EFFECT OF EARTHQUAKE DURATION AND SEQUENCES OF GROUND MOTIONS ON STRUCTURAL RESPONSES

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Abstract: This study analyses the behaviour of an unreinforced masonry wall under ground motions with different duration and intensities. The influence of successive ground motions with different intensities is also taken into account. The following factors are considered: i) records with the same intensity and different duration, ii) records with the same duration and different intensities, iii) a single long-duration ground motion event in comparison with a sequence of ground motions with equivalent duration, and iv) single main event in comparison with the same ground motions with several aftershocks.

The numerical results reveal the importance of the main event duration in structural responses and the accumulation of damages when the structure is affected by a sequence of ground motions.

Keywords: Time-history analysis, sequence of ground motions, ground motion duration.

1 INTRODUCTION

In the recent years, some researchers have investigated the effect of ground motion duration on the response and degradation of structures (Bommer et al., 2004; Bommer and Martinez-Pereira, 1999; Iervolino et al., 2006). However, studies on the effect of successive ground excitations are still very limited. In general, structural engineers consider the properties of the time-history of ground motions indirectly, and only some parameters such as peak ground acceleration are considered explicitly. Most of the studies on the effect of earthquake duration have demonstrated a correlation between earthquake duration and damage. They suggest that the seismic assessment of existing buildings can be improved by taking into account the earthquake duration. So far material degradation due to duration of longer earthquakes and earthquake sequence are often neglected.

This situation is especially critical for unreinforced masonry (URM) buildings, e.g. as damages observed during the most recent major earthquake in Sichuan province on 12th of May 2008. These structures have shown a poor performance attributable not only to their age (most of them are part of the architectural heritage building stock in many countries), but also, because of uncertainties in the fabrication process of their bricks (usually handcrafted) and in the construction process of the buildings (usually without engineering design). This situation has triggered studies on their characteristics and behaviour under seismic action (Griffith, 2008; Ingham, 2008; Lourenco, 2008).

The aim of this study is to analyse the response of URM structures under ground excitations with different durations and under a sequence of ground motions. In particular, the behaviour of an URM wall under earthquake ground motions is analysed using numerical models.

2 NUMERICAL MODEL AND RESPONSE PARAMETERS

2.1 Numerical model

The structure analysed corresponds to URM wall with 1000 mm high, 4240 mm long, 240 mm thick, and a total mass of 10000 kg. The compressive strength (f_c) and Young's modulus (E) shown in Table 1 correspond to the typical values for New Zealand stiff bricks with soft mortar (New Zealand Society for Earthquake Engineering, 2006).

The wall was modelled with the software Ruaumoko (Carr, 2004) as a single-degree-of-freedom system with structural properties according to the geometry and material characteristics (Figure 1), with a natural frequency of 2 Hz and a damping of 15 %.

The nonlinear behaviour of the structure was represented by a modified Takeda hysteretic model with parameters $\alpha = 0.4$ and $\beta = 0$. The yielding force of 22.8 kN is assumed for both directions, and the post yielding stiffness is 0.168 k_0 (Figure 2).

2.2 Earthquake Records

In the analysis, a series of real ground motion records were selected, however near-fault records were not considered in this research. The records were characterized by the following parameters:

- The peak ground acceleration (PGA) representing the intensity of the ground motion;
- The significant duration (D_s) defined as the time required to develop the Arias intensity in the range between 5 % and 95 % of its total value (Trifunac and Brady, 1975);
- The Arias intensity (IA) for quantifying characteristics of the record related to both intensity and duration (Arias, 1970).

2.3 Response parameters

The response of the structure is described by the largest displacement (u_{max}), the residual displacement (u_{res}), and the maximum ductility (μ_{max}).
The cumulative effect of the excitation on the structure is assessed by the damage index (DI) defined in the reference (Park and Ang, 1985) with parameter β equal to 0.05.
It is important to note that u_{res}, μ_{max} and DI are equal to zero when the structure behaves only in the elastic range.

Table 1. Material properties (MPa)

	f_c	E
Soft Mortar	1.0	7000
Stiff Brick	12.0	12000

3 CASES ANALIZED

Four cases were analysed in this study:

- Case 1: Comparison of records with the same intensity and different duration;
- Case 2: Comparison of records with the same duration but different intensities;
- Case 3: Comparison of a single long-duration ground motion event with a sequence of ground motions of an equivalent duration;
- Case 4: Comparison of the effect of a single ground motion event with the same ground excitation followed by aftershocks.

3.1 Case 1

The ground motions selected for Case 1 were the record HKD85 of the 2003 Hokkaido earthquake in Japan, and the record at the base of the Matahina Dam of the 1987 Edgecumbe earthquake in New Zealand. Both with the same intensity (PGA = 0.283 g), but significant different durations. The records are shown in Figure 3, and their characteristic in Table 2.

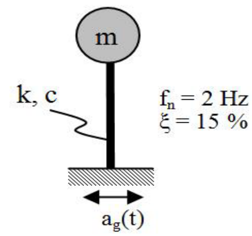

Figure 1: Single-degree-of-freedom model

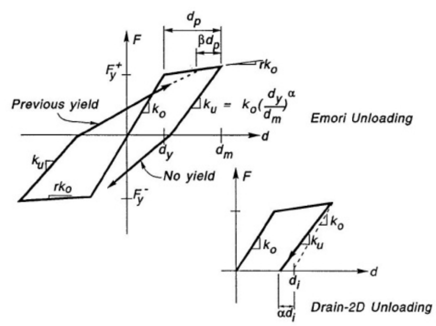

Figure 2: Modified Takeda hysteretic model (Carr, 2004)

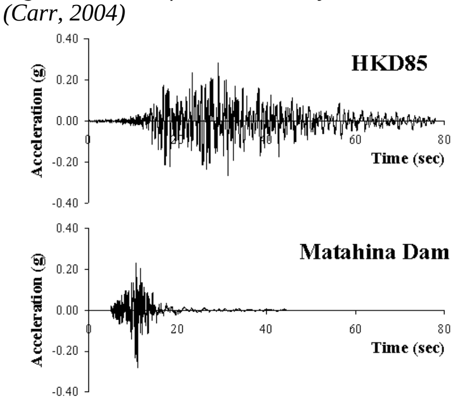

Figure 3: Records for Case 1

The results in Table 3 indicate that the response parameters were not able to detect any significant difference between the structural responses due each excitation. The parameters u_{max} and μ_{max} had similar values, and from the parameter u_{res} it was not possible to extract any conclusion. However, the parameter DI indicated that the structure suffered more degradation when it was excited by the ground excitation with a longer duration.

Table 2. Characteristics of records in Case 1

Record	PGA (g)	D_s (sec)	IA (m/sec)
HKD85	0.283	39.3	186
Matahina	0.283	6.2	33

Table 3. Structural response in Case 1

Record	u_{max} (mm)	u_{res} (mm)	μ_{max}	DI
HKD85	34.5	3.0	2.42	1.215
Matahina	30.3	5.78	2.13	0.877

3.2 Case 2

In the second case, the ground motions selected were the El Centro record of the 1940 Imperial Valley earthquake in California, and the La Union record of the 1985 Mexico earthquake. Both with the similar durations (D_s = 24.3 sec), but different intensities. The records are shown in Figure 4 and their characteristic in Table 4.

In this situation, the non-linear response was strongly related to the intensity of the ground motion. To be precise, the record with higher PGA induced higher values of u_{max}, u_{res}, μ_{max}, and DI. The greater magnitude of the displacements explains the higher DI, because the non-linear displacements induce an increment of the DI component related to the maximal non-linear response of the structure. Table 5 presents the response of the structure.

Table 4. Characteristics of records in Case 2

Record	PGA (g)	D_s (sec)	IA (m/sec)
El Centro	0.347	24.3	91
La Union	0.163	24.3	51

Table 5. Structural response in Case 2

Record	u_{max} (mm)	u_{res} (mm)	μ_{max}	DI
El Centro	44.6	7.9	31.4	1.281
La Union	16.2	0.1	1.14	0.473

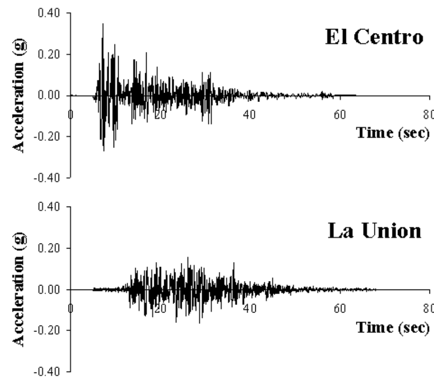

Figure 4: Records for Case 2

3.3 Case 3

In the third case, the HKD85 record of the 2003 Hokkaido earthquake and the record of Llolleo of the 1985 Chile earthquake are compared with a sequence of records (Series 1) with a total significant duration equivalent to the HKD85 and Llolleo (D_s≈35 sec).

The Series 1 is compounded of the Pacoima Dam record of the 1971 San Fernando earthquake (USA) scaled by a factor 0.528, followed by the original record of Matahina Dam and the original record of Matahina Dam, again. The scaling factor applied to the record of Pacoima Dam in the Series 1 was used to match the PGA of the record of Llolleo, and the IA of HKD85.

The records are shown in Figure 5, and their characteristic in Table 6.

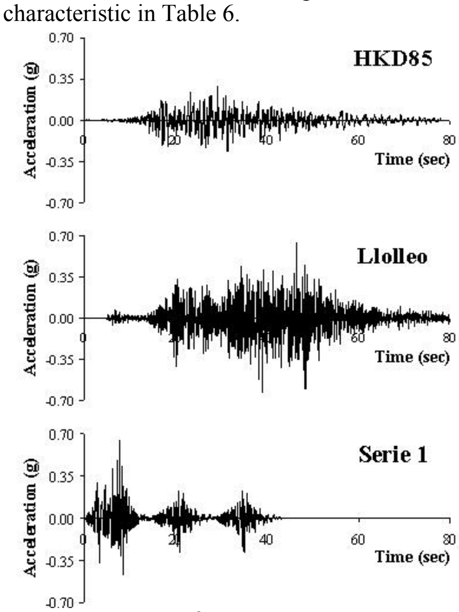

Figure 5: Records for Case 3

Comparing the results of Llolleo with those of Series 1, which have similar PGA (0.646 g) different responses can be observed. The excessive value recorded in every response parameters of Llolleo indicates total collapse of the structure.
In contrast, the HKD85 record has a lower PGA (0.283 g) compared to the Series 1 record (0.646 g), but their IA are the same (186). In this case, the responses of the structures were similar. These situations indicate that the IA is a better record parameter for predicting ground motion induced degradation of a structure, combining the whole acceleration history and not only the maximum value (PGA) or only the duration (D_s). The results of this analysis are presented in Table 7.

Table 6. Characteristics of records in Case 3

Record	PGA (g)	D_s (sec)	IA (m/sec)
HKD85	0.283	39.3	186
Llolleo	0.646	35.2	747
Series 1	0.647	33.0	186

Table 7. Structural response in Case 3

Record	u_{max} (mm)	u_{res} (mm)	μ_{max}	DI
HKD85	34.5	3.0	2.42	1.215
Llolleo	817	801	5734	22.9
Series 1	30.1	2.8	2.11	0.965

3.4 Case 4

The last case displays a comparison of the structural response due to the main event of an earthquake with the one due to the same main event (MGM) with several aftershocks (AS). The seismic event selected was the Chy074 record of the 1999 Chi-Chi earthquake in Taiwan, including four aftershocks recorded on the same station and named in the order of their recording. The records are shown separately in Figure 6 and their characteristic on Table 8.

Table 8. Characteristics of records in Case 4

Record Chy074	PGA (g)	D_s (sec)	IA (m/sec)
MGM	0.234	28.5	55
AS1	0.040	14.4	1
AS2	0.062	15.4	3
AS3	0.323	7.4	69
AS4	0.135	12.6	13

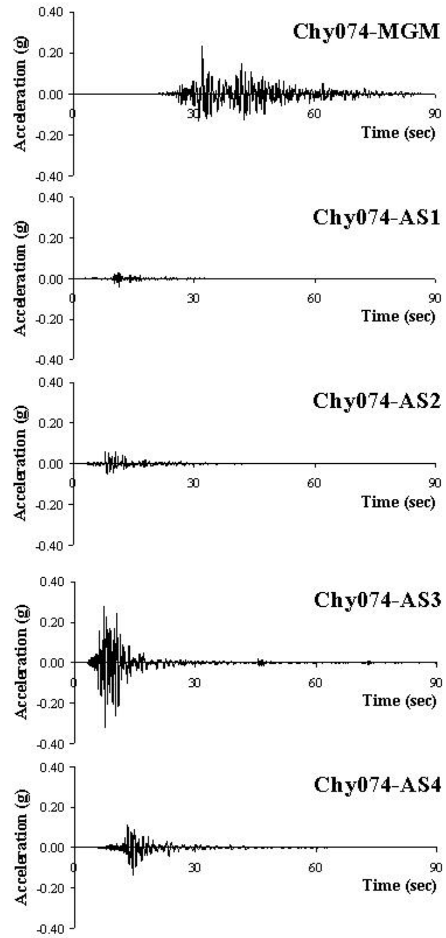

Figure 6: Records for Case 4

In Case 4a, the structure was excited by a sequence of records compounded of the main ground motion and the aftershocks AS1, AS2 and AS4 (Figure 7), excluding AS3, even thought a sequence including this aftershock is possible. This analysis had the goal to detect the damage increment due to ground motions of lower intensity recorded after the main event.
In case 4b, the sequence of records was compounded of the main event and the aftershocks AS1, AS2, AS3 and AS4 (Figure 8). In this case, the analysis focused on the variation in the structural response related to AS3 due to the precedent ground motions.
The characteristics of the record sequences after each ground motion are indicated in Table 9 (Case 4a) and Table 10 (Case 4b).

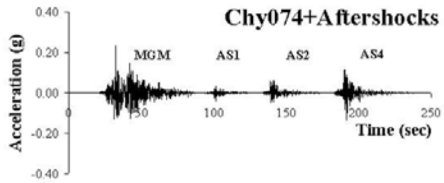

Figure 7: Records Sequence in Case 4a

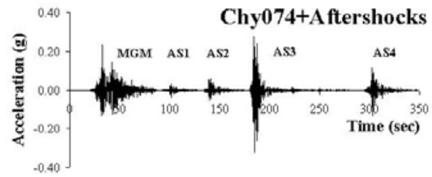

Figure 8: Records Sequence in Case 4b

Table 9. Characteristics of records in Case 4a

Record Chy074	PGA (g)	D_s (sec)	IA (m/sec)
MGM	0.234	28.5	55
+AS1	0.234	31.1	56
+AS2	0.234	107.8	58
+AS4	0.234	162.5	71
Note: + means that the record is added to the previous records to form a sequence			

Table 10. Characteristics of records in Case 4b

Record Chy074	PGA (g)	D_s (sec)	IA (m/sec)
MGM	0.234	28.5	55
+AS1	0.234	31.1	56
+AS2	0.234	107.8	58
+AS3	0.323	156.7	127
+AS4	0.323	271.6	140
Note: + means that the record is added to the previous records to form a sequence			

The results of the analysis of each record (Table 11) indicated that only MGM and AS3 were capable of inducing a non-linear response of the structure. It is interesting to remark, that the aftershock (AS3) seems to be more "destructive" than the main ground motions (MGM), with DIs equal to 0.677 and 1.032, respectively.
The case 4a (Table 12) shows that DI at the end of the whole considered excitation sequence is 2.4 % larger than DI after MGM. The records AS1, AS2 and AS4 also showed a non-linear response when they were analysed as a sequence, and showed just an elastic response when they were analysed as isolated records.
Focusing on the structural response of the strongest aftershock (AS3) and based on the results of case 4b (Table 13), it was possible to detect an increment of 11.6 % when the records was analysed as part of a sequence. It was larger than when it was analysed as a single isolated ground excitation. This result demonstrates the negative effect of the pre-existent damage due to previous incursions in the non-linear range, which is ignored if AS3 is analysed alone.
Finally, it is necessary to mention that the parameters u_{max}, u_{res} and μ_{max} are not capable to detect any significant difference in the response.

Table 11. Structural response in Case 4

Record Chy074	u_{max} (mm)	u_{res} (mm)	μ_{max}	DI
MGM	22.4	2.9	1.57	0.677
AS1	3.3	0	0	0
AS2	5.5	0	0	0
AS3	32.4	0.39	2.28	1.032
AS3	12.6	0	0	0

Table 12. Structural response in Case 4a

Record Chy074	u_{max} (mm)	u_{res} (mm)	μ_{max}	DI
MGM	22.4	2.9	1.57	0.677
+AS1	22.4	2.9	1.57	0.678
+AS2	22.4	3.1	1.57	0.681
+AS4	22.4	3.0	1.57	0.693

Table 13. Structural response in Case 4b

Record Chy074	u_{max} (mm)	u_{res} (mm)	μ_{max}	DI
MGM	22.4	2.9	1.57	0.677
+AS1	22.4	2.9	1.57	0.678
+AS2	22.4	2.9	1.57	0.681
+AS3	34.6	3.1	2.43	1.152
+AS4	34.6	0.9	2.43	1.180

4 CONCLUSIONS

The response parameters related to displacement (u_{max}, u_{res}, μ_{max}) were not capable to measure the effect of the earthquake duration, because they are manly related to the intensity of the record (PGA). In contrast, the accumulated damage (DI) depends strongly on the duration of the earthquake. The combined parameter IA offers a better measurement for the degradation capacity of the ground motion.
Once the structure had incurred in the non-linear range, all the subsequent excitations induced an

increment in the damage, no matter how small their intensities are.

When the effect of aftershocks was analysed, it was important to consider the possible incursions in the non-linear range of the structure due to precedent ground motions. The damage produced by the aftershock might be significantly larger, if the effect of the precedent ground excitations were considered. This situation is not detected by the response parameters related to displacement.

The damage development is a complex phenomenon that depends on the intensity and duration of the excitation, and also on the precedent and subsequent events.

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