Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Shaking table tests on strong motion damagingness upon unreinforced masonry

A. Pomonis, R.J.S. Spence & A.W. Coburn
The Martin Centre for Architectural and Urban Studies, University of Cambridge, Department of Architecture, UK
C. Taylor
Earthquake Engineering Research Centre, University of Bristol, UK

ABSTRACT: A series of 6 shaking table tests on various types of unreinforced masonry, was carried out in order to investigate the effects of strong ground motion, with varying characteristics. This was accomplished by estimating the damage potential of both actual and synthetic earthquake records, using various seismic motion parameters. The effects of amplitude, frequency content, duration and energy release of earthquake motion are correlated with observed damage in order to comment on their ability to represent the damage potential of shaking. The results showed that a combination of factors are contributing, with frequency content and amplitude being the most important. The root mean square acceleration during the strong phase of a record when combined with strong motion duration, proved to be a satisfactory indicator of damage.

1. INTRODUCTION

Unreinforced masonry is one of the most common construction types for residential buildings around the world. Many such buildings exist in areas of high seismic risk (South Europe, Central Asia and Latin America) as well in areas of lower risk (North Europe, Eastern North America). It is well known that unreinforced masonry has a poor behaviour under earthquake loads and its collapse has been the cause of a large proportion of loss of life in earthquakes. Knowledge on its seismic vulnerability has increased over the last 20 years, mainly as a result of post-earthquake damage surveys. However in most of these studies the vulnerability is usually expressed in terms of intensity scales (MSK or MM), or peak ground acceleration (PGA). Neither of these parameters is ideal to express the damagingness of ground shaking, as previous research has repeatedly shown (Housner, 1975; Sandi, 1988). One of the best ways to enhance our knowledge in this field, is to carry out detailed post-earthquake damage surveys, preferably in the vicinity of triggered strong motion instruments (as argued in another paper of this conference, Spence 1992). However in countries where masonry is still the predominant building type, the availability of digital records, that have a destructive potential is very scarce. Shaking table tests provide another means to study the effects of ground motion upon masonry structures. The Martin Centre has analysed damage distributions in the vicinity of 14 recording sites, after 7 destructive earthquakes but has also carried out 6 shaking table tests using full-scale masonry walls of various types in order to investigate the effect of various strong motion parameters upon damage occurrence.

2. SHAKING TABLE TESTS

Earthquake simulators, have two principal advantages in comparison to post-earthquake damage surveys :

* the possibility of controlling and repeating or incrementing the input motion on models that have well defined mechanical and dynamic properties and
* the ease in observing the response and damage pattern of each tested element.

The SERC shaking table in the University of Bristol (UK) is one of the few such facilities in Europe that can simulate motion in all 6 degrees of freedom. The platform of the table is 3.0 x 3.0 metres and is driven by eight hydraulic actuators each capable of generating a maximum thrust of 5 tonnes. The maximum specimen payload capacity is 15 tonnes. It is possible to reach an acceleration of 1.0g when the payload does not exceed 5 tonnes. Payloads of 15 tonnes can be tested up to 0.30g acceleration. The frequency range is 0.5 to 100 Hz. Maximum velocity is 0.50m/sec and peak displacement 150 mm.

2.1. The characteristics of the six models

Under these conditions, it was decided to build pairs of parallel walls using full-scale masonry units. Transverse walls were not included, but the walls were braced against transversal displacement. The purpose of these tests was to investigate the effect of different types of earthquake motion (in terms of strength, duration and frequency content) upon the behaviour of load-bearing wall models. The performance of the models was studied comparatively

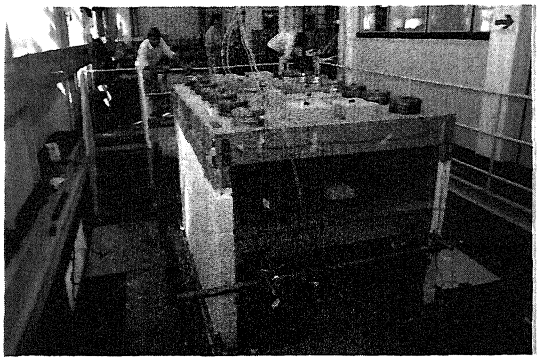

Photo 1. Model A2, before testing.

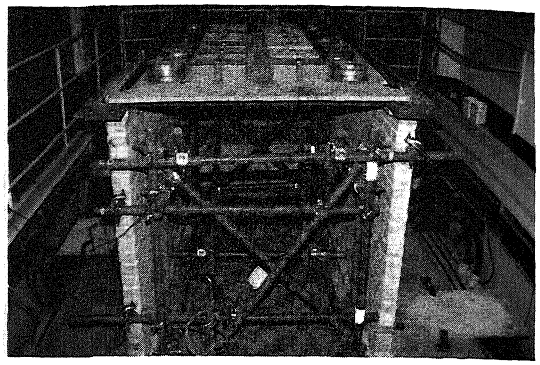

Photo 2. Model B1, before testing.

Table 1. The characteristics of the models

Model	Masonry Unit (mm)	Mortar mixes	Floor System	Model Size (cm)
A1	Lightweight Concr. Block (440x215x100)	1:1:6	Timber (joist hanger)	270 x 140x160
A2	Lightweight Concr. Block (440x215x100)	1:1:6	Timber (built-in joists)	270 x 160x160
B1	Lightweight Concr. Block (440x215x100)	1:1:6	Timber (built-in joists)	270 x 225x160
B2	Inner: as B1 Outer: Solid Clay Brick (215x65x100)	1:1:6	Timber (built-in joists)	270 x 226x160
C1	Sol.Concr.Brick (215x65x100)	1:2:9	None	225 x 157x160
C2	Sol.Concr.Brick (215x65x100)	1:2:9	None	225 x 157x160

Notes
Masonry Unit: length x height x width
Mortar Mixes: cement:lime:sand
Model Size: length x height x width

under actual and synthetic earthquake records. As shown in Table 1, several types of masonry materials were used.

The material used for the first 4 models (A1 to B2) was full-scale aerated lightweight concrete blocks (compr.strength=4N/mm^2;density=650 Kg/m^3). The models consisted of a pair of parallel walls of 2.70 metres length at a spacing of 1.35 metres. The height of each model varied. In three models, the system of the joists bearing directly on the wall was used, while in one model (A1) the joist hanger system was used for comparative reasons. Model B2, had in addition an outer leaf of typical clay bricks, with a 75 mm cavity separating it from the load-bearing wall. The two walls were tied together by means of wall ties (typical detail in some North European countries as well as in North America and Australia). Model B2 was identical to B1 except the addition of the outer skin. Simulated dead and live loads were also applied. The live load of 850 Kg was attached on the floor element (21 mm chipboard, supported by joists of 170x50 mm, placed at 600mm centres). The dead load simulating the weight of another storey and the roof above (950 Kg), was placed on a separate timber assembly on top of the walls, and strapped to them in order to avoid any out-of-plane movement (Photo 1). Thus models A1, A2,and B1 had less than 5 tonnes weight, while model B2 reached 6 tonnes.

Models C1 and C2 were made of solid concrete bricks (compressive strength=24 N/mm^2; density= 2100 Kg/m^3). The mortar mixture of 1:2:9 is the lowest grade accepted by the EC8 antiseismic code (Eurocode). Due to the higher density the walls were shortened to 2.25 metres. The spacing between the two parallel walls was kept 1.35 m. Both models were exactly the same, in contrast to the previous tests where some details were different. In tests C, a steel loading assembly was placed on top of the walls. The whole loading frame weighed 1200 Kg (Photo 2). The total weight of the models was 3.6 tonnes.

Two wallette specimens were constructed complying with the specifications required for standard masonry compression tests, for each of the two types of masonry unit. The value calculated from the mean of the maximum stresses achieved by the two wallettes was considered to be their characteristic compressive strength. For the aerated lightweight blocks the ultimate compression load was 3.6 N/mm^2, while for the concrete bricks it was 5.6 N/mm^2.

2.2. Input motion and instrumentation

All models were subjected to seismic loads acting in two directions, namely the in-plane horizontal (along the axis of the walls) and vertical. Out-of-plane horizontal input motion was not induced.

Two types of input motion were used in all models (except C1). The first was real earthquake accelerogram and the second was a synthetic record. The accelerogram recorded during the 1986 Kalamata, Greece, earthquake (M_L=5.7) was used in

Table 2. Characteristics of records used in the tests

Record	PHA (cm/s^2)	MRSA (cm/s^2)	Arias Intens (cm/s)	Signif Durat. (s)	RMSA (cm/s^2)
Kalamata	272	860	71	6.3	79
Calitri	177	340	132	57	36

Notes:
PHA: Peak horizontal acceleration
MRSA: Mean horiz. spectral accel. (5% damping). This is the average spectral acceleration in the period range of 0.1-0.3 seconds. The values shown are the mean of the two horizontal components.
Significant duration: the time interval during which 90% of the record's energy was released.
RMSA: Root mean square acceleration during the strong phase of the record (horizontal).

tests A1 to B2. In tests C, the accelerogram recorded in Calitri, during the 1980 Campania, Italy, earthquake (M_S=6.8) was used. The input motion severity was incrementally increased, until significant damage occurred, after which the tests were ceased. Some of the strong motion parameters of the two records are shown in Table 2.

The Kalamata record is typical of near field recording of a moderate magnitude shallow earthquake, with short duration. The total record duration was 20 seconds, from which only 5.8 seconds were more than 5%g. The peak horizontal spectral acceleration was nevertheless 125%g (at 0.34 seconds). The damage suffered by the old masonry buildings in the vicinity of the recording station was severe: 35% suffered damage degree 4 (MSK) and another 30% suffered damage degree 3. Also 20% of the RC buildings in the vicinity of the station suffered damage degree 3. The record was induced in increments starting from 25% of the amplitudes, and continuing with 50%, 100% and 125%. The displacements of the platform during the 125% input, were near the maximum allowed values of the shaking table, therefore no further increase of the real earthquake motion was possible. In order to continue testing, a synthetic motion was composed that permitted higher accelerations and frequencies without exceeding the maximum allowable displacement.

The Calitri record is much longer because the Campania earthquake was a double shock event (the second shock had its origin somewhat closer to Calitri). The instrument was located 19 Km from the surface rupture. The focal depth of the earthquake was 18 Km. The recorded motion was not particularly strong. The total record duration was 75 seconds, from which 30.6 seconds were more than 5%g, and 17.5 seconds were more than 10%g. The peak horizontal response spectral acceleration was 59%g at 0.33 seconds. The damage suffered by the old stone masonry buildings in the vicinity of the recording instrument in Calitri was not severe: 37% of stone masonry buildings suffered damage degree 2 and another 22%, damage degree 1. There were no seriously damaged buildings in the vicinity of the recording station in Calitri. In test C1 the whole Calitri record was induced, while in test C2 only the stronger second part of the record was induced. The Calitri trace was also induced in increments starting from 100% of the record and then 150%, 200%, 250% and 300% in test C1 and 100%, 200% and 300% of the short part in test C2.

The natural frequency of the models used in tests A and B ranged between 8 and 12 Hz (8% damping) as opposed to the predominant frequency of 3 Hz of the Kalamata record. For models C it was 11.5 Hz (7% damping) as opposed to the predominant frequency of 2.9 Hz of the Calitri record. Therefore additional synthetic records were prepared for all the tests. In tests A and B the synthetic record, consisted of a 12 second motion, containing frequencies of 5, 8 and 10 Hz for the horizontal component and 8.5, 11 and 15 Hz for the vertical component. There were some differences between the amplitudes and sequence of motion input between tests A and B. The synthetic record used in tests A had stronger vertical amplitudes than the one used in tests B. In test C2 and after the short Calitri inputs, the same synthetic record that was used for tests A1 and A2 was used again (no synthetic inputs for test C1).

Acceleration was measured in 6 positions, in each model. Four accelerometers at the top of the model measured the in-plane horizontal response of both walls in two directions. The other two measured out-of-plane horizontal and vertical response at the top. Horizontal and vertical accelerometers were also placed at the platform of the shaking table measuring the two components of the input motion.

3. DAMAGINGNESS OF SEISMIC MOTION

Intensity scales and their relationships with peak ground acceleration are the conventional means of expressing the vulnerability of buildings with similar structural attributes. These parameters although indicative of the strength of seismic motion, are by no means adequate for a reasonable description of its damage potential. Other factors, like the released energy, duration of shaking, frequency content of the motion are of equal or even higher importance. The destructivity of ground motion is well illustrated by two recent contrasting examples. During the 1986 earthquake in San Salvador (M_S=5.4 and M_w=5.6) the peak acceleration reached 60%g or more, while the duration was shorter than 10 seconds (Shakal et. al., 1987). The damage to adobe and bahareque houses as well as RC framed buildings was extensive (Anderson, 1987). By contrast during the 1985 earthquake in Mexico (M_S= 8.1and M_w=8.1) the peak horizontal accelerations recorded in the epicentral region did not exceed 25%g, but the shaking had a long duration of more than 70 seconds (Hudson, 1988). The damage to masonry buildings in the epicentral region of this earthquake was not as severe, as one would expect from such a large magnitude earthquake. It therefore seems reasonable to assume that most of buildings have a capacity to withstand long durations of shaking as long as the acceleration does not exceed their yielding level.

In order to have a further insight on the influence of all the parameters involved upon damage occurrence, a combination of factors has to be considered. One parameter that will incorporate several factors will be better for explaining ground motion severity. The root mean square acceleration (RMSA) is such a parameter. It is derived from the definition of the energy of a strong motion record that is equivalent to the area under envelope of the squared acceleration as in the following expression:

$$E(a,t) = \int_{t_0}^{t_f} a^2(t)\, dt \quad [cm^2/s^3] \qquad (1)$$

where t_0 and t_f are the beginning and end of the record. From this expression the Arias intensity can be obtained, by multiplying with the factor $\pi/2g$. By plotting the value of energy E(a,t) against ellapsed time in a cumulative form and taking the 0.05*E(a,t) and 0.95*E(a,t) values and the times in which they occurred (t_1 and t_2), we can obtain the part of the record during which most of the energy was released. Trifunac and Brady (1975) have proposed that the time interval between these two limits, is a better way to express the strong motion duration, than that of the "bracket duration" (amount of time during which the acceleration exceeded 5% or 10% g). This is usually called "significant duration" ($t_2 - t_1 = t_{sd}$). The root mean square acceleration during the strong phase of the record will then be:

$$RMSA = \left[\frac{1}{t_{sd}} \int_{t_1}^{t_2} a^2(t)\, dt\right]^{1/2} \quad [cm/s^2] \qquad (2)$$

Several recent studies have examined the relationship of such parameters as energy, Arias intensity, and RMSA versus macroseismic intensity and have, as expected, found that the correlation is significantly improved in comparison with previously published relationships between intensity and peak acceleration. Nevertheless the energy of the record, or its significant duration, sometimes can be quite misleading. This is illustrated in Table 2 where the parameters for the two actual records used in our tests are compared. We see that although the Arias intensity was much higher in Calitri, the damage to low-rise masonry buildings was by far more severe in Kalamata. On the other hand we notice that the MRSA and RMSA values are reversed, thus in better relation with the observed damage.

Most recently the product between RMS acceleration and strong motion duration has been proposed by Wen et. al. (1988) as a parameter of motion destructivity, based on laboratory tests on reinforced concrete structural elements. This parameter is called "characteristic intensity" and in our tests was calculated using the significant duration of each input and the root mean square acceleration during the same interval, as in the following expression:

$$I_{char.} = RMSA^{1.5} * \sqrt{t_2 - t_1} \qquad (3)$$

4. TEST RESULTS

In order to investigate the effect of strong motion upon our models the energy and Arias intensity of each input were calculated and the Husid plots were plotted so that the significant duration of the record could be obtained. After that, the Housner intensity and RMS acceleration were calculated for both in-plane horizontal (L) and vertical (V) components. The RMS acceleration taking into consideration both input

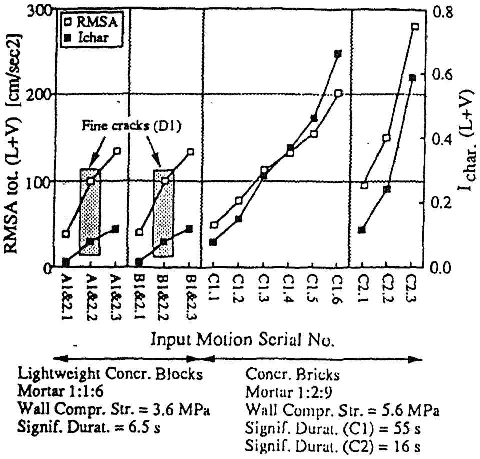

Fig.1 The behaviour of the models, during the actual earthquake inputs.

Table 3. Strong motion characteristics for all the real earthquake input motions, in all 6 tests

Test No.	Input No.	PHA (cm/s²)	PVA (cm/s²)	RMSA (cm/s²)	I_{Arias} (cm/s)	I_{char}
A1&2	1	121	150	40	12	0.02
(Kalam.)	2	315	271	100	58	0.08
	3	414	378	133	102	0.12
B1&2	1	121	150	40	12	0.02
(Kalam.)	2	315	271	100	58	0.08
	3	414	378	133	102	0.12
C1	1	204	257	49	120	0.08
(Calitri	2	339	334	76	281	0.15
whole)	3	449	565	112	610	0.28
	4	583	599	132	854	0.36
	5	777	667	156	1201	0.46
	6	723	995	200	1941	0.66
C2	1	218	240	95	118	0.12
(Calitri	2	488	385	153	349	0.24
short)	3	649	998	280	1039	0.59

Notes
PHA: Peak horizontal acceleration (in-plane)
PVA: Peak vertical acceleration
RMSA: total root mean square accel. (L+V)
I_{Arias}: mean between horiz. and vertical component
$I_{char.}$: characteristic intensity (L+V)

Table 4. Strong motion characteristics for all the synthetic input motions, in all 6 tests.

Test No.	Input No.	PHA (cm/s^2)	PVA (cm/s^2)	RMSA (cm/s^2)	I_{Arias} (cm/s)	$I_{char.}$
A1&2	4	186	361	115	86	0.10
	5	465	684	245	398	0.33
	6	617	1362	387	999	0.65
	7	855	1599	528	1861	1.04
B1	4	113	464	129	116	0.13
	5	227	456	148	140	0.15
	6	669	597	245	372	0.33
B2	4	84	385	132	115	0.13
	5	185	406	147	138	0.15
	6	582	804	248	369	0.33
	7	1121	1168	460	1648	0.82
C2	4	230	450	186	217	0.21
	5	400	656	283	506	0.40
	6	499	899	371	869	0.61
	7	657	1150	475	1417	0.88
	8	769	1550	589	2171	1.22

components (L+V) was also calculated. Some of the results are shown in Table 3 and 4 for real and synthetic inputs respectively.

Figure 1, shows the real earthquake inputs for all six models. The destructivity of motion is expressed by RMSA(L+V) and $I_{char.}$ As it is shown, in the case of tests A1 to B2, the highest RMSA was 13.3%g, and caused damage level D1 (without out-of-plane shaking). In tests C1 and C2, the strongest RMSA reached 28%g without causing any damage, despite the fact that the mortar was weaker. This maybe because the bonding strength was much higher in models C.

During the synthetic inputs, the levels of motion induced were much more substantial, with RMSA reaching 60%g and characteristic intensities exceeding 0.80 (Table 4). This in combination with the fact that the frequency content of the synthetic record, was much closer to the natural frequncy of the models, caused a range of damage levels, to all the models.

The level of input motions and damage are summarised in Figure 2. It is thus noticed that models A and B survived motions of up to 40%g, RMSA, without serious damage. The difference between A and B, is mainly because in tests A the inputs were predominantly vertical, while in B stronger horizontal amplitudes were also induced. In test B2, the cavity wall remained undamaged, despite the partial collapse of the load-bearing wall (without out-of-plane shaking). The Arias intensity of the input that caused the collapse of model B2 (B2.7), was lower than the highest actual earthquake input C1.7, which left the model undamaged. Notice though that in the case of characteristic intensity and RMSA this relationship is reversed. On the whole, as in the actual earthquake

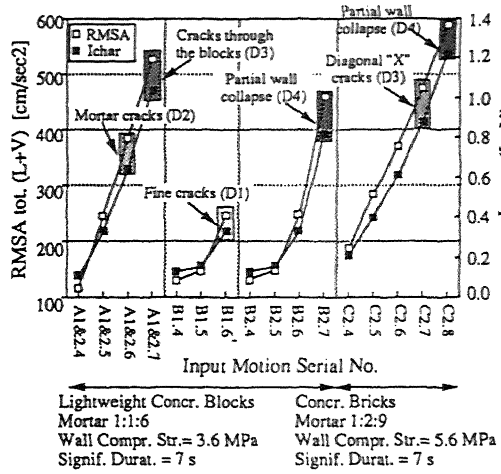

Fig. 2 The behaviour of the models, during the synthetic inputs.

inputs, the behaviour of the concrete brick walls was better, being able to withstand motions of around 40%g RMSA without significant damage.

5. CONCLUSIONS AND DISCUSSION

The frequency content of the motion proves to be an important factor along with the actual amplitude of motion in causing damage to masonry buildings. The RMS acceleration and characteristic intensity proved a much better parameter to explain the damage occurrence than the peak accelerations or the energy of the record. The energy of the record although a useful parameter, can be misleading when we compare the effect of records with different frequency characteristics. A parameter of motion damagingness that takes into consideration the natural period of the elements at risk is most important in order to improve our understanding and confidence in predicting damage to various types of structures. Analysis of earthquake damage surveys nearby recording stations showed that the response spectrum of the motion, is important in explaining the damage to buildings at different ranges of natural period. It was found that for the most common residential masonry buildings, the average value of response spectral acceleration in the period range of 0.1-0.3 seconds correlates the best with the observed damage (Spence et. al., 1992).
In a similar way because RMS acceleration is also connected to the spectral content of a record (Manic et. al., 1986), integration of the power spectrum of a record in the frequency range relevant to our building stock, will give us the RMS acceleration level that has affected the buildings in question. For common low-rise masonry the 3 - 10 Hz range of frequency can be used, as in the following equation.

$$\mathrm{RMSA} = \left[\int_{3}^{10\mathrm{Hz}} \mathrm{PSD(f)\,df}\right]^{1/2} \quad (4)$$

where PSD (f) = power spectral density of the record.

6. ACKNOWLEDGEMENTS

This research programme was partly funded by the Department of Environment and partly by the Science and Engineering Council of UK. We are also particularly indebted to Dr. J. Brownjohn, and Dave Ward of the Shaking Table Laboratory in Bristol, for their invaluable help.

7. REFERENCES

Anderson R.W. 1987. The San Salvador earthquake of October 10, 1986 - Review of building damage. Earthquake Spectra Vol. 3, No. 3.

Housner, G.W. 1975. Measures of severity of earthquake ground shaking. Proceed. of the U.S. Nat. Conf. on Earthq. Engineering, Ann Arbor, Michigan, June 1975.

Hudson, E.D. 1988. Some recent near-source strong motion accelerograms. Proceed. of the 9th W.C.E.E. Vol. 2, Tokyo, Japan.

Manic, M., Olumceva, T. and Stojkovic, M. 1986. Strong ground motion characteristics of the April 15, 1979 Montenegro earthquake in the epicentral area. Proceed. of the 8th E.C.E.E. Vol. 1, Lisbon, Portugal.

Sandi, H. 1986. An engineer's approach to the scaling of ground motion intensities. Proceed. of the 8th E.C.E.E., Vol. 1, Lisbon, Portugal.

Shakal, A. F., Huang, M., Linares, R. 1987. The San Salvador earthquake of October 10, 1986 - processed strong motion data. Earthquake Spectra Vol. 3, No. 3.

Spence, R.J.S., Coburn, A.W., Sakai, S. and Pomonis, A. 1992. The PSI scale: a scale of seismic intensity for use in vulnerability assessment and seismic risk analysis. To be presented at the 10th W.C.E.E., Madrid, Spain.

Trifunac, M.D. and Brady, A.G. 1975. A study on the duration of strong earthquake ground motion. BSSA, Vol. 65, No.3, June 1975.

Wen, Y.K., Ang, A.H.-S., and Park, Y.J. 1988. Seismic damage analysis and design of reinforced concrete buildings for tolerable damage. Proceed. of the 9th W.C.E.E. Vol. 8, Tokyo, Japan.