test houses is summarised in Tables 4.2(a),(b), and (c), and briefly described below: the condition at the end of the test is illustrated in figure 4.13.

Test 1 survived only four pulses before partial collapse occurred in the east (jack) wall; after a further two pulses the west wall partially collapsed. The cause of failure was overturning of the east and west walls; this was accompanied by skin splitting, or the separation of the outer and inner leaves of the wall. There were also large strains in the north and south walls at failure, principally in the horizontal direction.

In Test 2 partial collapse occurred on the inside face of the north wall in the ninth pulse. This collapse progressively developed until the thirteenth pulse, when the testing was terminated. At this stage the roof was still intact, and no significant failure had occurred in the east and west walls. Overturning of the east and west walls was effectively prevented by the horizontal reinforcement, which enabled the out-of-plane forces to be transmitted to the north and south walls. These were able to carry the load with little damage until much larger accelerations were reached; eventually

Structure No.3. Triple Timber Hatıl Strengthening L3

Pulse No.	Max Displ.	Max Acceln.	Observations	Damage State
1	18mm	0.65g	No significant damage	0
2	18mm	0.65g	No significant damage	0
3	18mm	0.60g	No significant damage	0
4	28mm	1.00g	Stones bulging below lintol, south wall, roof swaying	1
5	28mm	1.05g	Light cracking, south wall	1
6	28mm	1.05g	Cracks >10mm, N & S walls	2
7	38mm	1.60g	More extensive cracking	2
8	38mm	1.60g	Cracks widen, small stones dislodged, bulge grows	2
9	38mm	1.65g	Cracks widen, masonry spreading all walls bulging	2
10	48mm	1.60g	Local collapse of stones in S wall, below lintol	3
11	48mm	1.75g	Further collapse in S wall, bulging in E & W walls	3
12	48mm	1.50g	Further collapse in S wall, severe bulging	3
13	58mm	2.05g	More stone loss from N & S walls	3
14	58mm	2.05g	Major stone loss, all walls	4
15	58mm	2.05g	Stone collapse, lintols and roof remain supported	4
16	58mm	2.05g	More collapse, lintols and roof remain supported	4
17	58mm	NA	Few surviving structural elements but roof still in place. Test terminated.	4

NA: Not Available

Table 4.2(c) Impulse Table Test Results - III. Triple Timber Hatıls.

shear failure occurred in the north wall in the masonry panel between the first and second ringbeam, i.e. at the level of the window. At the termination of the test the north wall had begun to fail on the inside, but there was no external masonry collapse.

In Test 3 the first partial collapse occurred in the the in-plane south wall after 10 pulses; the north wall began to fail after 13 pulses, and a partial failure in the east (out-of-plane) wall occurred after 14 pulses. The test was terminated after 17 pulses with all four walls in a state of total disintegration, but with the roof still standing.

Test 3 followed a pattern very similar to Test 2, with little damage to the east and west walls in the early stages. The north and south walls began to show signs of shear failure at about the same level of loading as in Test 2, although the first local failure took place two pulses later. Testing was continued for a further 5 pulses in this test without causing a roof collapse, sufficient masonry remaining in both in-plane and out-of-plane walls to support the lintel. Local failures in the east and west walls began to occur only in the last 5 pulses.

Modes of failure

One way to compare the performance of the three test structures is by the level of damage caused by the same degree of shaking. Since all test structures experienced approximately the same sequence of pulses, the number of pulses experienced can be used as a crude measure of shaking severity. The level of damage can be assessed using the damage levels 0-5 defined in the MSK scale[6] and each damage level can be further subdivided according to the number of pulses in which it was recorded. Table 4.2 shows the damage level assessed after each pulse, and the comparison between the three tests is shown in figure 4.14.

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[6] Medvedev (1968).

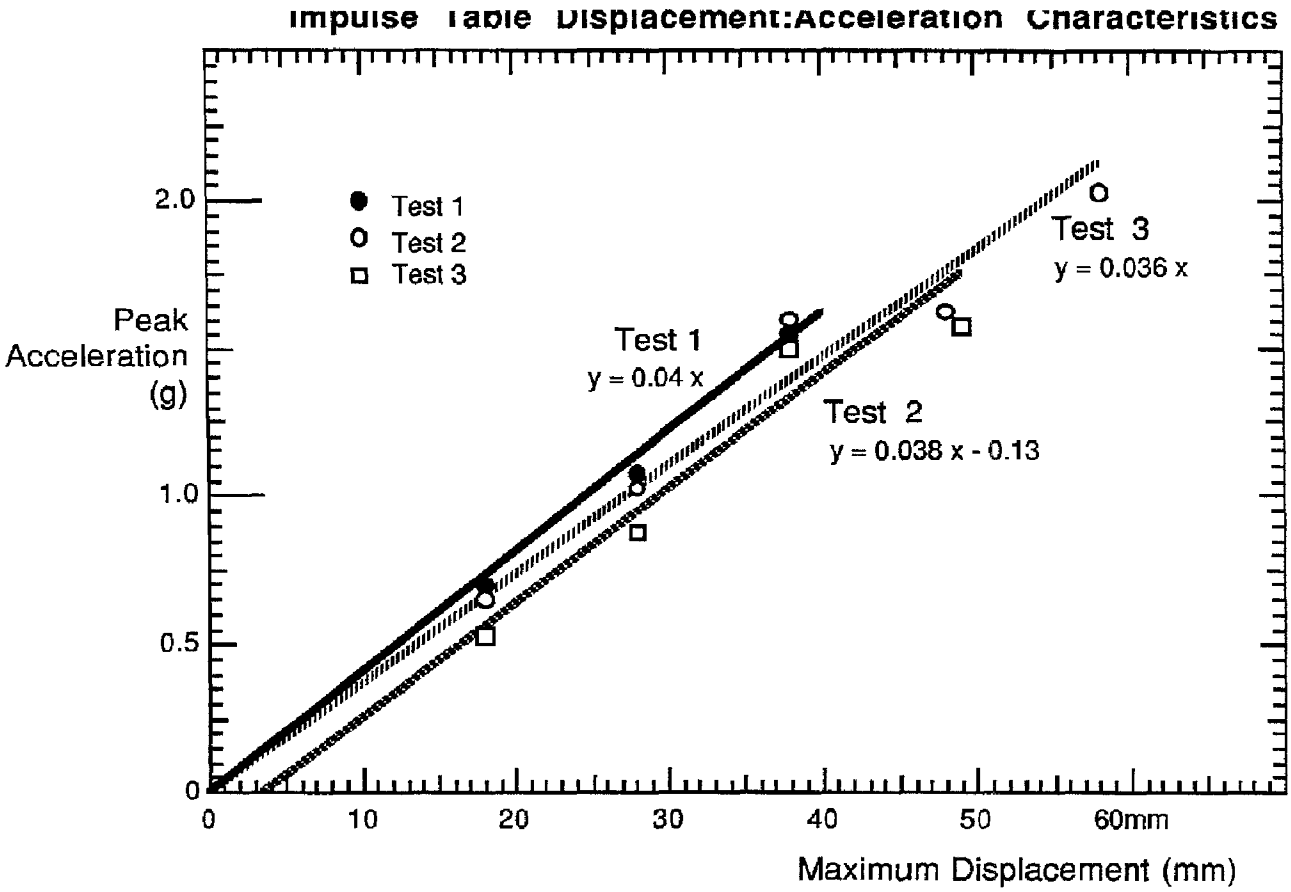

Figure 4.11 Imulse Table Tests: Peak Acceleration and Displacement

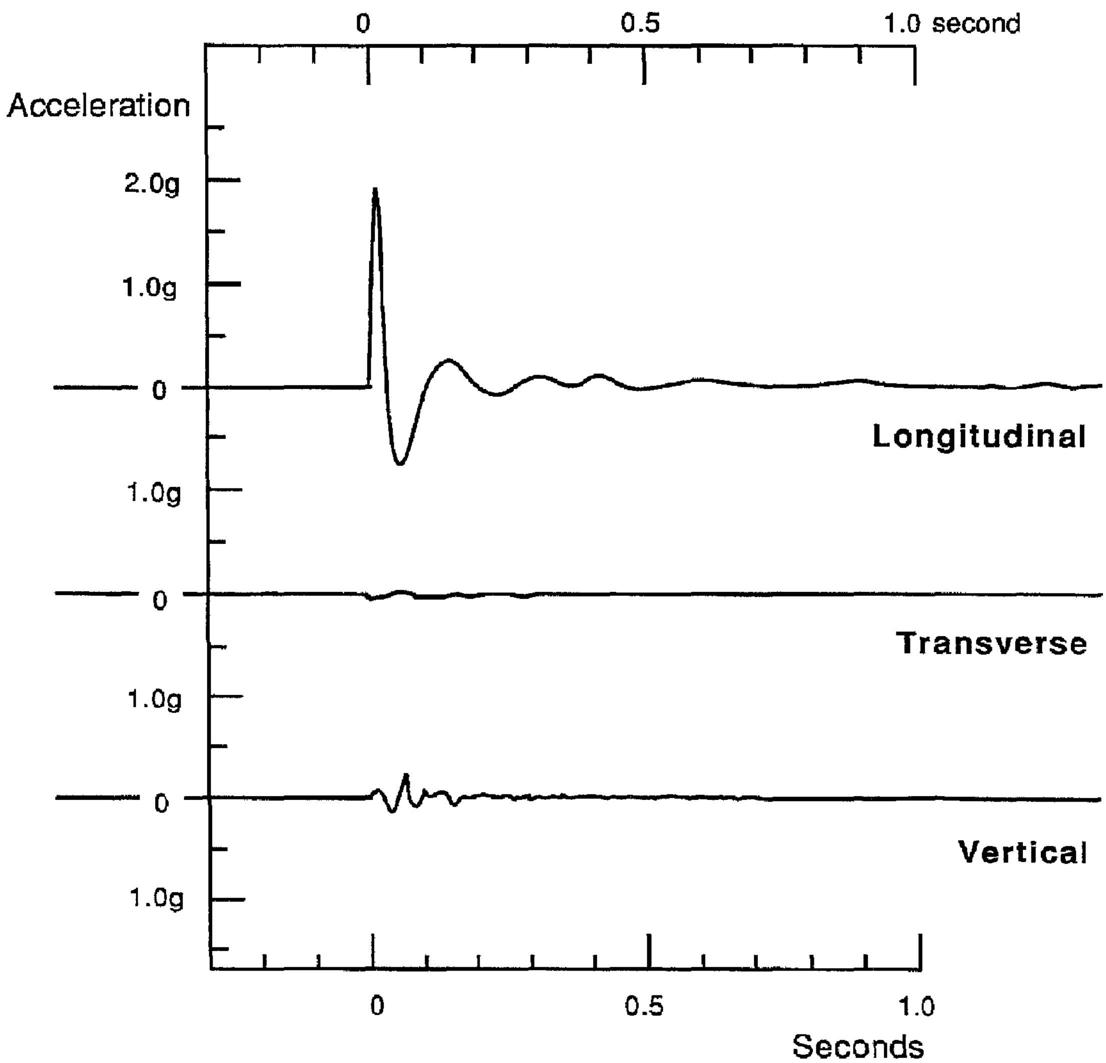

Figure 4.12 Impulse Table Tests: Typical Accelerometer Readings

Two observations can be made from figure 4.14. The first is that the two reinforced structures behaved in a very similar manner up to the point at which Test 2 was terminated. The much more disintegrated state of the walls of the third test house at the end of the test was solely due to the fact that it experienced five further pulses than Test 2. The second observation is that it shows very clearly the strengthening effect of the reinforcement. At pulse 4, the unreinforced structure experienced heavy damage, while the reinforced structures experienced only light damage; at pulse 5, the unreinforced structure experienced partial collapse, while the reinforced structures experienced moderate damage; and at pulse 7, when the partial collapse of the unreinforced structure took place, the reinforced structures were just at the onset of heavy damage.

It can be seen that under the same sequence of ground motions, the different constructions exhibited quite different characteristics of damage. The unstrengthened, traditional stone masonry building failed through out-of-plane, panel-type wall collapse very similar to observations of earthquake damage in the field. In the loadbearing walls there were also very large strains recorded, indicating the propensity of the unstable rubble masonry to spread out in all directions under dynamic excitation.

In both of the structures with horizontal reinforcement, the out-of-plane collapse of the non-loadbearing walls was effectively prevented. This enabled the out-of-plane forces to be transmitted to the north and south walls. They were able to carry the load with little damage until many more pulses had been experienced and acceleration levels had reached much higher values than needed to fail the unstrengthened structure. Shear failures in the loadbearing walls were the principal mechanism of failure under the uniaxial loads. Even when portions of masonry did collapse, the horizontal strengthening elements kept sufficient masonry remaining in plane to support the roof. The fact that the roof was tied to the top ringbeam meant that local collapses of masonry at the eaves could be spanned, preventing individual roof joists from falling.

The horizontal strengthening elements at regular heights up the wall reduced the vertical dimension of unsupported stonework and restricted its ability to spread and disintegrate. Strain measurements on the reinforced walls were greatly reduced. For this reason it appears that for a mud mortar wall, three weaker horizontal hatıls are probably more effective than a single, stronger ringbeam at eaves level, although this has not been explicitly tested.

The timber hatıl construction failed eventually by the stone masonry disintegrating and slipping out between them. The timber hatıls themselves did not fail and the nailed joints held firm. In the reinforced concrete hatıl construction, damage progressed principally through the failure of the concrete beams themselves. This was almost certainly due to faulty reinforcement connections at critical points. The bonding of the concrete to the stone masonry was, however, extremely good and stone masonry did not slip out or disintegrate anywhere where the beam itself remained continuous. Thus the reinforced concrete hatıl construction probably has the potential for much better earthquake reistance than the timber hatıl construction, but is much more difficult to construct without defects, especially in a village environment. From figure 4.14, the performance of the concrete hatıl construction and the timber hatıl construction can be seen to be broadly similar, deteriorating at much the same rate, compared with an extremely rapid decay for the unstrengthened traditional stone masonry house.

4.6 Conclusions on Testing

1. The impulse table was satisfactory for the purpose intended; the motion was repeatable with sufficient accuracy between one test and the next; each of the test houses was failed in a reasonably small number of pulses; resetting the trigger mechanism could be done much more quickly than the time needed to take complete set of readings; there was little vertical or horizontal acceleration. Some detailed improvements could be made to both table and instrumentation.[7]

2. The use of concrete and timber ring beams were equally effective in providing strengthening against the type of ground motion experienced on the impulse table. This motion, significantly, is

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7 Spence, Bayülke, Coburn, Hibbs (1987).

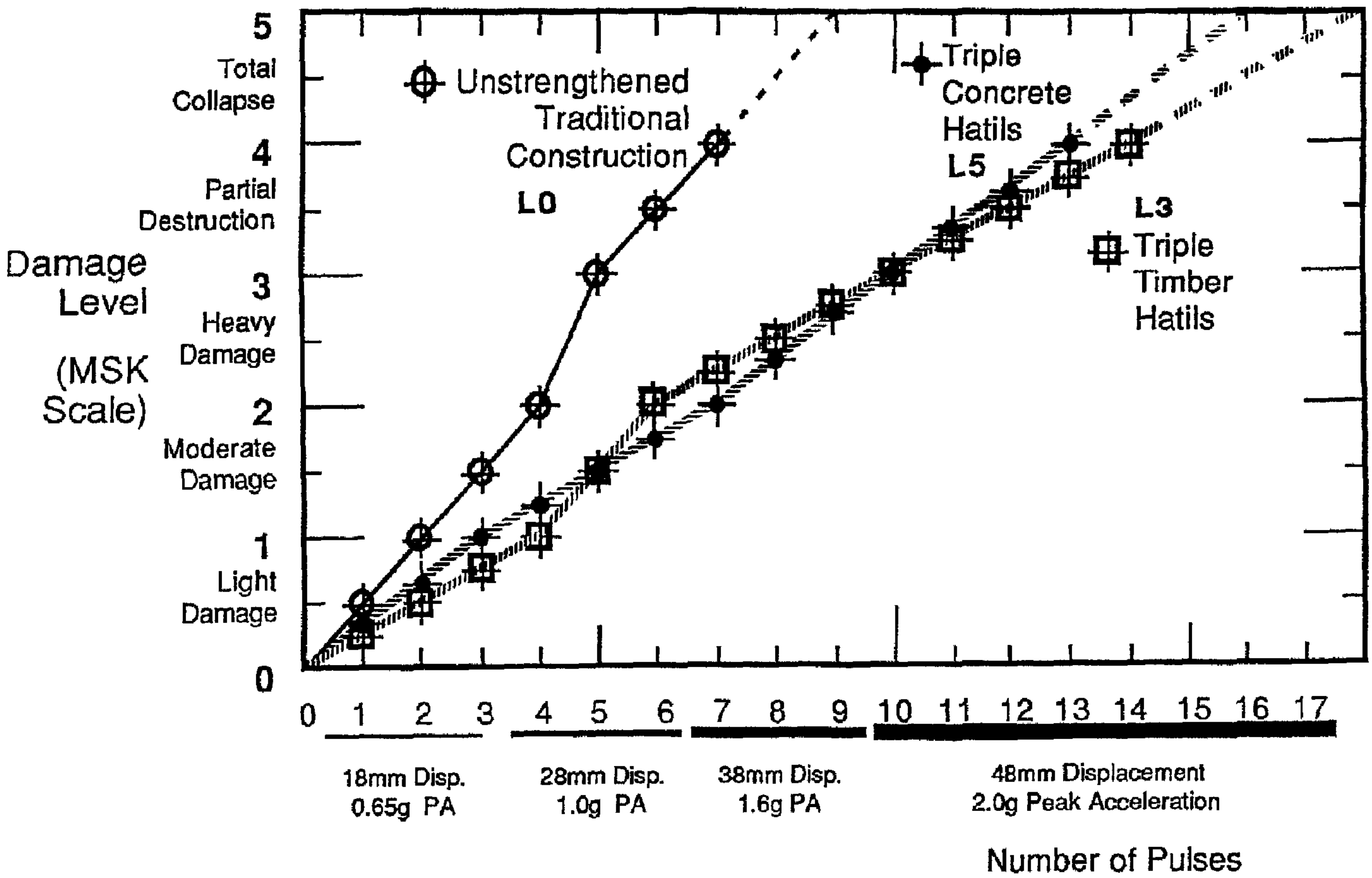

Figure 4.14 Impulse Table Tests: Comparative Performance of Test Structures

not unlike the horizontal component of ground motion in real earthquakes, which is thought to be the most damaging away from the immediate epicentral area; but relative performance under earthquake ground motion which contains a large vertical component cannot be assessed from these test results. The choice of concrete or timber for reinforcement could be made on the basis of cost and local experience. Timber, though scarce in some areas can more easily be checked to be satisfactory and properly connected; it would need treatment to prevent deterioration. The effectiveness of concrete ring-beams is very dependent on the proper execution of the reinforcement, which experience has shown is unreliable in rural construction. Even in this test programme faulty reinforcement detailing was not completely eliminated.

3. Both the reinforced test structures failed as a result of in-plane shear forces causing disintegration of the walls. Individual stones eventally became unstable leading to a progressive failure. Further strengthening could probably be achieved by the use of a wire mesh attached at intervals to the outside and inside[8] of each wall, and cross-connected at intervals . Addition of such reinforcement would however, be much more expensive than the addition of ring-beams. Another technical improvement which would certainly increase strength would be the use of cement or lime in place of mud mortar. This would add significantly to total cost.

4. Other points of difference in the details of the construction technique between the three test structures concern the bonding of the masonry and the connection of the roof. In Test 1, no dog-toothing or through-stones were used in the walls, and this evidently contributed to the rapid splitting and disintegration of the east and west walls in this test. Since the shear failures in Tests 2 and 3 ultimately took place by the splitting and disintegration of the masonry panels between the walls, it may be assumed that these panels would have failed sooner had no through-bonding been used, but this was not tested.

5. In none of the three tests was there a roof failure. In all cases, the roof was supported on the north and south (in-plane) walls; in Test 1 these walls did not fail; in Tests 2 and 3 the lintol level ring beam survived the test and continued to support the roof even after almost total disintegration of those walls. The survival of the roof after such severe shaking must be attributable in part to its secure fixing to the top ring-beam; but no comparative information was gained.

6. The comparative performance of the three test houses, measured in terms of damage levels, may be used to obtain some idea of the relative vulnerability of actual structures built using these methods. Given that only one test was performed on each structure however, the results of such analysis must be treated with caution.

[8] Scawthorn (1986).