Structural Engineering and Mechanics, Vol. 41, No. 4 (2012) 479-494 479
DOI: http://dx.doi.org/10.12989/sem.2012.41.4.479

Seismic spectral acceleration assessment of masonry in-filled reinforced concrete buildings by a coefficient-based method

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(Received September 17, 2010, Revised January 11, 2012, Accepted January 28, 2012)

Abstract. This study explores a coefficient-based seismic capacity assessment method with a special emphasis on low-rise masonry in-filled (MI) reinforced concrete (RC) buildings subjected to earthquake motion. The coefficient-based method without requiring any complicated finite element analysis is a simplified procedure to assess the maximum spectral acceleration capacity of buildings. This paper first compares the fundamental periods of MI RC structures obtained, respectively, from experimental period data and empirical period-height formulas. The coefficient-based method for low-rise masonry buildings is then calibrated by the published experimental results obtained from shaking table tests. The comparison of the experimental and estimated results indicates that the simplified coefficient-based method can provide good approximations of the maximum spectral accelerations at peak loads of the low-rise masonry reinforced concrete buildings if a proper set of drift factors and initial fundamental vibration periods of structures are used.

Keywords: seismic capacity; masonry; earthquake; inter-storey drift; shaking table tests

1. Introduction

Seismic performance evaluations of building structures are normally conducted through nonlinear time history analysis and pushover analysis (ATC 1996, Fajfar and Gašperšič 1996, Chopra and Goel 1999, Fajfar 2000, Chopra and Goel 2002, Kalkan and Kunnath 2007) which is likely to obtain the best estimation of the seismic capacity of the structures. However, a detailed and well-calibrated analytical finite element model of the building, together with the inelastic force-displacement behaviour of each of the structural components, should be fully constructed and simulated prior to performing nonlinear seismic assessment analysis. In many cases, there are uncertainties associated with the generation of analytical models, especially for old building structures without detailed design information and for structures with masonry infills whose inelastic dynamic behaviours cannot be easily determined and simulated. Moreover, if seismic evaluations of

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a large number of buildings are needed, it is a time-consuming task. From a practical point of view, a simplified and manual procedure for rapid estimation of the maximum spectral acceleration and maximum inter-storey drift capacity may be a good alternative during the preliminary design of new buildings and for quick evaluation of existing buildings.

Recently, simplified coefficient-based procedures for assessing the seismic inter-storey drift demand and capacity of structures have been studied extensively. The seismic inter-storey drift demand and capacity can be obtained by simply multiplying several prescribed drift-related factors defined in coefficient-based methods. Miranda (1999) adopted an approximate building model integrated with a coefficient-based method to estimate the maximum lateral displacement demands of multi-storey buildings subjected to earthquake motions. The effects of the distribution (uniform, parabolic and triangular) of lateral forces along the building height and the ratio of overall flexural and shear deformations were examined. Moreover, four drift-related parameters that account for the linear and non-linear behaviour of the structure, were determined based on the assumed lateral displacement solution functions for various lateral force distributions. It was concluded that the simplified model for a ten-storey steel moment-resisting frame (MRF) building provides good approximations of the maximum roof displacement ratio as well as the maximum inter-storey drift ratio (with errors smaller than 26%) with specific displacement ductility ratios. Lu *et al*. (2009) proposed a new storey capacity factor to take into account both storey shear resistance and stiffness with different beam-column hinging mechanisms to predict the storey drift distribution and the critical drift concentration in multi-storey reinforced concrete (RC) frames. The proposed method was verified by performing a pushover analysis and dynamic time history analysis on three 7-storey and three 12-storey RC building frames subjected to earthquake motions. The results indicated that the inverse of the storey capacity factor correlates well with the storey drift distributions. Gupta and Krawinkler (2000) proposed a similar procedure to estimate the seismic roof and inter-storey drift demands of regular steel MRF structures by multiplying the modification factors, which include the multiple degrees of freedom (MDOF) modification factor, the inelasticity modification factor, the P-delta modification factor and the storey drift modification factor, by the given spectral displacement demand at the fundamental period of the structure. Zhu *et al*. (2007), Su *et al*. (2008) and Tsang *et al*. (2009) also proposed coefficient-based methods to evaluate the seismic drift demands of existing RC buildings under estimated displacement response spectra for Hong Kong soil sites. The coefficients used in these papers that take stiffness degradation, period shifting, non-linear damping and higher mode effects into account were calibrated by using the dynamic simulation results of a large number of buildings with heights varying from 35 m to 430 m in Hong Kong. Lee and Su (2011) further adopted a calibrated coefficient-based method to conduct a seismic fragility analysis for low-rise MI RC buildings.

The aforementioned coefficient-based methods normally focused on determining the seismic capacity or demand of structures in terms of the inter-storey drift ratio or the global roof drift ratio. The accuracy of these coefficient-based seismic assessment methods is strongly dependent on the proposed modification or drift-related factors, which are unexceptionally determined and calibrated through the numerical simulation results obtained from non-linear time history analysis of building structures subjected to various earthquake motions. Moreover, most finite element building models discussed in the studies are those medium- to high-rise RC or steel bare frames without masonry or shear wall (SW) infills. The results of the post-earthquake field study conducted by Su (2009) in the Wenchuan Earthquake in 2008 in China concluded that, for those low-rise masonry buildings that survived in the earthquake, the inherent strength, rather than ductility, protected the confined masonry buildings from collapse or serious damage. However, the relationship between the deformation-based parameters and the seismic inherent strength (or the spectral acceleration capacity) at the peak load state of structures has rarely been explored. Therefore, the present study intends to propose a coefficient-based seismic capacity assessment method, which integrates the drift-related factors and the spectral acceleration capacity of structures. The proposed coefficient-based method will be calibrated by using available shaking table test results with a special emphasis on the low-rise masonry in-filled (MI) RC buildings. This paper begins with a comparsion of the fundamental periods of MI RC structures obtained from experimental results and the empirical period-height formulas, respectively. The coefficient-based method is then calibrated by the published experimental results obtained from shaking table tests where applicable. Finally, the accuracy of the seismic spectral acceleration at peak load states of low-rise MI RC buildings estimated by the coefficient-based method is discussed.

2. Coefficient-based spectral acceleration assessment method

According to the building model illustrated in Fig. 1, the maximum roof displacement and the maximum inter-storey drift ratio at the peak load state of the structure can be represented in Eq. (1) and Eq. (2), respectively, as

$$u_{roof} = \lambda_1 S_d \tag{1}$$

where S_d is the spectral displacement at the peak load of the structure and λ_1 is the ratio of the maximum roof displacement to the spectral displacement, and

$$\theta_{\max} = \lambda_2 \theta_{avg} \tag{2}$$

where λ_2 is the ratio of the maximum inter-storey drift ratio to the maximum roof drift ratio, $\theta_{avg} = u_{roof}/H_b$, and H_b is the height of the building.

From Eq. (1) and Eq. (2), the maximum spectral displacement related to the maximum inter-storey drift ratio can be represented as

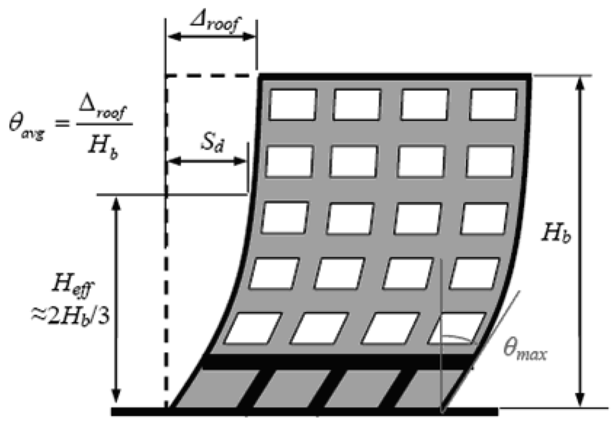


Fig. 1 Deformation of low-rise MI RC building model at peak load state

$$S_d = \frac{H_b \theta_{\max}}{\lambda} \tag{3}$$

where $\lambda = \lambda_1 \lambda_2$ is defined as the drift factor in this paper.

Considering a first-mode dominant MI RC building subjected to ground motion and using secant (equivalent) stiffness and damping properties of the structure at maximum base shear capacity, rather than based on initial elastic properties, the inherent strength or the maximum spectral acceleration at the peak load of the building can be calculated as

$$S_a = \left(\frac{2\pi}{T_p}\right)^2 S_d \tag{4}$$

where $T_p = \beta T_0$ is the fundamental vibration period at the peak load of the structure, β is the period shift factor that accounts for the effect of period lengthening with strong ground shaking, and T_0 is the initial fundamental vibration period of the undamaged structure.

Substituting Eq. (4) into Eq. (3), the maximum spectral acceleration and the drift factor at the peak load of the structure can be represented, respectively, as

$$S_a = \frac{H_b (2\pi)^2}{T_0^2} \frac{\theta_{\max}}{\lambda} \frac{1}{\beta^2} \tag{5a}$$

$$\lambda = \frac{H_b (2\pi)^2}{T_0^2} \frac{\theta_{\max}}{S_a} \frac{1}{\beta^2} \tag{5b}$$

It should be noted that the above formulation has assumed that floor diaphragms have been provided with sufficient steel reinforcement to prevent premature tearing failure of floors from happening (Su 2009, Su *et al*. 2011). This study adopts Eq. (5a) to estimate the maximum spectral accelerations at peak loads of the low-rise MI RC structures, and uses Eq. (5b) to calibrate the drift factor of the simplified coefficient-based approach through the published experimental results obtained from shaking table tests where applicable.

3. Fundamental periods of structures

A reliable and simplified estimate of the fundamental period of the structure is important both to determine the period-dependent seismic forces in accordance with the design response spectra and to perform rapid seismic capacity assessment of the existing buildings. The estimation of the fundamental periods of structures is usually carried out through ambient vibration tests, force vibration tests or earthquake events. Many empirical formulas that determine the fundamental vibration periods of structures have been proposed in design building codes and studies (ATC 1978, Goel and Chopra 1997, 1998, 2000, Hong and Hwang 2000, Su *et al*. 2003, BSSC 2004, Crowley and Pinho 2004, NZSEE 2006, Pinho and Crowley 2006, Vona and Masi 2009) all over the world. These empirical formulas normally provide simple period-height relationships that depend on building material (concrete, steel, or masonry), building type (e.g., frame or shear wall), and overall dimensions. The typical form of the period-height expression denoted in the literature or building codes is as follows

Table 1 Coefficients of period-height relationship for various empirical formulas

Code/Author	C	γ
ACT 3-06 (ATC 1978)	0.075	0.75
NZSEE (NZSEE 2006)	0.09	0.75
NEHRP (BSSC 2004)	0.0466	0.90
Hong and Huang (2000)	0.0294	0.804
Chopra and Goel (2000)	0.067	0.90
Crowley and Pinho (2004)	0.10	1
Su *et al*. (2003)	0.013	1

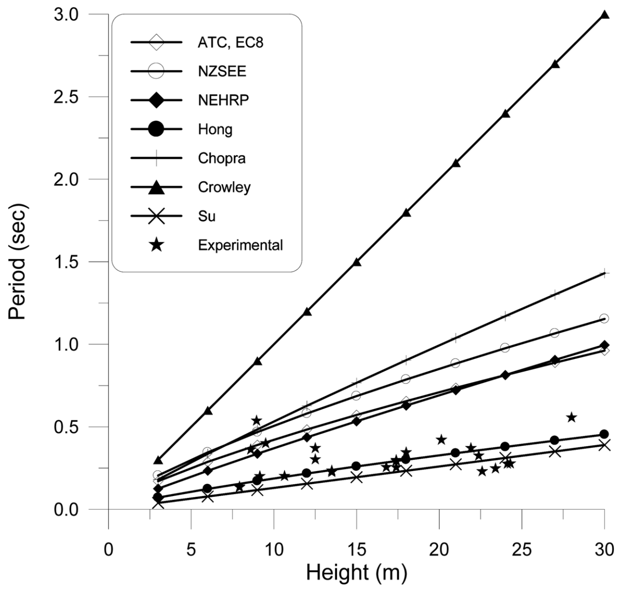


Fig. 2 Comparison of experimental and predicted initial fundamental vibration periods

$$T_0 = CH_b^{\gamma} \tag{6}$$

where C and γ are the coefficients derived from the regression analysis of the heights and the measured fundamental vibration periods of a number of building structures.

In order to accurately estimate the fundamental vibration periods of low-rise MI RC buildings for the seismic capacity evaluation using the coefficient-based method, this paper collected experimental vibration period data of MI, CM (confined masonry) and SW RC buildings through a literature review, and these data were further compared with those estimated by seven empirical formulas, as summarized in Table 1, where the two coefficients were usually obtained from the measured periods of buildings under earthquake motions. The comparison of the published experimental periods obtained from the literature (Tomaževič and Weiss 1994, Huang *et al*. 1994, Xia 1995, Xia *et al*. 1996, Kwan and Xia 1996, Negro and Colombo 1997, Shen *et al*. 1997, Tomaževič and Klemenc 1997, Dou 1997, Fardis *et al*. 1997, Tsionis *et al*. 2001, Lu 2002a, Lu 2002b, Lee and Woo 2002, Zheng *et al*. 2004, Dolce *et al*. 2005, Matsumori *et al*. 2005, Pinto and

Taucer 2006, Sun *et al*. 2007, Barecchia 2007, Pujol *et al*. 2008, Xiong *et al*. 2008) and those estimated from the empirical formulas is shown in Fig. 2, where most of the experimental periods of the low-rise MI and SW buildings are bounded within those estimated by ATC 3-06 (ATC 1978) or NEHRP (BSSC 2004). Moreover, a good correlation between the experimental periods and those estimated from Hong and Hwang (2000) and Su *et al*. (2003) is observed. The comparison result reveals that the fundamental periods of low-rise regular MI and SW RC buildings are relatively lower than those of medium- to high-rise buildings and those of bare frame structures because of the contribution of the lateral stiffness of masonry bricks or shear walls. The experimental periods of the structures reviewed in this paper are summarized in Table 2, in which the period data were measured with impact tests, force vibration tests and shaking table tests of full-scale or scaled-down low-rise RC buildings. It is noted that the periods shown in Table 2 have been converted to those of the prototype buildings using the period similitude factors mentioned in the literature. However, if

Table 2 Summary of building models used for evaluation of fundamental vibration periods

Building model	Number of storeys (N)	Model scale ($1/S_l$)	Period T_0 (sec)
1 (MI) (Tomaževič and Weiss 1994)	3	1:5	0.3618
2 (MI) (Tomaževič and Weiss 1994)	3	1:5	0.3621
3 (MI) (Huang *et al*. 1994)	8	1:4	0.3704
4 (SW) (Xia 1995, Xiong *et al*. 2008)	8	1:4	0.2299
5 (SW) (Xia 1995, Xiong *et al*. 2008)	8	1:4	0.2475
6 (MI) (Xia 1995, Xiong *et al*. 2008)	8	1:4	0.2778
7 (MI) (Kwan and Xia 1996)	4	1:3	0.2255
8 (SW) (Xia *et al*. 1996)	4	1:3	0.2341
9 (MI) (Xia *et al*. 1996)	4	1:3	0.2255
10 (MI) (Negro and Colombo 1997)	4	1:1	0.3030
11 (MI) (Shen *et al*. 1997)	5	1:6	0.2544
12 (MI) (Shen *et al*. 1997)	5	1:6	0.2923
13 (CM) (Tomaževič and Klemenc 1997)	3	1:5	0.1337
14 (CM) (Tomaževič and Klemenc 1997)	3	1:5	0.1374
15 (MI) (Dou 1997)	7	1:6	0.3252
16 (MI) (Fardis *et al*. 1997)	3	1:1	0.4000
17 (SW) (Tsionis *et al*. 2001)	4	1:1	0.3704
18 (SW) (Lu 2002a, Lu 2002b)	6	1:5.5	0.4210
19 (MI) (Zheng *et al*. 2004)	5	1:6	0.2977
20 (MI) (Zheng *et al*. 2004)	5	1:6	0.2555
21 (SW) (Zheng *et al*. 2004)	7	1:6	0.2760
22 (MI) (Dolce *et al*. 2005)	3	1:3.3	0.2012
23 (SW) (Matsumori *et al*. 2005)	6	1:3	0.3447
24 (SW) (Sun *et al*. 2007)	9	1:6	0.5565
25 (MI) (Barecchia 2007)	2	1:1	0.5370
26 (MI) (Pujol *et al*. 2008)	3	1:1	0.2000

the period similitude factors were not provided in the literature, the value of $\sqrt{S_l}$ was used in this paper ($T_{prototype} = \sqrt{S_l}T_{model}$, in which the model scale is $1/S_l$).

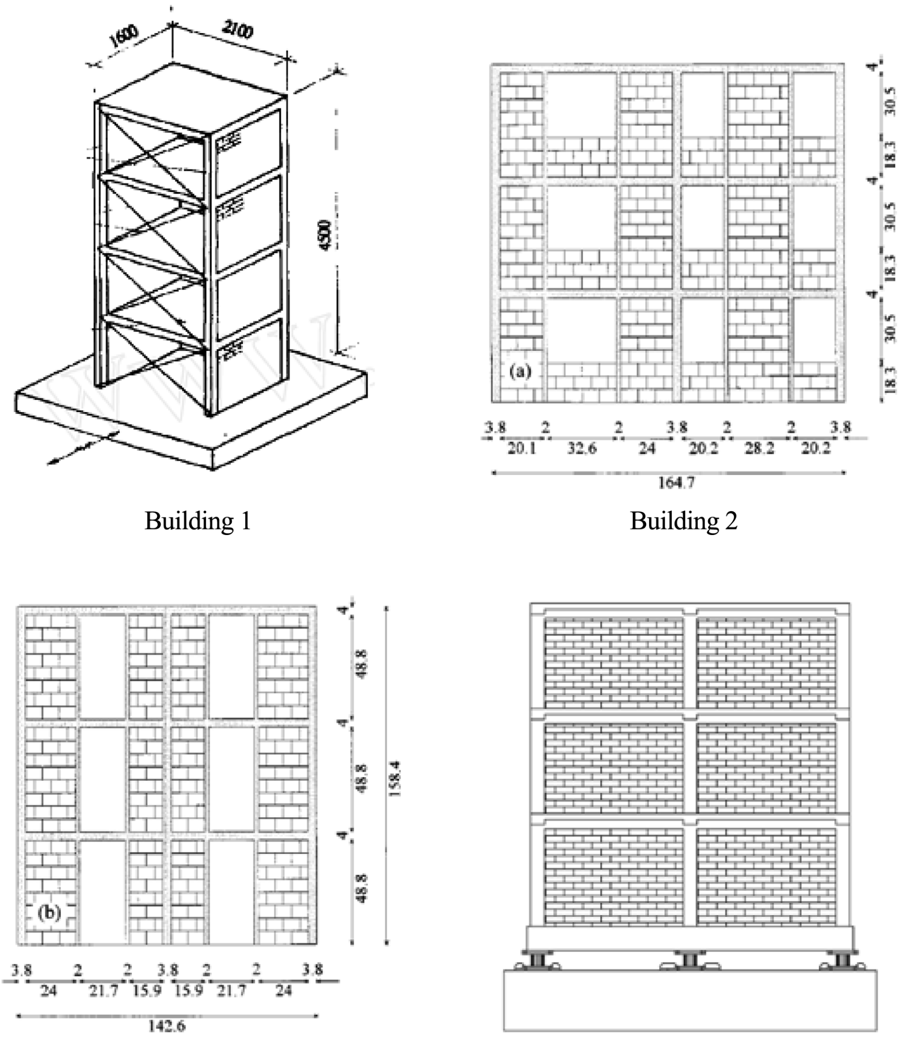
4. Calibration of the coefficient-based method

In order to calibrate the drift ratio and the coefficient-based approach for seismic capacity assessment of low-rise MI RC buildings, the published experimental results obtained from shaking table tests (or pseudo-dynamic tests) were reviewed. Although numerous shaking table tests of low-rise RC buildings have been conducted all over the world, many of the building models considered in the tests were bare frames without masonry or SW; furthermore, many of these studies presented only the acceleration responses or maximum roof displacement responses without demonstrating the maximum inter-storey drift ratios (local deformation) and the lengthened vibration periods at peak loads of the buildings. While not all the shaking table tests obtained the complete drift-related factors defined in this paper, some meaningful data have been found in the literature (Xia *et al*. 1996, Kwan and Xia 1996, Negro and Colombo 1997, Tomaževič and Klemenc 1997, Lu 2002a, b, Lee and Woo 2002, Zheng *et al*. 2004, Dolce *et al*. 2005, Matsumori *et al*. 2005, Pinto and Taucer 2006, Barecchia 2007), such as λ_2 varying between 1.2105 and 2.1649 with an average of 1.6948 (Table 3), the period shift ratio (β) ranging from 1.40 to 2.70 and the ratio of $\theta_{\max}/\beta^2$ varying between 0.001 and 0.005 at peak loads of low-rise MI and SW buildings, which may be useful references for other seismic capacity assessment applications. In this paper, shaking table tests of four low-rise MI RC or CM buildings (Fig. 3) (Kwan and Xia 1996, Tomaževič and Klemenc 1997, Dolce *et al*. 2005) are described, and the accuracy of the coefficient-based approach for estimating the maximum spectral accelerations at peak loads of these four low-rise MI RC buildings is then verified through the published experimental results.

Because of the limitation of the paper length, only the relevant experimental data of these four building models are described in this paper. The first-mode natural frequency of the building 1 at the initial (linear or intact) state was 7.68 Hz and was shortened to 3.92 Hz at the peak load state, where the maximum inter-storey drift ratio ($\theta_{\max}$) was 1/141 (0.0071). The fundamental periods of the prototype structures at the initial and peak load states were estimated from the experimental periods of the tested model: $T_0 = 1/(7.68/\sqrt{3}) = 0.2255$ and $T_p = 1/(3.92/\sqrt{3}) = 0.4413$,

Table 3 Summary of drift-related factors of building models

Building model	N	T_0 (sec)	θ_{avg}	$\theta_{\max}$	λ_2
1 (MI) (Negro & Colombo 1997)	4	0.3030	0.0064	0.0110	1.7188
2 (SW) (Lu 2002a, Lu 2002b)	6	0.4210	0.0190	0.0230	1.2105
3 (SW) (Zheng *et al*. 2004)	7	0.2760	0.0035	0.0065	1.8646
4 (MI) (Dolce *et al*. 2005)	3	0.2012	0.0051	0.0090	1.7647
5 (SW) (Matsumori *et al*. 2005)	6	0.3447	0.0259	0.0357	1.3801
6 (MI) (Barecchia 2007)	2	0.5370	0.0034	0.0050	1.4706
7 (MI) (Lee & Woo 2002)	3	0.3800	0.0194	0.0420	2.1649
8 (MI) (Pinto & Taucer 2006)	4	NA	0.0021	0.0043	2.0476
9 (MI) (Pinto & Taucer 2006)	4	NA	0.0019	0.0031	1.6316



Building 1 Building 2

Building 3 Building 4

Fig. 3 Building models for calibration of the coefficient-based method

respectively. Thus, the period shift factor can be calculated as $\beta = T_p/T_0 = 1.96$. Moreover, the maximum experimental spectral acceleration of the El Centro Earthquake at the lengthened fundamental period of T_p was calculated as S_a = 1.395 g considering a damping ratio of 5% for the peak ground acceleration (PGA) of 0.64 g. With these experimental parameters, the drift factor can be estimated from Eq. (5b) to be λ = 1.42, as shown in Table 4. It should be noted that the lengthened fundamental periods of the four building models in the literature were determined from the Fourier spectra (or power spectral density spectra) of the structural acceleration responses obtained by impact hammers (after the tests), random white noise excitation with small intensities (after the tests), or earthquake signals (during the tests).

Building 2 (tested longitudinally) and building 3 (tested along its transverse direction) have the

Table 4 Summary of structural parameters used for seismic capacity assessment

Building Model	N	Model Scale ($1/S_l$)	Model Height h (m)	PGA at Peak Load State (g)	β	θ_{max}	θ_{max}/β^2	T_0 (sec)	T_p (sec)	S_a (g)	λ
1 (MI) (Kwan and Xia 1996) (1940 El Centro Earthquake)	4	1:3	4.500	0.64	1.96	0.0071	0.0019	0.2255	0.4413	1.395	1.42
2 (CM) (Tomaževič and Klemenc 1997) (1979 Montenegro Earthquake)	3	1:5	1.584	0.84	1.41	0.0050	0.0025	0.1337	0.1886	1.524	2.94
3 (CM) (Tomaževič and Klemenc 1997) (1979 Montenegro Earthquake)	3	1:5	1.584	0.82	2.52	0.0150	0.0024	0.1374	0.3459	1.550	2.58
4 (MI) (Dolce *et al*. 2005) (Acceleration compatible with EC8)	3	1:3.3	3.225	0.63	1.94	0.0090	0.0024	0.2012	0.3907	1.302	1.94
Average Value					1.96	0.009	0.0023				2.22

Table 5 Summary of experimental and estimated spectral accelerations at peak loads of structures

Building Model	N	Building Height $H_b = S_l h$ (m)	T_0 (sec)	S_a (g) (Experimental)	$\bar{S}_a(g)$ ($\lambda = 2.22$, $\beta = 1.96$, $\theta_{max} = 0.009$)
1 (MI) (Kwan and Xia 1996)	4	13.5	0.2255	1.395	1.128 (19%)
2 (CM) (Tomaževič and Klemenc 1997)	3	7.92	0.1337	1.524	1.882 (23%)
3 (CM) (Tomaževič and Klemenc 1997)	3	7.92	0.1374	1.550	1.782 (15%)
4 (MI) (Dolce *et al*. 2005)	3	10.64	0.2012	1.302	1.117 (14%)

The number in ( ) representing the prediction error.

same design and construction and meet the requirements of Eurocode 8. The natural frequencies of the models obtained from impact hammer tests at the initial states were 22 Hz and 21.4 Hz, respectively, and shortened to 15.6 Hz (with damping ratio of 9.8%) and 8.5 Hz (with damping ratio of 10.1%), respectively, at the peak load states. The initial fundamental periods of the prototype structures at the initial states were $T_0 = 1/(22 \times 0.34) = 0.1337$ and $T_0 = 1/(21.4 \times 0.34) = 0.1374$, which were obtained by the frequency modelling scale factor (0.34), and those at peak load states were $T_p = 1/(15.6 \times 0.34) = 0.1886$ and $T_p = 1/(8.5 \times 0.34) = 0.3459$, respectively. Because the maximum experimental inter-storey drift ratio ($\theta_{\max}$) was not mentioned in the study, this paper assumed that the maximum inter-storey drift ratios are 0.005 and 0.015, respectively, based on the period shift ratios of these two buildings. Moreover, the spectral accelerations of the Montenegro Earthquake at the lengthened fundamental periods of T_p were obtained from Fig. 3 (Tomaževič and Klemenc 1997) to be S_a = 1.524 g and S_a = 1.550 g, respectively, by considering damping ratios of 10% for both the first modes of the two structures. Similarly, the drift factors for the two buildings can be estimated from Eq. (5b) to be $\lambda = 2.94$ and $\lambda = 2.58$, respectively, as shown in Table 4.

The natural frequency of the building 4 at initial state was 9.03 Hz and shortened to 4.65 Hz at the peak load state, where the maximum inter-storey drift ratio ($\theta_{\max}$) was 0.009. The fundamental periods of the prototype structures at the initial and peak load states were estimated from the experimental periods to be $T_0 = 1/(9.03/\sqrt{3.3}) = 0.2012$ and $T_p = 1/(4.65/\sqrt{3.3}) = 0.3907$, respectively. Moreover, the experimental spectral acceleration of the MI RC building at the lengthened fundamental period of T_p is obtained as S_a = 1.302 g using Fig. 11(b) (Dolce *et al.* 2005) and considering a damping ratio of 5%. From Eq. (5b), the drift factor can be estimated to be $\lambda = 1.94$, as shown in Table 4.

As the equivalent damping ratio of a building may vary with its lengthened structural period and the frequency contents of earthquakes, it is worth exploring the spectral amplification factor and the equivalent damping ratio of buildings at peak load states. In order to obtain a reasonable equivalent damping ratio of structures at peak load states, the spectral amplification factor defined as the ratio of spectral acceleration to peak ground acceleration (S_a/PGA) is further obtained in this study. The spectral amplification factors of the four buildings (Table 4) were found to be 2.18, 1.81, 1.89, and 2.07, with an average of 1.99 (≈2.0). From the spectral amplification factors as suggested by Newmark and Hall (1982), the equivalent damping ratio that corresponds to the spectral amplification factor of 2.0 is around 6% (median), or 10% (median with one standard deviation), which is consistent with the damping ratios of 9.8% and 10.1% for the Building 2 and Building 3, respectively. Hence, the spectral amplification factor of 2.0 or the equivalent damping ratio of 6-10% for the low-rise MI RC buildings at peak load states are recommended for the spectral acceleration assessment of buildings by using the proposed coefficient-based method.

From Table 4, the parameters obtained from the published shaking table tests are the period shift ratio, β, which ranges from 1.41 to 2.52 with an average of 1.96; the maximum inter-storey drift ratio, $\theta_{\max}$, which varies from 0.005 to 0.015 with an average of 0.009; the ratio, $\theta_{\max}/\beta^2$, which ranges from 0.0019 to 0.0025 with an average of 0.0023; and the drift factor, λ, which ranges from 1.42 to 2.94 with an average of 2.22. The maximum spectral accelerations ($\bar{S}_a$) of the four buildings estimated using the average values of $\lambda = 2.22$, $\beta = 1.96$ and $\theta_{\max} = 0.009$ that correspond to the peak load state of low-rise MI RC buildings obtained by this study, are summarized in Table 5. It can be observed that the errors between the experimental and estimated spectral accelerations at peak loads of the structures are 14% to 23% using the averaged drift factor obtained from test data. The experimental and estimated spectral accelerations are demonstrated in Fig. 4, where good

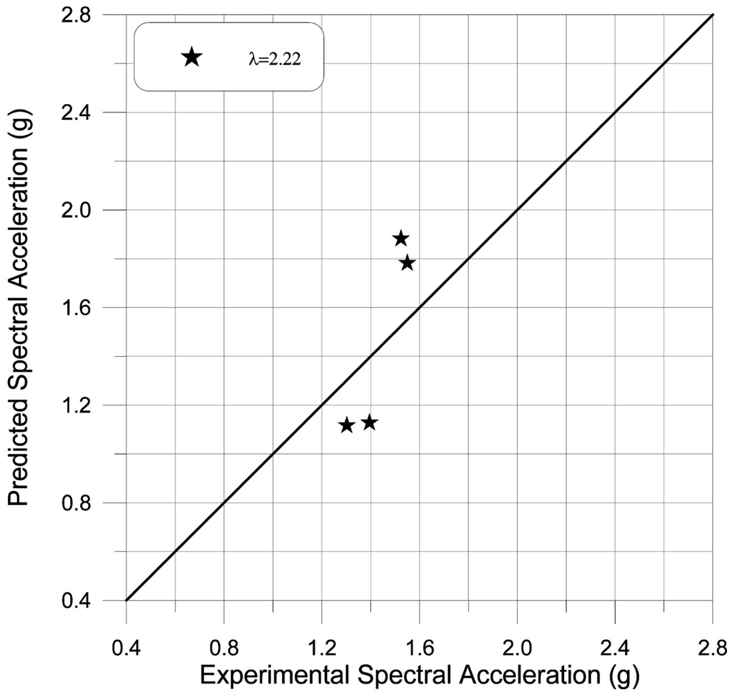


Fig. 4 Comparsion between the predicted and experimental spectral accelerations

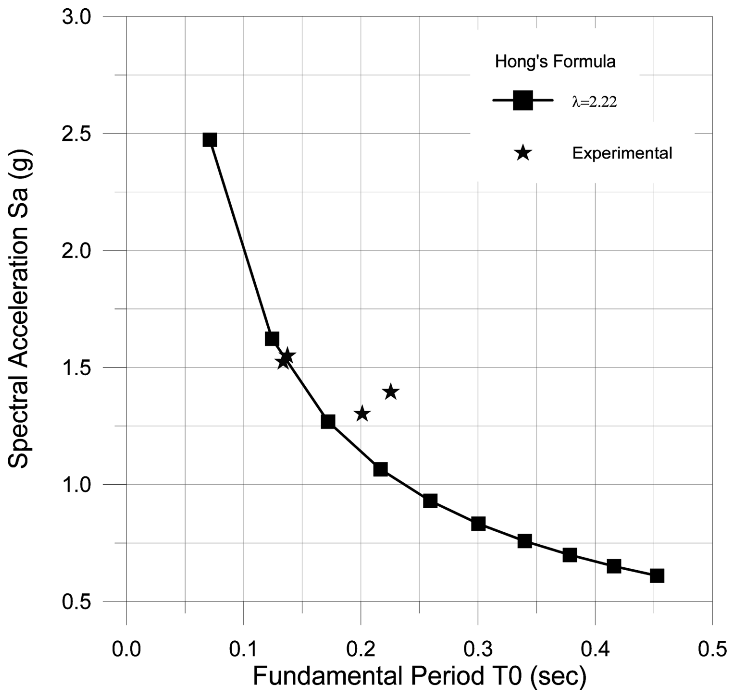


Fig. 5 Comparison of experimental and predicted spectral accelerations (Hong's Formula)

correlation has been achieved using the simplified coefficient-based, or inherent strength-based, approach for seismic capacity assessment of low-rise MI RC buildings if the proper drift factors are used. Moreover, the spectral acceleration capacities of low-rise MI RC buildings obtained using the initial fundamental periods estimated by Hong's empirical formula combined with $\theta_{max}/\beta^2 = 0.0023$ and $\lambda = 2.22$, are illustrated in Fig. 5, where the trend of the predicted spectral accelerations is consistent with that obtained from the experimental tests. This indicates that, if the initial fundamental period of the MI RC building can be accurately obtained from proper empirical formulas or from ambient vibration tests, the proposed coefficient-based method can provide a good approximation of the maximum spectral acceleration capacity (inherent seismic strength) at the peak load of the MI RC structure.

Fig. 6 illustrates a comparison between the inherent seismic strengths predicted by the proposed coefficient-based method and the spectral acceleration demands under various earthquakes with different intensities (assuming 5% damping ratio). It should be noted that the low-rise MI RC

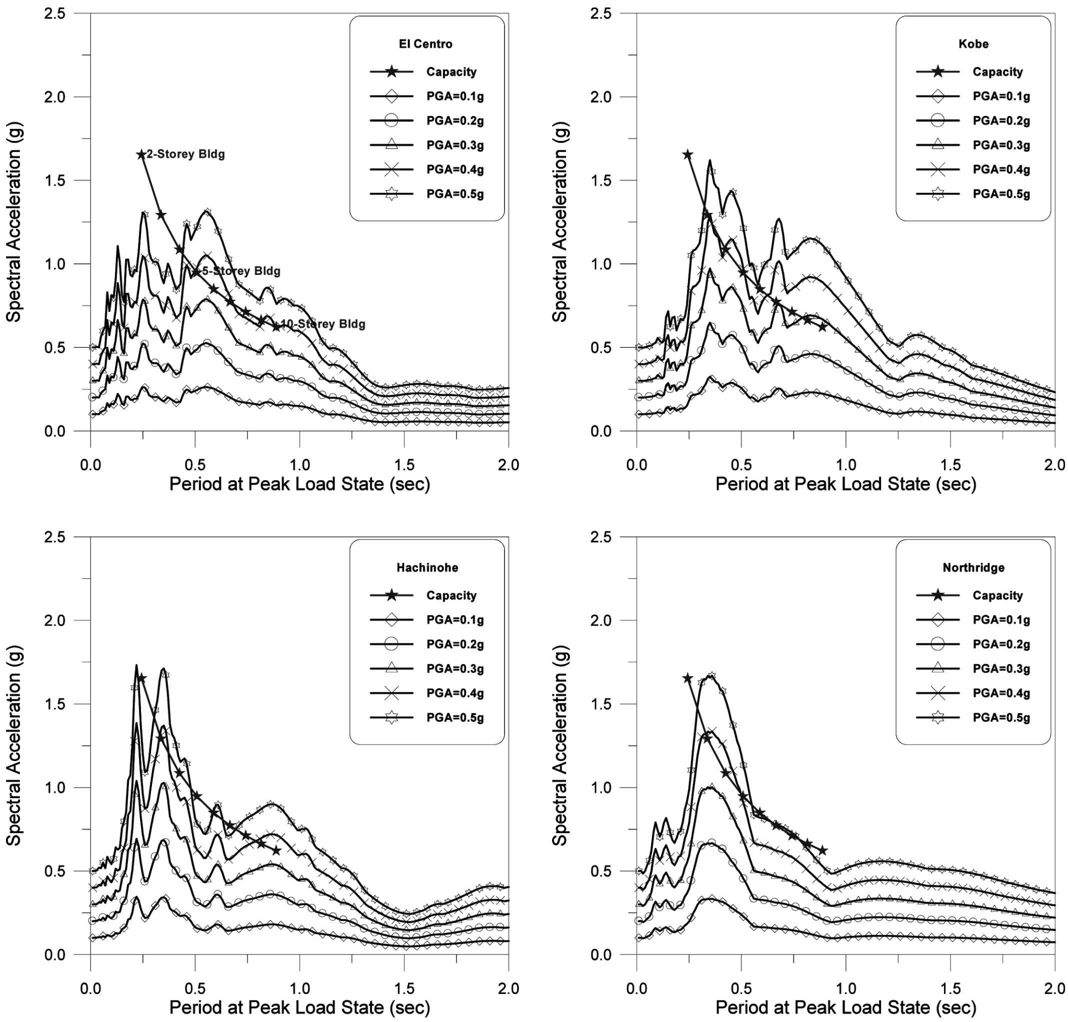


Fig. 6 Comparison between the inherent strengths and the seismic spectral demands

buildings with less than N = 10 storeys and with a uniform storey height of 3 m were considered. Moreover, the predicted spectral acceleration capacities were obtained by using λ = 2.22, β = 1.96, θ_{max} = 0.009 and the initial period T_0 of structures estimated by Hong's empirical period-height formula. The results show that buildings with $N \leq 2$ storeys generally have sufficient inherent strength under these earthquakes with PGA smaller than 0.5 g considered in this paper, while buildings with $N \geq 3$ storeys could have insufficient inherent strength, such as the 3-storey building subjected to Kobe, Hachinohe and Northridge earthquakes with the intensity of 0.4 g, and buildings with $N \geq 7$ storeys under Kobe earthquake with the intensity of larger than 0.3 g. From a practical point of view, the simplified procedure for rapid estimation of the maximum spectral acceleration capacity is a good alternative during the preliminary design of new MI RC buildings and for quick seismic performance evaluation of existing MI RC buildings under specific earthquakes or code-based design acceleration spectra.

5. Conclusions

This study calibrated a coefficient-based approach for seismic inherent strength assessment of low-rise masonry in-filled reinforced concrete buildings, which are subjected to higher earthquake forces than those of medium- to high-rise buildings in rock and soil conditions. The coefficient-based method that does not require any finite element analysis is a promising simplified and rapid manual procedure to estimate the maximum spectral acceleration capacity at the peak load of building structures, especially for the quick evaluation of existing buildings and conceptual design phase of new buildings. This paper first compared the fundamental periods of MI RC structures obtained, respectively, from experimental data and empirical period-height formulas, followed by a verification of the coefficient-based method for low-rise masonry in-filled reinforced concrete buildings using the published experimental results obtained from shaking table tests, where applicable. In accordance with the experimental verification processes, the following conclusions may be drawn.

1. The averaged drift factor λ of the coefficient-based method obtained from the published experimental data of four masonry in-filled reinforced concrete buildings via shaking table tests is 2.22 in this study.
2. Good correlation between the estimated and experimental maximum spectral accelerations at peak loads of four low-rise masonry in-filled reinforced concrete buildings has been achieved (with a maximum error of 23%), which indicates that the simplified coefficient-based approach can provide a satisfactory approximation of the inherent seismic strength of low-rise masonry in-filled reinforced concrete structures if the proper inter-storey drift-related factors, such as λ = 2.22, β = 1.96 and θ_{max} = 0.009 are used in this paper.
3. The accuracy of the maximum spectral acceleration of a low-rise masonry in-filled reinforced concrete building also depends on estimating the initial fundamental vibration period of the structure. The simplified and rapid manual seismic capacity assessment approach, which integrates the coefficient-based method and the initial fundamental vibration period estimated from proper empirical period-height formulas (such as Hong's formula used in this study), has proven to be capable of estimating the spectral acceleration capacities at peak loads of the low-rise masonry in-filled reinforced concrete buildings with good accuracy.

Acknowledgements

The research described here was supported by the Sichuan Earthquake Roundtable Fund of The University of Hong Kong.

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