SEISMIC PERFORMANCE-BASED ANALYSIS OF CONFINED MASONRY STRUCTURES

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ABSTRACT :
Confined masonry buildings (CMBs) have been widely used in China. The field survey conducted during the recent great Wenchuan Earthquake had been proved that the CMBs well designed and constructed performed well even in severely affected areas. However, the performance-based design for the CMBs has been few reported. In this paper based on the previous work conducted by the author a four-linear skeleton curve with negative stiffness for the confined masonry walls along with the parameters characterizing the different performance levels is developed. Nonlinear time history analysis (NTHA) and nonlinear static analysis -- pushover analysis (POA) for four typical 5- to 7-story confined concrete/ brick masonry buildings are performed. From comparisons of the numerical results some conclusions are made, they could be helpful for further investigation and promotion of a wider application of the CMBs.

KEYWORDS:
Confined masonry building, nonlinear time history analysis, pushover analysis, performance-based design

1. INTRODUCTION

The seismic performance of confined masonry walls, which consist of brick/concrete masonry walls confined by reinforced concrete (RC) tie-columns and tie-beams, were extensively studied in China after the 1976 Great Tangshan Earthquake. Buildings of confined masonry walls combined with RC floors of cast-in-situ plate or pre-casted slabs, in the later case the tie-beams become ring-beams, have been used more and more in recent 30 years in both China and Latin America countries etc. It has been recognized that the confinement contributed by the introduction of the RC tie-columns and tie(ring)-beams significantly increases the integral rigidity of the building. Due to the limited land source the maximum height of such buildings has reached 7 or even 8-stories in China. The field investigation in the recent 512 Wenchuan Earthquake shows that many buildings, including the CMBs, might suffer moderate to severe damage but without collapse even the shaking intensity being higher than the seismic design fortification intensity by III or IV (Wang YY, 2008). From the field survey conducted in the Dujiangyan City by Zhang MZ et al.(2008), where the design fortification intensity is VII, while the sustained intensity during the 512 earthquake was VIII to IX, it is concluded that for instance all the 5-story CMBs surveyed has an average damage index of 0.38 (near the lower bound of the moderate damage index ranging from 0.3 to 0.55) and no building collapsed. From the author's personal experience in the latest half month-field investigation similar conclusion can be made. Both from the engineering practice and the lessons learned from the Wenchuan earthquake the seismic design needs to be based on achieving multiple performance objectives. Up to date the performance-based design has been investigated intensively and adopted in engineering practice wider and wider for RC and steel structures. While for masonry structures there are few reported. The static nonlinear analysis is the key in the performance-based design, the main purpose of the present paper is to verify its applicability to the CMBs. In this paper based on the previous work conducted by the author a four-linear skeleton curve with negative stiffness for the confined masonry walls along with the

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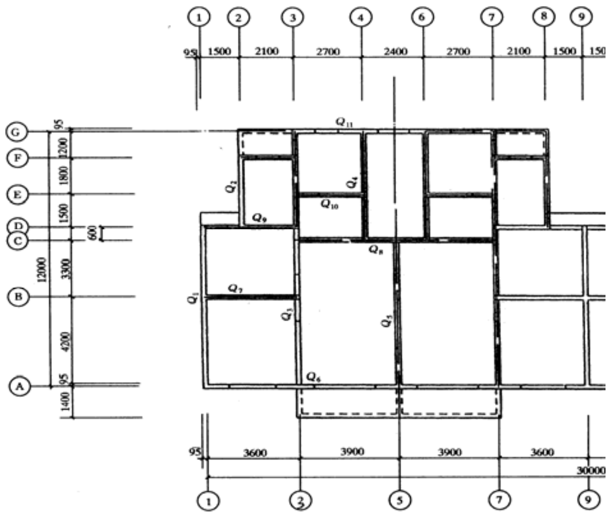
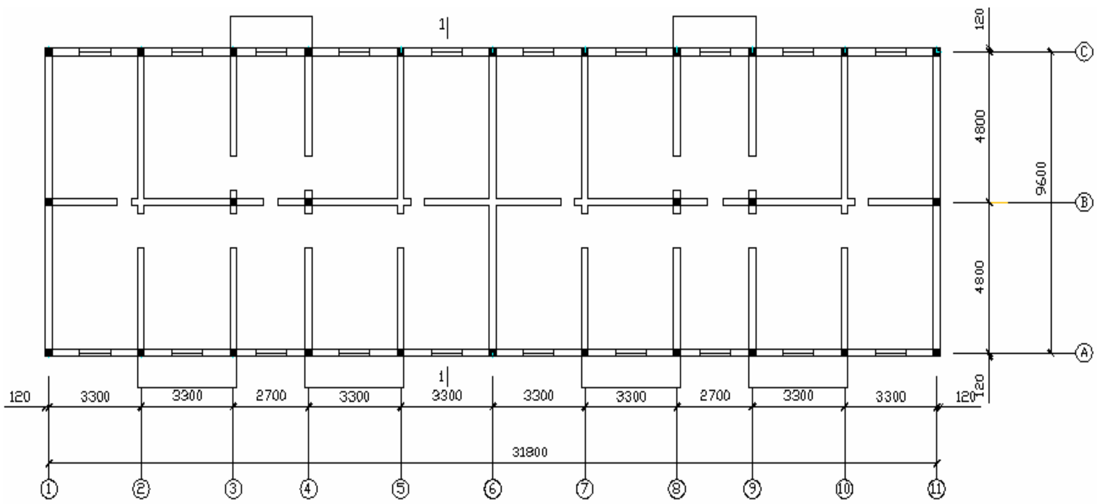
parameters characterizing the different performance levels is developed. Nonlinear time history analysis (NTHA) and nonlinear static analysis -- pushover analysis (POA) for four typical 5- to 7-story masonry concrete/ brick masonry buildings are performed. From comparisons of the numerical results some conclusions are made, they could be helpful for the further investigation and promotion of a more wider application of the CMBs.

2. DESCRIPTION OF THE BUILDINGS

Four confined concrete/brick masonry building designed according to Design Code for Masonry Buildings (GB 50003-2001）(PRC National Standard, 2001-a) representing the most common types of masonry buildings in China, including different constructional detailing, building heights etc. are selected in the analyses. Figure 1 shows their plane, their fundamental properties are listed in Table 1



(a) Structure 1#, 2# and 3#



(b) Structure 4#

Figure 1 Plane of the CMBs used in the analysis

Table 1 Fundamental properties of sample structures

	Structure 1#	Structure 2#	Structure 3#	Structure 4#
Wall materials	Small concret hollow blocks	Small concrete hollow blocks	Small concrete hollow blocks	Type KP1 bricks of multiple cells
Story number	5-stories for one unit	6-stories for one unit	7-stories for one unit	6-stories
Bearing system	Longitudinal walls	Longitudinal and transverse walls	Longitudinal and transverse walls	Transverse walls
Story height（mm）	2800	2800	2800	2800
Wall thickness（mm）	190	190	190	190
Construction detailing	RC core-or tie-columns installed at all intersection of long. and trans. walls, and the corners of external walls, the four corners of the stair shaft. RC tie-columns installed at both sides of the window and door openings	The same as for for Structure 1#, In addition, core-columns of space less than 2m be installed in the transverse walls		The same as for Structure 1# except for no core-columns installed
Concrete strength of tie-and core-columns	C20	C20	C20	C20
Longitudinal reinforcement steel bars in tie-columns	4Φ14	4Φ14	4Φ14	4Φ14
Stirrups in tie-columns	Φ6	Φ6	Φ6	Φ6
Longitudinal reinforcement steel bars in core-columns	1Φ14	1Φ14	1Φ14	
Mortar strength	M10（1st and 2nd floors）, M7.5 (3rd-5th floors）	M10（1st - 3rd floors）, M7.5 (4th - 6th floors）	M10（1st- 4th floors）, M7.5 (5th - 7th floors）	M10（1st -2nd）floor), M7.5 (3rd - 4th floor）, M5（5th - 6th floor）
Seismic fortification intensity	VIII	VIII	VII	VIII

3. RESTORING FORCE-DISPLACEMENT MODEL AND THE RELATIONSHIP BETWEEN THE PERFORMANCE OBJECTIVES AND THE LEVEL OF DAMAGE TO THE STRUCTURE

Based on the results obtained by XIA, et al.(1988) and XIONG (2004）from a great amount of pseudo-static tests of masonry walls and shaking table tests of building models the skeleton curve of their restoring force-displacement relationship can be represented by using a four-linear modeling as shown in Figure 1, where the points A, B, C, D and E characterize elastic limit, slight damage, moderate damage, serious damage, and collapse, respectively. The V_i and Δ_i for each of these characteristic points can be evaluated by using expressions suggested by Xia et al.(1988) and Xiong (2004). Table 2 gives these values normalized to the ultimate strength (V_u) and ultimate displacement(Δ_u), respectively at point D for walls with both RC core-columns and tie-columns and walls with RC tie-columns, respectively.

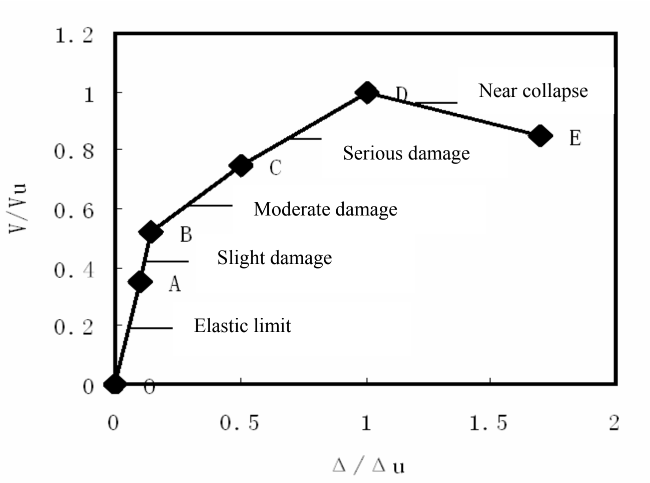


Figure 2 Skeleton curve and damage levels for confined masonry walls

Table 2 Characteristic parameters in restoring force-displacement relationship of confined masonry walls

	Walls with both RC core-columns and tie-columns					Walls with RC tie-columns				
	A	*B*	*C*	*D*	*E*	*A*	*B*	*C*	*D*	*E*
Δ/Δ_u	0.14	0.19	0.50	1.00	1.80	0.2	0.35	0.60	1.00	1.30
V/V_u	0.4	0.55	0.75	1.00	0.85	0.4	0.65	0.80	1.00	0.85

4. NONLINEAR TIME HISTORY ANALYSIS (NTHA) OF STRUCTURES

4.1. Basic equations and expressions

In the seismic response analyses of CMBs in general the multi-degree-of-freedom (MDOF) lamped mass shear model is used and the equation of motions can be written in Eq. (1), if the iteration technique with constant stiffness in the solution of the equation of motion is adopted.

$$[M]\{\Delta\ddot{x}\}+[C_0]\{\Delta\dot{x}\}+[K]\{\Delta x\}=-[M]\{\Delta\ddot{x}_g(t)\}-\{\Delta R\} \tag{1}$$

$$\{\Delta R\}=([K_0]-[K])\{\Delta x\}+([C_0]-[C])\{\Delta\dot{x}\} \tag{2}$$

Where $[M],[K]$ and $[C]$ are the mass, stiffness and damping matrix, respectively; $[K_0]$ and $[C_0]$ are the initial stiffness and damping matrices, respectively; $\{\Delta x\}$, $\{\Delta\dot{x}\}$ and $\{\Delta\ddot{x}\}$ are the displacement, velocity and acceleration vector increments, respectively; and $\{\ \Delta R\ \}$ the nonlinear force vector modification term.
In the numerical solution the Newmark integral technique was adopted.

4.2. Hysteretic curves

The skeleton curve of the confined masonry walls has been given in the section 3, the hysteretic curves for the dynamic analysis is shown in Figure 3.

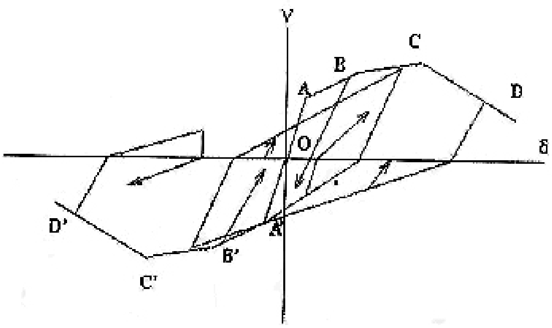


Figure3 Representative hysteretic characteristics of confined concrete / block masonry structure

In the NTHAs a total of 77 accelerograms recorded on different site conditions were used as the input motions. The peak ground accelerations (PGAs) of all recordings used are scaled to the value corresponding to that of the frequently, design protected or rarely encountered earthquake specified by the current China design code PRC National Standard (2001-a).

5. NONLINEAR STATIC(PUSHOVER) ANALYSIS

In the pushover analysis (POA) the analysis model is the same as in the NTHA, the governing equation can be written in the incremental form as follows

$$\mathbf{K}\Delta x = \Delta\mathbf{F} \qquad (4)$$

where $\boldsymbol{K}$, Δx and $\Delta\boldsymbol{F}$ denote the stiffness matrix of the structure, the incremental displacement vector and the externally incremental force, respectively. During the analysis the stiffness $\boldsymbol{K}$ should be adjusted so as to associate with the value of the current displacement as shown in Figure 2.

In general the lateral force profiles in the static POAs will influence the structural response in different extent depending on the structural characteristics. Two different load patterns, i.e., the inverse triangular, it is the most simple, and the uniform load distribution shape had been used to represent the distribution of the inertial forces imposed on the building model. But the results comparison conducted illustrate that the inverse triangular distribution yielded excellent results, therefore only the results related with the inverse triangular distribution will be provided.

In this paper the modified capacity spectrum method is used to determine the target displacement, in which the strength reduction factor is determined by adopting the method proposed by Nassar and Krawinkler (1992)

6. NUMERICAL RESULTS OF THE NTHA AND POA, COMPARISONS AND DISCUSSIONS

Note that due to limited space in the following subsections the results presented by figures and tables are referred to response under rarely encountered earthquakes.

6.1. Comparisons between capacity curves

The capacity curve of a structure is used in general to assess whether the predicted performance of the structure can be achieved when it suffers earthquake. Figure 4 provides a comparison of the capacity curves obtained by the NTHAs and POAs described in the previous sections for structure 2# under different site conditions. For the NTHAs results of both the individual earthquake recordings and the average of all the 77 recordings are given.

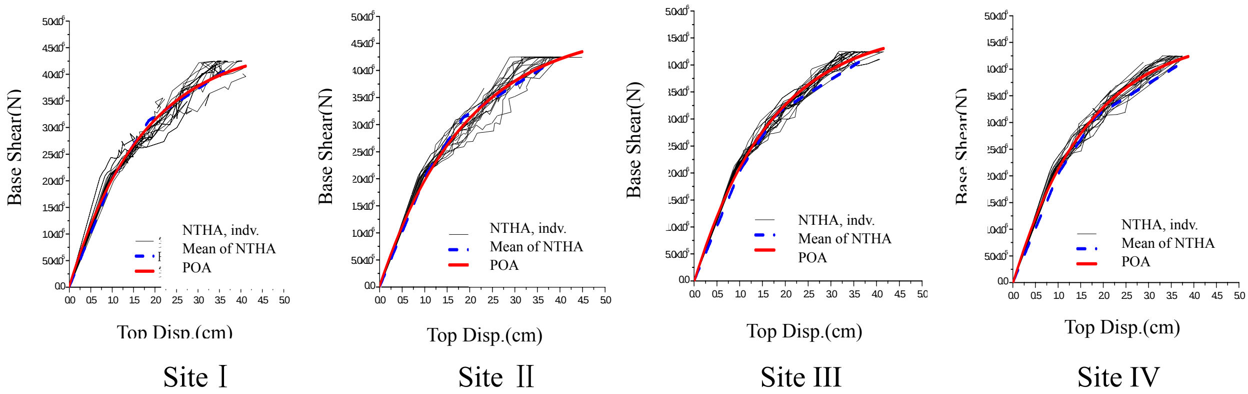


Figure 4 Comparisons between capacity curves for different site conditions for structure 2#

From the Figure 4 following conclusions can be made:

(1) It is evidently that the capacity curves obtained from the POAa consist of three linear segments. It is related with the restoring force-displacement curve used in the analysis;

(2) The capacity curves obtained from the NTHA for different input motions differ very small in the elastic stage，the difference increases slightly with increasing displacement. Note that the more soft of the soil site is the less is the difference between the capacity curves for different input motions. It reveals that on soft soil site the acceleration time histories have little influence on the capacity curve; and

(3) The capacity curves obtained from the POA fit well in the elastic range with those obtained from the NTHAs. In the inelastic range the POA fits the NTHA relatively well for site classifications of I and II. However, for the site classifications of III and IV the POA gives approximately the lower bound of the NTHAs.

6.2. Comparisons between target displacements

Under the rarely encountered earthquakes the results of the target displacements are listed in Table 3. The following features can be observed:

(1) For site classification II the POA underestimates the target displacement for all of the four structures considered. This can be explained as follows:

(a) The predominant period of the site classification II is relatively close the fundamental natural period of the masonry buildings, leading to a larger structural response. To consider such effect is unable for POA.

(b) There are some difference between the response spectra of China and US. The inelastic demand spectrum was generated in this paper based on the method of strength reduce factor proposed by abroad researchers, and the values of the predominant period, T_g in the site classification in the China specification (PRC National Standard, 2001) are less than those in the abroad about 15% to 30%, therefore the inelastic spectrum derived from the elastic spectrum gives lower displacement, and leads to a reduced target displacement after iteration.

(2) For site classifications I, III and IV the target displacements of structures 1#, 2# and 3# obtained by two methods differ relatively small.

(3) For structure 4# the modified capacity spectrum always underestimates the target displacement for all site classifications.

In summary, the modified capacity spectrum in most cases yields less target displacement than the NTHA with a maximum deviation of about 20%.

Table 3 Comparison between target displacements (mm) of structures obtained by capacity spectra and NTHA under rarely encountered earthquakes

Site classification	Method	Structure 1#	Structure 2#	Structure 3#	Structure 4#
I	Capacity spectra	17.59	24.54	13.46	24.79
	NTHA	17.19	23.07	14.42	30.92
	Deviation (%)	2.34	6.36	-6.65	-19.82
II	Capacity spectra	17.59	27.26	17.39	29.75
	NTHA	22.94	31.98	21.82	37.46
	Deviation (%)	-23.31	-14.74	-20.32	-20.59
III	Capacity spectra	17.59	27.26	17.39	29.75
	NTHA	19.44	28.91	17.70	40.78
	Deviation (%)	-9.5	-5.7	-1.77	-27.05
IV	Capacity spectra	17.59	27.26	17.39	29.75
	NTHA	18.99	26.50	16.15	38.66
	Deviation (%)	-7.36	2.87	7.68	-23.04

6.3. Comparisons between interstory drifts

Figure 4 plots the maximum interstory draft distribution for structure 2#.

Note that the ground floor has the largest draft due to the relatively uniform distribution of the mass and stiffness along the height of the building. Comparing the results of the NTHA and POA the maximum deviation is within 28%.

Due to similar reasons as for target displacement described in the subsection 6.2, CMSs on site classification II gives larger the interstory draft than other site classifications, besides, largest deviation occurs between the NTHA and POA, and in most cases the POA underestimates the draft.

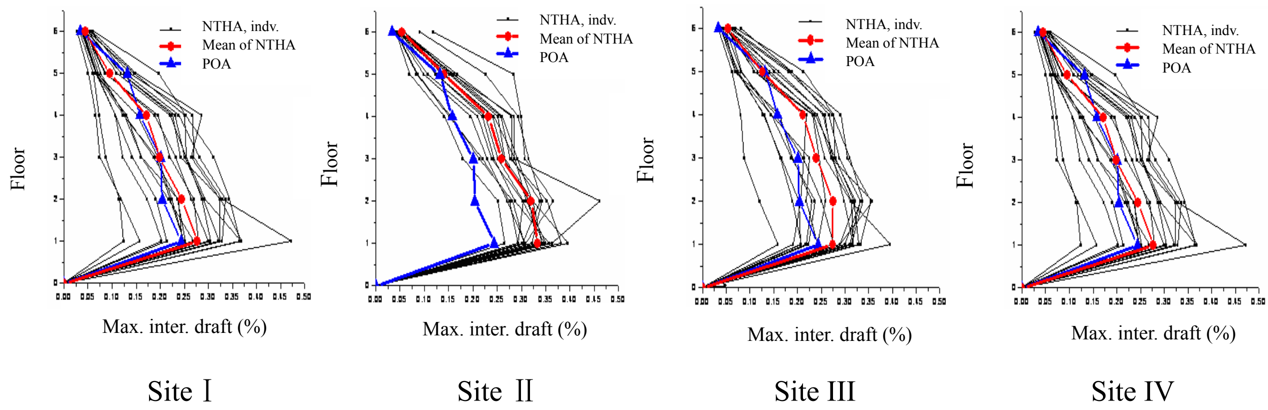
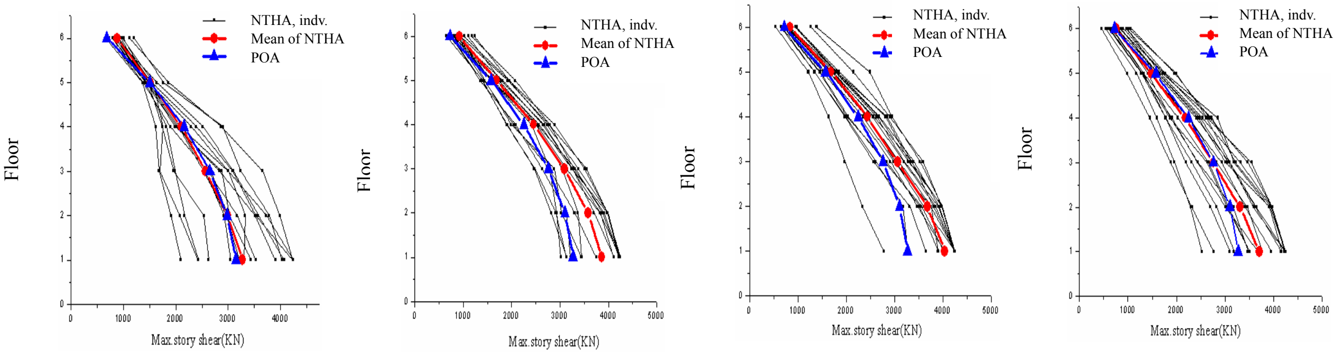


Figure 5 Maximum interstory drift distribution of structure 2#

6.4. Comparisons between base shears

Similar to the feature observed from the results of the top displacement and the interstory draft, the maximum interstory shear forces obtained from POA agree relatively well with the NTHA as shown in Figure 5. The maximum deviation is no more than 15% under frequently encountered earthquake and 23% under the design protected and rarely encountered earthquakes. From the point view of the site condition, on site classification II due to the same reason as above the the maximum interstory shear forces is larger and the difference between the results of the NTHA and POA is also larger than on other site classifications, and in most cases the POA underestimates the maximum interstory shear forces.



Site Ⅰ Site Ⅱ Site III Site IV

Figure 6 Distribution of the interstory shear for structure 2#

7. CONCLUSSIONS

Based on the previous work performed by the author a four-linear skeleton curve with a negative stiffness for the confined masonry walls along with the parameters characterizing the different performance levels was developed in the paper. Nonlinear time history analysis (NTHA) and nonlinear static analysis -- pushover analysis (POA) for four typical confined concrete/ brick masonry buildings were performed. From the comparisons between the results of the NTHAs and POAs following conclusions can be drawn:

（1） The capacity curve obtained from the POA consists three linear segments with two evident turning points, it is due to the four-linear skeleton curve used.

The capacity curves obtained from both the POA and NTHA agree well in the elastic range. In the inelastic range the POA fits the NTHA relatively well for site classifications of I and II. However, for the site classifications of III and IV the POA gives approximately the lower bound of the NTHAs.

（2） In most cases the POA underestimates the target displacement, the maximum interstory draft and the interstory shear force. Therefore, the POA as an approximate method could overestimate the seismic resistance of masonry buildings.

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