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Analysis of Seismic Performance of Confined Masonry Buildings during the Wenchuan Earthquake

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Abstract

Abstract: In order to study the seismic performance of masonry structure, nonlinear dynamic analysis of 12 building models was carried out. Totally 6 models marked A1-A6 were chosen from affected region in Wenchuan earthquake, and the other 6 models marked B1-B6 were fictitious unconfined masonry structures having the same configuration and other characteristics as A1 to A6 except for in confinement details. Totally 20 accelerograms were collected on different soil conditions in Wenchuan earthquake were selected as input accelerations. The study focuses on two aspects: seismic response comparison between the numerical analysis results and the observations from the earthquake investigation; and the response of the confined structures and the unconfined structures to the same inputting PGA. It implied that the ductility of confined masonry buildings was significantly improved and anti-collapse capacity was enhanced comparing with the unconfined ones.

Keywords:confined masonry buildings; nonlinear dynamic analysis; Wenchuan earthquake, damage; seismic performance

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1. Introduction

On May 12, 2008，a Ms 8.0 earthquake occurred in Wenchuan County, Sichuan Province, which induced great damage to buildings especially for masonry buildings (MBs). However, there were still some confined masonry buildings were not collapsed in areas of intensity 11.

In this paper three 3-story MBs, two 4-story MBs and one 7-story MBs were studied by nonlinear dynamic analysis. The analysis results were compared to the observations of actual damages in order to study the effect of structural column (SC) and ring beam to the seismic performance of masonry buildings. Non-linear study of other 6 models of unconfined MBs with similar geometric details as previous models were also carried out.

2.The selection of accelerograms of near-filed ground motion

Totally 20 accelerogram records of Wenchuan near-filed ground motion of different site conditions were selected. The average acceleration response spectrum was given in Fig.1. It shows that in bedrock the amplification factor of MB whose natural vibration period is less than 0.21s is larger than that in soft soil, while that between 0.21s and 0.6s was reversed. The average predominant periods of accelerograms on bedrock and soft soil site are summarized in Table 1.

Table 1 – Characteristics of near-fault ground motion recordings in Wenchuan Earthquake used in the study

Site type	near-fault ground motion($R \leq 20$ km)		
	number	PGA (Gal)	Tg (sec)
bed rock	3	120,142,306	0.14,0.18,0.19
soil site	17	105-950	0.22-0.42

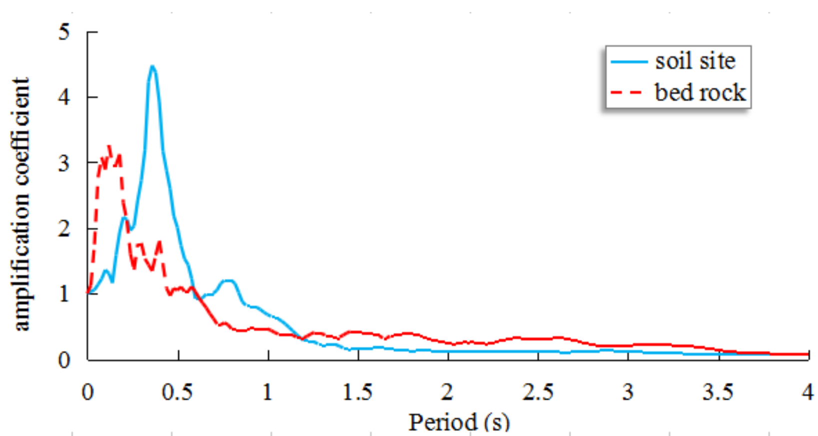


Fig. 1 – Acceleration spectra of the Wenchuan earthquake records selected in the analysis

3. Nonlinear dynamic analysis and comparison

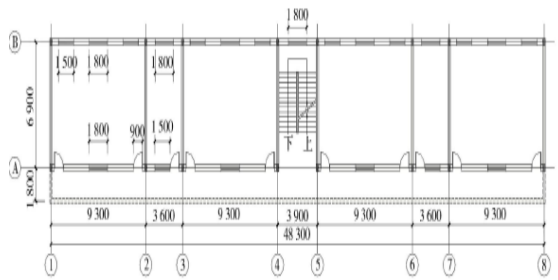
3.1 Description of examples

The characteristics and parameters of the models marked A1-A6 and the compared unconfined models marked B1-B6 were shown in table 2. A1, A2, A3 are 3-story buildings, A4 and A5 are 4-story buildings and A6 is 7-story building. The plan of each model is shown in Fig.2

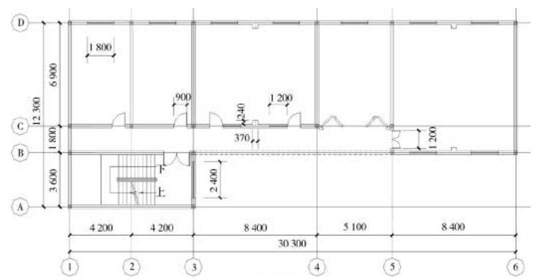
Table 2 – Parameters of the building models used in the paper

Mark	Height of	Fortification /seismic	Wall thickness & mortar strength	Arrangement of Structural columns (SCs) and ring beams
A1	3.6	7/Ⅹ	370mm in short direction, and 240mm in long direction; mortar strength: M5	SCs: Joints of transverse and longitudinal walls; Ring beams: every story at floor level
B1				No arrangement
A2	3.6	7/Ⅹ	Thickness: 240mm， mortar strength: M5	SCs: Joints of transverse and longitudinal walls, Ring beams: every story at floor level
B2				No arrangement
A3	3.2	7/Ⅸ	Thickness: 240mm， mortar strength: M5	SCs： joints of transverse and external walls and both sides of big openings Ring beams: every story at floor level
B3				No arrangement
A4	3	7/Ⅹ	Thickness: 240mm， mortar strength: M2.5	SCs： joints of transverse and external walls and both sides of big openings Ring beams: every story at floor level
B4				No arrangement
A5	3.6	7/Ⅸ	Thickness: 240mm， mortar strength: M5	SCs： joints of transverse and external walls and both sides of big openings Ring beams: every story at floor level
B5				No arrangement
A6	3	7/Ⅷ	Thickness: 240mm， mortar strength: M10 L1 M7.5 L2 & L3, M5 other levels	SCs: Joints of transverse and longitudinal walls; Ring beams: every story at floor level
B6				No arrangement

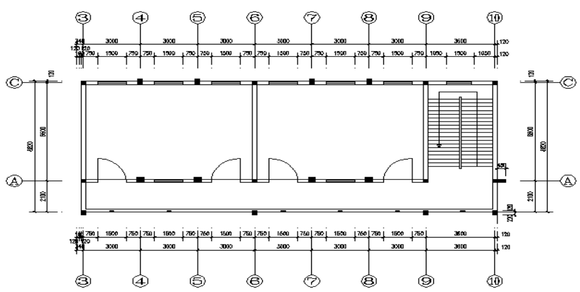
Table 2 Values of parameters for the sample buildings used in the paper



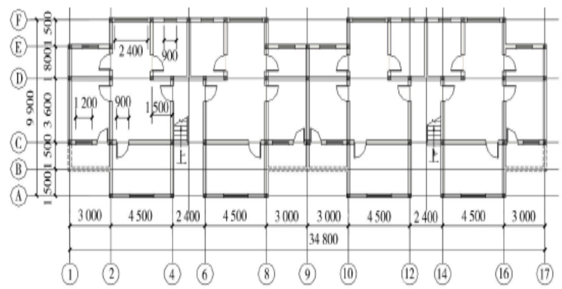
A1，B1



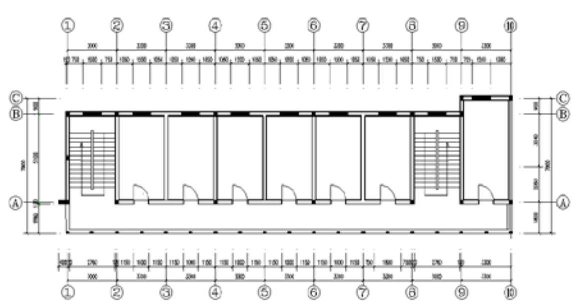
A2，B2



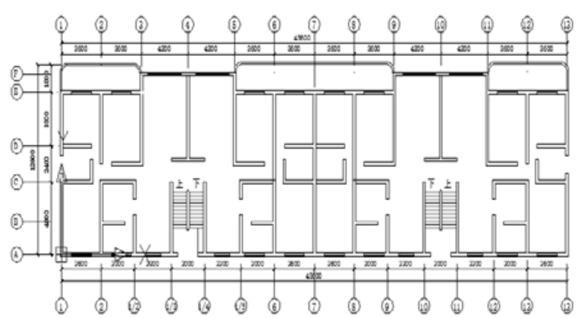
A3，B3



A4，B4



A5，B5



A6，B6

Fig. 2 – Plan of the samples

3.2 Nonlinear dynamic analysis model and characteristic curve

Multi-degree of freedom series model was used to simulate the masonry building model in this paper. According to the references [2-6], skeleton curve with negative stiffness(Fig.3) was used to simulate the interstory stiffness of masonry building in nonlinear dynamic analysis (reference 1). Calculation of the ultimate loading capacity of confined masonry wall is referred to reference 2, in which the contribution of SCs on masonry wall is taken into consideration. Relationship between wall performance and interstory drift ratio is summarized in table 3.

3.3 Nonlinear dynamic analysis results

The calculation results of ground acceleration corresponding to different building performances during earthquake are summarized on Table 4. Typical shear force, displacement and maximum acceleration of building models under action of different PGA inputs (220Gal , 400Gal and 620Gal) at bedrock and soft soil site conditions were given in Figure 4.

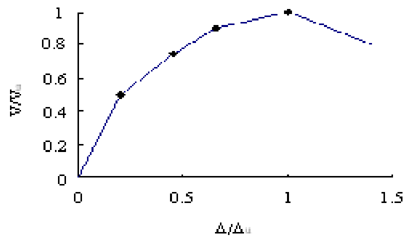


Fig. 3 – Skeleton curve for masonry walls

Table 3 – Values of basic characteristics for the studied confined buildings under different damage levels

Damage degree	intact	Slight damage	Moderate-damage	Severe damage	Near collapse
characteristics（interstory drift ratio）	1/2000	1/1600	1/700	1/350	1/200

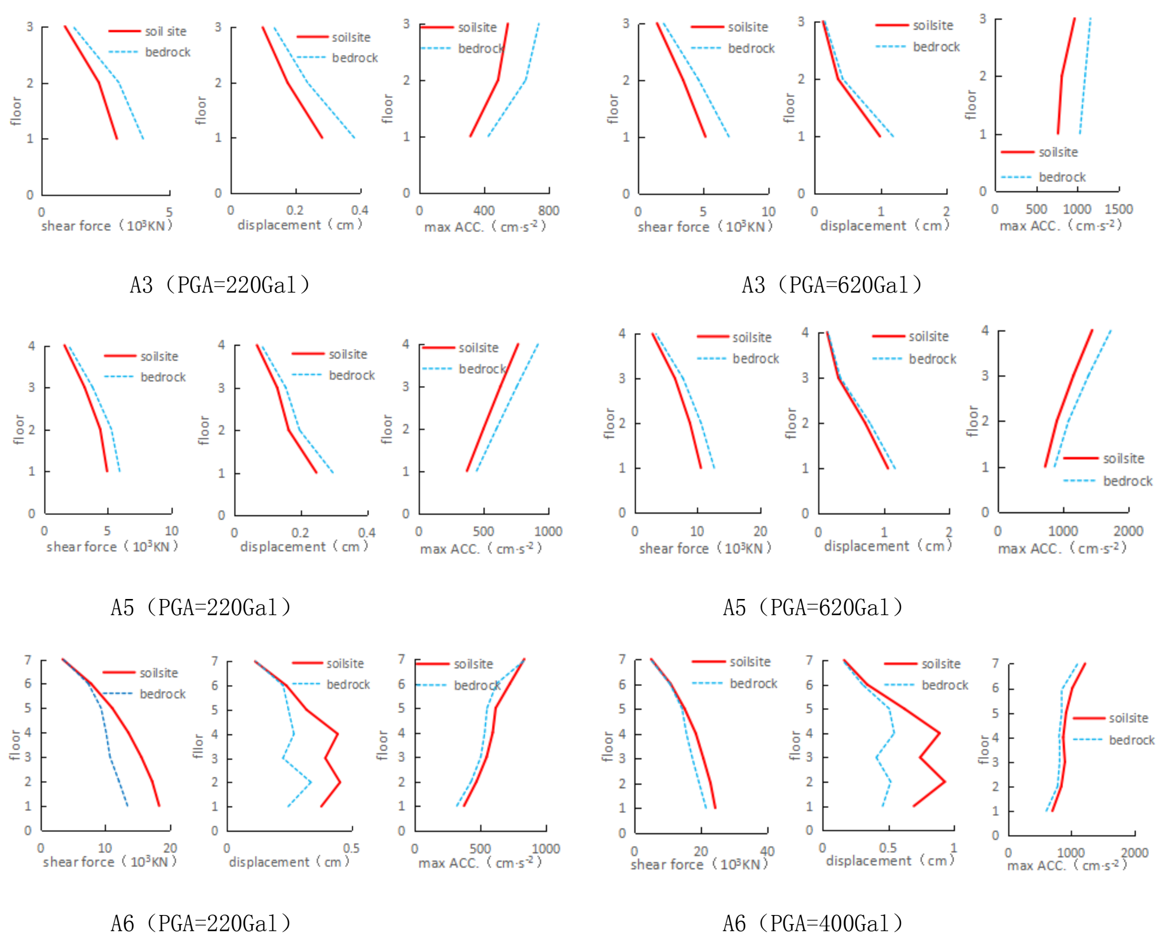


Fig. 4 – Distributions of the maximum seismic responses of the selected buildings along their height

Below characters can be found from Fig.4:

1. For 3-story and 4-story MBs (A1-A5,B1-B5), responses on bedrock were more intensive than that on soil site and for 7-story MBs (A6,B6) it reversed in the same input of PGA. This is mainly due to the predominant period of bed rock accelerograms being closer to the natural vibration period of 3-story and 4-story MBs (0.16s-0.19s) and the predominant period of the soft soil being closer to the natural vibration period of 7-story MB (0.29s) in this paper.

2. Comparing the results of unconfined MBs with those of confined MBs under the same configuration and other characteristics, the responses of unconfined MBs were much more intensive than confined ones in the same input of PGAs. For example, in bedrock situation at the condition of input PGA=400Gal, the peak acceleration response of confined MBs was average 25% lower than that of unconfined ones. Results can be read that confined MBs A1-A6 reached the state of severe damage at 385-578Gal, while unconfined MBs B1-B6 reached same state at 279-430Gal.

Table 4(a) – Seismic intensities corresponding to different damage states of the studied buildings on bedrock

Acc(Gal) / degree	A1	B1	A2	B2	A3	B3	A4	B4	A5	B5	A6	B6
Slight damage	141	110	163	121	160	122	170	124	165	131	185	131
Moderate damage	326	220	341	267	326	261	322	199	326	250	341	222
Severe damage	520	311	539	371	508	324	505	294	468	327	489	349
Near collapse	697	395	711	426	685	401	650	377	645	389	662	439

Table 4(b) – Seismic intensities corresponding to different damage states of the studied buildings on soil site

Acc(Gal) / degree	A1	B1	A2	B2	A3	B3	A4	B4	A5	B5	A6	B6
Slight damage	168	135	190	153	185	154	182	130	173	140	145	114
Moderate damage	352	258	372	285	360	280	330	210	340	263	261	177
Severe damage	565	356	578	430	541	371	523	309	521	342	385	279
Near collapse	726	481	746	508	716	479	681	405	670	404	591	372

3.4 Results compared with actual building damage in earthquake

Three typical examples (A3, A5 and A6) with different number of stories were selected to compare with actual damage during earthquake. According to the investigation results, all these three buildings were at the stage of severe damage.

Distributions of maximum interstory drift ratio along building height were given in Fig.5. According to GB 50011-2011, the input value of PGA=220Gal/400Gal/620Gal is corresponding to intensity Ⅶ/Ⅷ/Ⅸ.

Based on calculation, it can be found from Fig.5 that:

1. When the input PGA is 220Gal and the building located at soft soil site, the 1st floor of A3 and A5 were slightly damaged and the other floors were at the stage of intact, The 1st floor of A6 was intact but the 2nd and 4th floor were moderate-damage, and the other floors were slight damage.

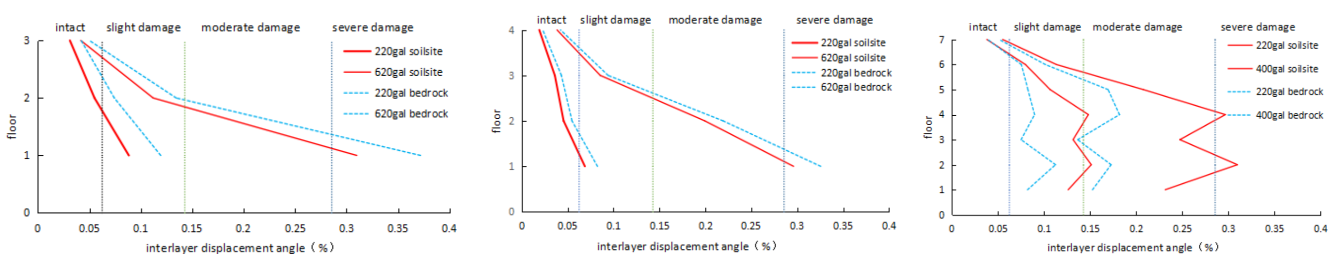
2.When the input PGA is 220Gal and the building located at bedrock, the 1st and the 2nd floors of A3 were slight damage and the other floors were at the stage of intact. The 1st floor of A5 were slight damage and the others were intact. The 7th floor of A6 were intact but other floors were slight damage.

3. When the input PGA is 400Gal and the building located at soft soil site, the 2nd and the 4th floors of A6 were severe damage. It is mainly due to the strength of mortar in the damaged floor was lower than that in adjacent lower floor; The model building performance fitted well in with the real responding in earthquake site.

4. When the input PGA is 400Gal and the building located at bedrock, the 1st, the 2nd, the 4th and the 5th floors of A6 were moderate-damaged; the 3rd and the 6th floors of A6 were slight damage, and the 7nd was at the stage of intact.

5.When the input PGA was 620Gal, the 1st floor of both A3 and A5 was severe damage. It fitted in well with the real building performance at earthquake site.

Some typical seismic performances of the buildings were selected in figures 6 to 8.



(a) A3 (b) A5 (c) A 6

Fig. 5 – Distributions of the maximum interstory shift of the selected buildings along their height

Fig6. – Damage of A3

Fig7. – Damage of A5

Fig8. – Damage of A6

3.5 Seismic resistance of confined MBs

1.SCs and ring beams significantly improved the integrity and ductility of MBs and also enhanced the loading capacity. Therefore confined MBs presented good performance under strong earthquake. The results from the nonlinear time history analysis showed that at bedrock, confined MBs A1 to A6 would be at the stage of severe damage when the input acceleration reached 520Gal 、539Gal、50
separately. While for unconfined MBs B1-B6, the building would be at the stage of severe damage when input acceleration reached 311Gal, 371Gal, 324Gal, 294Gal, 327Gal and 349Gal; If the building was at soft soil site, confined MBs A1-A6 would be severe damage when ground acceleration was above 565Gal, 578Gal, 541Gal, 523gal, 521Gal and 385Gal separately. While for unconfined MBs B1-B6, the corresponding acceleration was above 356Gal, 430Gal, 371Gal, 309Gal, 342Gal and 279Gal separately.

2.The contribution of SCs to the building resistance was ignored at ultimately limit state in Current code[1]. The calculation of the additional resistance from SCs and ring beams were considered in this paper and the results were verified by earthquake disaster observation.

3. A1, A2 and A4 were chosen from seismic intensity Ⅹ; A3 and A5 were chosen from seismic intensity Ⅸ; and A6 was chosen from seismic intensity Ⅷ, all of them were higher than the fortification intensity 7. However, due to the contribution of SCs, the buildings were seriously damaged but no one collapsed. It verified that confined MBs had good seismic performance.

4.Conclusions

In this paper, totally 20 typical near-field ground motion accelerograms on both bedrock and soft soil sites were selected for the nonlinear dynamic analysis of 12 confined and unconfined MBs. Comparing with the seismic disaster investigation results,, below conclusions can be drawn:

1. The damage levels of model buildings were generally coordinated with the investigated seismic performances of studied buildings, which proved that the model and parameter chosen for the analysis were reasonable.

2. Comparing with unconfined MBs, the seismic capacity of confined MBs were improved significantly due to the contribution of SCs and ring beams. For example, in bedrock situation at the condition of input PGA=400Gal, the peak acceleration response of confined MBs was average 25% lower than that of unconfined ones.

3. The calculation in the paper considered the contribution of SCs to the shear resistance of walls. And the results were closer to the actual seismic response of MBs comparing with the calculation without the consideration of SCs. Suggestion recommended that SCs should be considered when calculating shearing capacity of confined masonry walls.

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