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# Seismic performance evaluation of partially staggered-wall apartment buildings

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**In this study the seismic performance of apartment buildings with partially staggered walls was evaluated using non-linear static and dynamic analysis, and was compared with the responses of conventional apartment buildings that use the shear wall system. Buildings of rectangular and square plan were selected as analysis model structures and the responses were compared with corresponding regular buildings. According to the analysis results, the structure with staggered walls designed with smaller response modification factor showed a similar or higher strength than the shear-wall-type structure. The structure with partial staggered-wall system satisfied the collapse prevention performance objective required by FEMA-356 and thus was considered to have sufficient capacity for design level seismic load.**

## Introduction

Recently spatial flexibility has become more and more important in planning of residential buildings, not only for special planning but also for ease of remodelling. New structural systems are required to meet growing demands for spatial flexibility, and one good alternative is the staggered-truss/wall system. The system consists of a series of storey-high trusses/walls spanning the total width between two rows of exterior columns and arranged in a staggered pattern on adjacent column lines. The systems were developed to achieve a more efficient structural frame to resist wind loads and provide versatility of floor layout. The staggered-truss system (STS) is known to be appropriate for use in such buildings as apartments, condominiums, dormitories and hotels (Taranath, 1998). The STS has the advantage that large clear span open areas are possible at the first floor level, because columns are located only on the exterior faces of the building. Other benefits include minimum deflection and greater stiffness in the structure while reducing seismic loads and foundation costs (Scalzi, 1971). It has also been reported that the structural costs per unit building area, on a relative basis, turned out to be lowest in STS (Cohen, 1986). The staggered truss systems have been successfully applied to many large-scale building projects and their efficiency and economy have been reported (McNamara, 1999; Pollak and Gustafson, 2004).

The STS, however, has not been considered as one of the basic seismic-force-resisting systems in most of the design codes, because of the vertical discontinuity of the lateral load-resisting system. FEMA-450 (FEMA, 2003) requires that lateral systems that are not listed as the basic seismic-force-resisting systems shall be permitted if analytical and test data are submitted to demonstrate the lateral force resistance and energy dissipation capacity. To facilitate the application of the STSs, the American Institute of Steel Construction published the *Design Guide 14: Staggered Truss System Framing Systems* (AISC, 2002), in which some recommendations and examples for structural design are provided. The design guide recommends the use of 3·0 for the response modification factor for seismic design; however, the other seismic behaviour factors, such as overstrength and ductility factors, to define inelastic behaviour of the structure, are not specified. Further research is still necessary in order for the system to be accepted as a standard seismic load-resisting structure system.

Compared with the STS, the staggered-wall system in reinforced concrete (RC) has not been widely investigated. In the present

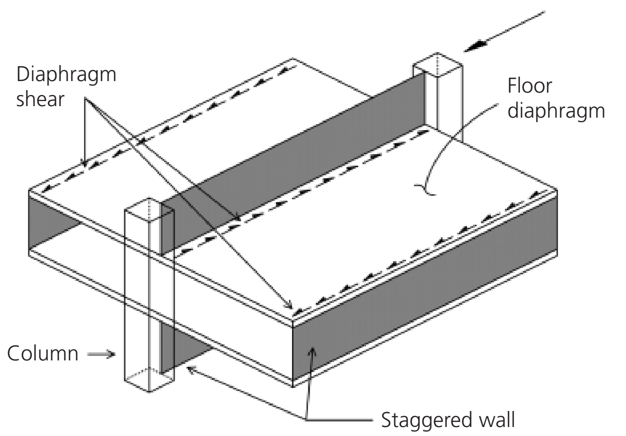


Figure 1. Transfer of lateral load in a staggered-wall system

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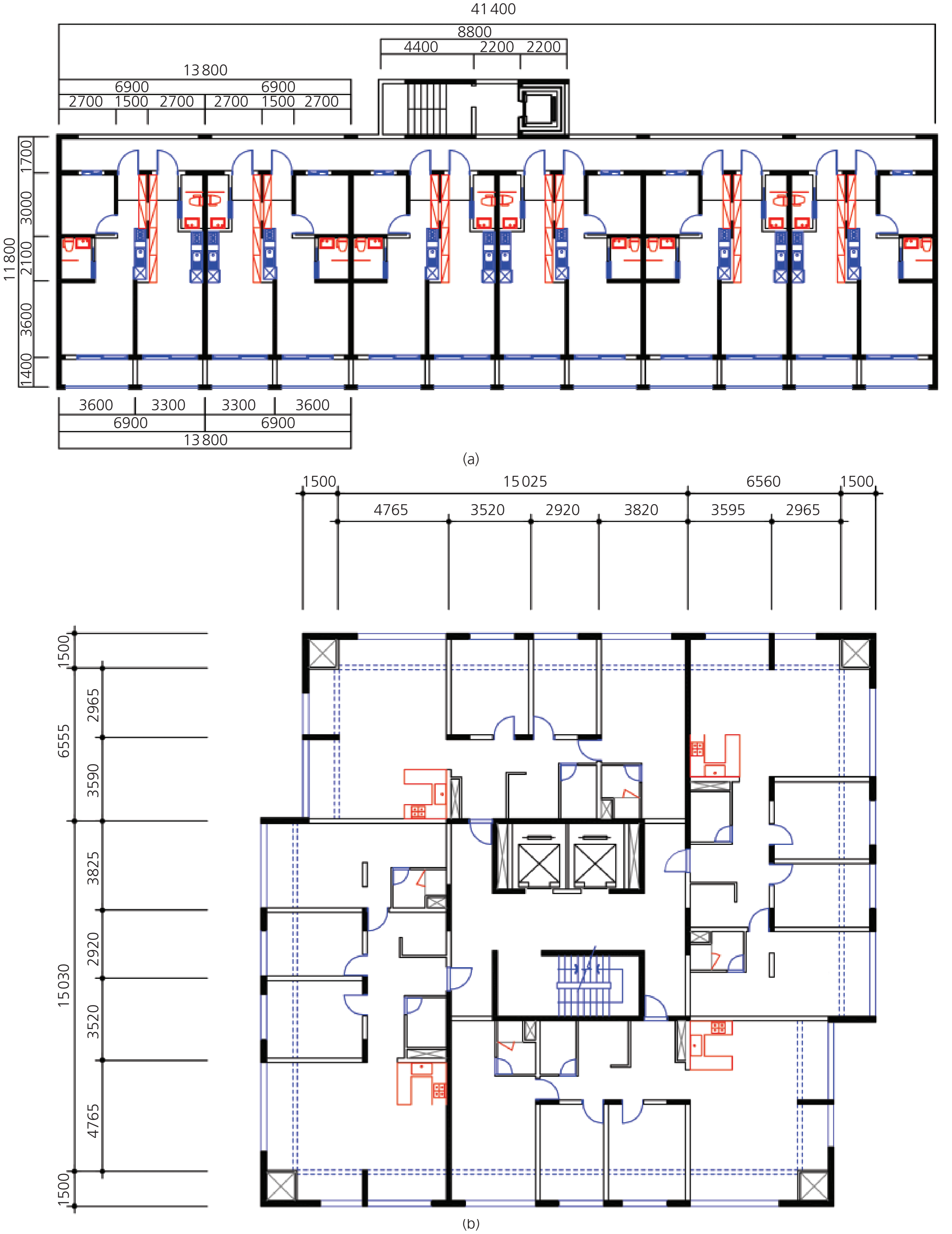


Figure 2. Architectural plans of analysis model structures (unit: mm): (a) rectangular structure; (b) square structure



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study 12-storey structures with regular RC shear walls and structures with some of their shear walls vertically staggered were designed, and their seismic behaviours were compared through non-linear static and dynamic analyses. The responses of two types of staggered-wall systems, rectangular-plan buildings with parallel shear walls and square-plan buildings with orthogonal shear walls, were compared.

## Staggered-truss/wall systems

In a staggered-truss/wall system, shown in Figure 1, the storey-high trusses/walls that span the width of the building are located in a staggered pattern. Columns have minimum bending moments due to gravity and lateral loads, because of the cantilever action of the double-planar system of framing. With the columns only on the exterior of the building, the usual interior columns are omitted, thus providing a full width of column-free area on the first floor. Columns are oriented with their strong axis resisting lateral forces in the longitudinal direction of the building (AISC, 2002).

In a STS the total frame is acting as a stiff truss with direct axial loads acting in most structural members. The staggered arrangement of the floor-deep trusses placed at alternate levels on adjacent column lines allows an interior floor space of twice the column spacing to be available for freedom of floor arrangements. The floor system spans from the top chord of one truss to the bottom chord of the adjacent truss, serving as a diaphragm transferring the lateral shears from one column line to another. This enables the structure to perform as a single braced frame, even though the trusses lie in two parallel planes. However, secondary bending occurs at the chords in the Vierendeel panels of the trusses, which may become a weak link of the system. Kim *et al.* (2007) carried out non-linear analysis of STS systems and found that plastic hinges formed at horizontal and vertical chords of Vierendeel panels in STS, which subsequently led to brittle collapse of the structure. Based on the analysis results they recommended reinforcement of Vierendeel panels to increase ductility of the system.

In a staggered-wall system with RC walls located in alternate floors, flexibility in spatial planning can be achieved when compared with conventional wall-type structures with vertically continuous shear walls. As in the staggered truss system, the horizontal load is transferred to staggered walls below through diaphragm action of floor slabs. The staggered walls are designed as storey-high deep beams, and at the ends of the walls are columns continuous throughout the stories. The columns in this case are actually short walls located at the ends of the staggered walls. In the rectangular staggered-wall structures, the exterior shear walls and some of the interior walls remain continuous throughout the storeys, and in the square structures only the partition walls between apartment units are staggered, while the core walls are continuous throughout the building height. Therefore, the systems analysed in the current paper are referred to as partially staggered-wall (PSW) systems.

## Design of analysis model structures

The architectural plans of typical RC shear-wall apartment buildings that are widely used in Korea and other countries are shown in Figure 2. Two types of wall-type apartment buildings, rectangular-plan buildings with perpendicular parallel shear walls and square-shaped buildings with orthogonal shear walls, were used for analysis. Each long shear wall separates two apartment units. In the staggered wall systems, apartments with twice the unit area can be provided. The model buildings have 12 storeys and the storey height is 2·7 m. The thickness of the walls, both shear walls and staggered walls, is 20 cm throughout the storeys. The thickness of the floor slabs is 21 cm, which is the minimum thickness required for wall-type apartment buildings in Korea to prevent transmission of excessive noise and vibration through the floors. In PSW system structures, the long shear walls located between apartment units are placed alternately along the height. The regular rectangular RC wall structure shown in Figure 2(a) has 12 apartment units, whereas the number of units can be arranged from four to 12 in the rectangular structure with

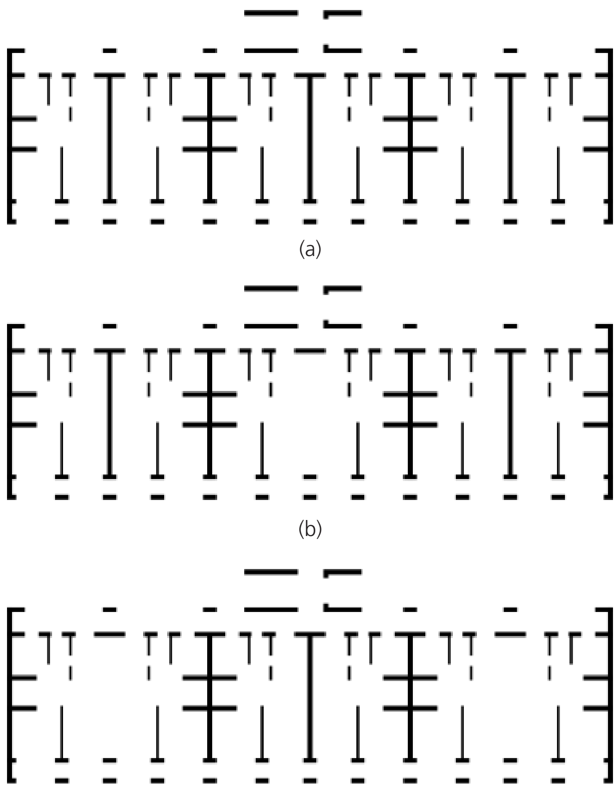
(a)
(b)
(c)

Figure 3. Structural plans of rectangular structures: (a) rectangular structure; (b) staggered-wall structure (above); (c) staggered-wall structure (below)

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staggered walls. The tower-shaped building shown in Figure 2(b) has four apartment units in each storey, which can be arranged to two units in the staggered system. Figures 3 and 4 depict the structural framing plan for the rectangular and square analysis model structures, respectively, and Figure 5 depicts the three-dimensional view of the PSW buildings.

In the structural design for seismic load, the regular RC shear-wall structures were considered as the 'special RC shear-wall systems' specified in the IBC 2006 (ICC, 2009) and the response modification factor of 5·0 was used to derive design base shear. In the PSW systems, both with rectangular and square plans, the staggered walls act like storey-high deep beams and as a result the structures correspond to a typical weak column–strong beam system. Therefore, a smaller response modification factor of 3·0 was used in the structural design of the staggered-wall systems. The soil type was assumed to be the site class B, which corresponds to a rock site, in all cases. The design spectral response acceleration parameters, $S_{DS}$ and $S_{D1}$, were computed as 0·37 and 0·15, respectively, in the IBC 2006 format, based on the

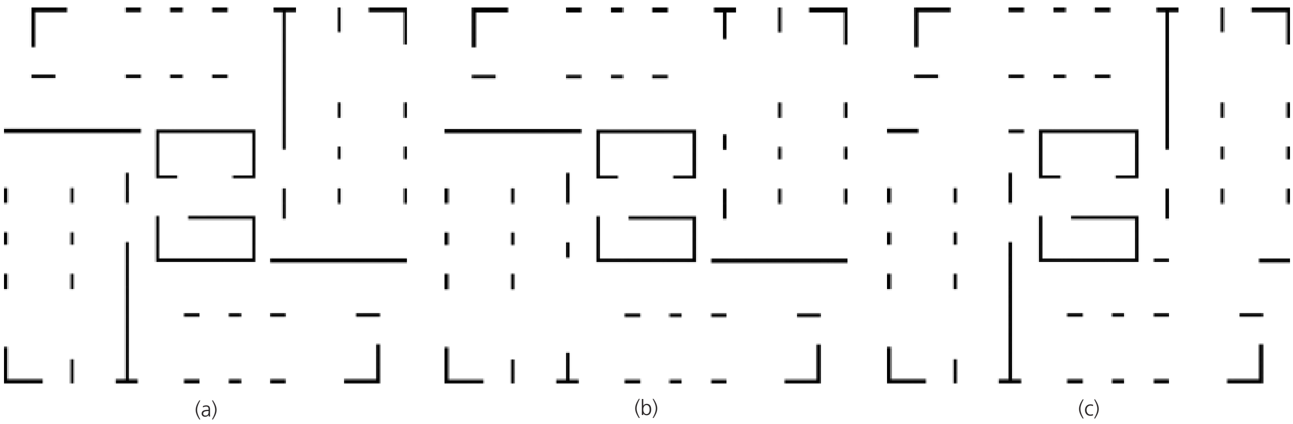


Figure 4. Structural plans of square buildings: (a) regular shear-wall structure; (b) staggered-wall structure (above); (c) staggered-wall structure (below)

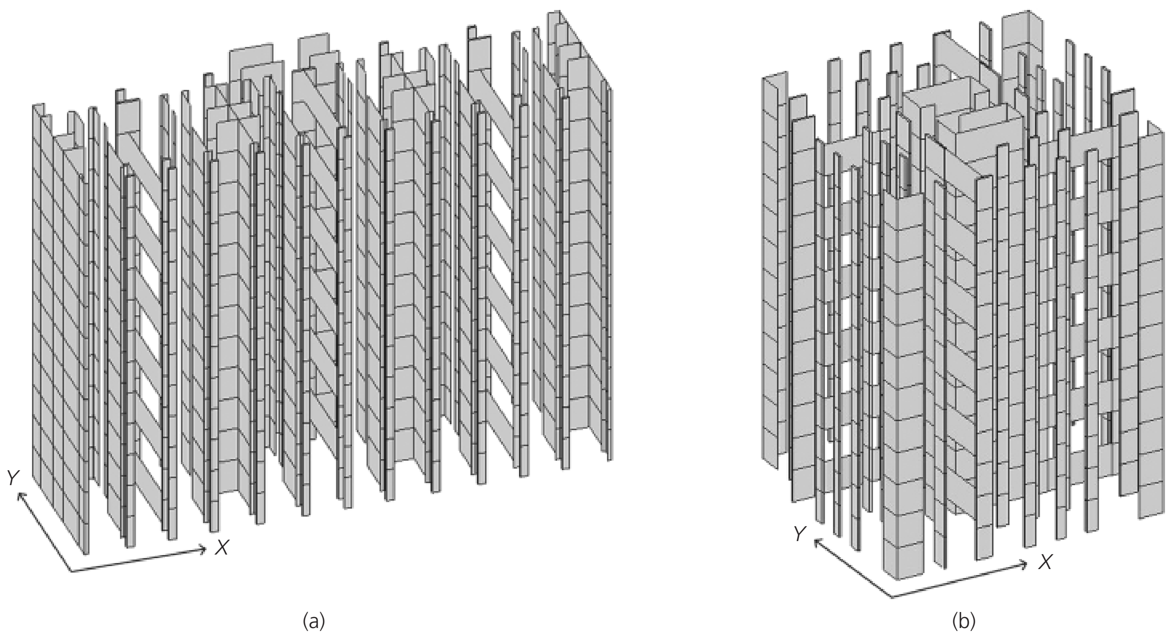


Figure 5. Three-dimensional view of the partially staggered-wall structures: (a) rectangular structure; (b) square structure

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| Structural system | | | $V_d$: kN | Mode 1 | | Mode 2 | |
|---|---|---|---|---|---|---|---|
| | | | | Period: s | Modal mass: % | Period: s | Modal mass: % |
| Rectangular | Shear wall | *X*-direction | 2650·5 | 2·19 | 63·8 | 0·36 | 19·8 |
| | | *Y*-direction | 2650·5 | 0·41 | 66·2 | 0·09 | 21·4 |
| | Staggered wall | *X*-direction | 4289·2 | 2·17 | 63·8 | 0·36 | 19·6 |
| | | *Y*-direction | 4289·2 | 0·44 | 66·2 | 0·09 | 21·3 |
| Square | Shear wall | *X*-direction | 2226·0 | 0·83 | 58·2 | 0·16 | 20·1 |
| | | *Y*-direction | 2226·0 | 0·90 | 62·7 | 0·17 | 20·8 |
| | Staggered wall | *X*-direction | 3568·5 | 0·90 | 56·4 | 0·18 | 18·9 |
| | | *Y*-direction | 3568·5 | 1·02 | 63·3 | 0·20 | 19·9 |

**Table 1.** Design base shears and modal characteristics of the model structures

maximum considered earthquake ground acceleration of 0·22***g***, where ***g*** is the acceleration of gravity. The design base shear, $V_d$, and the first two natural periods and modal masses are listed in Table 1. The ultimate strength of concrete is 24 MPa up to the fourth storey and 21 MPa above the fourth storey. The rebars have tensile strength of 400 MPa.

The deflections of the floor slabs in the rectangular model structures under service load conditions were computed using the finite-element analysis code Midas SDS (Midas, 2007) and they are depicted in Figure 6. The maximum floor deflections of the regular and the staggered-wall systems were computed as 0·613 mm and 0·648 mm, respectively. Even though the vertical deflections of the slabs in the PSW structure are slightly higher than those of the regular structure, they satisfy the requirement for vertical deflection.

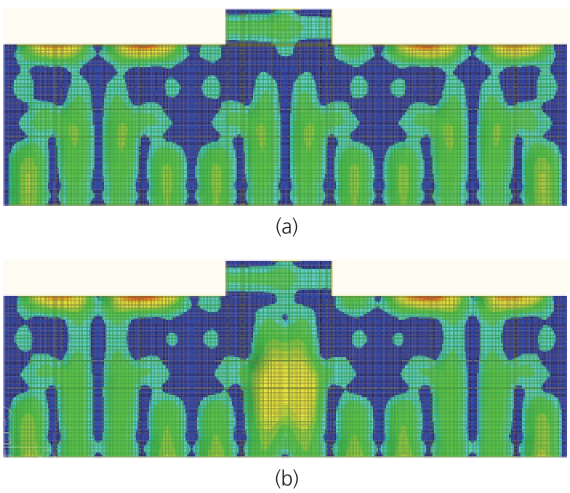


**Figure 6.** Vertical deflection of slabs under service load: (a) regular shear-wall structure; (b) staggered-wall structure

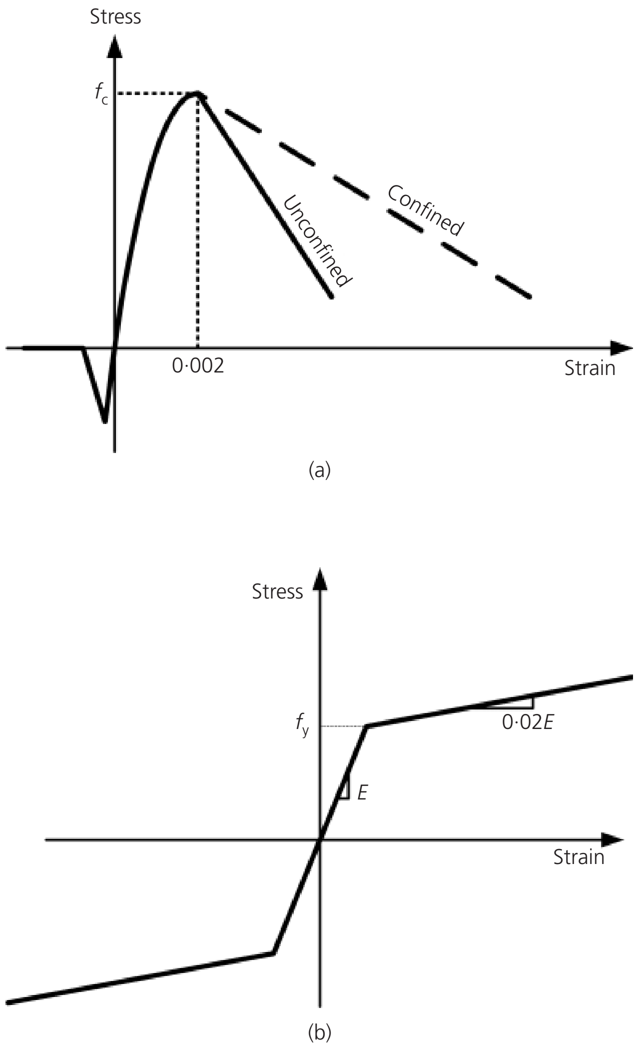


**Figure 7.** Stress–strain relationships of structural materials: (a) concrete; (b) steel

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## Seismic performance evaluation

### Modelling for analysis

The seismic performances of the model structures were evaluated using the non-linear analysis program CANNY (Li, 2004). The expected ultimate strengths of the concrete and steel were taken to be 1·5 and 1·25 times the nominal strengths based on the recommendation of FEMA-356 (FEMA, 2000). The non-linear force–displacement relationships of the materials are shown in Figure 7, where $f_c$ and $f_y$ represent the compressive strength of concrete and the yield stress of rebars. $E$ is the elastic modulus. The behaviour of the core concrete of the columns and walls was modelled using the work of Kent and Park (1971). The reinforcing steel was modelled by bilinear lines with 2% of post-yield stiffness. The shear walls were modelled by fibre elements with shear springs, as shown in Figure 8, where the in-plane shear

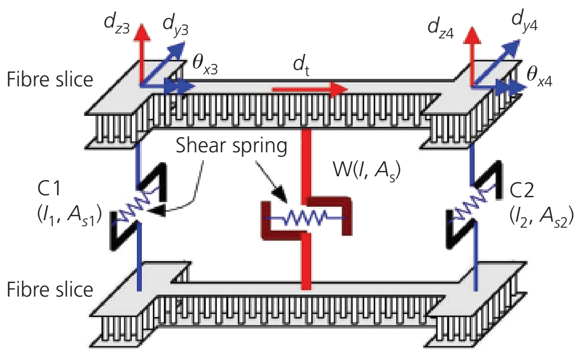


Figure 8. Analysis modelling of shear walls using fibre elements and shear springs

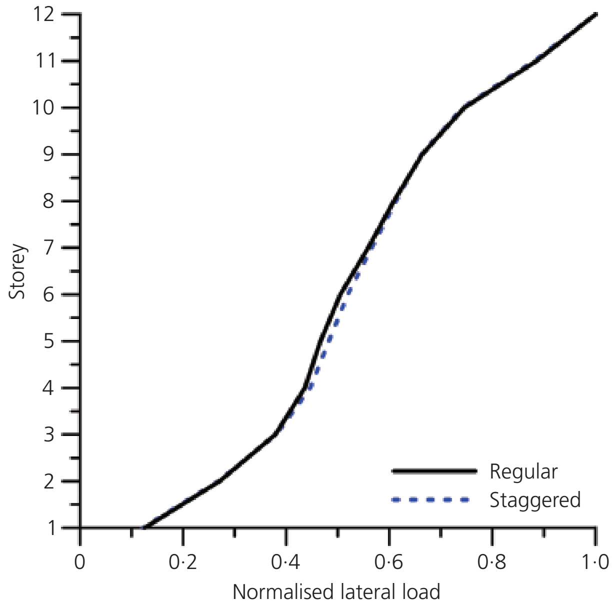


Figure 9. Vertical distribution of lateral load along the transverse direction of the rectangular structures

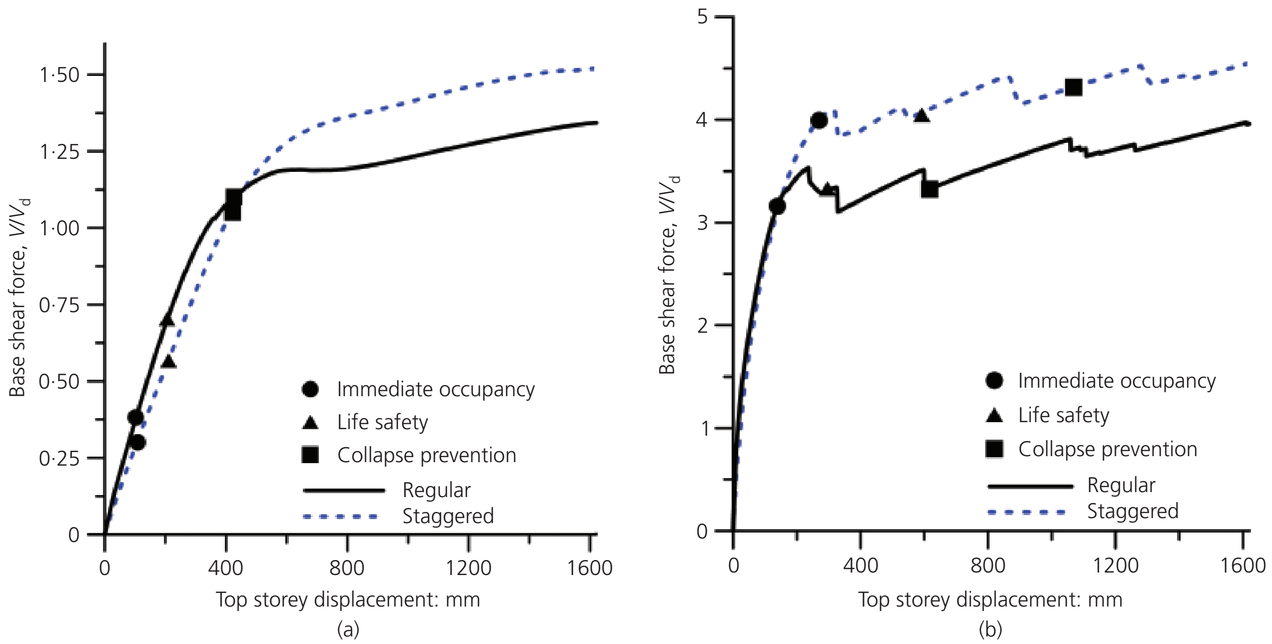


Figure 10. Pushover curves of the rectangular buildings: (a) longitudinal direction; (b) transverse direction

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force is resisted by the spring, W, and the out-of-plane shear is resisted by the springs C1 and C2. The symbols *I, A, d* and $\theta$, denote the moment of inertia, cross-sectional area, displacement and the rotation at a joint, respectively. The slabs were considered as a rigid diaphragm.

## Non-linear static analysis results

Non-linear static analyses of the model structures were performed to obtain non-linear force–displacement relationships. The vertical distribution of the lateral load for pushover analysis was determined combining the first three vibration mode shapes, and the normalised lateral load patterns applied along the transverse direction are presented in Figure 9. The non-linear analyses were carried out until the maximum displacement reached 5% of the building height. Figure 10 shows the pushover curves of the rectangular model structures along the longitudinal and the transverse directions, and Figure 11 shows the pushover curves of the square buildings along the transverse direction. The vertical axis of the pushover curve represents the base shear divided by the design base shear, and the maximum values correspond to the overstrength factors of the structure. In the rectangular structures the maximum strengths of the PSW structure turned out to be higher than those of the regular shear-wall structure in both directions. The maximum strengths of the square buildings are almost the same. As the PSW systems were designed with a lower response modification factor, the design base shears for PSW are larger than those of the regular shear-wall system buildings. This results in higher (rectangular) or similar (square) strength of PSW systems compared with those of the regular shear-wall systems. In the curves the performance points corresponding to the immediate occupancy (IO), life safety (LS) and collapse prevention (CP) limit states specified in FEMA-356 for shear walls are indicated. The limit state for each performance point is provided as 0·5%, 1·0% and 2·0% of the storey height, respectively. For the longitudinal direction of the rectangular plan structure, the limit states are specified as 1·0%, 2·0% and 4·0%, respectively. The analysis results show

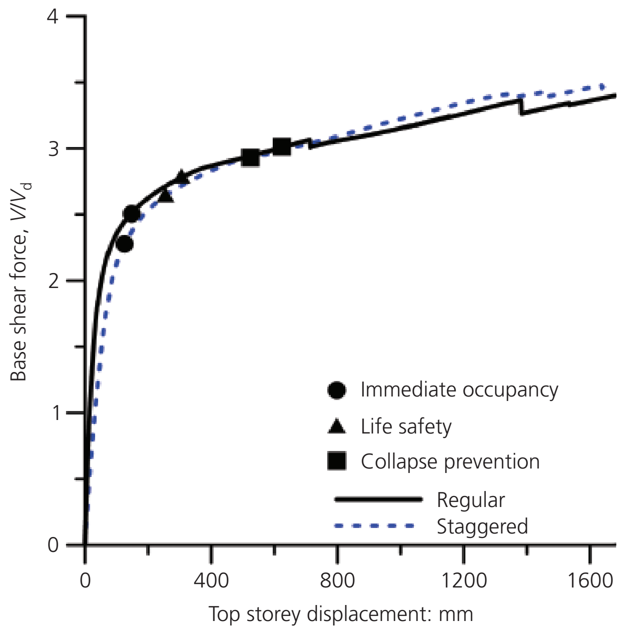


Figure 11. Pushover curves of the square buildings

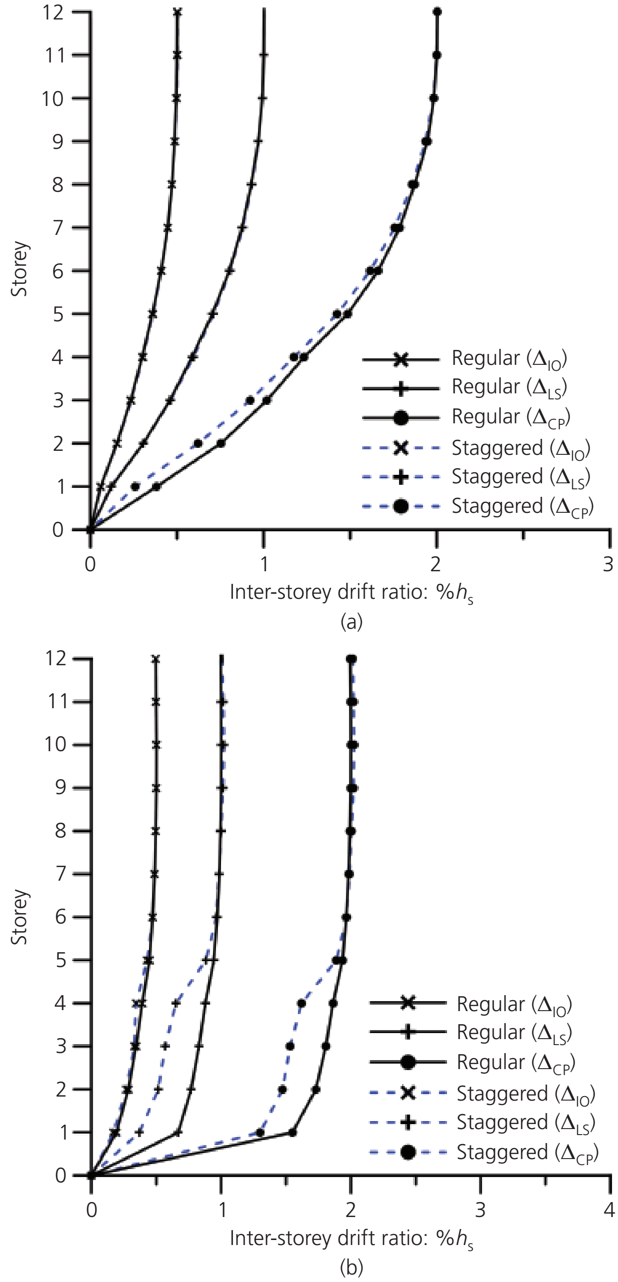


Figure 12. Inter-storey drifts of the rectangular buildings: (a) longitudinal direction; (b) transverse direction

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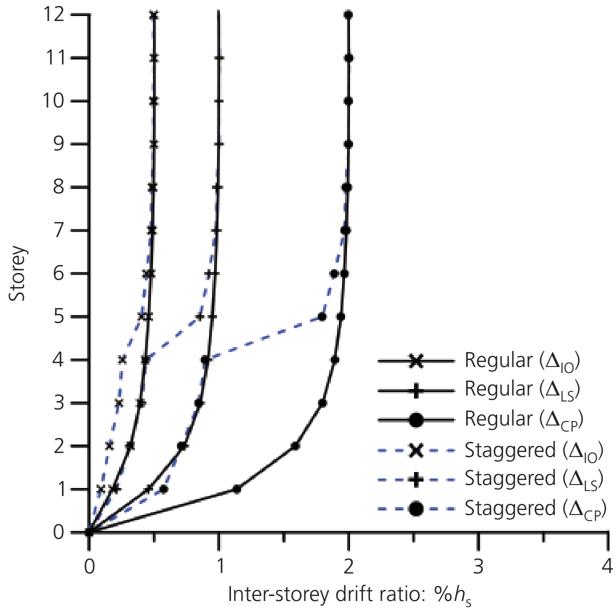


**Figure 13.** Inter-storey drifts of the square buildings

that all the model structures remained stable until the collapse-prevention limit state was satisfied.

Figures 12 and 13 depict the inter-storey drifts of the model structures when the maximum inter-storey drift reached the specified limit state. It can be noticed that the vertical distribution of the inter-storey drifts of the regular shear-wall structures are more or less smooth, whereas the inter-storey drifts are concentrated in a lower few storeys in the PSW structures.

The plastic hinge formations in staggered walls at each performance point are presented in Figures 14 and 15. It can be observed that as the lateral deformation increases from the IO to CP stages the amount of plastic deformation also gradually increases. In both the rectangular and the square models, plastic hinges were concentrated in the first and the fifth storeys, which matches well with the large inter-storey drifts in those storeys as observed in Figures 12 and 13.

The plastic hinge formation of the PSW structures indicates that most plastic deformations are concentrated in columns located at

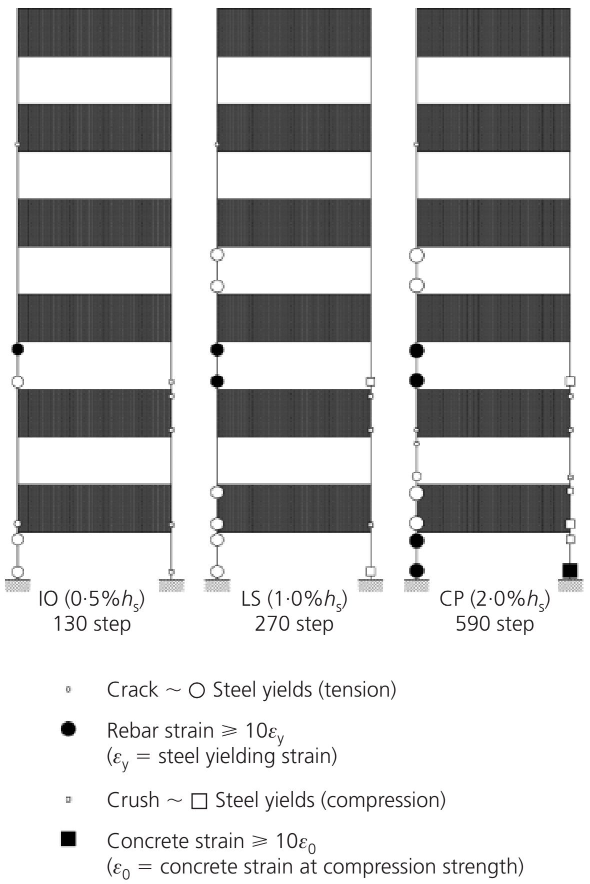


**Figure 14.** Plastic hinge formation in a staggered wall in the rectangular PSW building in three different performance stages

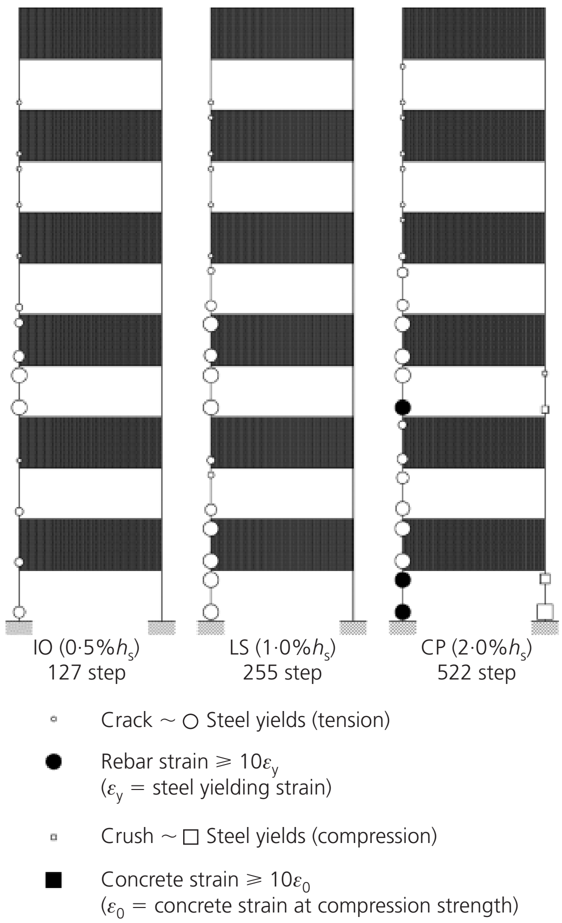


**Figure 15.** Plastic hinge formation in a staggered wall in the square PSW building in three different performance stages

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the ends of the staggered walls. Therefore, it can be expected that if the columns are reinforced, the seismic performance of the structures would be enhanced. Figure 16 depicts the pushover curves of the PSW structures with their column thickness increased to 1·5 and 2 times the original thickness. It can be observed that when the column thickness increased from 20 cm to 30 cm the maximum strengths of the rectangular and the square PSW structures increased to about 23% and 15%, respectively. However, as the thickness further increased to 40 cm, the increase in the strength was not significant.

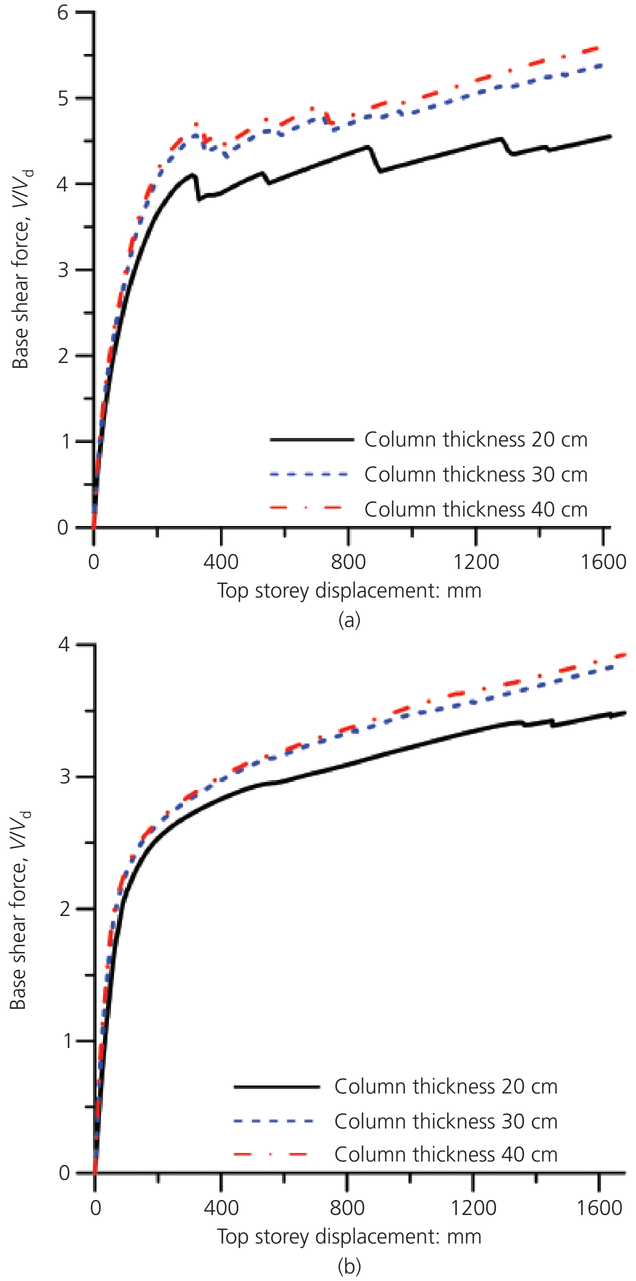


**Figure 16.** Pushover curves of the PSW buildings with varying column thickness: (a) rectangular structure; (b) square structure

## Time history analysis results

Non-linear dynamic time history analyses of model structures were carried out using the seven earthquake records developed for the SAC phase II programme (Somerville *et al.*, 1997). The earthquake records were scaled to fit the design spectrum of the Korea building code (KBC, 2005). Figure 17 shows the response spectra of the earthquake records used in the analysis and Table 2 shows their characteristics. In the analyses, the damping ratios of the model structures were assumed to be 5% of the critical damping in the first and second modes of vibration. The mean inter-storey drifts of the rectangular and the square buildings obtained from the time history analyses using the seven earthquake records are presented in Figure 18 and Figure 19, respectively. The horizontal axis represents the percentage of the storey height $h_s$. In the rectangular buildings the mean inter-storey drifts of the PSW structure turned out to be slightly larger than those of the regular shear-wall buildings in both longitudinal and transverse directions. It can be observed that the drifts along the transverse direction, where more shear walls were located, are significantly smaller than those along the longitudinal direction. Differently from the results of the rectangular PSW structure, the mean inter-storey drifts of the square PSW structure were slightly smaller than those of the regular shear-wall buildings. In both the rectangular and the square structures the mean inter-storey drifts turned out to be smaller than 1·5% of the storey height specified in the design code as a limit state for design seismic load. Therefore it can be concluded that the PSW system buildings retain enough strength and stiffness to satisfy the requirement of the current design code.

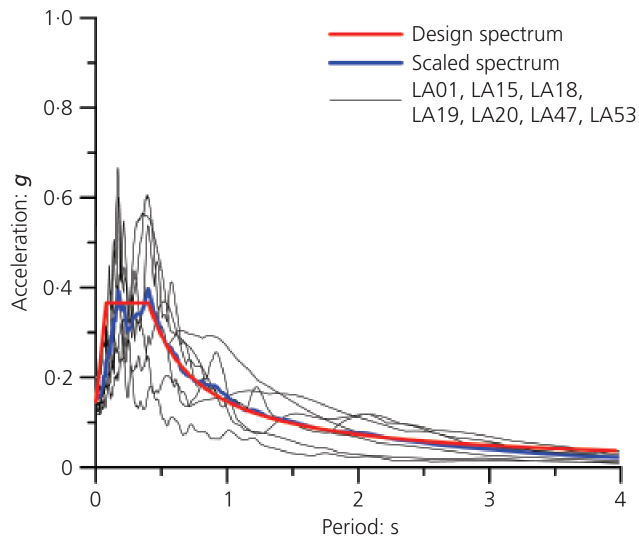


**Figure 17.** Response spectra of seven earthquake records and design spectrum

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| No. | Ground motion record (year) | Time step: s | Duration: s | PGA: cm/s$^2$ | Scaled PGA: cm/s$^2$ |
|---|---|---|---|---|---|
| LA01 | Imperial Valley (1940) | 0·020 | 53·48 | 452·03 | 111·4 |
| LA15 | Northridge (1994) | 0·005 | 14·945 | 523·30 | 136·2 |
| LA18 | Northridge (1994) | 0·020 | 59·98 | 801·44 | 162·9 |
| LA19 | North Palm Springs (1986) | 0·020 | 59·98 | 999·43 | 198·3 |
| LA20 | North Palm Springs (1986) | 0·020 | 59·98 | 967·61 | 134·4 |
| LA47 | Landers (1992) | 0·020 | 79·98 | 331·22 | 172·5 |
| LA53 | Parkfield (1996) | 0·020 | 26·14 | 680·01 | 120·6 |

**Table 2.** Lists of earthquake ground motions used in the dynamic analysis

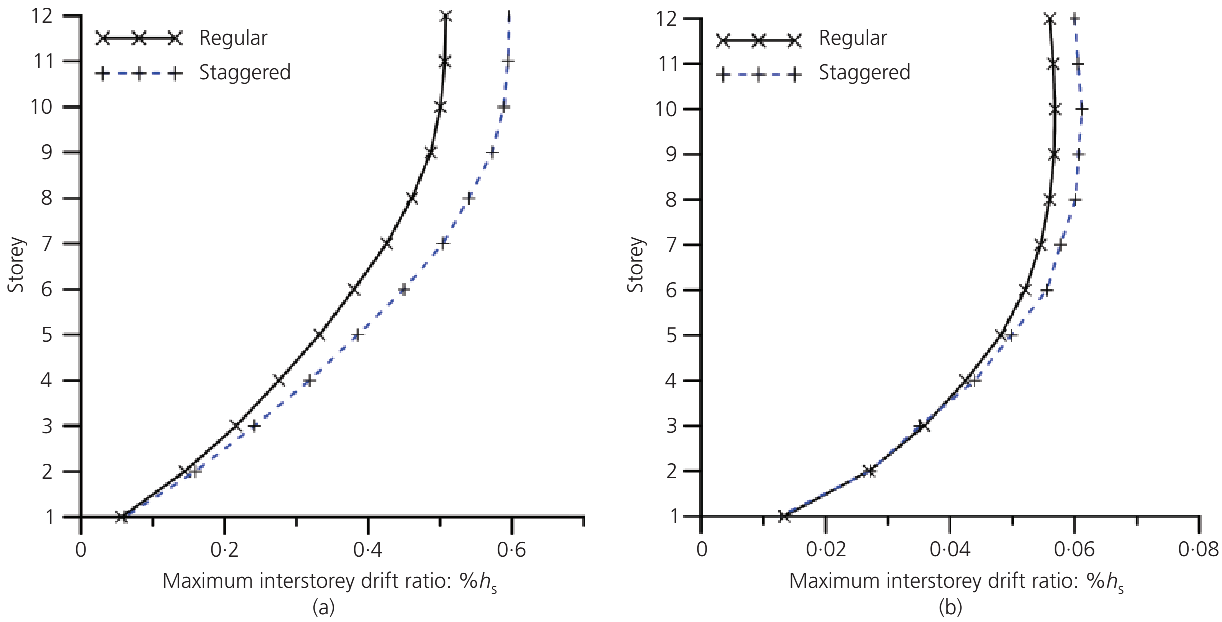


**Figure 18.** Mean inter-storey drifts of the rectangular buildings obtained from non-linear dynamic analysis: (a) longitudinal direction; (b) transverse direction

The structural responses subjected to the scaled earthquake ground motions turned out to be nearly in the elastic range. To observe inelastic behaviour of the model structures when subjected to dynamic motion, the rectangular model structures were analysed using the El Centro earthquake ground motion with its peak ground acceleration (PGA) equal to 0·342$g$ as shown in Figure 20. Figure 21 depicts the time histories of the top storey displacements of the regular and the PSW structures. It can be observed that the responses are quite similar to each other along the longitudinal direction; however, along the transverse direction the PSW structure experienced slightly smaller inelastic deformation than the regular structure, judging from the amount of residual displacements.

### Amount of structural materials

Figure 22 shows the change in the amount of concrete and rebars in the PSW structures in comparison with the regular structures. As the PSW structures were designed with fewer shear walls and with smaller response modification factors (therefore with larger seismic base shear), the amount of rebars required for columns (short shear walls located perpendicular to the staggered walls), walls (both continuous and staggered) and slabs increased significantly compared with the amount of rebars used for regular structures. However, the reduction in concrete is relatively small in comparison with the increase in rebars. Considering the fact that the cost of rebars is generally higher than that of concrete, let alone more sophisticated formwork placement, the PSW

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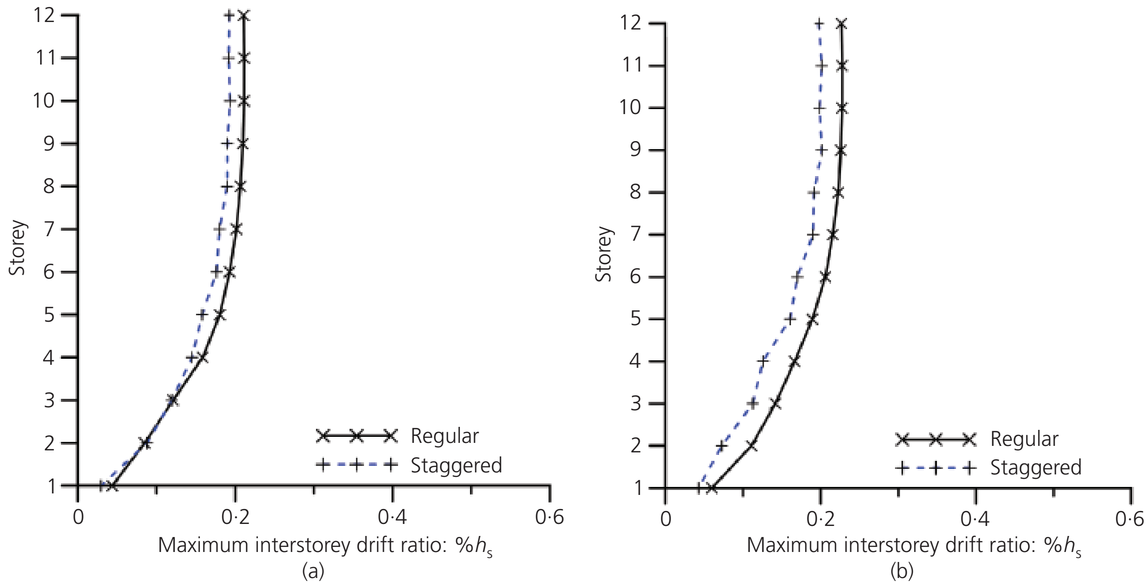


**Figure 19.** Mean inter-storey drifts of the square buildings obtained from non-linear dynamic analysis: (a) longitudinal direction; (b) transverse direction

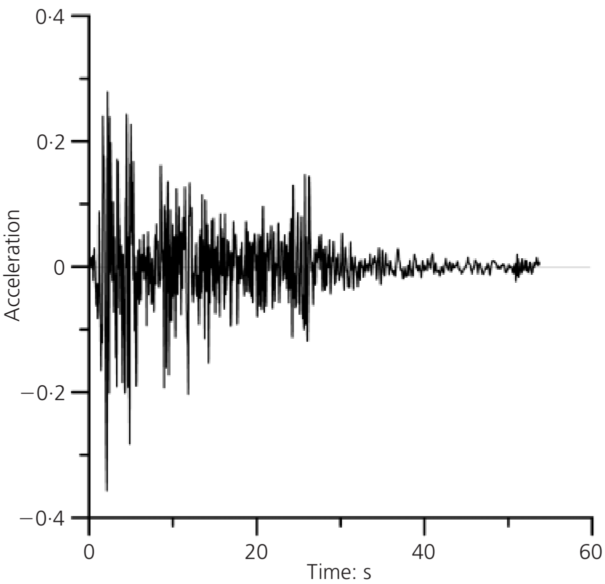


**Figure 20.** Time history of the El Centro earthquake

systems may not be economical compared with conventional shear-wall structures. However, the justification for the staggered-wall systems may be found in the fact that spatial flexibility can be provided in shear-wall-type residential buildings.

## Conclusions

In this study the seismic performances of 12-storey regular and staggered-RC-wall-type structures were compared through non-linear static and dynamic analyses. Both rectangular and square plan buildings were selected as analysis model structures. According to the analysis results, the structure with partially staggered walls designed with smaller response modification factor showed a similar or higher strength than the shear-wall-type structure. The structure with PSW system satisfied the collapse prevention performance objective required by FEMA-356 and thus was considered to have sufficient capacity for design level seismic load. It was observed that the amount of concrete used for PSW structures decreased significantly owing to the staggering of some of the concrete walls. However, the amount of rebars did not decrease significantly compared with the amount of rebars used in regular shear-wall structures because PSW structures were designed with smaller response modification factors. It also should be noted that, as all conclusions have been drawn based on numerical analysis results, proper experiments need to be conducted to verify the effectiveness of the PSW structures.

## Acknowledgement

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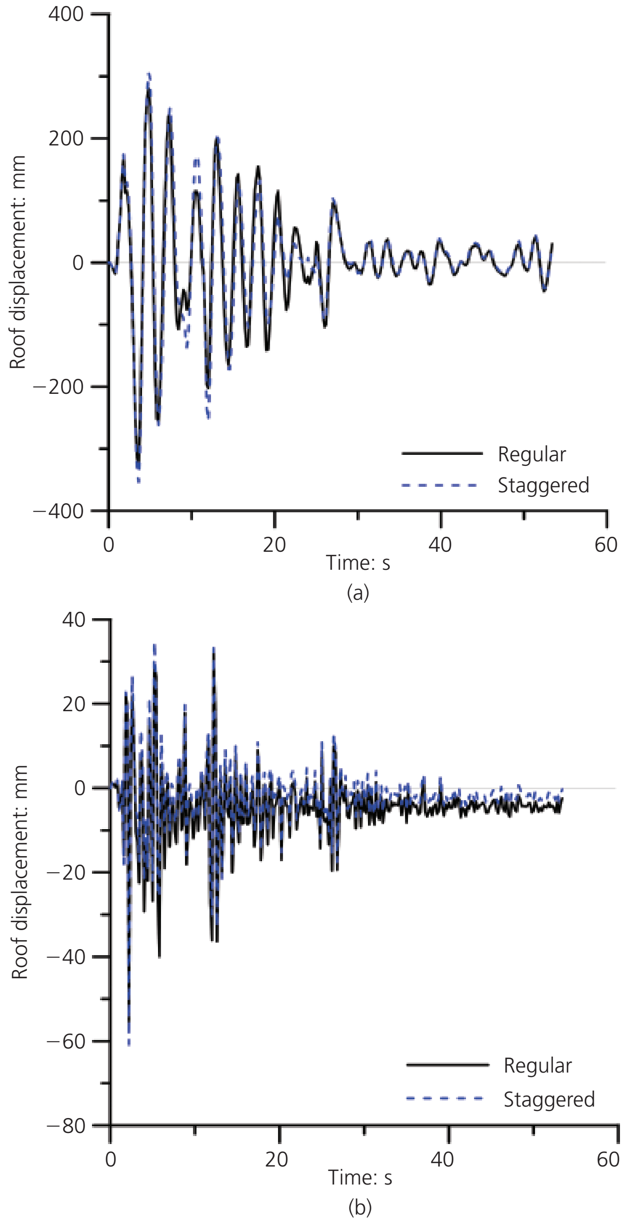


Figure 21. Maximum displacements of the rectangular structures subjected to the El Centro earthquake: (a) longitudinal direction; (b) transverse direction

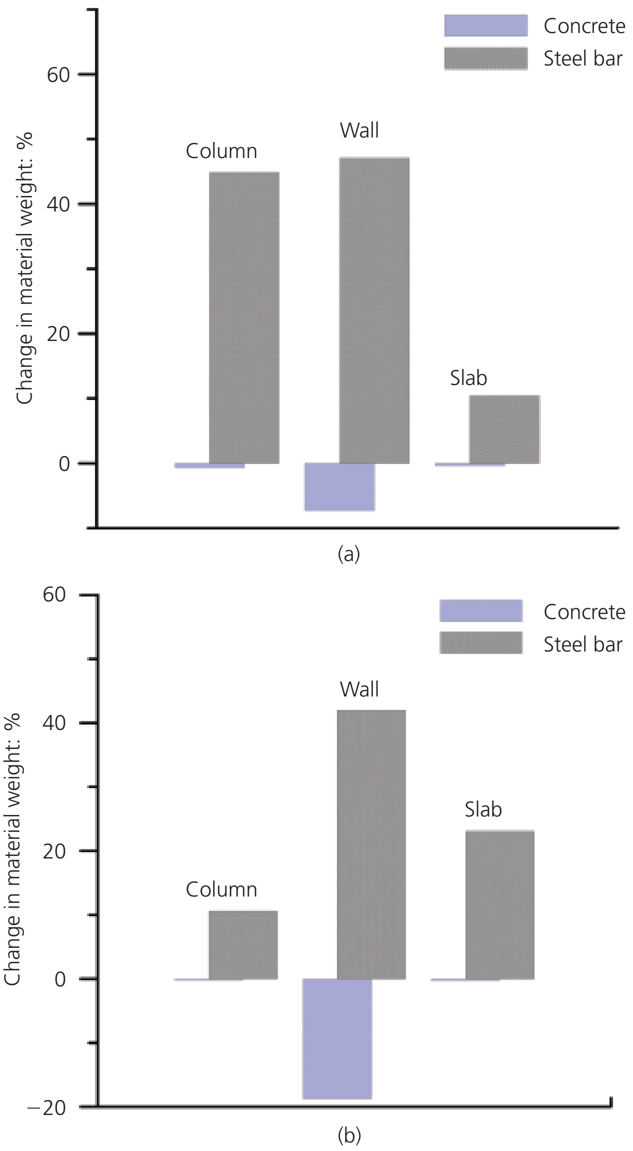


Figure 22. Change of structural materials in the partially staggered-wall buildings: (a) rectangular structure; (b) square structure

REFERENCES

AISC (American Institute of Steel Construction) (2002) *Staggered Truss Framing System, AISC Steel Design Guide 14*. AISC, Chicago, IL, USA.

Cohen MP (1986) Design solutions utilizing the staggered-steel truss system. *AISC Engineering Journal* **23(3)**: 97–106.

FEMA (Federal Emergency Management Agency) (2000) *FEMA 356: Pre-standard and Commentary for the Seismic Rehabilitation of Building*. Federal Emergency Management Agency, Washington, DC, USA.

FEMA (2003) *FEMA 450: NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. Federal Emergency Management Agency, Washington, DC, USA.

ICC (2009) *International Building Code*. International Code Council, Falls Church, VA, USA.

KBC (2005) *KBC 2005, Korea Building Code*. Korea Institute of Architectural Engineering.

Kent DC and Park R (1971) Flexural members with confined concrete. *Trans. ASCE* **97(ST7)**: 1969–1990.

Kim J, Lee J and Kim Y (2007) Inelastic behavior of staggered truss systems. *The Structural Design of Tall and Special Buildings* **16(1)**: 85–105.

Li K-N (2004) *CANNY: 3-Dimensional Nonlinear Static/Dynamic*

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*Structural Analysis Computer Program-User Manual*. CANNY Structural Analysis, Canada.

McNamara RJ (1999) Aladdin Hotel. *American Institute of Steel Construction, Modern Steel Construction*, May.

Midas (2007) *MIDAS SDS, General Structure Design System for Window*, Version 3.4.0.

Pollak BS and Gustafson M (2004) Complex apartments. *American Institute of Steel Construction, Modern Steel Construction*, September.

Scalzi JB (1971) The staggered-truss system – structural considerations. *AISC Engineering Journal* October.

Somerville P, Smith H, Puriyamurthala S and Sun J (1997) *Development of Ground Motion Time Histories for Phase 2 of the FEMA/SAC Steel Project*. SAC Steel Project, Richmond, California, SAC Joint Venture, SAC/BD 97/04.

Taranath BS (1998) *Steel, Concrete, and Composite Design of Tall Buildings*. McGraw Hill.

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