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Fundamental period formulae for RC staggered wall buildings

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The objective of this work was to derive empirical formulae for the fundamental natural period of reinforced concrete staggered wall system structures using modal analysis results. To this end, 32 model structures of varying height and span length were designed and their fundamental natural periods were computed by modal analysis. The proposed formulae for the natural period of staggered wall structures with and without a middle corridor were determined from regression analysis. To validate the proposed formulae, the seismic performance of selected model structures designed using the formulae was investigated. According to the regression analysis, the computed best-fit formulae for the model structures represent the natural periods precisely. The structures designed using the proposed formulae showed satisfactory performance against the design seismic load.

Notation

A_b	area of the base of a structure
A_c	total effective area of the shear walls in the first storey of the building
A_e	effective cross-sectional area of the ith shear wall in the first storey of the building
A_i	web area of the ith shear wall
C_t	regression coefficient
D_e	dimension of the ith shear wall in the first storey of the structure
D_i	length of the ith shear wall
F_R	residual strength of fibre element
F_y	yield strength of fibre element
h_i	height of the ith shear wall
h_n	height of the building
K_0	initial stiffness of concrete
N	number of storeys
N_w	total number of shear walls
S_DS	design spectral response acceleration parameter at short period
S_D1	design spectral response acceleration parameter at period of 1 s
T	fundamental period of the structure
β	regression coefficient

Introduction

Many residential buildings have been designed with a large number of shear walls located in both longitudinal and transverse directions which act as partition walls as well as lateral and gravity load resisting systems. These days, however, all shear wall buildings are not preferred, mainly because fixed plan layouts divided into many small spaces by vertical shear walls fail to meet the diverse needs of people in terms of spatial planning. In this regard, apartment buildings with vertical walls placed on alternate levels have the advantage of enhanced spatial flexibility, economics and constructability. This type of structural system has already been widely applied in steel residential buildings and is typically called a staggered truss system. Although they have not yet been realised in reinforced concrete (RC) buildings, the idea was suggested many years ago.

Fintel (1968) proposed an RC staggered wall–beam structure in which staggered walls with attached slabs resist gravity as well as lateral loads as H-shaped storey-high deep beams. He conducted experiments on a half-scale staggered wall structure subjected to gravity load and found that the system was competitive with conventional forms of construction and, in many cases, would be more economical. Mee *et al.* (1975) investigated the structural performance of staggered wall systems subjected to dynamic load by carrying out shaking table tests of 1/15 scaled models.

More recently, Kim and Jun (2011) evaluated the seismic performance of partially staggered wall apartment buildings using non-linear static and dynamic analysis. They found that a structure with partially staggered walls satisfied the collapse prevention performance objective required by Federal Emergency Management Agency code FEMA 356 (Fema, 2000) and thus was considered to have sufficient capacity for design-level seismic load. Lee and Kim (2013) investigated the seismic performance of six- and twelve-storey staggered wall structures with a middle corridor based on the FEMA P695 (Fema, 2009) procedure and found that the model structures had enough safety margin for collapse against design-level earthquakes.

For seismic design of structures using current design codes, the formula for the fundamental natural period is essential. Goel and Chopra (1998) evaluated the formulae specified in design codes using available data on the fundamental period of buildings measured from the ground motions recorded during eight Californian earthquakes. They proposed an improved formula by calibrating a theoretical formula against the measured period data through regression analysis. Lee *et al.* (2000) measured the fundamental natural periods of 50 RC shear wall apartment buildings and compared them with those computed by formulae from various design codes. They found that no code formula examined in their study was sufficient to estimate the fundamental period of the shear wall structures and proposed an improved formula by regression analysis on the basis of ambient vibration data. Zalka (2001) applied a simplified equivalent column concept for calculation of the natural frequencies of multi-storey buildings. Balkaya and Kalkan (2003) analysed 80 RC shear wall structures constructed using tunnel forms and compared their natural periods with those obtained by code formulae. They proposed an empirical formula that corresponded well with the analysis results. Amanat and Hoque (2006) investigated the fundamental periods of vibration of a series of regular RC framed buildings using modal analysis, including the effects of infill. It was found that when the effect of infill was included in the models, the time periods determined from eigenvalue analysis were close to those predicted by the code formulae. Crowley and Pinho (2009) compared formulae for natural frequency specified in various design codes and suggested a recommendation for the future development of the formulae in Eurocode 8 (CEN, 2004). Kwon and Kim (2010) evaluated building period formulae in seismic design codes for over 800 apparent building periods from 191 building stations and 67 earthquake events. The evaluation was carried out for steel and RC moment resisting frames (MRFs), shear wall buildings, braced frames and other structural types. A qualitative comparison of the measured periods and periods calculated using code formulae showed that the code formula for RC MRFs described the lower bound of measured periods well but the formula for shear wall buildings overestimated periods for all building heights.

The staggered wall system is currently not categorised as a seismic load resisting system in design codes. Therefore, to evaluate the design seismic load for such a system it is necessary to use the guidelines recommended for other structural systems such as shear wall systems, MRF systems, etc. As the staggered wall system has not yet been applied in a real structure, no field measurements for natural periods are available for use in developing formula to be used for seismic design.

Gate and Foth (1978) reported that the period of a building during an earthquake approaches the natural period computed from a theoretical model of the pure structural system, neglecting all non-structural elements. Based on this observation, the aim of this study was to derive empirical formulae for fundamental natural periods of staggered wall system structures using modal analysis results of model structures. To this end, 32 model structures of varying height and span length were designed and their fundamental natural periods were theoretically computed. The proposed natural period formulae were determined from separate regression analyses for staggered wall structures with and without a middle corridor. The seismic performances of selected model structures designed using the proposed formulae were investigated to validate the applicability of the formulae.

Staggered wall system model structures

Design and analytical modelling of model structures

Figure 1 shows a perspective view of a staggered wall structure with a middle corridor. In RC staggered wall systems, the storey-high RC walls that span the whole width of the building are located along the short direction in a staggered pattern. The horizontal shear force from the staggered walls above flows to the columns and staggered walls below through the floor diaphragm. This study investigated the fundamental natural frequencies of staggered wall buildings

- without a middle corridor (type A)
- with a middle corridor (type B).

For each type, the length of the staggered walls located along the transverse direction and the building height were selected as design parameters: the staggered wall lengths were 6, 8, 10 and 12 m and the number of storeys was 8, 12, 16 and 20. In all models, the number of bays in the longitudinal direction was six, with a span length of 6 m, and the storey height was 3 m. In total, 32 buildings (16 for each structure type) were thus prepared for the computation of natural periods. Side views of the 8-storey type A and type B model structures are shown in Figure 2(a) and 2(b) respectively. Along the longitudinal direction, the column–beam combination resists lateral load as an MRF, and this study is focused on the dynamic characteristics along the transverse direction.

The model structures were designed using a dead load of 7·6 kN/m^2 (2·6 kN/m^2 for the panel heating system and 5 kN/m^2 for the slab weight) and a live load of 2 kN/m^2. As the response modification factor for a staggered wall system is not specified in current design codes, a factor of 3·0 was used in the structural

Figure 1. Configuration of RC staggered wall structure with middle corridor

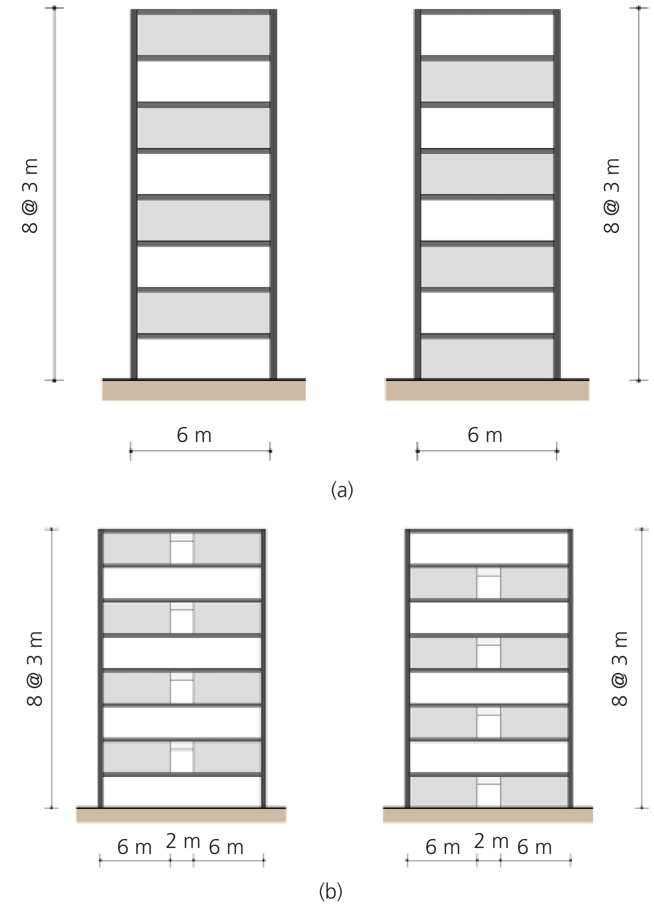


Figure 2. Side view of eight-storey model structures (wall length = 6 m): (a) type A (without a middle corridor); (b) type B (with a middle corridor)

design of the model structures; this value is generally used for structures not specified as a seismic load resisting system. Along the longitudinal direction, the structures were designed as RC ordinary MRFs with a response modification factor of 3·0. Design spectral response acceleration parameters at short period (S_{DS}) and at a period of 1 s (S_{D1}) of 0·37 and 0·15 respectively were used to form the design spectrum. The soil type was site class S_B, which corresponds to a normal rock site.

The compressive strength of the concrete was 27 MPa up to a quarter of the building height and 24 MPa above that level. The yield strength of the rebars was 400 MPa. In the model structures without a middle corridor (type A), the thickness of the staggered walls was 20 cm. In buildings with a middle corridor (type B), the thickness of the staggered wall was 20, 20, 24 and 26 cm in structures with 6, 8, 10 and 12 m long staggered walls respectively. The thickness of the floor slab was 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 floor. The size of the first-storey columns in the type A structures varied from 46 × 46 cm in the eight-storey structure to 76 × 76 cm in the 20-storey structure. In the type B structures with a middle corridor, the first-storey column size varied from 58 × 58 cm in the eight-storey structure to 94 × 94 cm in the 20-storey structure.

Structural design and modal analysis of the 32 model structures were carried out using the program code Midas Gen (Midas, 2013), which is a general-purpose finite-element based structural analysis and optimal design program having a graphic user interface. The fundamental periods of the model structures obtained from eigenvalue analyses are listed in Table 1. In type A structures, the fundamental period ranges from 0·196 s in the eight-storey structure with 12 m long staggered walls to 1·388 s in the 20-storey structure with 6 m staggered walls; in type B,

Number of storeys	Short side dimension: m	Fundamental period: s
Type A, without middle corridor		
8	6	0·351
8	8	0·271
8	10	0·225
8	12	0·196
12	6	0·651
12	8	0·502
12	10	0·413
12	12	0·354
16	6	1·001
16	8	0·777
16	10	0·638
16	12	0·548
20	6	1·388
20	8	1·084
20	10	0·899
20	12	0·767
Type B, with middle corridor		
8	14	0·348
8	18	0·301
8	22	0·267
8	26	0·246
12	14	0·565
12	18	0·439
12	22	0·386
12	26	0·351
16	14	0·798
16	18	0·626
16	22	0·548
16	26	0·470
20	14	0·970
20	18	0·825
20	22	0·720
20	26	0·618

Table 1. Dimensions and natural periods of model structures

structures, the fundamental period ranges from 0·246 s to 0·970 s. The table shows that the natural period increases as the number of storeys increases and the length of the staggered walls decreases.

Modal characteristics of staggered wall structures

To investigate the modal characteristics of the staggered wall structures, 16 MRFs and shear wall structures with the same dimensions as the staggered wall structures were designed using the same loading conditions and their natural frequencies and mode shapes were compared with those of the staggered wall structures. The plans of the three different structure systems are shown in Figure 3 and Figure 4 for type A and type B structures respectively. Figure 5 compares the fundamental natural periods of the three different structure systems along the transverse direction. It can be observed that the fundamental periods of the staggered wall structures are similar to those of the shear wall structures regardless of the existence of the middle corridor. In the 16- and 20-storey structures, the periods of the staggered wall structures are even slightly smaller than those of the shear wall structures. This is due to the contribution of the increased column size in those structures. The natural periods of the MRFs are significantly higher than those of the other systems. Even though not shown in Figure 5, the second- and third-mode natural periods of the three different structural systems showed similar trends to the fundamental periods. Figures 6 and 7 show the fundamental mode shapes of the eight-storey type A and type B

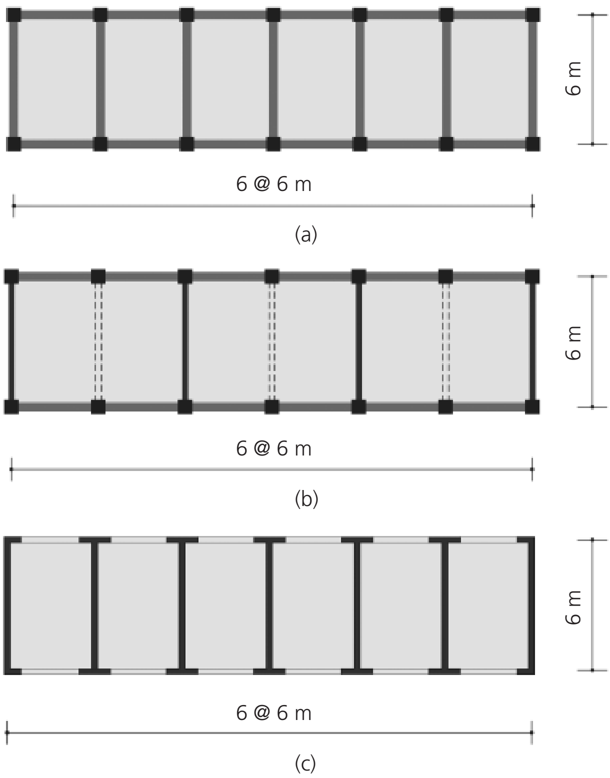


Figure 3. Plans of structures without middle corridor (type A): (a) MRF; (b) staggered wall system; (c) shear wall system

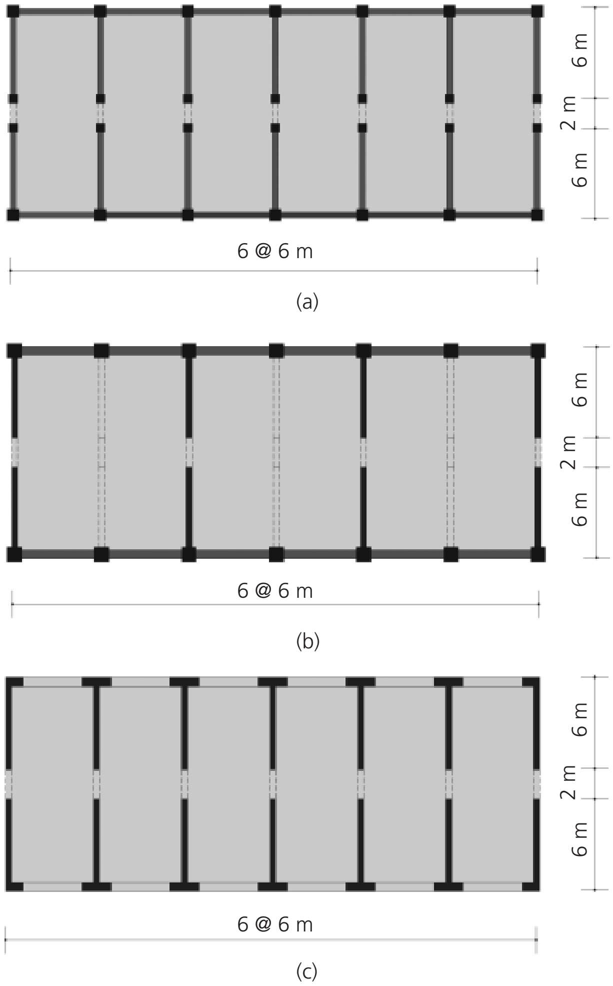


Figure 4. Plans of structures with middle corridor (type B): (a) MRF; (b) staggered wall system; (c) coupled shear wall system

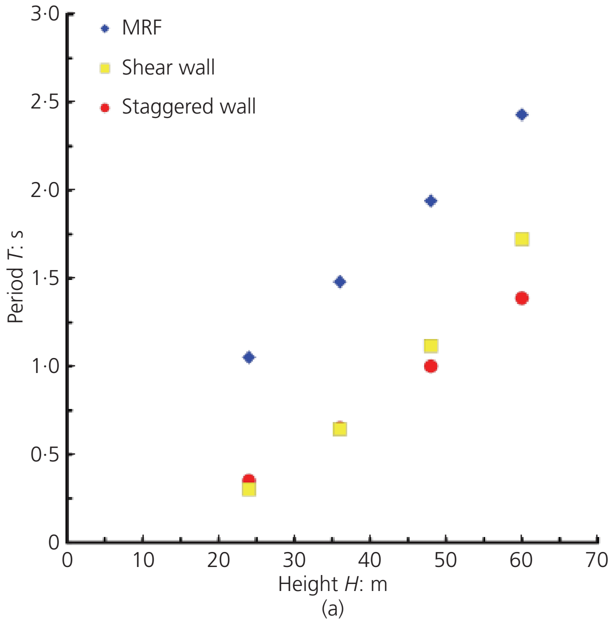
structures respectively, along the transverse direction. It can be observed that the mode shape of the type A staggered wall structure resembles the cantilever deformation mode of the shear wall structure, whereas the mode shape of the staggered wall structure with a middle corridor (type B) is somewhat similar to that of the MRF.

Comparison of fundamental periods

The computed fundamental natural periods of the staggered wall model structures obtained from modal analysis are now compared with those estimated using the empirical formulae of three building design codes

- Eurocode 8 (CEN, 2004)
- ASCE 7-10 (ASCE, 2010)
- Korea Building Code (KBC2009) (MCT, 2009).

In these design codes, the following form is generally used as an empirical formula for the fundamental period T of building structures



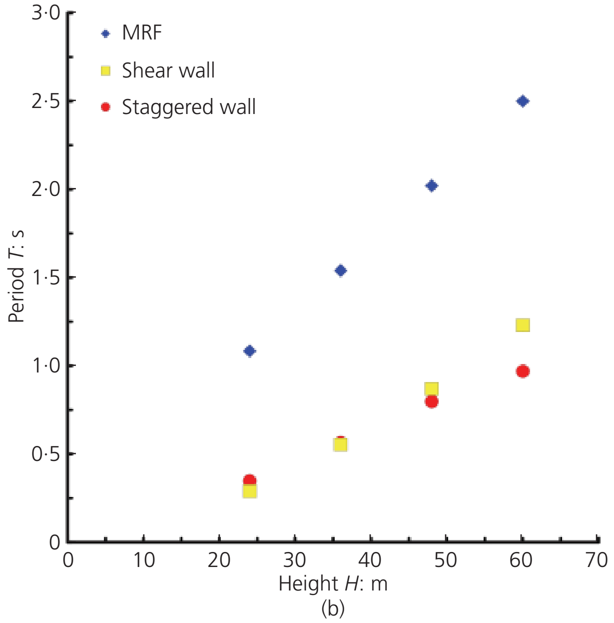


Figure 5. Comparison of the fundamental periods of the MRF, shear wall and staggered wall structures: (a) type A structures; (b) type B structures

$$T = C_t h_n^{\beta} \tag{1}$$

where C_t and β are the regression coefficients and h_n is the height of the building. For RC MRFs, the numerical coefficient C_t is 0·073 in Eurocode 8 and KBC2009, and 0·0466 in ASCE 7-10. The exponent β is 0·75 in Eurocode 8 and KBC2009, and 0·9 in ASCE 7-10. All three codes recommend $C_t = 0{\cdot}049$ and $\beta = 0{\cdot}75$ to calculate the natural period of all other structural systems. In Eurocode 8, Equation 1 is applied only for buildings of height up to 40 m.

Alternatively, in ASCE 7-10 and KBC2009, the following simple formula for MRF buildings is also given

$$T = 0{\cdot}1N \tag{2}$$

where N is the number of storeys. This simple formula is permitted for structures not exceeding 12 storeys above the base and having an average storey height of 3 m.

For RC shear wall buildings, KBC2009 recommends the following value of C_t in Equation 1

$$C_t = \frac{0{\cdot}0743}{A_c^{0{\cdot}5}} \tag{3a}$$

in which

$$A_c = \sum_{i=1}^{N_w} A_e \left[0{\cdot}2 + \left(\frac{D_i}{h_n}\right)^2\right] \tag{3b}$$

where A_c is the total effective area of the shear walls in the first storey of the building (in m^2), A_e is the effective cross-sectional area of the ith shear wall in the first storey of the building (in m^2), D_i is the dimension in the direction under consideration of the ith shear wall in the first storey of the structure and N_w is the total number of shear walls. The value of D_i/h_n used in Equation 3 shall not exceed 0·9. The KBC2009 formula for the natural period shown above is taken from UBC 97 (UBC, 1997).

In Eurocode 8, a similar form of the equation is used for structures with RC shear walls except that the formula for A_c is slightly different and is given by

$$A_c = \sum_{i=1}^{N_w} A_e \left[0{\cdot}2 + \left(\frac{D_i}{h_n}\right)\right]^2 \tag{4}$$

In ASCE 7-10, the approximate fundamental period for RC wall structures is determined from

$$T = \frac{0{\cdot}0019}{A_e^{0{\cdot}5}} h_n \tag{5a}$$

in which

$$A_e = \frac{100}{A_b} \sum_{i=1}^{N_w} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{[1 + 0{\cdot}83(h_i/D_i)^2]} \tag{5b}$$

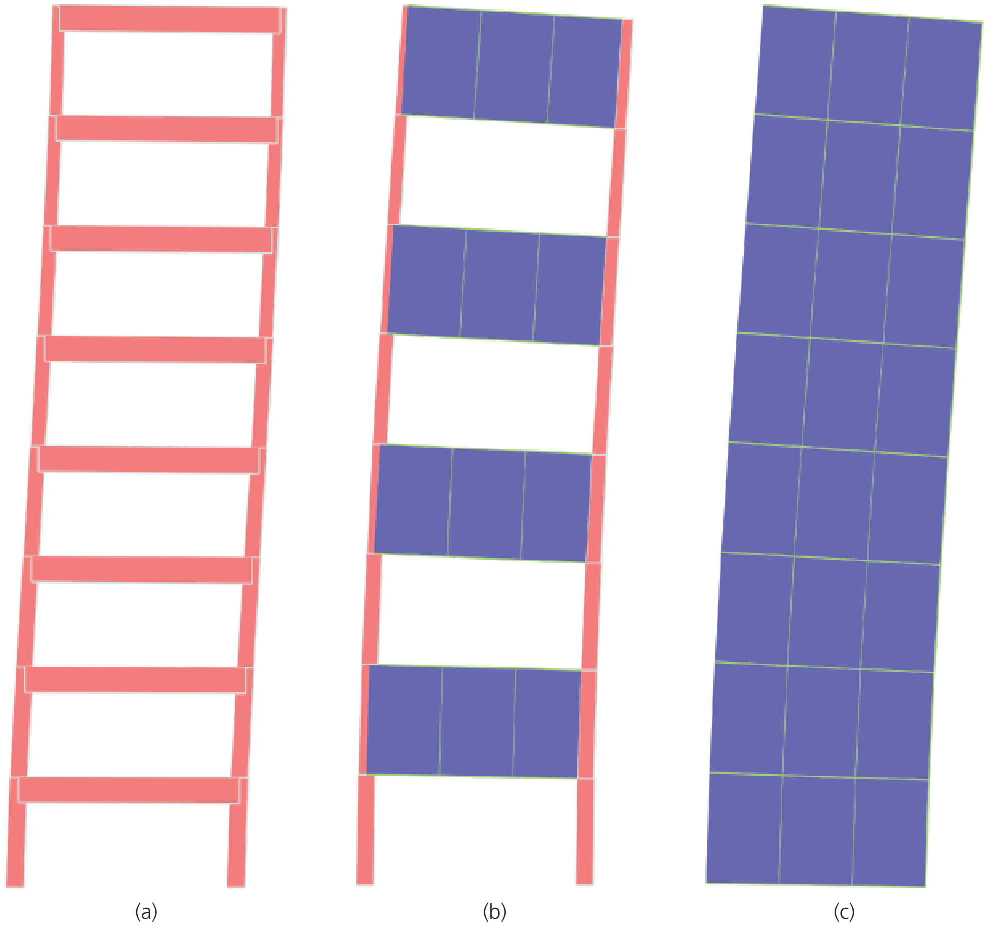


Figure 6. Mode shape of type A structures: (a) moment frame; (b) staggered wall; (c) shear wall

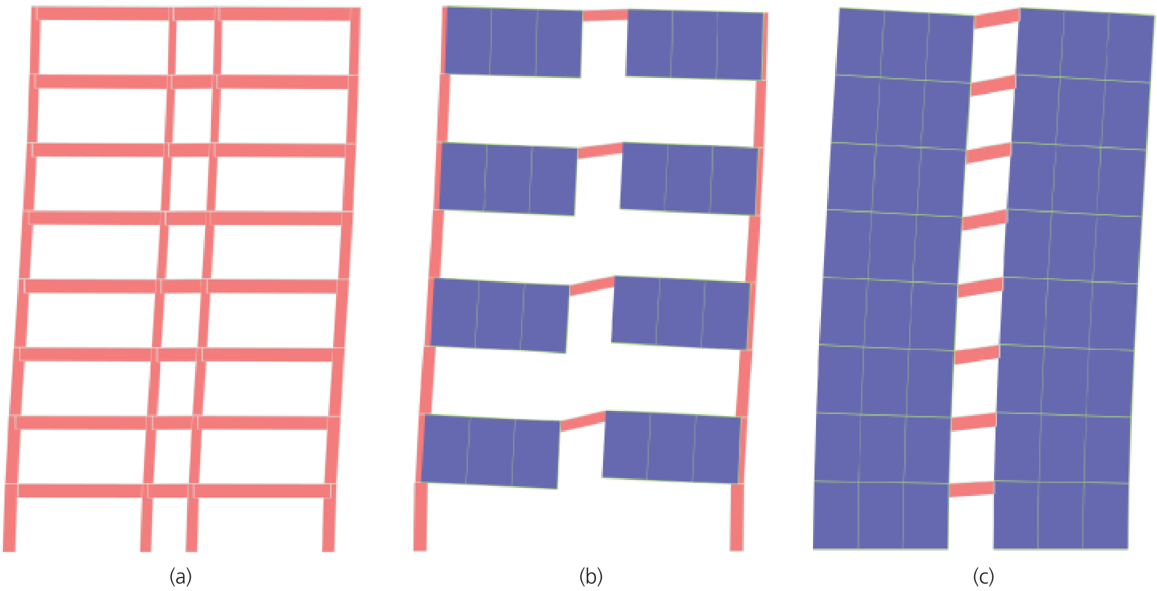


Figure 7. Mode shape of type B structures: (a) moment frame; (b) staggered wall; (c) shear wall

where h_n is the height of the buildings (in ft), A_b is the area of the base of structure (in ft^2), A_i is the web area of the ith shear wall (in ft^2), D_i is the length of the ith shear wall (in ft) and h_i is the height of the ith shear wall (in ft).

Figure 8 compares the natural periods of staggered wall model structures obtained from modal analysis and those obtained from code formulae for RC MRFs. The figure also shows the periods predicted by the formulae for all other structures. Figure 8(a) shows that the fundamental natural periods of the model structures predicted by the empirical formulae for MRFs in the design codes form an upper bound of the values obtained by modal analysis for both type A and type B structures. It can also be noted that the formulae applicable for all structures other than MRFs overestimate the natural periods of all the staggered wall structures with a middle corridor (type B) and most of the type A structures without a middle corridor. The simplified formula for MRFs, $T = 0{\cdot}1N$, significantly overestimates the natural periods of both types of staggered wall structures, as shown in Figure 8(b).

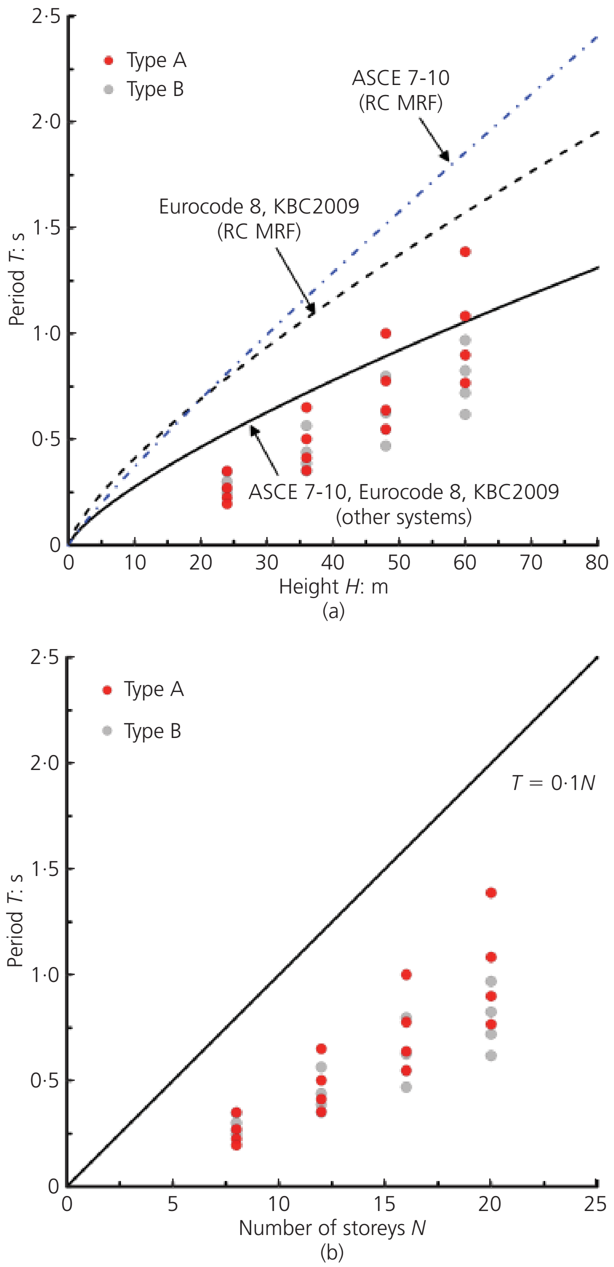


Figure 8. Comparison of natural periods of model structures obtained from modal analysis and code formulae for RC MRFs and other systems: (a) Equation 1 with different regression coefficients; (b) simplified formula specified in KBC2009 and ASCE 7-10

Figure 9 compares the natural periods obtained from modal analysis and those obtained from code formulae for RC shear wall structures. Figures 9(a) and 9(c) show that the data produced by the Eurocode 8 and KBC2009 formulae overestimate the periods of the type A and type B structures obtained by modal analyses. The results of ASCE 7-10, however, correspond well with the natural periods of both type A and type B structures (Figure 9(b)).

Regression analysis

Regression analyses were carried out to determine the regression coefficients of the empirical formula for the fundamental natural period of a staggered wall structure. The regression analysis for the natural period started from the following form based on the code formulae shown in Equations 3–5

$$T = C_t \frac{H^{\beta}}{A_e^{0{\cdot}5}} \tag{6}$$

in which H is the building height, C_t and β are the regression coefficients to be determined and A_e is the effective area of walls along the direction under consideration. Using the above form of the period formula, the following three different regression cases were used to find the best fit for each type of staggered wall structure.

(*a*) Case I: Best-fit regression coefficients were determined using the 32 natural periods of both type A and type B staggered wall structures. The effective area used in ASCE 7-10 and shown in Equation 5b was used in Equation 6 based on the observation in the preceding section that the ASCE 7-10 formula for shear wall structures produced the most conservative natural periods for both types of staggered wall structures.
(*b*) Case II: Regression analyses were conducted separately for type A and type B structures using each set of 16 data points. The effective area of Equation 5b was also used in this case.
(*c*) Case III: Regression analyses were carried out separately for type A and type B structures using the total shear wall area given by

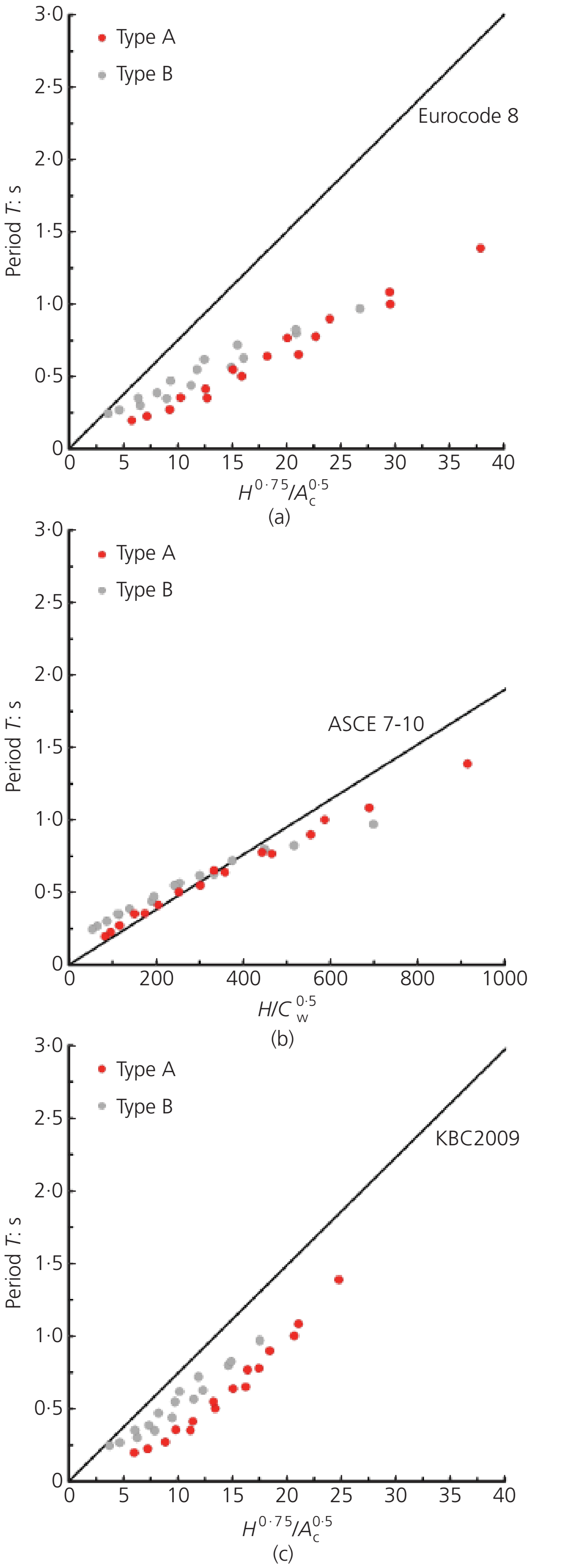


Figure 9. Comparison of natural periods of model structures obtained from modal analysis and code formulae for RC shear wall structures: (a) Eurocode 8 (Equations 1 and 4); (b) ASCE 7-10 (Equation 5); (c) KBC2009 (Equations 1 and 3)

$$A = \sum_{i=1}^{N_w} A_i \tag{7}$$

According to Goel and Chopra (1998), the use of the total shear wall area shown in Equation 7 better represents the shear deformation mode of vibration than the effective shear wall area A_e of Equation 5b, which was derived considering both shear and bending modes of deformation.

For the purpose of regression analysis, Equation 6 can be rewritten as

$$\log T = \log C_t + \beta \log H - 0.5 \log A_e \tag{8}$$

where the constants $\log C_t$ and $\log \beta$ are determined by minimising the squared error between the data obtained from modal analysis and those estimated by the formula in the least-squared sense. As mentioned previously, both combined (case I) and separate (cases II and III) regression analyses were carried out for each of the type A and type B staggered wall systems. Both the best fit and the best fit $\pm 1\sigma$ formulae were obtained to provide conservative natural formula for code application.

The formulae obtained from the unconstrained regression analyses using all the 32 natural periods of type A and B structures (case I) are depicted in Figure 10. The best-fit regression coefficients C_t and β turned out to be 0·04 and 0·37 respectively, and the associated standard error was 0·062. Figure 11 shows the

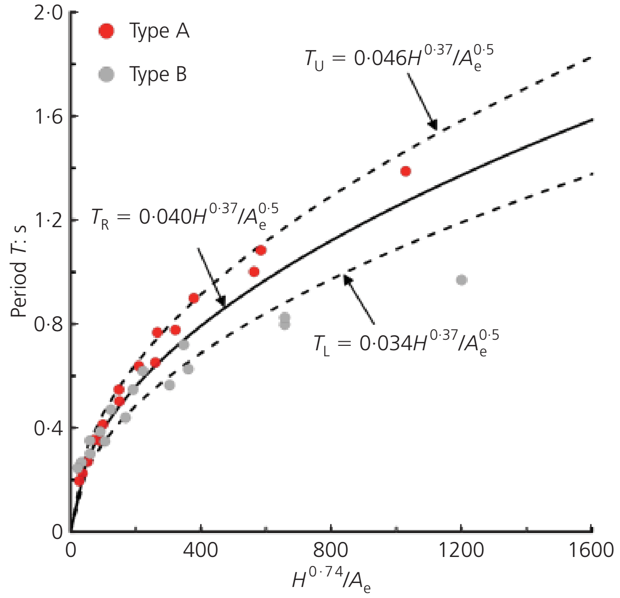


Figure 10. Best-fit formula for combined fundamental natural periods of type A and type B staggered wall structures (case I)

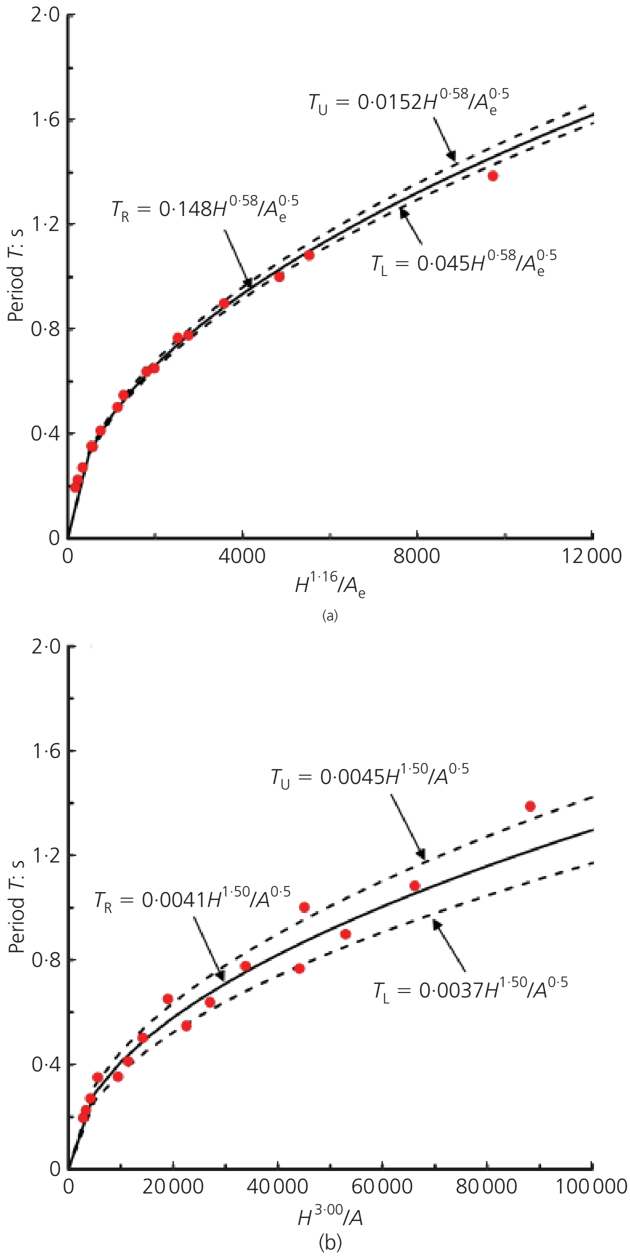


Figure 11. Best-fit formula for fundamental natural period of type A structures obtained using different shear wall areas: (a) case (II); (b) case (III)

regression analysis results obtained exclusively for natural periods of type A structures using Equation 5b (case II) and Equation 7 (case III) as the effective shear wall area. As shown in Figure 11(a), the best-fit regression coefficients C_t and β obtained using Equation 5b for A_e were determined as 0·0148 and 0·58 respectively, with a standard error of 0·010, which turned out to be the smallest error of all the cases analysed. Unconstrained analysis using the total shear wall area of Equation 7 (case III) resulted in regression coefficients with significantly larger standard errors of 0·044. This illustrates that use of the effective shear wall area presented in Equation 5b is preferable to the use of the total area of Equation 7 to formulate a regression formula for type A structures without a middle corridor.

The best-fit regression analysis results for type B structures (with a middle corridor) using the two different expressions for shear wall area show a different trend. Figure 12(b) shows that for type B structures, the case III unconstrained regression analysis using the total shear wall area of Equation 7 results in a best-fit regression coefficient $\beta = 1{\cdot}08$ with a smaller standard error of

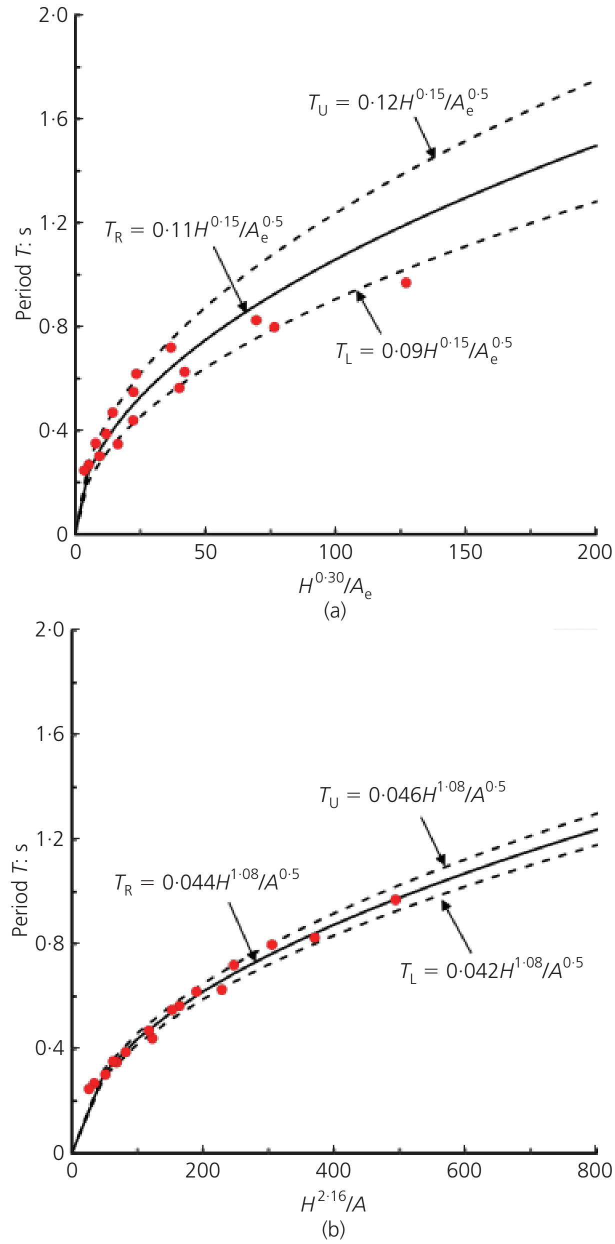


Figure 12. Best-fit formula for fundamental natural period of type B structures obtained using different shear wall areas: (a) case II; (b) case III

0·021. As Equation 7 is known to be appropriate for structures with shear mode vibration, the use of the total shear wall area is recommended to be used for estimation of the natural period of type B structures with a middle corridor, the mode shape of which resembles that of shear frames as can be seen in Figure 7.

Figure 13 plots the standard errors associated with the best-fit formulae obtained from constrained regression analysis using various values of β. The standard errors corresponding to the best-fit regression coefficients obtained from unconstrained regression analyses are also indicated in the figure. The figure shows that the standard errors are minima when the regression coefficients are obtained from unconstrained regression analysis, and increase as the regression coefficients deviate from optimum values. It also can be observed that, in the type A structures, the unconstrained regression analysis using the effective area of Equation 5b results in a formula with a very small standard error, and that the unconstrained regression analysis using the total shear wall area of Equation 7 provides the best-fit formula. The standard error associated with the formula obtained from the case I regression analysis using the combined data of type A and type B structures turned out to be reasonably small.

Seismic performance of model structures designed with the proposed formulae

The seismic performance of eight-storey type A and type B model structures with 6 m span designed using the proposed formulae for natural periods was investigated to confirm the validity of the proposed formulae. Type A and type B structures were designed using the formulae obtained by case II and case III regression analyses respectively, which produced the best-fit formulae. In the case of the type A structure, the natural period

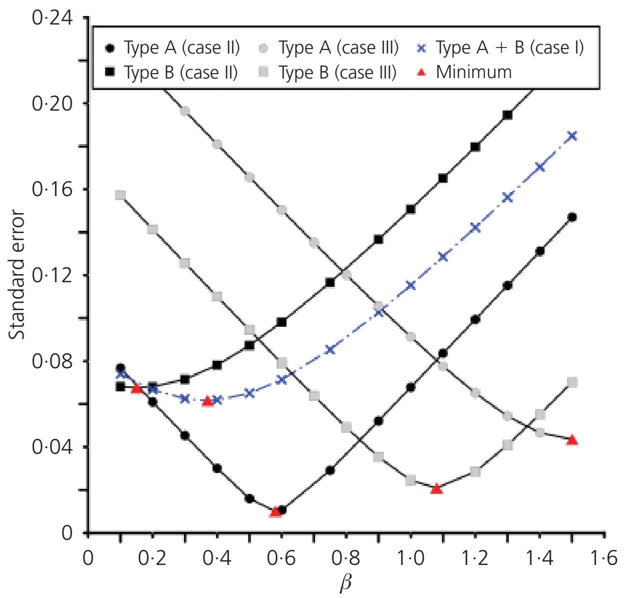


Figure 13. Standard errors corresponding to various regression coefficients β

obtained by the proposed formula is 0·35 s, while the natural period estimated by the ASCE 7-10 formula for 'other structures' is 0·531 s. The design base shears corresponding to the proposed and the code natural periods are 2000 kN and 2677 kN respectively. Therefore, use of the proposed formula resulted in a 34% increase in the design base shear. The natural periods of the type B structure obtained by the proposed and the code formulae are similar to those of the type A structure, and the corresponding design base shears are 3013 kN and 5964 kN respectively.

Non-linear analyses of the model structures were carried out using the program code Perform-3D (Perform-3D, 2006). The force–displacement relationship of the columns was modelled using the 'FEMA column, concrete type' elements provided in Perform-3D, which is illustrated in Figure 14. The staggered walls were modelled by the 'general wall' fibre elements. The stress–strain relationship of concrete fibre elements was defined as trilinear lines, as shown in Figure 15(a). The tensile strength of concrete is neglected. The yield strength (F_Y) and residual strength (F_R) were defined as 60% and 20% of the ultimate strength respectively. The strain at ultimate strength is 0·002 and the ultimate strain was defined as 0·004. The fibre elements for reinforcing steel were modelled with trilinear lines, as depicted in Figure 15(b). The initial stiffness of concrete K_0 is 20 000 kN/cm^2. The expected ultimate strengths of the concrete and steel were taken to be 1·25 times the nominal strengths. The model for coupling beams located between the two staggered walls is composed of two end-rotation-type moment hinges defined based on ASCE/SEI 41-06 (ASCE, 2005). As the general wall element has no in-plane rotational stiffness at its nodes, a beam element was embedded in the wall (Figure 16) to specify a moment-resisting connection between beam and wall.

Figure 17 shows the pushover curves of the model structures obtained by gradually increasing the lateral load distributed vertically proportional to the fundamental mode shape of the structures. The pushover curves of the structures designed using

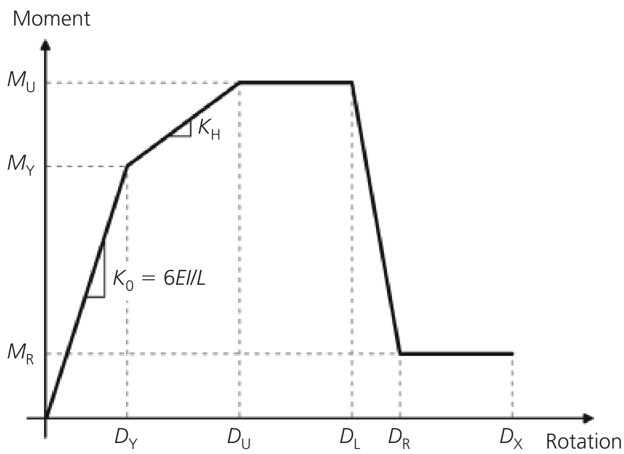


Figure 14. Force–deformation relationship of columns

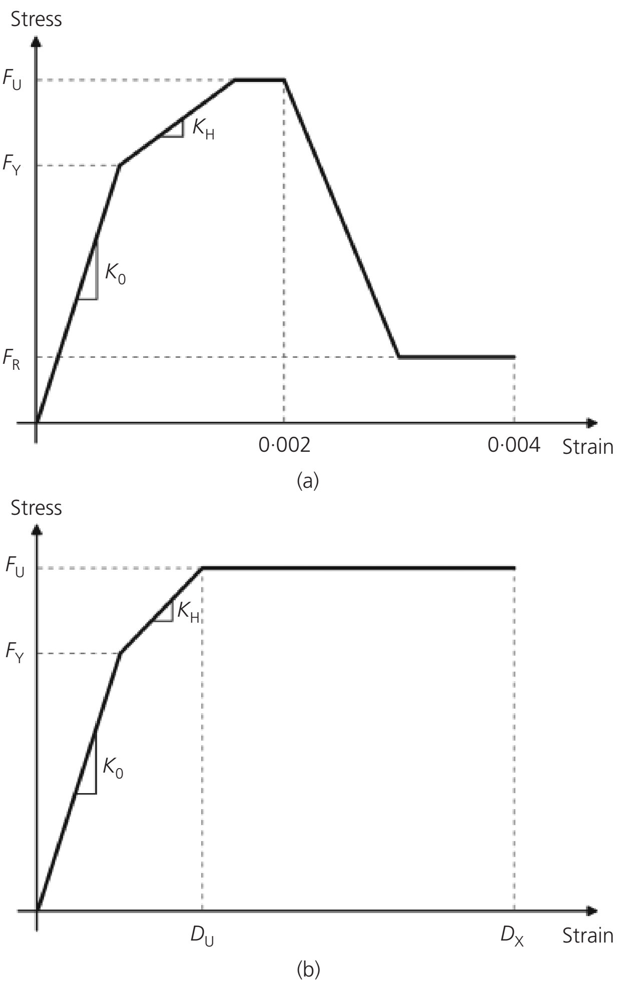


Figure 15. Stress–strain relationship of general wall fibre elements: (a) concrete; (b) steel

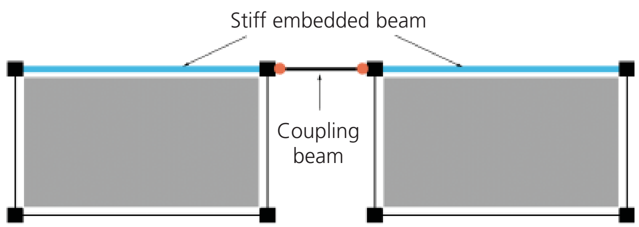


Figure 16. Installation of embedded beams at top of staggered walls

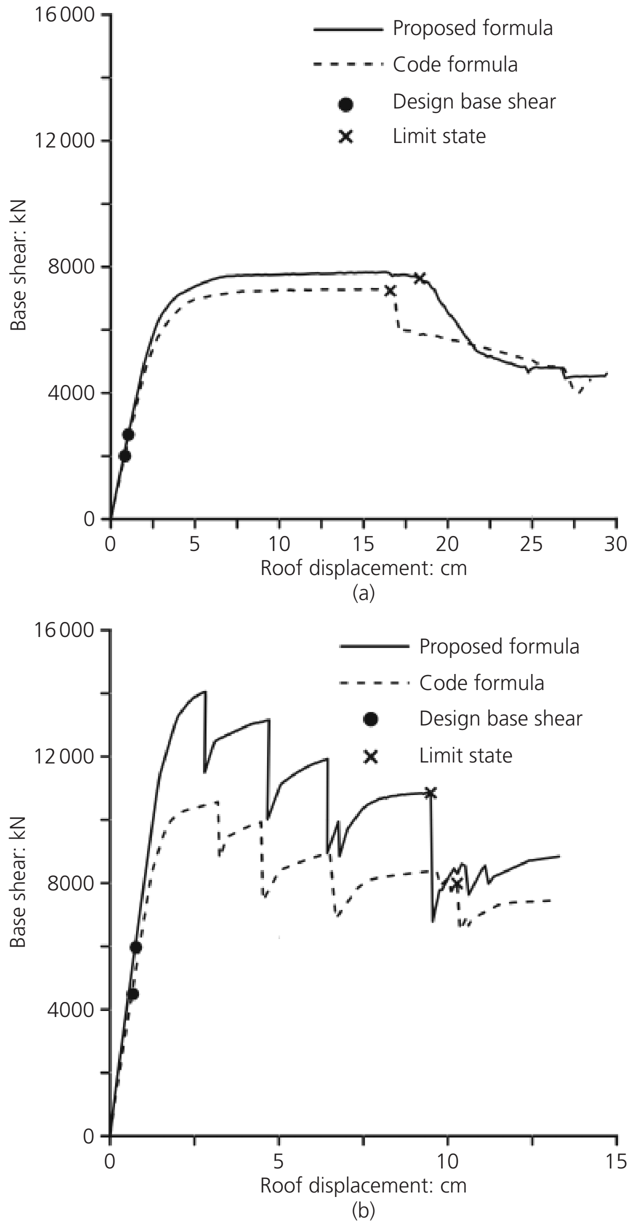


Figure 17. Pushover curves of the eight-storey staggered wall analysis model structures: (a) type A; (b) type B

the proposed formula and the code formula suggested for 'other structures' are depicted in the figures. It can be observed that the structures designed using the proposed formula, which resulted in smaller fundamental periods, showed slightly larger stiffness and strength than the structures designed with the code formula. The maximum strengths of the structures are much higher than the design base shears, and the strengths of the structures with a middle corridor (Figure 17(b)) are higher than those of the type A structures (Figure 17(a)). It was observed that failure of the type A structures was initiated by the formation of plastic hinges at the lower storey columns, whereas the sudden strength drop in the type B structures occurred due to the formation of plastic hinges at the connection beams.

Figure 18 shows the seismic behaviour factors of the model structures designed using the proposed natural period formula. The response modification factors (Figure 18(c)), computed by multiplying the overstrength and ductility factors, were 9·3 and 6·9 for type A and type B structures respectively. These values

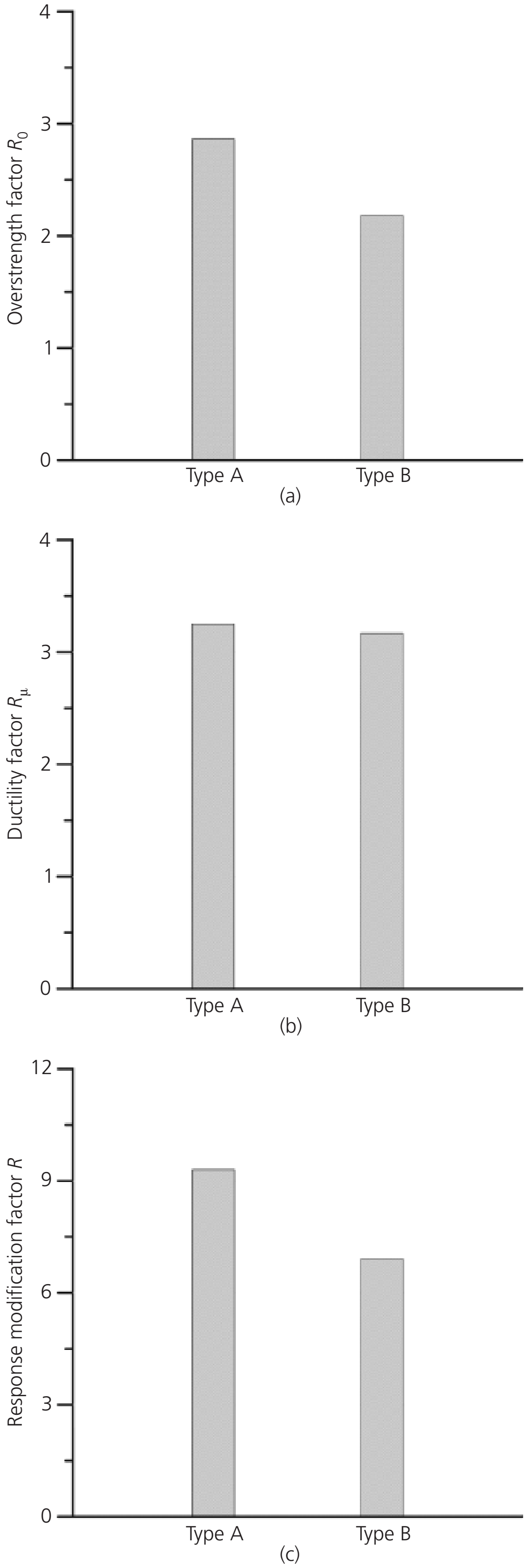


Figure 18. Seismic behaviour factors of the eight-storey model structures

are significantly higher than the value of 3·0 generally used for structures not defined as a seismic load resisting system.

The maximum inter-storey drifts of the model structures were computed by non-linear dynamic analyses using seven earthquake ground motions selected from records provided by the Pacific Earthquake Engineering Research Center. Details of these records are presented in Table 2, and Figure 19 shows the response spectra of the seven earthquake records scaled to the design spectrum for earthquakes with a return period of 2400 years. Figure 20 shows the mean maximum inter-storey drifts of the model structures obtained from the seven non-linear dynamic analyses. It can be observed that the maximum inter-storey drifts are well below the limit state of 1·5% of the storey height.

Conclusions

Empirical formulae were derived for the fundamental period of staggered wall system structures using modal analysis results of

Record number	Name	Peak ground acceleration: g	Peak ground velocity: cm/s
953	Northridge	0·52	63
960	Northridge	0·48	45
1602	Duzce, Turkey	0·82	62
1787	Hector Mine	0·34	42
169	Imperial Valley	0·35	33
174	Imperial Valley	0·38	42
111	Kobe, Japan	0·51	37

Table 2. Earthquake records used in dynamic analysis

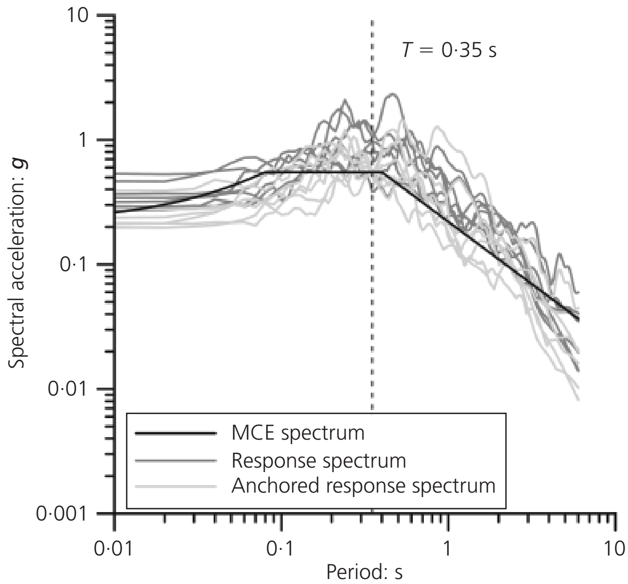


Figure 19. Seven earthquake records scaled to the design spectrum for earthquakes with return period of 2400 years (MCE, maximum considered earthquake)

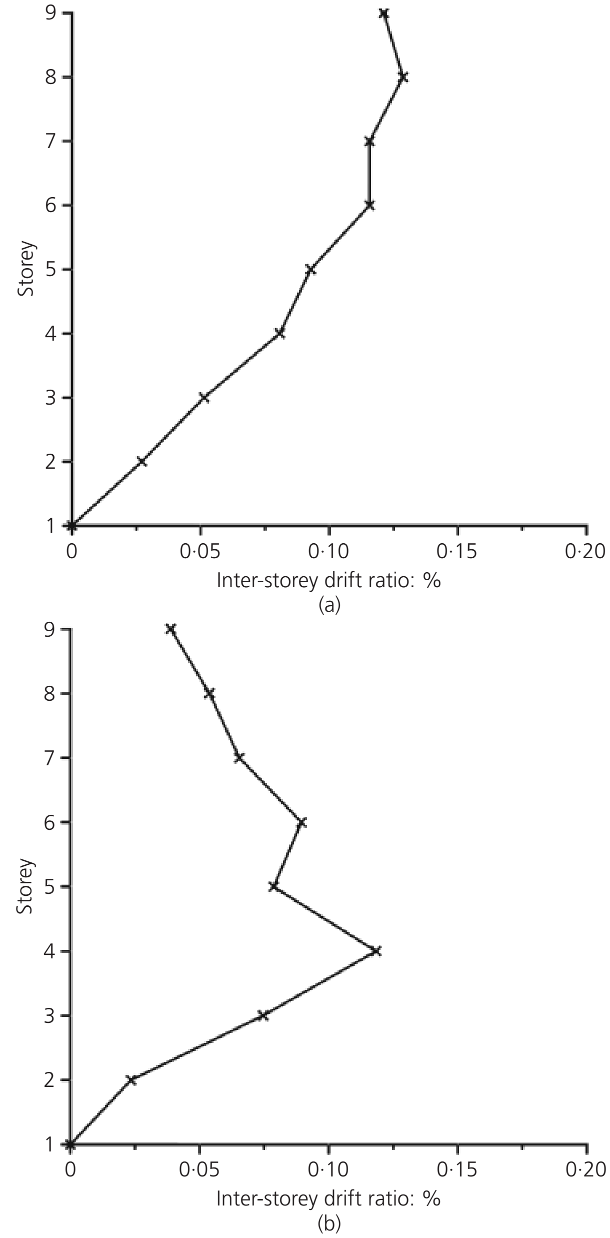


Figure 20. Mean inter-storey drifts of the model structures subjected to seven ground motions with a return period of 2400 years: (a) type A; (b) type B

32 model structures. The proposed natural period formulae were determined from separate regression analyses for staggered wall structures with and without a middle corridor. The seismic performance of selected model structures designed using the proposed formula was investigated to verify the validity of the proposed formulae. The findings are as follows.

(*a*) The fundamental periods of staggered wall structures are similar to those of shear wall structures with similar dimensions. The mode shape of a staggered wall structure without a middle corridor resembles the cantilever deformation mode of a shear wall structure, whereas the mode shape of a staggered wall structure with a middle corridor is similar to the shear deformation mode of a moment resisting frame (MRF).
(*b*) The fundamental natural periods of the staggered wall structures predicted by the code formulae for MRFs form an upper bound of the values obtained by modal analysis. The natural periods predicted by Eurocode 8 match reasonably well those of the staggered wall structures with a middle corridor obtained from modal analysis, but overestimate the natural periods of structures without a middle corridor. The results of ASCE 7-10 correspond well with the natural periods of structures without a middle corridor but fall far below those of structures with a middle corridor. The natural periods estimated by KBC2009 fall between those of structures with and without a middle corridor.
(*c*) Best-fit regression analyses for the natural periods obtained by modal analyses resulted in regression coefficients of $C_t = 0{\cdot}015$ and $\beta = 0{\cdot}58$ for structures without a middle corridor and $C_t = 0{\cdot}004$ and $\beta = 1{\cdot}50$ for structures with a middle corridor. The significant difference between the coefficients and exponents of the best-fit formulae for the two types of staggered wall system is mainly due to the difference in the shear wall area used in the formulation. The best fit $- 1\sigma$ may be used for code formulae.
(*d*) The use of the effective shear wall area is preferable for estimation of the fundamental natural period of structures without a middle corridor, but the total shear wall area is recommended for structures with a middle corridor.
(*e*) The eight-storey model structures designed using the proposed formulae for natural period showed sufficient capacity for resisting design-level seismic load.

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