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SEISMIC SHEAR DEMANDS OF RC CORE WALLS FOR DESIGN OF TALL BUILDINGS IN THAILAND

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Abstract

ASCE 7-10 allows practical engineers to use Response Spectrum Analysis (RSA) procedure to compute the design forces of the structures. However, it has been found to be inappropriate for seismic shear demands of reinforced concrete (RC) walls. This research aims to investigate the seismic shear demands of RC core walls from low-rise to high-rise buildings. Generic RC split core walls in 5 buildings varying from 5 to 25 stories subjected to ground motions in Bangkok and Chiang Mai of Thailand are first designed by RSA procedure in ASCE 7-10. Then, nonlinear response history analysis (NLRHA) is conducted to compute more accurate seismic demands of the structures. Two real tall buildings which represent common types of existing tall buildings in Bangkok are selected to study in order to confirm the finding of generic buildings. The results from both generic and real buildings demonstrate that shear demands of core walls from NLRHA are significantly larger than those from RSA procedure. The ratio between shear force from NLRHA and RSA procedure is defined as shear amplification. The shear amplifications of core walls in cantilever direction are larger than those in coupled direction. The two building locations having different spectrum shapes lead to different shear amplifications. Previous researchers' equations can estimate shear forces of core walls determined by NLRHA only for buildings lower than 25 stories. They are no longer applicable for tall buildings in Bangkok except that shear magnification factor equation in EC8 can predict well the shear force from NLRHA even for tall building. In Bangkok, it is found that Rejec et al. (2012)'s equation can well estimate shear forces in cantilever direction of the core walls lower than 25 stories. In Chiang Mai, Luu et al. (2014)'s equation provides good estimation of shear forces in both directions of the core walls lower than 25 stories. Beside these two equations, the shear magnification factor equation in EC8 is found acceptable to be adopted to multiply with shear force from RSA procedure before using it as design shear force of RC core wall in both Bangkok and Chiang Mai.

Keywords: Seismic shear demand; RC core wall; response spectrum analysis; nonlinear response history analysis

1. Introduction

Usefulness of structural walls in medium-rise and high-rise buildings has long been recognized both structural and functional requirements. They can form an efficient lateral resisting system to control the lateral deflection and to reduce the possibility of excessive deformations of the building subjected to earthquake loading. Reinforced concrete (RC) split core walls where elevators are built inside are widely used in Thailand. Practical engineers usually employ modal response spectrum analysis (RSA) procedure in ASCE 7-10 [1] to determine the design forces of the walls. This method employs a single response modification factor (R) for all modes of response to reduce the elastic forces computed by RSA procedure. However, previous researchers have found that flexural yielding at the base region of the wall reduced mainly the first-mode shear but higher-mode shears were not significantly affected by inelastic action [2-4]. When flexural capacity at the base of the wall was reached, the flexural over-strength inherent in the design could increase shear force of the wall. This increase was predominately related to the first-mode shear response [5]. On the other hand, ASCE 7-10 uses the same R for reducing shear force and bending moment expecting that flexural yielding at the base region of the wall limits shear force in the same way as it limits bending moment of the wall. In contrast, nonlinear response history analysis (NLRHA) results show that shear force keeps increasing after flexural yielding occurred at the base of the wall. The ratio between shear force from NLRHA and from RSA procedure is regarded as shear amplification. Therefore, when contribution of higher modes is significant and flexural over-strength of the wall is large, RSA procedure leads to non-conservative estimation of shear demands in nonlinear RC walls.

This shear amplification has been recognized by some seismic design codes. National Building Code of Canada (NBCC) [6] ,Canadian Standard Association (CSA) [7]; Eurocode 8 (EC8) [8] and New Zealand Standard (NZS) [9] have already been modified to account for shear amplification in structural wall, but there is no such shear amplification in current ASCE 7-10 yet based on [10]. Regarding to this problem, it should be kept in mind that United States commonly design tall buildings by using NLRHA which consumes much effort and time to conduct the analysis. However, Thailand adopting mostly the ASCE 7-10 still uses RSA procedure to determine the design shear force for RC wall which can lead to unsafe result. Therefore, this paper attempts to address this concern.

2. Review on higher-mode shear in RC wall

Seismic shear demands of RC walls have captured many interests from researchers around the world. Several equations for estimating shear forces in RC cantilever walls have been developed based on their parametric studies such as structural configurations and ground motions. Obviously, the results depend on these choices.

NZS [9] outlines that the design shear force of the wall shall not be less than the shear force determined by multiplying the shear force from equivalent static analysis, V_E, with base shear amplification factor, ω_v, proposed by [11] and flexural over-strength factor.

$$V_o^* = \omega_v \phi_o V_E \tag{1}$$

$$\omega_v = \begin{cases} 0.9 + \dfrac{n}{10} & \text{for } n \le 6 \\ 1.3 + \dfrac{n}{30} \le 1.8 & \text{for } n > 6 \end{cases}$$

where V_o^* is the design shear force at any level of the wall, ϕ_o is the over-strength factor related to flexural action at any level of the wall and n is number of stories of the building.

The design shear force of RC wall, V_{Ed}, in EC8 [8] is computed by amplifying total shear force, V'_{Ed}, obtained from RSA with a shear magnification factor, ε. The value of ε is taken as 1.5 for moderately ductile wall (q<3). For highly ductile wall, it is calculated from Eq. (2), which was based on formula proposed by [3].

The value of ε has to be at least 1.5, but needs not be larger than behavior factor, q. EC8 uses ε as constant factor along the height of the wall.

$$V_{Ed} = \varepsilon V_{Ed}^{'} \qquad (2)$$

$$\varepsilon = q\sqrt{\left(\frac{\gamma_{Rd}}{q}\frac{M_{Rd}}{M_{Ed}}\right)^2 + 0.1\left(\frac{S_e(T_C)}{S_e(T_1)}\right)^2}$$

where ε is the base shear magnification factor, q is the behavior factor (force reduction factor used in design), M_{Rd} is the design flexural strength at the base of the wall, M_{Ed} is the design bending moment obtained from RSA at the base of the wall, γ_{Rd} is the over-strength factor accounted for strain-hardening of rebar, T_c is the upper-limit period of constant spectral acceleration region, T_1 is the fundamental period in the direction of shear force considered and $S_e(T)$ is ordinate of the elastic response spectrum.

Modified Modal Superposition (MMS) approach was proposed by [4] to determine the design shear force of RC cantilever wall, as shown in Eq. (3). This method was developed based on the assumption that inelastic action only limited the shear force from the first mode and shear force from higher modes were not affected by inelastic action.

$$V_i = \left(V_{1i}^2 + V_{2Ei}^2 + V_{3Ei}^2 + ...\right)^{0.5} \qquad (3)$$

where V_i is the design shear force of the wall at level i, V_{1i} is the lesser of elastic first-mode and ductile first-mode shear computed by direct displacement base design (DDBD) at level i, V_{2Ei} and V_{3Ei} are elastic modal shear at level i for second and third mode, respectively.

Another study was conducted by [5] to propose possible improvement to EC8 provision for computing the design shear force of RC wall. They found that the design shear force in EC8 should be computed from the first-mode shear, $V^{'}_{Ed,1}$, amplified by the magnification factor, ε_a. The factor, ε_a, in their proposed formula was derived in the same manner as Eq. (2) but they did not limit ε_a by factor, q. They limited the base shear force of the wall by total elastic shear force as included in the first term of ε_a. So, they modified Eq. (2) as:

$$V_{Ed} = \varepsilon_a V_{Ed,1}^{'} \qquad (4)$$

$$\varepsilon_a = q\sqrt{\left(\min\left[\frac{\gamma_{Rd}}{q}\frac{M_{Rd}}{M_{Ed}};1\right]\right)^2 + 0.1\left(\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \geq 1.5$$

Eq. (4) was applicable only to the base shear force of RC cantilever wall. They further proposed formula to compute shear force along the height of the wall by replacing the constant ratio between the contribution of second mode to that of the first mode with a variable ratio, $m(z)$, as shown in Eq. (5).

$$\varepsilon_a(z) = q\sqrt{\left(\min\left[\frac{\gamma_{Rd}}{q}\frac{M_{Rd}}{M_{Ed}};1\right]\right)^2 + m(z)^2\left(\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \geq 1.5 \qquad (5)$$

where z is the vertical coordinate of the wall and $m(z)$ is the ratio between the higher mode normalized shear and the first mode normalized shear.

NBCC [6] has explicitly considered higher-mode effects when using equivalent static force procedure by applying higher-mode factor to increase the base shear force and by applying base overturning reduction factor to reduce overturning moment. These factors depend on the structural type, fundamental period of the structure and the shape of design spectrum. In addition to NBCC, CSA [7] contains specific provision for seismic design of shear wall. For ductile wall, it requires that the base shear resistance must be increased by the ratio of probable base bending moment capacity to the base bending moment obtained from RSA, which is actually the base flexural over-strength of the wall. For moderately ductile wall, the same calculation is followed by using nominal base bending moment capacity instead of probable base bending moment capacity.

Recently, seismic demands of moderately ductile (MD) RC shear walls was studied by [12]. From their parametric study, they included that flexural over-strength and fundamental period have significant effects on base shear amplification factor. A new base shear amplification factor, ω_v, applied to the base shear, V_d, obtained from RSA was proposed for NBCC [6] and CSA [7] for MD shear walls.

$$V_b = \omega_v V_d \qquad (6)$$
$$\omega_v = \begin{cases} 1.6+0.7(\gamma_w - 1)+0.2(T-0.5) & \text{if } 0.5 \le T \le 1.5 \\ 1.8+0.7(\gamma_w - 1)-0.1(T-1.5) & \text{if } 1.5 < T \le 3.5 \end{cases}$$

where V_b is the design base shear force of the wall, T is the fundamental period and γ_w is the base flexural over-strength factor of the wall.

Although, previous researchers have studied higher-mode shear in RC wall, most of them have mainly focused on RC cantilever wall designed for a concentrated plastic hinge at the base of the wall. None of these studies focused on split core-wall systems which are extensively constructed in Thailand. Thus, this study attempts to investigate the shear demands in such systems and aims to explore the accuracy of various codes and previous researchers' formulas if applied to the core-wall structures.

3. Selected buildings and ground motions

3.1 Selected buildings

Five generic RC split core-wall buildings ranging from 5- to 25-story having core walls located at the center of a square-shape floor plan were employed. The orientation of core-wall cross section of each building was presented in Fig.1. Each building consists of two core walls coupled by long coupling beams (LCB) in X direction with the exception that 5-story building has only one central core wall. In Y direction, there are short coupling beams (SCB) above the opening doors. For the 5-story buildings, the core wall behaves like a cantilever wall in both directions. For 10- to 25-story buildings, the behaviors of core walls in both directions are different as they are coupled in X direction, while they behaves like a cantilever wall in Y direction. Core walls were assumed to have uniform cross section. The coupling beams were considered to have uniform stiffness and strength along the height of each building. Wind load was not considered in the design of core-wall strength.

In addition to the generic core-wall buildings, two real existing buildings which represent common type of existing tall buildings in Bangkok were selected as case study to confirm the results of the generic buildings. The first building has 19 stories and the second building has 38 stories. They consist of podium and tower zones. Their typical floor plans and 3D pictures were presented in Fig.2. Only the tower floor plans were shown. The characteristics of the selected building were summarized in the Table 1. The two real buildings were properly designed to resist wind load required by Thai building code.

3.2 Ground motions

The maximum considered earthquake (MCE) ground motions having two percent probability of exceedance in 50 years were employed in this study. It should be noted that RSA procedure in ASCE 7-10 uses the design spectrum that is referred to as design basic earthquake (DBE) having ten percent probability of exceedance in 50 years. MCE is used in NLRHA to evaluate the response of the structural system against collapse. However, in this study, MCE was used for both RSA and NLRHA for Bangkok (low seismic zone) because the structural systems did not yield under DBE. For Chiang Mai (high seismic zone), DBE was employed for both RSA and NLRHA. Using the same intensity of ground motions for RSA and NLRHA is appropriate for the purpose of comparing analysis methods.

For Bangkok, a set of seven ground motions selected from PEER Ground Motion Database was used. They were modified and scaled such that their spectra matched the bed-rock target spectrum of [13]. Then, those spectral-matching ground motions were input in ProShake program [14] to simulate the wave propagation through layers of soft soil in Bangkok. For Chiang Mai, ten pairs of ground motions were employed. Those ground motions were multiplied by a scaling factor such that the mean SRSS spectrum of the ten pairs of ground motions was not less than 1.17 times the design spectrum per ASCE 7-10. The component with larger PGA was selected from each of the ten pairs to make a set of the ten ground motions to be employed in NLRHA. The elastic spectrum in Bangkok and Chiang Mai was shown in Fig.3.

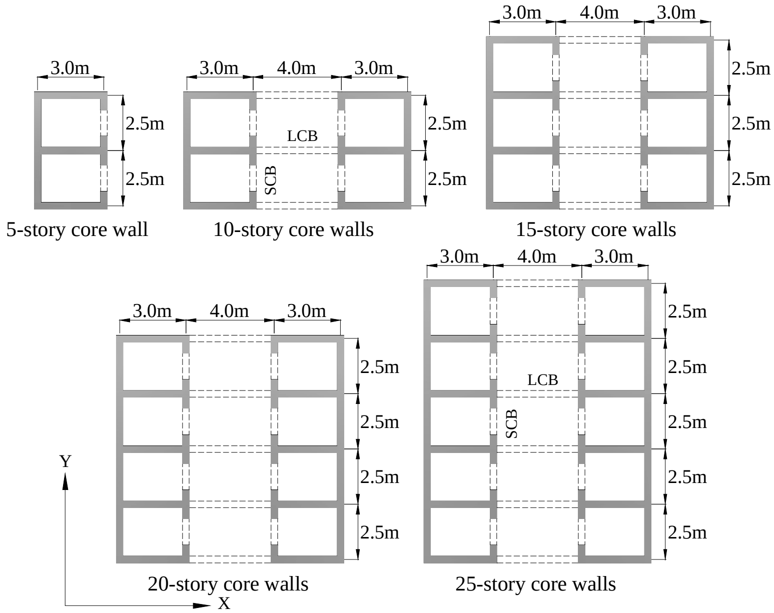
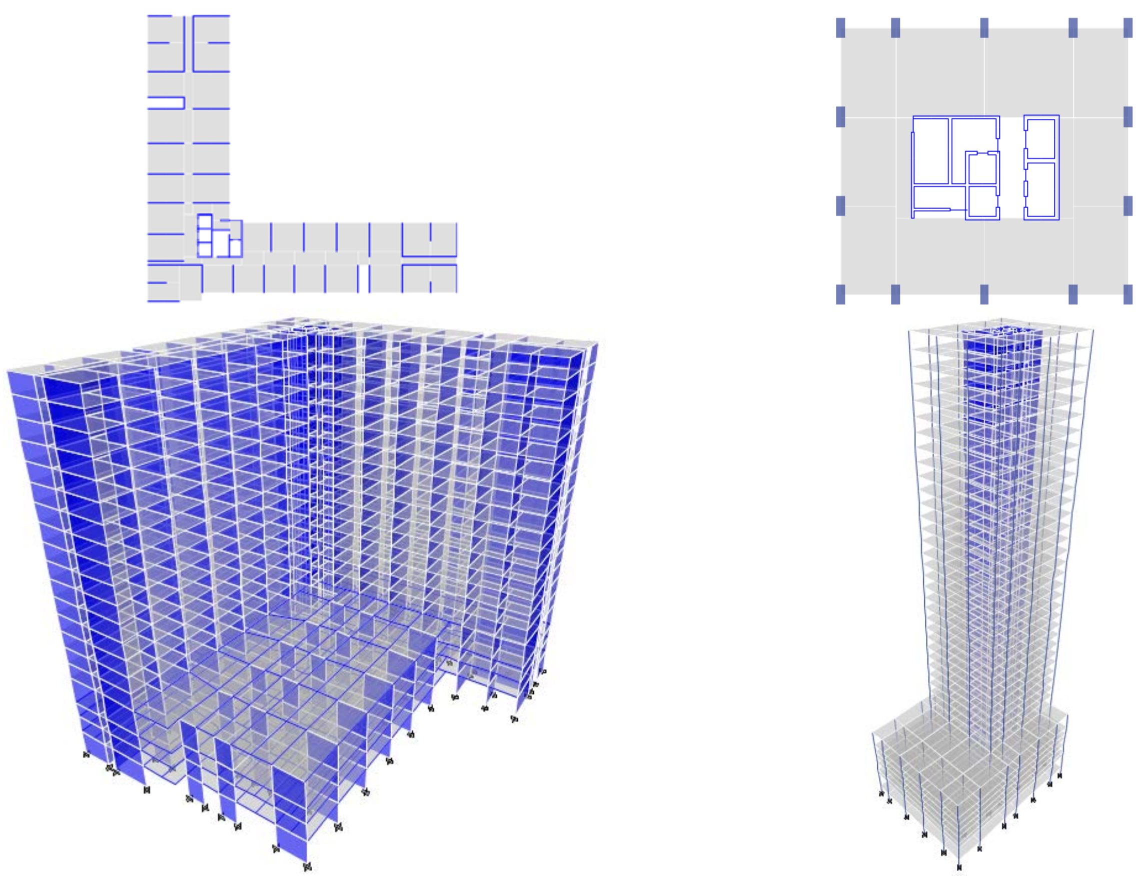


Fig. 1 – Core-wall cross sections of the five generic buildings



(a) 19-story building

(b) 38-story building

Fig. 2 – Typical floor plans and 3D pictures of two real buildings: (a) 19-story and (b) 38-story

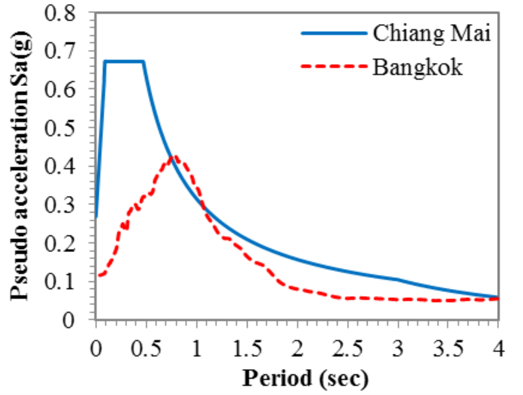


Fig. 3 – Elastic spectrum for 5% damping ratio

Table 1 – Characteristics of the case study buildings

Building type	**No. story**	***H*** (m)	***CFR***	***WFR***	t_w (m)	f'_c (MPa)	f_y (MPa)	***T*** (sec)	
								X	**Y**
Five generic buildings	5	15	-	0.01	0.20	30	390	0.57	0.43
	10	30	-	0.012	0.25	30	390	1.62	1.29
	15	45	-	0.015	0.30	35	390	2.68	1.71
	20	60	-	0.02	0.35	35	390	3.72	2.05
	25	75	-	0.025	0.40	40	390	4.60	2.27
Two real buildings	19	55	-	0.022	0.2	28	400	1.43	1.71
	38	123	0.013	0.015	0.35	32	400	6.82	5.13

H is building height, *CFR* is RC column cross-section area/floor area, *WFR* is RC wall corss-section area/floor area, t_w is wall thickness, f'_c is compressive strength of concrete, f_y is yield strength of rebar, *T* is fundamental period of the building.

4. Analytical model and analysis consideration

For the generic buildings, RC spilt core walls were designed to resist the entire lateral loads; hence, only the structural split core-wall system (core wall and coupling beam) was modelled in the analysis. However, p-delta effects due to gravity columns were included by creating a dummy column with no lateral stiffness at the centre of each core wall. For the real buildings, all structural elements (wall, column, beam, slab, and masonry infill wall) were included in the analysis. Rigid diaphragm was assigned to the floor slab. Foundation was assumed to be fixed support. Floor masses were assigned to the centre of mass at each floor level.

RSA procedure was carried out by ETABS program [15]. The effective stiffness values of the structural members given in ACI 318-11 (Table 2) were used to account for cracked sections of RC members. Constant modal damping ratio of 5% was employed for RSA procedure. The seismic load was applied in each direction separately at a time. The vertical seismic load effect was not considered in this study. Basic load combinations for strength design in ASCE 7-10 were adopted.

NLRHA was conducted by using PERFORM-3D program [16]. Core walls were modelled using inelastic fibre shear wall elements. The in-plane shear stiffness was modelled using elastic shear properties. The material stress-stain relationship for confined and unconfined concrete proposed by [17] was adopted such that it was represented by a tri-linear relationship in PERFORM-3D. The steel material was modeled with tri-linear stress-strain relationship with strain-hardening ratio of 3%. Nonlinear fibre model was incorporated over the entire height of the wall because flexural yielding may occur at any location due to higher-mode effects. Short coupling beams (SCB) were modelled using a nonlinear shear displacement-hinge at the centre of the beam. The long coupling beams (LCB) included rotational plastic hinge elements at both ends. The coupling beams were connected to the walls using embedded rigid beam to ensure the rigid connection between the coupled walls and coupling beams. Columns were modelled by combination of linear elastic element with nonlinear plastic hinge zone at both ends. The plastic zones were defined by inelastic fibre element in similar manner to structural wall. The plastic zone was assumed to have length of half the shortest dimension of the column. The concrete slabs were assumed to be elastic and were modelled by elastic shell elements with the effective stiffness of 25% of the uncrack stiffness. The masonry infill walls were included in the nonlinear structural model. They were modelled by equivalent diagonal compressive struts. The axial stiffness, strength, and inelastic deformation capacity of these struts were determined from the geometry and material properties of the masonry wall by following the ASCE 41-06 [18] guidelines. The effective stiffness values for elastic portion of structural elements were listed in Table 2. Rayleigh damping was implemented with 3% damping ratio specified in the first and third modes for NLRHA. The gravity load of all dead loads plus 25% of live loads was applied before NLRHA.

Table 2 – Effective stiffness of structural members used in RSA and NLRHA

Elements	**RSA**		**NLRHA**	
	Flexural	**Shear**	**Flexural**	**Shear**
Wall	0.70 EI_g	1.0 GA_g	Fibre section	1.0 GA_g
Beam	0.35 EI_g	1.0 GA_g	0.35 EI_g	1.0 GA_g
Column	0.70 $\mathrm{EI_g}$	1.0 GA_g	0.70 EI_g	1.0 GA_g
Slab	0.25 EI_g	1.0 GA_g	0.25 EI_g	1.0 GA_g

EI_g is flexural rigidity of gross section and GA_g is shear rigidity of gross section

5. Comparison of seismic demands computed by RSA procedure and NLRHA

5.1. Base shear and base moment amplification

Base shear amplification (BSA) is defined as the ratio between base shear force from NLRHA and RSA procedures. The results demonstrated that the BSA in Bangkok and Chiang Mai is significantly different in cantilever direction (EQy) of core wall (Fig.4b). In this direction, BSA is as large as 5 for 20- and 25-story buildings in Bangkok while in Chiang Mai, BSA is about 3 for 10- to 25-story buildings. In coupled direction (EQx), BSA is smaller than in cantilever direction and BSA in Bangkok is a little bit larger than in Chiang Mai (Fig.4a). From the design of reinforcing steel in core walls, flexural over-strength in coupled direction is smaller than in cantilever direction. From detail investigation, it was found that core walls in coupled direction did not yield much at the base while the coupling beams sustained wide spread yielding at several plastic hinges in the upper stories dissipating much energy. These are maybe the reason why core walls suffered little yielding and smaller shear amplification in coupled direction.

Base moment amplification (BMA) is defined as the ratio between base bending moment from NLRHA and RSA procedures. The base bending moment is limited by actual flexural strength of core wall which is dependent on the axial load. Due to flexural over-strength inherent in the design of core wall and different axial load (DL+0.25LL for NLRHA and 0.9DL, which governed the design, for RSA procedure), bending moments from NLRHA are larger than those from RSA procedure. In cantilever direction (Fig.5b), BMA could be as high as 3 in Bangkok and 2 in Chiang Mai. Whereas in coupled direction (Fig.5a), BMA is relatively smaller than in cantilever direction because bending moments in coupled direction governed the design of core wall, which resulted in smaller flexural over-strength in this direction.

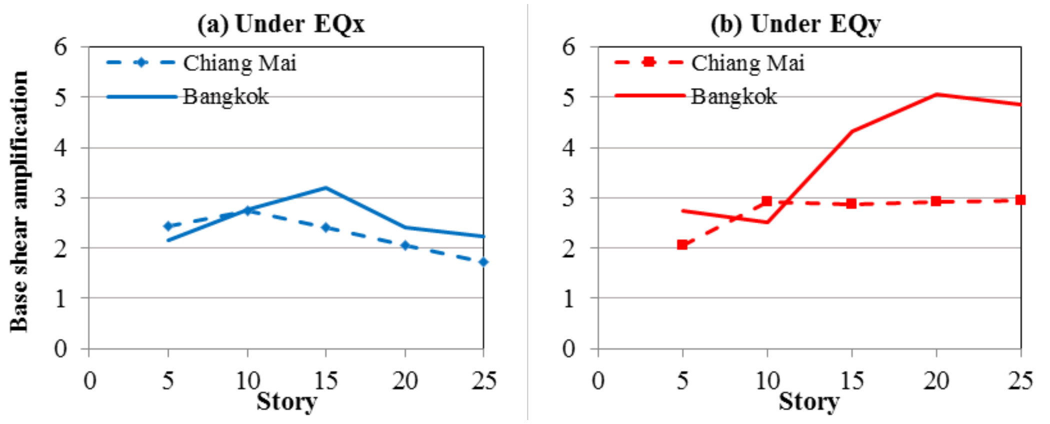


Fig. 4 – Base shear amplification of core wall: (a) under EQx and (b) under EQy

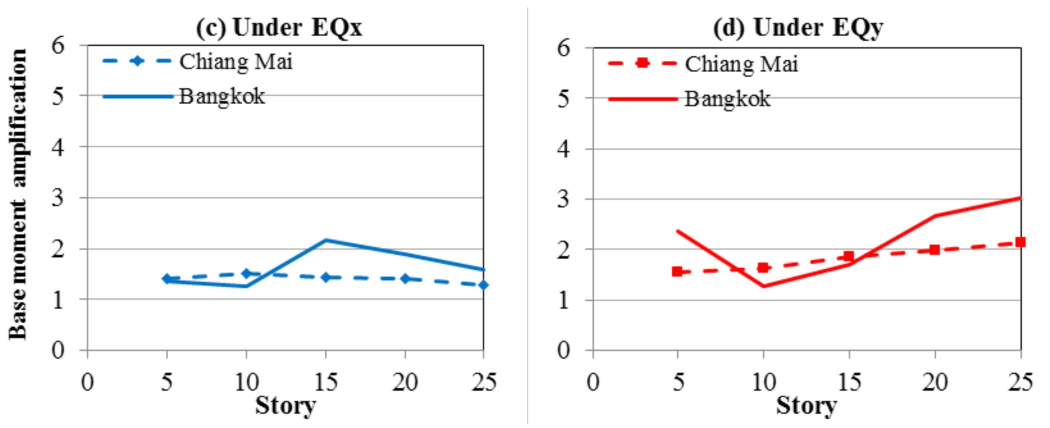


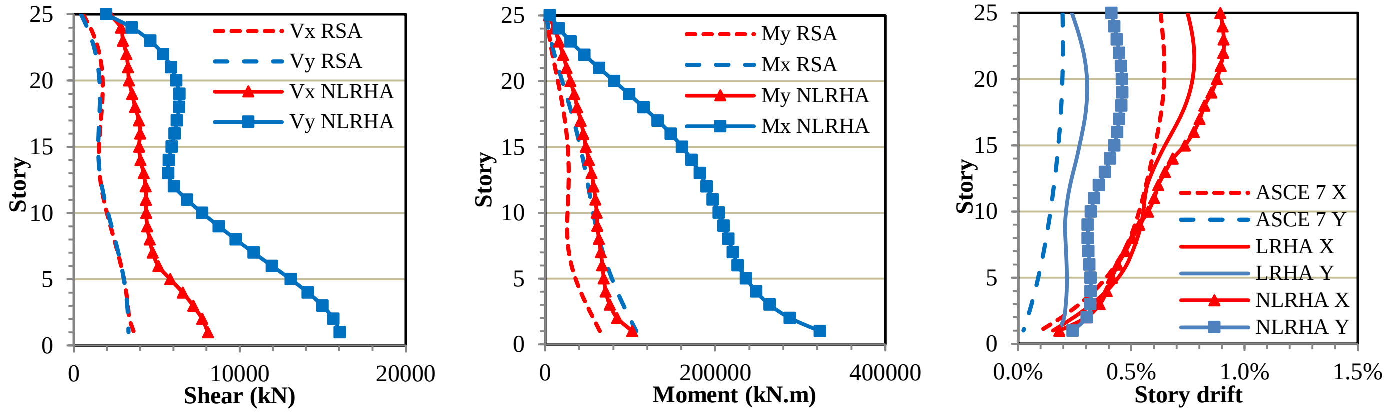
Fig. 5 – Base moment amplification of core wall: (a) under EQx and (b) under EQy

5.2 Shear, bending moment and story drift along the height of structure

The core wall is utilized to resist lateral load in both X and Y directions. The stiffness in resisting earthquake in Y direction, which behaves as cantilever wall, is larger than in X direction, which behaves as coupled walls; thus, Y direction has shorter period and attracts more force (V_y) and bending moment (M_x) as observed in Figs.6a and 6b. For generic buildings, shear demands from NLRHA are significantly larger than the design shear forces from RSA procedure along the height of core wall (Fig.6a). The shear amplification of core wall in coupled direction (V_x) is less than that in cantilever direction (V_y) and rather uniform along the height of core wall. Bending moment demands from NLRHA are much larger than the design bending moments from RSA procedure along the height of core wall (Fig.6b). These large bending moments from NLRHA are mainly due to different axial load and flexural over-strength, as explained in base moment amplification in Section 5.1. The moment amplification in coupled direction (M_y) is smaller than in cantilever direction (M_x) because the flexural demands in coupled direction governed the design of core wall.

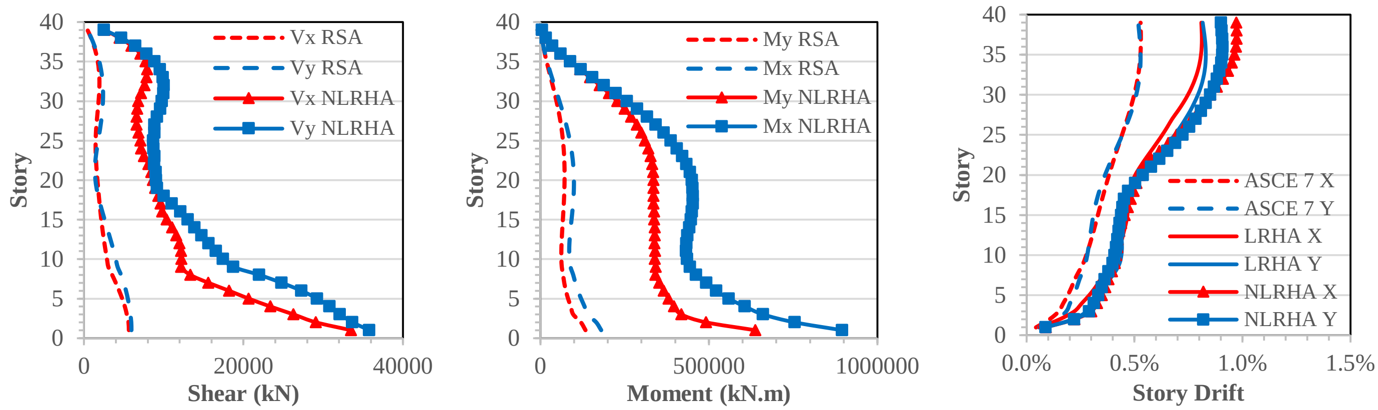
Regarding story drifts, ASCE 7 employs deflection amplification factor (C_d) not larger than response modification factor, R, implying that expected inelastic story drifts are not larger than elastic story drifts. However, Fig.6c shows that inelastic story drifts from NLRHA are larger than elastic story drifts from linear response history analysis (LRHA), while those computed by ASCE 7 procedure using C_d are the smallest. Therefore, elastic story drifts seem more accurate than those obtained by ASCE 7 procedure.

Seismic demands of the 38-story real building are presented in Fig.7. Similar trends as in generic buildings can be observed in the real-building case study with the exception that bending moments from NLRHA are much larger than those from RSA procedure in both directions because the wind load governed the design of this building and leaded to large flexural over-strength of the structure.



(a) (b) (c)

Fig. 6 – Comparison of seismic demands from RSA procedure and NLRHA: (a) shear force; (b) bending moment and (c) story drift of the 25-story generic core-wall building



(a) (b) (c)

Fig. 7 – Comparison of seismic demands from RSA procedure and NLRHA: (a) shear force; (b) bending moment and (c) story drift of the 38-story real building

6. Accuracy of previously proposed formulas for estimating shear forces of RC core walls

The results from NLRHA are the mean values of responses to the set of ground motions. The mean (designated as NLRHA in Figs. 8 to 10) is presented along with the mean plus and minus one standard deviation (mean±std.dev.) to show the variability of results among different ground motions.

6.1 Base shear amplification

For Bangkok, Rejec [5]'s equation provided good agreement with BSA in cantilever direction of generic core-wall buildings (Fig.8b) but it conservatively overestimated BSA in coupled direction (Fig.8a). For Chiang Mai, Luu [12]'s equation could well estimate the BSA in both directions. Beside these two equations, EC8 [8]'s equation generally provided conservative results for both cantilever and coupled directions and both locations, Bangkok and Chiang Mai, with the exception that it slightly underestimated the base shear force in cantilever direction of 20- and 25-story core walls in Bangkok (Fig.8b). It should be noted that this observation came from only the case of generic core-wall buildings.

6.2 Shear force along the height of structure

For real buildings as illustrated in Fig.9, Luu's and Rejec's equations could provide good comparison with NLRHA's result for 19-story building (Fig.9a) but they are no longer applicable for 38-story building (Fig.9b). Whereas, EC8's equation provided good agreement with NLRHA's results in both buildings except that it slightly underestimated at the base region.

For generic buildings as shown in Fig.10, in Bangkok, Rejec [5]'s formula could well estimate the shear forces along the height of the core wall in cantilever direction (Fig.10b), while it largely overestimated in couple direction (Fig.10a). In Chiang Mai, Luu [12]'s formula provided good estimation of shear forces along the height of core walls in both directions (Figs.10c, d), while other formulas over-predicted significantly shear forces at the base of core walls. EC8 [8]'s equation resulted in conservative results in most of the cases but it underestimated the base shear force in cantilever direction of 25-story core wall (Fig.10b).

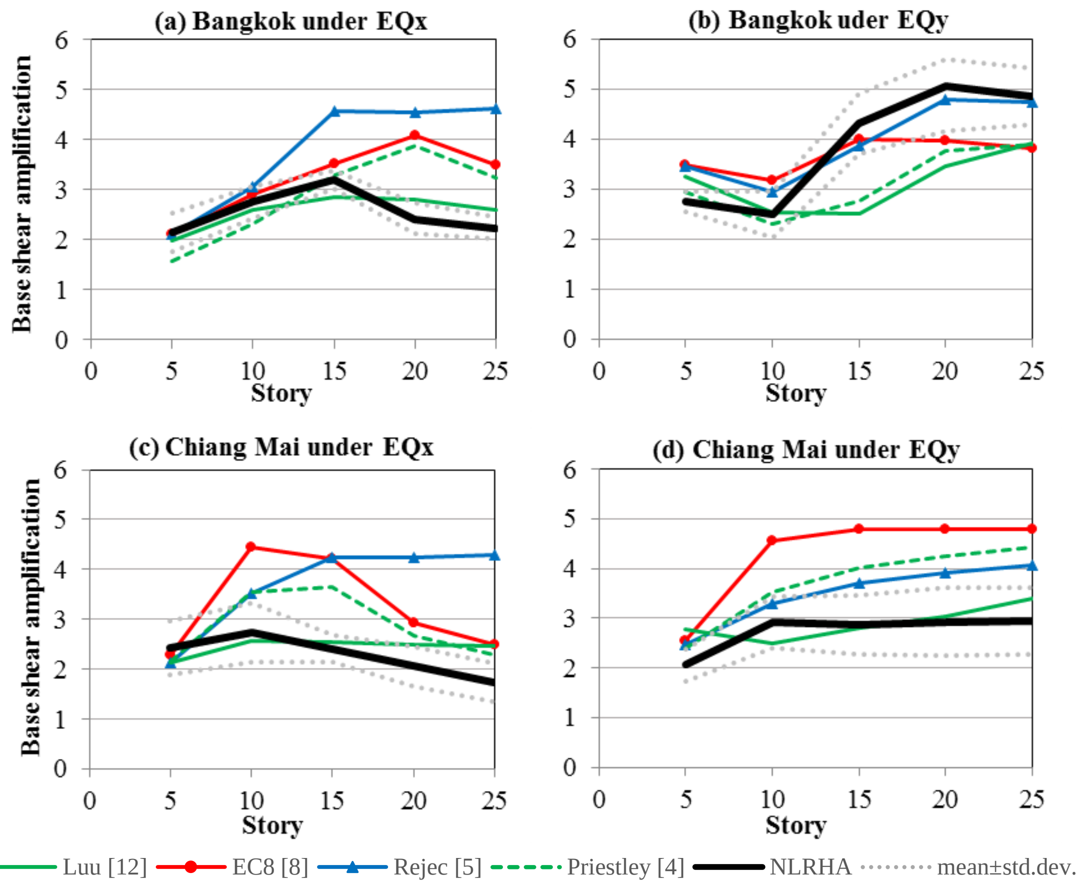


Fig. 8 – Comparison of BSA from NLRHA in this study and previously proposed equations: (a) Bangkok under EQx; (b) Bangkok under EQy; (c) Chiang Mai under EQx and (d) Chiang Mai under EQy

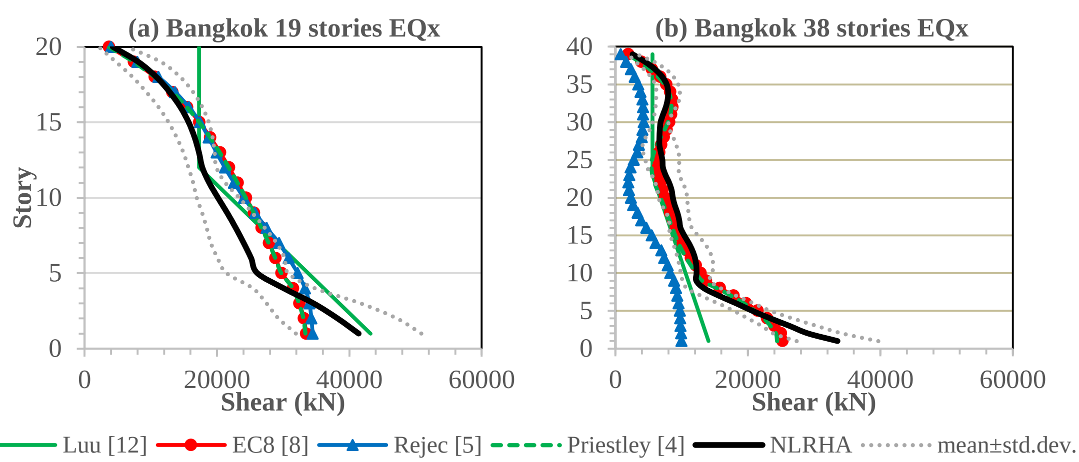


Fig. 9 – Comparison of story shear forces from NLRHA in this study and previously proposed equations: (a) Bangkok 19-story real building under EQx and (b) Bangkok 38-story real building under EQx

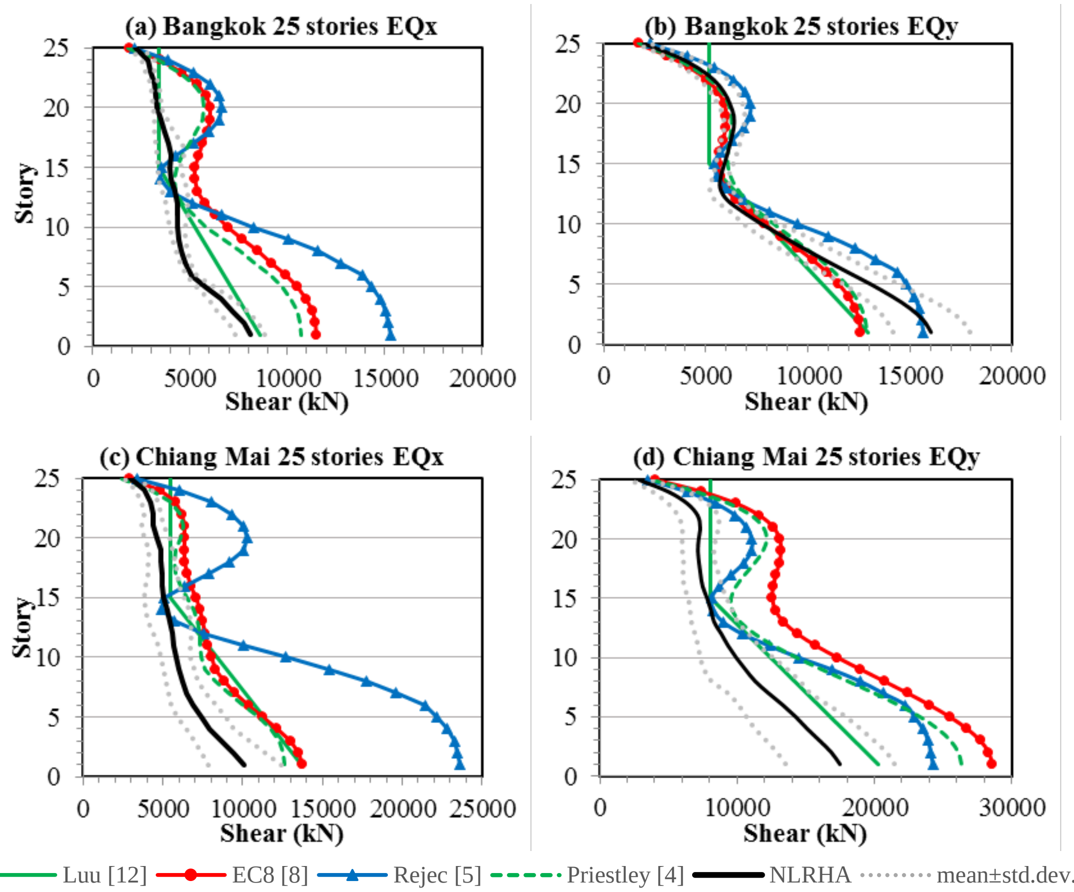


Fig. 10 – Comparison of shear forces along the height of 25-story generic core-wall building from NLRHA in this study and previously proposed equations: (a) Bangkok under EQx; (b) Bangkok under EQy; (c) Chiang Mai under EQx and (d) Chiang Mai under EQy

7. Conclusions

The seismic shear demands of RC core walls determined by response spectrum analysis (RSA) procedure and nonlinear response history analysis (NLRHA) have been compared. It was found that seismic shear demands determined by NLRHA were significantly larger than the design shear forces used in RSA procedure. This result was confirmed by both five generic buildings and two real buildings. The base shear amplification in cantilever core wall was larger than that in coupled core wall. The two building locations, Bangkok and Chiang Mai, having different spectrum shapes leaded to different shear amplification. Therefore, designing the wall to resist shear force from RSA procedure can lead to undesired shear failure.

Previous researcher's equation could estimate shear force from NLRHA only in buildings lower than 25 stories except EC8's equation that could well predict shear force from NLRHA even for tall buildings. In Bangkok, Rejec [5]'s equation could well predict shear force in cantilever core wall lower than 25 stories but it significantly overestimated in coupled direction. In Chiang Mai, Luu [12]'s equation could well estimate shear force in both cantilever and coupled core wall lower than 25 stories. But these two equations are no longer applicable for taller buildings. For taller buildings, the shear magnification factor equation in EC8 is recommended to be adopted to multiply with the shear forces from RSA procedure before using them as design shear forces of RC core wall. EC8 generally provides conservative results for both cantilever and coupled core walls with the exception that it slightly underestimates the base shear force in generic core-wall buildings of 20 stories and 25 stories, and in real building of 19 stories and 38 stories in Bangkok.

8. Acknowledgements

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