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Estimation of fundamental natural period of vibration for reinforced concrete shear walls systems

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Abstract. This study attempts to develop new simplified approximate formulas to predict the fundamental natural periods of vibration (*T*) for bearing wall systems engaged with special reinforced concrete shear walls (RCSW) under seismic loads. Commonly, seismic codes suggested empirical formulas established by regression analysis of measured *T* for buildings during earthquake motions. These formulas depend on structure type, building height, number, height and length of SW, and ratio of SW area to base area of structure. In this study, a parametric investigation is performed for *T* of 110 selected models of bearing RCSW systems with varying structural height, configuration of horizontal plans including building width, number and width of bays, presence of middle corridors and core SWs. For this purpose, a 3D non-linear response time history (TH) analysis is implemented using ETABS v16.2.1. New formulas to estimate *T* are anticipated and compared with those obtained from formulas of IBC 2012 and ASCE/SEI 7-10. Moreover, the study examines responses of an arbitrarily two selected test model of 60 m and 80 m in height with presence of SWs having middle corridors. It is observed that the performance of the tested buildings is different through arising of considerable errors when using codes' formulas for estimating *T*. Accordingly, using the present proposed formulas exhibits more reasonable and safer design compared to codes' formulas. The results showed that equitable enhancement is promising to improve *T* formulas approaching enhanced and accurate estimation of *T* with reliable analysis, design, and evaluation of bearing RCSW systems.

Keywords: seismic codes; fundamental period; bearing walls, RC shear walls, non-linear time history

1. Introduction

The fundamental natural period of vibration (*T*) is an important dynamic property of structures since it governs the behavior and responses of the structure during seismic actions. For instance, *T* has significant influence on the seismic base shear and lateral seismic forces acting on structures, especially in bearing RCSW systems where lateral forces are resisted only by SWs. Fundamental natural period of a structure is controlled by the mass, stiffness and strength of the structure. Thus, it is affected by many parameters, which include the height of the building, characteristics of shear walls, number of stories, number of bays, dimensions of member sections, regularity of structure, and others (Asteris *et al.* 2015).

However, most offered empirical formulas found in research to estimate *T* are derived from testing data for existing structures, which have experienced strong earthquakes but not deformed into the inelastic range (Ricci *et al*., 2011). Therefore, determining realistic and precise *T* for RCSW systems is an essential step towards reliable seismic analysis, design and evaluation (Asteris *et al.* 2016). Hence, it is desirable to properly associate the available experimental fundamental periods of vibration evaluated for existing structures during strong motions with those numerically computed values. Still, this is hard to accomplish since very limited number of existing buildings are in fact equipped with measurement instruments, in addition to the infrequent incidences of strong motion of buildings due to earthquakes (Goel and Chopra 1998). Moreover, the collection of accumulated measured data of *T* for existing structures essentially requires years exceeding the age of the structure. Herein, it is worth mentioning the wide variation in measured vibration amplitudes, frequency, and periods due to scattered database for different structures with various construction materials and wide-ranging types of tested structural systems (Michel *et al.* 2010).

In that regard, most of seismic design codes and provisions (e.g., IBC 2012 and ASCE/SEI 7-10) specify approximate empirical formulas to estimate *T* for several types of structural systems with different materials, which explicitly include bearing RCSW systems. Accordingly, the estimated *T* using these approximate empirical expressions, as they are available in seismic codes, may lead to more conservative periods than the real values during earthquake excitation. This may result in imprecise values for the necessary seismic design parameters and subsequently affect the anticipated responses of structure including the design base shear, story displacement, lateral drift, deflections of members, and others. Hence, an accurate estimation of *T* using trustworthy formulas for use in seismic design would permit an enhanced judgment of the global elastic seismic demands for bearing RCSW systems

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and, ultimately, the required inelastic performance in static procedures.

Many researchers provided various empirical formulas by using different methods and taking into account geometrical characteristics of the structure, but with restriction of narrow configuration data of buildings. For example, Goel and Chopra (l997a) evaluated the empirical formulas specified in U.S. codes using measured data of fundamental periods for building motions recorded during specified eight California earthquakes. Their study showed that code formulas for estimating *T* of RCSW buildings were grossly inadequate. Subsequently, they proposed improved expression for *T* by calibrating a theoretical formula, derived using Dunkerley's method in conjunction with measured period data by using regression analysis. In addition, they recommended a factor to limit the period calculated by a "rational" analysis method, such as Rayleigh's method.

Generally, research stated three significant procedures of estimating the dynamic properties of structural systems including data measurement of actual systems, analytical methods, and empirical formulas. Based on the first procedure, the current available formulas in seismic codes for the fundamental period, presented as simple empirical expressions, relate the periods of buildings with their type, material, and geometry.

Chun *et al*. (2000a, 2000b) assessed the approximate estimation of *T* for apartment buildings with SW dominant systems according to the Korean Building Code of 1988. Their study aimed to evaluate the reliability of code formula based on full-scale ambient vibration measurements of 50 RC apartment buildings with wall-slab configurations. They observed that the stiffness of the tested buildings is very different from code estimation and quite large errors occurred when using code formula for estimation of *T*. Consequently, they proposed an improved empirical formula to estimate *T* for such structural systems. However, their proposal pointed only to the improvement of the serviceability condition without considering the influence of *T* on the other seismic design key parameters, such as the base shear and floors seismic lateral forces.

Moreover, Balkaya and Kalkan (2003) investigated the consistency of equations for fundamental periods of RCSW structures and their related dynamic properties as found in UBC 97, and the Turkish Seismic Code 1998. Using three-dimensional finite-element modelling, they analyzed 80 different building configurations for SW dominant multi-storey RC structures, constructed by using a special tunnel form technique. They concluded that codes empirical equations for the prediction of *T* for that type of structures yielded inaccurate results, and accordingly, codes presented limited information for their design criteria. Their results demonstrated that different proposed formulas accurately improved predictions for a broad range of different configurations, and led them to recommend such formulas as efficient tool for the implicit design of such structures.

Other studies by Lee *et al*. (2000) and Gilles *et al*. (2012) showed that code formulas are grossly inadequate when comparing with conducted full-scale on-site vibration tests. Such studies showed inappropriate behavior of structures under dynamic loads where mathematical models of dynamic structural systems based on measured data have a significant potential for ambient vibration.

Draganić *et al*. (2010) performed an analytical research on 600 different models of RC Moment Resisting Framed (MRF) structures with different column dimensions in order to confirm empirical expressions for determining *T*, as given by several authors and the EN1998-1 of Eurocode 8 (2004). Consequently, using a similar databank of modelled RC MRF structures, Hadzima-Nyarko *et al*. (2012) delivered new expressions for *T* of regular RC frames by implementing nonlinear regression analysis using genetic algorithm and taking into consideration the direction of the structure. Moreover, Kwon and Kim (2010) evaluated building period formulas from a selected seismic design code for 800 apparent periods from 191 buildings and 67 earthquake records. Their calculations were carried out using the formulas taken from ASCE 7-05 for steel and RC MRF, SW buildings, braced frames, and other structural types. They addressed that the differences between the estimated periods from code formulas and these of measured periods of low-to-medium rise buildings were found to be relatively high. They concentrated on the fact that code formulas for SW buildings noticeably overestimated periods for all tested building heights.

Based on the Colombian seismic design code, Castellanos *et al*. (2013) studied *T* of SW buildings in Calif. Colombia by using ambient vibration tests and modal identification methods and taking into account factors such as the foundation system, constructive system and seismic zone. As a result, they suggested simple equations for estimating *T* based on the building height and type of structural system. However, they showed that these estimations often differ greatly from the experimental values. Similarly, Chalah *et al*. (2014) proposed a simplified expression for *T* of RCSW systems considering the geometrical and mechanical characteristics of the structure and based on an elastic behavior and the Dunkerley's formula. Their suggested period values as function of the number of floors and their stiffness. Similarly, an earlier study by Dym *et al*. (2007) examined the empirical estimates of the fundamental frequency (the inverse of period) for tall buildings by analyzing the consequences of using two beam models to estimate such natural frequencies. Their study showed that the Timoshenko beam model is appropriate for describing the behavior of shear-wall buildings, while a coupled two-beam model is appropriate for shear-wall-frame (e.g., tube-and-core) buildings where it comes much closer to replicate the parametric dependence of building frequency (or period) on height. Thus, one may conclude that the analysis procedure and its assumptions would greatly affect the estimation of natural periods for tall buildings.

In a similar manner, Hadzima-Nyarko *et al*. (2015) carried out a parametric study on 480 different RC building models with SWs, taking into account different influencing parameters such as the number of bays. They compared *T* of RCSW dominant building models with several empirical expressions of *T* given by different selected seismic design codes, and their results emphasized the essential need to improve the available expressions for T estimation of RCSW structures allowing for the effect of number of bays. Later, Abo El-saad and Salama (2017) carried out a study on estimation of T for RCSW buildings and developed new formulas by regression analysis of measured period data. Comparisons showed good agreement between the periods determined using their proposed formula and measured values. Moreover, their results indicated that the value of coefficient C_t in formulas of the ASCE, and similarly Egyptian building code (2012) need to be decreased and a factor to limit the period calculated by Rayleigh's method should be considered.

Recently, based on measured period data, Badkoubeh and Massumi (2017) proposed a simple closed-form expression to estimate T for RCSW buildings in low, moderate, and high seismicity regions. Their proposed expression provided reasonable estimates of the lower bounds of fundamental periods, and thus, led them to ascertain that US code formulas were inadequate for estimation of T for concrete SW buildings since they depend significantly on the displacement and mechanical properties of the SW.

Generally, current seismic codes offer empirical approximate formulas to estimate T for RCSW systems mainly depending on building height; number, heights and lengths of SWs; and ratio of SW area to base area of the structure. As mentioned earlier, these code formulas have been derived from regression analysis of empirical data of measured T for buildings during seismic actions. However, it is clearly evident from previous literature that the approximate equations available in many current codes have notable and concealed variances between "code-estimated" and "measured" period values for actual structures. Thus, this study aims to establish new improved and simplified formulas to estimate the fundamental periods T for bearing RCSW systems subjected to earthquake excitations, contributing in attempts for better identification of T and thus, accurately evaluates the consequent responses of structure. In that regard, this paper considers the effect of additional influencing parameters such as configuration of horizontal plans, building width, number and width of bays, and presence of SWs with middle corridors and/or core SWs.

In this study, A hundred and ten selected models of bearing RCSW systems were established and analyzed using a 3D non-linear response time history (TH) procedure utilizing the software ETABS v16.2.1. Finite element models of the structures were developed with the nonlinear "Takeda" hysteretic behavior for elements. The dynamics 3D non-linear TH analysis keeps drift limitations for a serviceable structure to be built through taking *P-delta* effect into consideration.

Regression analysis was employed to achieve accurate and comprehensive simplified formulas for estimation of T for bearing RCSW systems towards improving their seismic analysis and design results**.** Results were presented and compared with those obtained from using formulas of seismic code IBC 2012 associated with the provisions of ASCE-SEI 7-10. The seismic performance for two arbitrarily selected test models of 60 m (20-story) and 240 m (80-story) in height with middle corridors were investigated. The responses are presented by design base shear, story shears, maximum lateral displacement, story drift and drift ratio, and are compared with these obtained from implementation of T computed from code formulas. It was observed that the seismic performance of the tested buildings was different with the presence of obvious errors when using T estimated by code formulas. The current proposed formulas led to a reasonable and safe design with success in having reasonable enhancement over the existing code approximate T formulas. Thus, an enhanced accuracy and reliable design may be accomplished when using the proposed expressions by this study to estimate the fundamental natural periods of vibration for bearing RCSW systems.

2. Fundamental period by building design codes

As per the IBC 2012 and the ASCE-SEI 7-10 provisions, the fundamental period of the structure, T, shall be established by properly substantiated analysis procedures using the structural properties and deformational characteristics of the resisting elements. As an alternative to performing analysis, these codes permitted a direct use of given simple formulas for an approximate period, T_a. However, in case of determining T using analysis procedures, these codes assigned an upper limit for that calculated period depending on the design spectral response acceleration at 1-second period, S_{D1}, and an upper limit coefficient, C_u.

In principle, the approximate formulas aimed to introduce simplified periods leading to conservative evaluation of responses required for analysis and design purposes. However, these formulas offered by seismic design codes frequently provide overestimated periods while in other cases giving underestimated values. This questionable estimation refers to lack in the limited parameters included in the formulas imposed by design codes. Generally, the IBC 2012/ASCE-SEI 7-10 proposed two methods for T_a of RCSW structures. The most common approximate formula given by Method-1 in these codes to obtain T_a (in seconds) is as following

$$T_{a1} = C_t h_n^x \tag{1}$$

Where T_{a1} represent the approximate fundamental period using the main simplified code equation of Method-1, and h_n is the structural height (in meters or feet) representing the vertical distance from the base to the highest level of the seismic force-resisting system of the structure. The C_t and x are building period coefficients determined from Table 1 below.

Generally, Eq. (1), as found in these codes, is a semi-empirical equation which was established based on Rayleigh's method in a form of an exponential equation as $T = \alpha H^{\beta}$, where H is the structural height, and α and β are numerical variables. Early study by Goel and Chopra (1997b) concluded that the variables α and β depend on the properties and type of the structure. Later, the code replaced these variables α and β by the coefficients C_t

Table 1 Values of Approximate Period Coefficients C_t and x, as per IBC 2012/ASCE-SEI 7-10

Structure Type	C_t	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
- Steel moment-resisting frames	0.028 (0.0724)[a]	0.8
- Concrete moment-resisting frames	0.016 (0.0466)[a]	0.9
Steel eccentrically braced frames in accordance with ASCE 7-10 requirements.	0.030 (0.0731)[a]	0.75
Steel buckling-restrained braced frames	0.030 (0.0731)[a]	0.75
All other structural systems	0.020 (0.0488)[a]	0.75

[a] Metric equivalents are shown in parentheses, for h_n in meters.

and x as illustrated in Table 1 below. Herein, it is worth mentioning that despite listing of some structure types in Table 1 with exclusive values for C_t and x, curiously the RCSW system and many other systems are not explicitly included except within the extensive structural types of "*all other structural systems*". Thus, it seems that there is a serious need for in-depth research about this concern.

However, the IBC 2012/ASCE-SEI 7-10 permitted using alternative approach for calculating T_a (in seconds) for masonry or concrete shear wall structures using the following equation

$$T_{a2} = \frac{0.00623}{\sqrt{C_w}} h_n \qquad (\text{if } h_n \text{in meter}) \qquad (2)$$

Where T_{a2} represent the approximate fundamental period using the alternative code equation, h_n is the structural height, and C_w is a constant calculated from Eq. (3) as follows

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]} \qquad (3)$$

Where

A_B = area of the base of structure (m^2)
A_i = web area of shear wall "i" (m^2).
D_i = length of shear wall "i" (m).
h_n= structural height (m)
h_i = height of shear wall "i" (m).
x = the number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

The second code alternative Eq. (2) has been based on the Dunkerley's method along with many previous related studies. Among them is that of Goel and Chopra (1998), who employed regression analysis on measured periods for SW buildings recorded in California during earthquake actions. Essentially, the coefficient C_w reflects three important factors that influence the values of T. One of these factors is the ratios between the summation of web cross sectional area of SWs in the direction under consideration and the base area of the building ($\sum_{i=1}^{x} A_i/A_B$), the building height and height of SW (h_n/h_i) and the height and length of SW (h_i/D_i). Accordingly, one may notice that as either ($\sum_{i=1}^{x} A_i/A_B$) or (h_n/h_i) for a building in certain direction decreases, then T for that building in that direction increases, whereas when (h_i/D_i) for a building in certain direction decreases then T for the building in that direction decreases.

Table 2 Members Geometry and Modeling Design Parameters*

Building design parameter	Value
Concrete compressive strength, $f_c^{'}$ (MPa)	28
Concrete unit weight, w_c (kg/m^3)	2400
Modulus of Elasticity, E_c (MPa)	24870
Dead load	Seismic weight
Live load (kN/m^2)	2.0
Size of drop beams (mm × mm)	200 × 500 to 800 × 500
Wall thickness (mm)	200 to 400
Solid Slab thickness (mm)	210 to 350

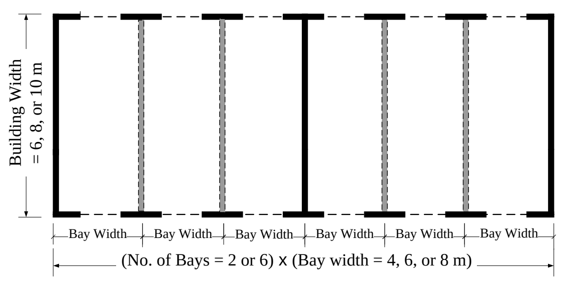
* Data is adopted from Kim and Lee (2014).

3. Description of structural models and analysis

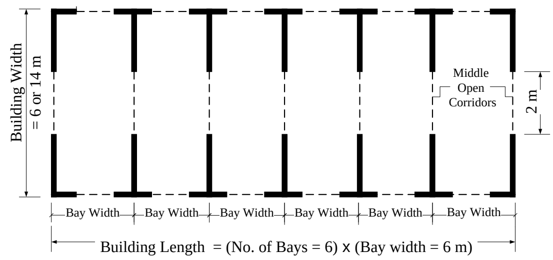
In this section, the examined cases of the structural models for which the fundamental periods are evaluated are thoroughly described. All building models are designed according to the requirement of ACI 318-14 and IBC 2012 codes. All modeled structures are assumed to have a fixed base without presence of columns, considering that all diaphragms are rigid with neglecting the effect of any infill walls within analysis. Summary of members' geometry and modeling design parameters are presented in Table 2.

Dynamic analysis was performed on 110 selected models of bearing RCSW systems using the 3D non-linear response time history (TH) procedure. Analysis is carried out through utilizing the CSI software ETABS v16.2.1 with employing El-Centro (1940) N-S record and (5%) modal damping ratio. Initially, a static design preceded the dynamic analysis for selected models to ensure that their elements have satisfactory dimensions and reinforcements and thus have adequate design that satisfies the code requirements. The 3D nonlinear response TH analysis consists of establishing finite element models which directly account for the nonlinear "Takeda" hysteretic behavior of the structure's elements. Types of finite elements selected were shell elements for slabs and walls and 3D solid type for beams. This is to determine responses through methods of numerical techniques to suite ground motion acceleration histories compatible with the design response spectrum for the site. Moreover, performing a nonlinear TH analysis assures solving for *P-delta* effect, and thus keeps drift limitations for a serviceable structure to be built.

A parametric investigation on T of the selected systems with varying building height (i.e., number of stories), layout configuration of horizontal plans including building width, number and width of bays, and presence of SWs with

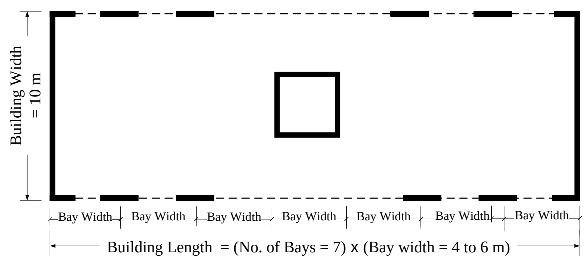


(a) SWs without Middle Corridors

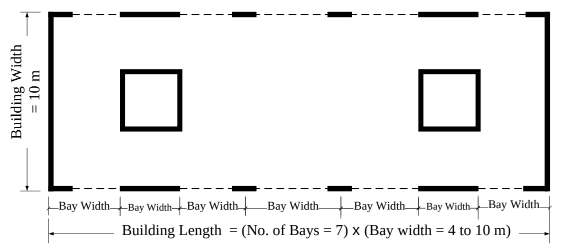


(b) SWs with Middle Corridors

Fig. 1 Horizontal plans for RCSW models with different No. of bays, bay width or building width



(a) Building with Central Core SW



(b) Building with Two Core SWs

Fig. 2 Horizontal plans for RCSW models with one or Two Core SWs

middle corridor and/or core SWs have been studied. The models examined are shown schematically in Figs. 1-2.

A total of eleven horizontal plans of the structural models were constructed and analyzed with different number of stories (i.e.; 4, 8, 12, 16, 20, 30, 40, 50, 60, 70, and 80). The height of each story is taken to be constant and equal to 3.0 m. Thus the seismic structural height considered was varying from 12 m up to 240 m. Different layout configurations are investigated using nine horizontal plans. Moreover, to study the effect of number of bays, two horizontal plans with a different number of bays are examined; one has two bays and the other has six bays as shown last in Fig. 1(a). Both plans have building width equal to 6 m with a regular width of each bay equal to 6 m.

The effect of the width of bay is investigated by using three different horizontal plans, each of them with 6 bays. The bay width has been varied for each case of study as 4, 6, and 8 m with layouts have a regular building width equal to 6 m as showed in Fig. 1(a).

Similarly, to study the effect of the geometry of the transverse y-direction on T, the transverse width of buildings has been varied and its effect is examined. To achieve this purpose, three different horizontal plans were investigated with different building's width for each case of study taken as 6, 8, and 10 m as showed in Fig. 1(a). For each of these cases of study, a regular bay width is assumed equal to 6 m.

To study the effect of the presence of SWs with middle open corridors, two cases of study are considered, with a 2 m wide middle corridor. Both plans have six bays with regular bay width equal to 6 m. One of them has 6 m building width, while the other has 14 m building width as showed in Fig. 1(b). The effect of core SWs has been studied considering two horizontal plans; one of them has 4 m×4 m central core, with building length and width equal to 36 m and 10 m, respectively, as showed in Fig. 2(a). The other model has two 4 m×4 m cores, with building length and width equal to 48 m and 10 m, respectively, as shown in Fig. 2(b). Both plans have seven bays with irregular bay width ranging from 4 to 10 m. The geometry data of all selected models is presented in Table 3 below.

4. Regression analysis

The results of the T values were used as input data for a rational unconstrained linear regression analysis to identify the most accurate formulas for predicting T for the bearing RCSW systems.

4.1 Forms of proposed equations

In this study, two forms of proposed equations for predicting T were considered. The first equation has the following form

$$T = \alpha h_n^{\beta} \tag{4}$$

where h_n is the height of the building in meter, and α and β are numerical coefficients determined from unconstrained linear regression analysis. For purposes of regression analysis, Eq. (4) is rewritten as

$$y = a + \beta x \tag{5}$$

where $y = \log(T)$, $a = \log(\alpha)$ and $x = \log(h_n)$

Alternatively, second form of equation has been used as

$$T = \alpha \left(\frac{h_n}{\sqrt{C_w}} \right)^{\beta} \tag{6}$$

where C_w is same as defined previously in Eq. (3).

It may be noticed that Eq. (5) is linear equation with intercept constant a and slope β. However, for purposes of regression analysis, Eq. (6) may also be rewritten in the form

Table 4 Values of α, β, and S_e for Each Empirical Equation based on Eqs. (4)-(6)

Proposed Equation	Studied Parameter	α	β	S_e%
First Proposed Equation $T_R = \alpha h_n^{\beta}$	Number of bays	0.00073	1.90	2.9
	Building width	0.00066	1.88	10.5
	Bay width	0.0007	1.90	9.4
	Presence of middle corridor	0.0022	1.63	14.9
	Presence of core SWs	0.0013	1.76	3.2
	All Parameters Together	0.001	1.82	7.9
Second Proposed Equation $T = \alpha\left(\frac{h_n}{\sqrt{C_w}}\right)^{\beta}$	Presence of middle corridor	0.0045	0.79	10.3
	Presence of core SWs	0.0032	0.90	4.3
	All Parameters Together	0.0038	0.87	15.3

of Eq. (5) above but with $x = \log\left(\frac{h_n}{\sqrt{C_w}}\right)$.

4.2 Procedure of regression analysis

In regression analysis, the constants a and β are determined by minimizing the squared root error between the direct computed periods and the these obtained from analytical models for each studied parameter separately to obtain individual equations based on each parameter. In Table 4, the second column displays all parameters that have individual equations based on the first and the second proposed equations.

For both proposed equations, α is calculated back from the relationship $a = \log(\alpha)$ for each individual equation separately.

Then, the standard error (S_e), is calculated from the following equation

$$S_e = \sqrt{\frac{\sum_{i=1}^{n}[y_i - (a + \beta x_i)]^2}{(n-2)}} \tag{7}$$

where n is the number of periods considered in regression analysis, and $y_i = \log(T_i)$ with T_i represents the period obtained from dynamic analysis. The term $(a + \beta x_i)$ is as defined in Eq. (5) for both proposed equations, respectively.

Finally, to obtain two general forms of equations based on the first and second proposed equations of Eqs (4)-(6), the values a, β, α and S_e are determined by minimizing the error between the computed periods and the periods obtained from dynamic analysis for all cases of study. Based on the two forms of Eqs. (4)-(6), these values of α, β, and S_e which are obtained to count for the effect of each considered parameter individually and consequently for the proposed generalized equation, are established and summarized below in Table 4.

4.3 Lower and upper limits of periods

The coefficient α which is calculated from rational analysis would have lower value α_L and upper value α_U. These values are calculated from the following equations

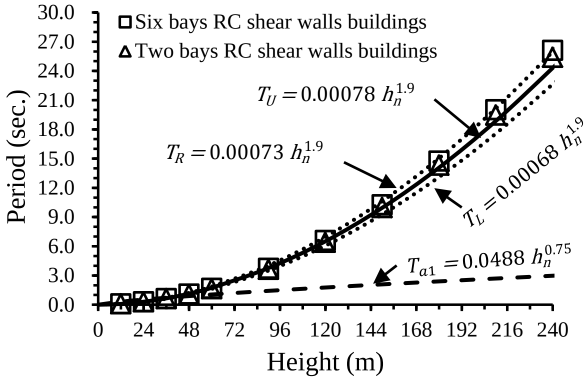


Fig. 3 Natural periods of models with different number of bays and building heights, obtained by code Eq. (1) and the analytical-regression procedure

$$log(\alpha_L) = log(\alpha) - S_e \tag{8}$$

$$log(\alpha_U) = log(\alpha) + S_e \tag{9}$$

The lower and upper periods T_L and T_U for the first proposed Eq. (4) will have the following forms, respectively

$$T_L = \alpha_L h_n^{\beta} \tag{10}$$

$$T_U = \alpha_U h_n^{\beta} \tag{11}$$

On the other side, the T_L and T_U for the second proposed Eq. (6), will have the following forms, respectively

$$T_L = \alpha_L\left(\frac{h_n}{\sqrt{C_w}}\right)^{\beta} \tag{12}$$

$$T_U = \alpha_U\left(\frac{h_n}{\sqrt{C_w}}\right)^{\beta} \tag{13}$$

Herein, the lower and upper limits on T make the results more conservative and reasonable, where the over or under estimated periods are removed.

5. Results and discussions

For purposes of comparison, all the period equations obtained analytically from using the ETABS 3D non-linear time history analysis along with the performed regression analysis (i.e., T_R) and their upper (T_U) and lower bounds (T_L) are plotted, together with the comparable periods computed by code's Eqs. (1) or (2), (i.e., T_{a1} or T_{a2}, respectively).

5.1 Effect of number of bays

As shown in Fig. 3, the variation of number of bays has slightly effect values of T for buildings having the same height, where T for the two bays buildings are found slightly smaller than that obtained for six bays buildings. Fig. 3 shows clearly that the code Eq. (1) does not show compatibility with the determined analytical periods.

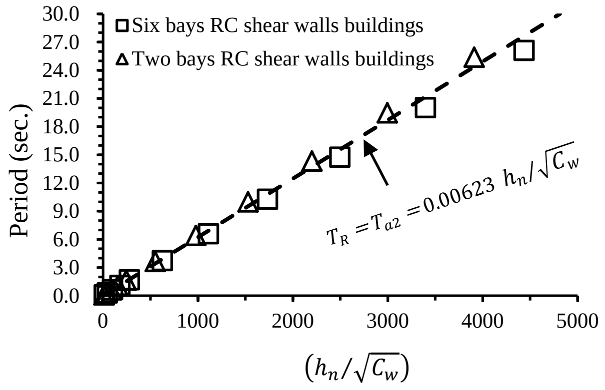


Fig. 4 Natural periods of building models with different number of bays and heights, obtained by code Eq. (2) and the analytical-regression procedure

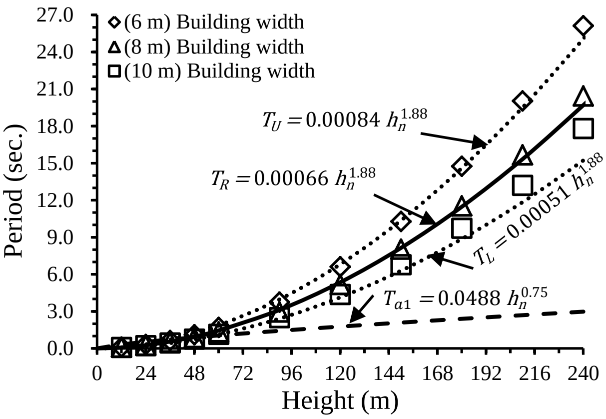


Fig. 5 Natural periods of models with different building width and heights, obtained by code Eq. (1) and the analytical-regression procedure

Therefore, a modified equation is proposed by the rational regression analysis, and represented as T_R in Fig. 3 with their upper and lower bounds T_L and T_U, respectively.

The standard error S_e between the analytical periods and the periods obtained from code Eq. (1) is equal to 60.0%, whereas is limited to 2.9% between the analytical periods and the periods obtained from the modified equation T_R.

Fig. 4 shows that the buildings with two and six bays with the same height have closed values for $(h_n/\sqrt{C_w})$. This indicates that the number of bays does not have a remarkable effect on values of T. In fact this due to the interactive influence of the coefficient C_w on the values of T reflected by the two important factors $(\sum_{i=1}^{x} A_i/A_B)$ and (h_i/D_i). Also, Fig. 4 shows clearly that period from code Eq. (2) agrees well with the analytical computed periods. Therefore, no further modification is appeared to be necessary on period from code Eq. (2) due to effect of number of bays.

5.2 Effect of building width

Fig. 5 shows noticeably that the code Eq. (1) is not compatible with the computed analytical periods, where a large percentage of standard error of 54.0% between their values is observed. Therefore, a modified equation is seemed to be essential and proposed by employing the rational regression analysis. The modified equation with its upper and lower bounds are appearing scattered on Fig. 5, where the percentage of standard error S_e between the analytical periods and the periods obtained from the modified equation is limited to 10.5%. As shown from Fig. 5, the buildings with the largest width have the lowest fundamental periods of vibration.

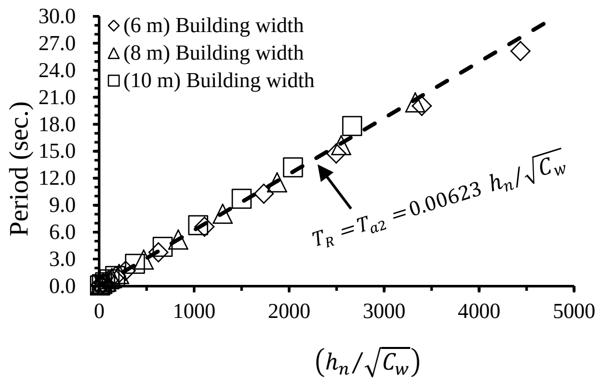


Fig. 6 Natural periods of models with different building width and heights, obtained by code Eq. (2) and the analytical-regression procedure

Fig. 6 represents the code alternative Eq. (2) and the computed analytical periods for buildings with different building width, where the percentage of standard error between their values is found to be 4.1%, which is appeared to be a reasonable deviation. Therefore, there is no need for further modification on code Eq. (2) due to effect of building width. However, Fig. 6 shows a slight difference between periods for buildings with different width due to the fact that the buildings with 6 m width have the largest values of (h_i/D_i), whereas the buildings with 10 m width have the smallest values of (h_i/D_i). The buildings with 8 m width have intermediate values of (h_i/D_i) compared to the former two buildings. These values explain why T for buildings having the same seismic heights, but with 6 m width are a little larger than the periods of the buildings with 8 m width, and followed by the smallest periods for buildings with 10 m width.

5.3 Effect of bay width

Fig. 7 shows the analytically computed periods for buildings with bay width equals 4 m, 6 m and 8 m, respectively. As noted from Fig. 7, the variation of bay width has a noticable effect on T for buildings with the same height, where it increases as the bay width increases. On the one hand, it is appeared from Fig. 7 that the code Eq. (1) does not show good compatibility with T obtained by analytical procedure, with percentage of standard error between their periods reaches 59.3%. Instead, the proposed modified equation shows more agreement with the analytical periods, with a standard error percentage between their values limited to 9.4%.

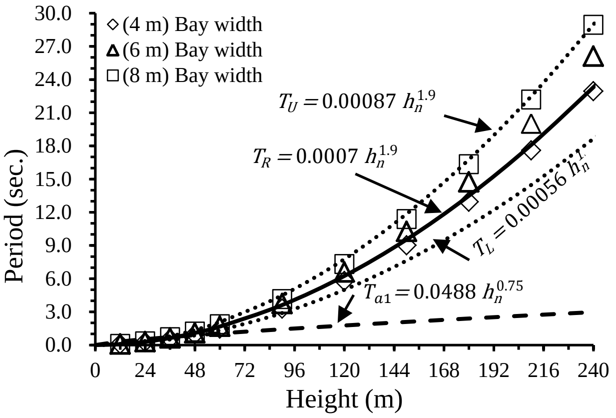


Fig. 7 Natural periods of models with different bay width and building heights, obtained by code Eq. (1) and the analytical-regression procedure

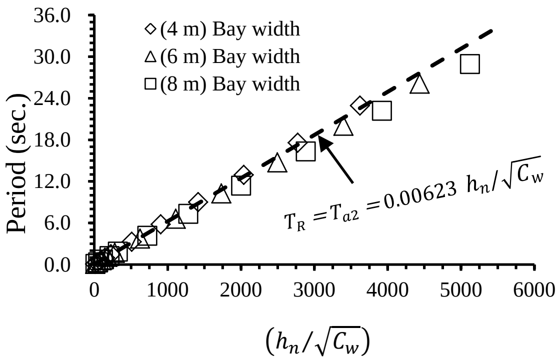


Fig. 8 Natural periods of models with different bay width and building heights, obtained by code Eq. (2) and the analytical-regression procedure

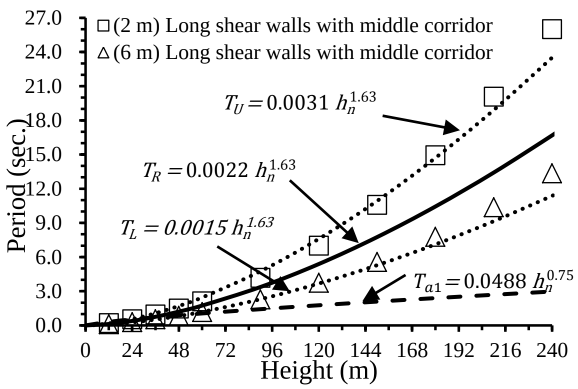


Fig. 9 Natural periods of models with different heights and SW's length with presence of middle corridor, obtained by code Eq. (1) and the analytical-regression procedure

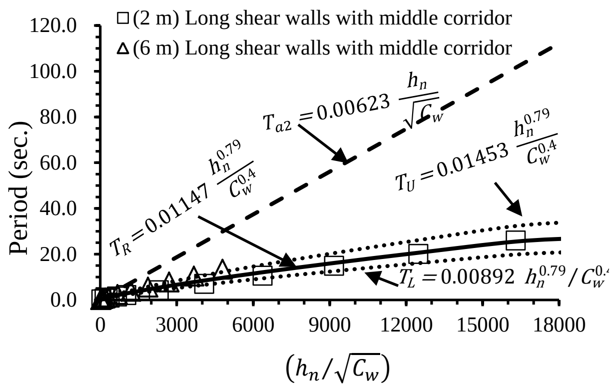


Fig. 10 Natural periods of models with different heights and SW's length with presence of middle corridor, obtained by code Eq. (2) and the analytical-regression procedure

In comparison with code Eq. (1), the percentage of standard error for Eq. (2) is reduced down to 3.7%. As shown in Fig. 8, the code Eq. (2) introduces compatible periods with the analytically computed periods. Due to this fact, no further modification is needed for code Eq. (2) due to the effect of bay width. The difference of bay width between the three horizontal plans does not have significant effect on the value of (h_i/D_i). Furthermore, the buildings with 4 m , 6 m, and 8 m bay width have a descending values for ratio of SW area to base area of structure (*i.e.*, $\sum_{i=1}^{x} A_i/A_B$), respectively. Moreover, Fig. 8 shows that buildings with the largest ratio of $(h_n/\sqrt{C_w})$ have the longest period. These ratios clarify why T for buildings having the same height but with different bay width are varying. The buildings with 8 m bay width have the longest period followed by these with 6 m bay width, and then buildings with 4 m bay width.

5.4 Effect of presence of middle corridor

Fig. 9 presents computed periods for models with different heights and SW's length with presence of middle corridor. These periods are obtained by Code Eq. (1) and the modified analytical regression equation (T_R) with its upper and lower bounds. In case of buildings with two SWs having length of 2 m, among them 2 m middle open corridor, the standard error between the analytically computed periods and these from code Eq. (1) is found to be 62.7%. In comparison, a standard error of 43.2% is obtained for the case of two SWs of having length of 6 m. In addition, Fig. 9 shows that the current modified equation agrees well with the analytically computed periods, where the standard errors for both cases are limited to 15.7% and 15.4%, respectively. A shown in Fig. 9 the periods for buildings with two SWs having length of 2 m, with middle open corridor, is larger than these for buildings with two SWs having length of 2 m, with middle open corridor.

Fig. 10 shows that the value of $(h_n/\sqrt{C_w})$ for buildings have the same height, with two SWs of 2 m length and having 2 m middle open corridor between them, is larger than that for building with two SWs of 6 m length, with 2 m middle corridor. Also, it is clearly appeared that code Eq. (2) does not show good compatiblity with the analytical periods, where it led to large standard error of 40.7%. In comparison, the standard error between the current modified equation and the analytical periods is limited to 10.3%.

5.5 Effect of presence of core shear walls

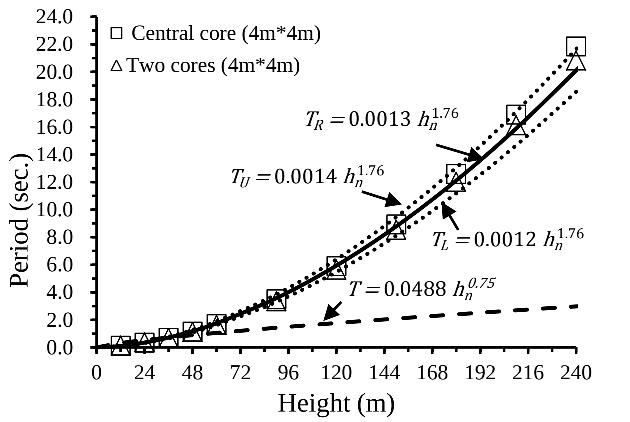


Fig. 11 Natural periods of models with core SW and different building heights, obtained by code Eq. (1) and analytical-regression procedure

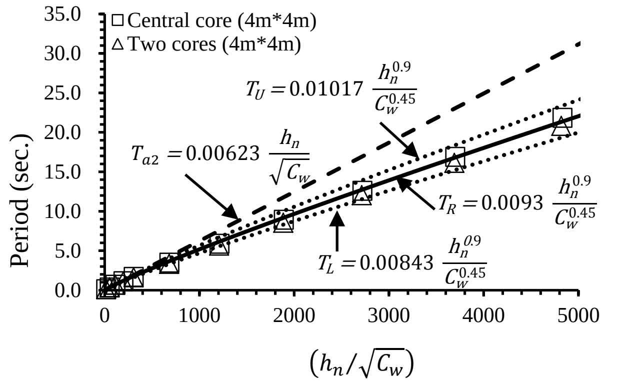


Fig. 12 Natural periods of models with core SW and different building heights, obtained by code Eq. (2) and the analytical-regression procedure

Fig. 11 shows that the periods of vibration for buildings with two core SWs are larger than these for obtained buildings with one central core SW. In addition, it appears obviously that code Eq. (1) does not show proper compatibility with the analytical periods, where the standard error is found 53.9%. So, a modified equation by rational regression analysis is essentially proposed. It is shown that the modified equation and its upper and lower limitations are scattered on Fig. 11. The standard error between the analytical periods and these from modified equation is found 3.2%.

Fig. 12 shows that the buildings with central single core SW and have different values of $(h_n/\sqrt{C_w})$ compared to buildings with two core SW's, at same height. The buildings with central core SWs have smaller values of $(\sum_{i=1}^{x} A_i/A_B)$ and $(\sum_{i=1}^{x}(D_i/h_i)^2)$ compared with the buildings having two cores SWs. These values explain why T for buildings with central core SWs is larger than that for buildings with two core SWs. Also, Fig. 12 shows that code Eq. (2) does not give good compatibility with the analytical periods for buildings higher than 60 m, where the standard error between their values is found to be 64.7%. So, a modified equation with its upper and lower bounds are essentially proposed using the rational regression analysis. The percentage of standard error between the analytical periods and the periods from modified equations is limited to 10.7%.

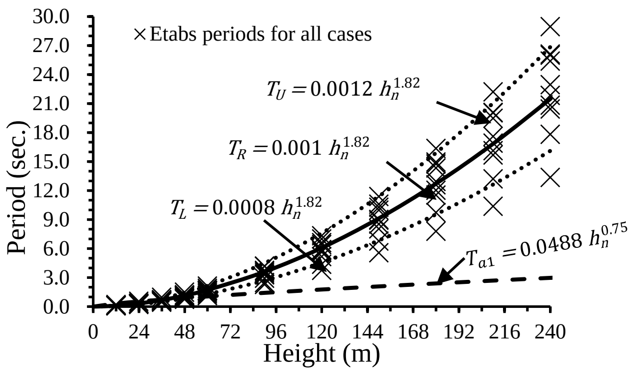


Fig. 13 Natural periods of all models with different building heights, obtained by model code Eq. (1) and the analytical-regression procedure

5.6 General forms of the modified period equations

Due to the imprecise periods obtained by code Eq. (1), regression analysis is conducted on all of the analytically computed periods for all of proposed models based on the code empirical form of Eq. (4). Thus, a first form of the general modified equation including the effects of all studied parameters altogether is formulated. The first form of the general equation for periods with its upper and lower bounds are shown in Fig. 13. As shown in Fig. 13, the most proper equations obtained from regression fitting, with lower and upper bounds, are given below in Eqs. (14)-(16).

The percentage of standard error between the analytical periods and these obtained from first general modified equation is narrowed to 7.9%.

$$T_R = 0.001\ h_n^{1.82} \tag{14}$$

$$T_L = 0.0008\ h_n^{1.82} \tag{15}$$

$$T_U = 0.0012\ h_n^{1.82} \tag{16}$$

Since few periods fall above T_U or under T_L, it is deemed to be suitable to use the equation of T_L to estimate conservative periods. T_U is used particularly to limit the periods computed from rational analysis. So, the periods from rational analysis should not be longer than the following limit: $(\alpha_U/\alpha_L)T_L = (0.0012/0.0008)T_L = 1.5\ T_L$.

Previously, it is shown that code Eq. (2) is not compatible with the analytically computed periods for buildings that have core SW's or middle open corridor. Thus, an essential need arises for more reasonable equation to estimate T basing on the alternative form of code Eq. (6). Thus, a regression analysis is conducted on the analytically computed periods obtained from ETABS non-linear time history analysis for all cases together considering different affecting parameters. Fig. 14 shows the obtained periods with their bounds, and these obtained by code Eq. (2). A second form of general proposed modified equation including the effects of all parameters together with its upper and lower bounds are given in Eqs. (17)-(19) below.

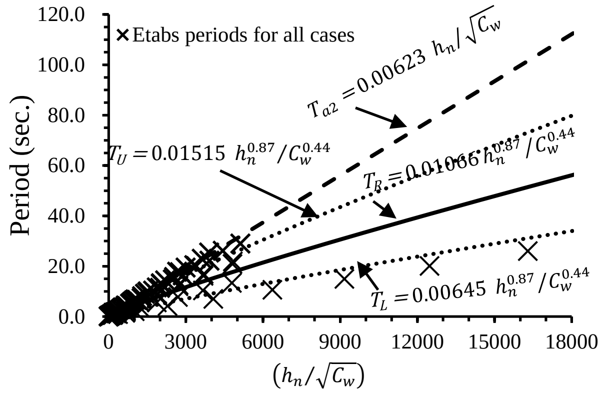


Fig. 14 Natural periods of all models with different building heights, obtained by code Eq. (2) and the analytical-regression procedure

Table 5 Modeling design parameters* for 20- and 80- story building test models

Modeling Design Parameters	20-Story Model	80-Story Model
$f_c^{'}$ (MPa)	28	28
Live load, (kN/m^2)	2	2
Story height (m)	3	3
Number of stories	20	80
Seismic Structural Height (m)	60	240
Building width (m)	15	15
Building length (m)	40	40
Walls' Thickness (mm)	200	400
Solid Slabs' Thickness (mm)	210	350
Beams (mm×mm)	200×500	400×500
Width of Wall Openings (m)	5	5

The second modified general form shows that periods from rational analysis should not be longer than the limit of $(0.01515/0.00645)T_L = 2.35\ T_L$.

$$T_R = \frac{0.01066 h_n^{0.87}}{C_w^{0.44}} \tag{17}$$

$$T_L = \frac{0.00645 h_n^{0.87}}{C_w^{0.44}} \tag{18}$$

$$T_U = \frac{0.01515 h_n^{0.87}}{C_w^{0.44}} \tag{19}$$

6. Seismic performance of buildings test models

To validate the current proposed modified equations, investigation is carried out for the seismic performance of two selected test models to ensure a satisfactory performance against the design seismic loads. The test models are analyzed and designed depending on five alternative equations for periods, that are: 3D non-linear time history dynamic analysis using ETABS v16.2.1; code Eq. (1); code Eq. (2); modified Eq. (15); and modified Eq. (18). In order to achieve this target, an arbitrary horizontal plan with middle open corridor is chosen for two building test models with different structural heights.

6.1 Description of the selected test models

In purpose of verification of the results, two test building models of bearing wall systems with special RC shear walls have been selected and analyzed for different responses. The selected building test models have 20- and 80-story with reqular story height of 3 m (i.e., structural heights of 60 m and 240 m, respectively).

The modeling design parameters for both building test models, including geometrical dimensioning of all shear walls, solid slabs, and beams are presented in Table 5. Besides, exterior and interior drop beams are provided with using coupling beams connecting shear walls at openings where they exist. Types of finite elements selected were shell elements for slabs and walls and 3D solid type for beams. The length of each wall in both *X* and *Y* directions is exposed in Fig. 15.

Comparable arbitrarly selected horizontal plan is proposed for both test models as shown in Fig. 15 with width and length for both buildings equal to 15 m and 40 m, respectively. The test building models have six bays with irregular bay width varying from 5 to 7 m, in addition to presence of middle open corridors of 5 m wide. Both buildings are chosen not to have any irregularity type.

6.2 Essential seismic parameters for design

Using the response spectrum procedure, the mapped spectral response acceleration parameters at short period, (S_S), one-second period, (S_1), and all other necessary material properties and load calculations for the test models are taken from (Kim and Lee, 2014). The values of S_S and S_1 are $0.555g$ and $0.225g$, respectively, where g is the gravity acceleration ($g = 9.81\ \mathrm{m/sec^2}$). The site class is considered class (B), thus there is no need to further modification on the spectral accelerations S_S and S_1. The risk category of the studied model is chosen to be (II) with the importance factor, (I_C), taken as (1.0).

Referring to IBC 2012/ASCE/SEI 7-10, with computed S_{DS} equals to 0.37 g and with the computed S_{D1} equals to 0.15 g and the assigned risk category II, the Seismic Design Category of the test models is determined to be (SDC C). The test models are designed as a bearing wall lateral-force-resisting system with selection of special reinforced concrete shear walls to resist the lateral forces. The response modification factor, (*R*), for this system is slected to be (5.0).

6.3 Alternative periods used in analysis

The most important parameter that must be defined in this type of analysis is the fundamental period of vibration, *T*. The test models are analyzed five consequent times based on the aforementioned five alternative procedures for period estimation, which leads to periods for five cases of study as presented in Table 6 below.

As shown from the above different estimated periods presented in Table 6, the code Eq. (1) and the modified Eqs.

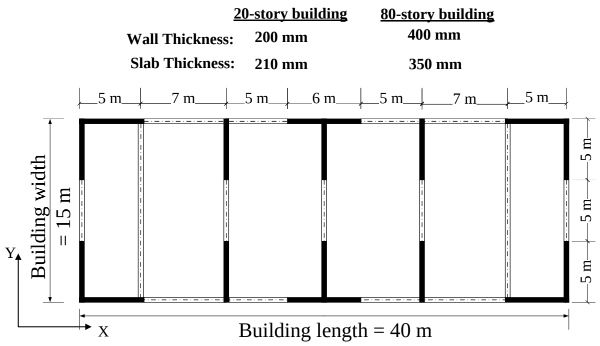


Fig. 15 The horizontal proposed plan for the 20- and 80-story bearing wall building test models

Table 6 Values of fundamental periods and absolute errors for test models using different methods

Case Study for Different Periods	Period (sec)			
	20-Story Model	Error %	80-Story Model	Error%
3D Non-linear time history analysis (ETABS)	1.70	0	20.17	0
Code Eq. (1)	1.05	-38	2.98	-85.3
Code Eq. (2)	3.08	+81.4	35.85	+77.7
Modified Eq. (15)	1.38	-18.8	17.18	-14.8
Modified Eq. (18)	1.50	-11.9	12.43	-38.4

(15) and (18) introduce periods less than the period of non-linear time history analysis. That means they are probable to introduce a conservative analysis results and design, whereas the code Eq. (2) introduces a period larger than the period obtained by ETABS non-linear time history analysis, which means that it will introduce non-conservative analysis results and insufficient design to resist the seismic lateral forces.

Table 6 also shows the percentages of errors in period for both the 20- and 80-story buildings obtained by different methods measured with respect to the period obtained by non-linear time history as benchmark value. The modified Eq. (18) produces the least absolute percentage of error for the case of 20-story building, where Eq. (15) gives the least absolute error for the 80-story building. In both cases, code Eqs. (1) and (2) give under- and over-estimated periods, respectively, with considerable absolute percentage of errors, which may lead to significant inaccuracy in targeted responses. Also, error calculations indicate that both modified Eq. (15) and (18) give values of periods lower than these of TH procedure with insignificant absolute percentage of errors. Thus, they lead to optimized conservative responses in analysis. Besides, it is observed that Eq. (15) seems more suitable to predict periods for high-rise buildings, while Eq. (18) is appropriate to be employed for the intermediate to low-rise buildings.

6.4 Analysis results and seismic responses

The seismic base shear, story shears, maximum lateral displacements, story drifts and drift ratios are investigated and discussed for tested buildings using the above-mentioned approaches of different estimated periods.

Table 7 Values of base shear for test models using different computed periods

Case Study for Different Periods	Base Shear (kN)			
	20-Story Model	Error %	80-Story Model	Error%
Non-linear time history analysis (ETABS)	2087.9	0	10010.0	0
Code Eq. (1)	3368.0	+61.3	12890.0	+28.8
Code Eq. (2)	1922.7	-7.9	6198.3	-38.1
Modified Eq. (15)	2570.9	+23.1	10641.3	+6.3
Modified Eq. (18)	2370.2	+13.5	11507.0	+15.0

6.4.1 Base shear forces

The values of base shear for all periods are represented below in Table 7. It appears that the period of the code Eq. (2) does not introduce a sufficient base shear for purposes of adequate design. code Eq. (2) gives base shear for both of the 20- and 80-story buildings lower than that obtained by non-linear TH procedure with negative percentage of errors measured with respect to the TH analysis. However, the period obtained from code Eq. (1) introduces base shear higher than that obtained from using the period of non-linear time history analysis, but with highly considerable percentage of errors. This means a very conservative design and higher cost. On the contrary, the periods obtained from the modified Eqs. (15) and (18) introduce reasonably higher values of base shear with acceptable percent of increase for both of the 20- and 80-story buildings.

The base shears obtained from using the proposed Eqs. (15) and (18) for both of the 20- and 80-story buildings are found to be higher than that of TH analysis with percent of increase equal to (+23.13%) and (+13.5%), respectively. However, Eq. (15) introduces much economical increase limited to (+6.3%) in base shear presenting a slightly more conservative design for the 80-story building. Eq. (18) gives (+15%) increase in base shear for the 80-story building, which is almost similar to that obtained in the 20-story-building. Therefore, the current modified equations seem to introduce realistic, conservative, and better design cost. However, Eq. (15) seems more suitable to predict base shear for high-rise buildings, while Eq. (18) is more applicable for the intermediate to low-rise buildings.

6.4.2 The story shears

Figs. 16-17 show vertical distribution of seismic shear forces into each story of the 20- and 80-story building models using different methods of predicting periods. Both figures show noticeably that the periods of the proposed modified Eqs. (15) and (18) introduce the most reasonable vertical distribution of story seismic shear forces for purposes of reputable design compared to these obtained from non-linear TH analysis. On the other hand, the period of the code Eq. (1) introduces very conservative and uneconomic design, while the period of the code Eq. (2) introduces non-conservative low shears, and thus less reliable design.

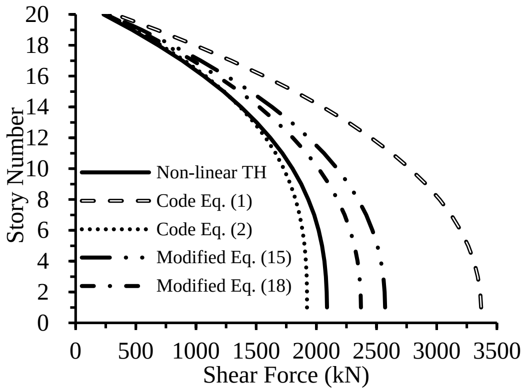


Fig. 16 Vertical distribution of seismic shear forces into each story of the 20-story model using different periods

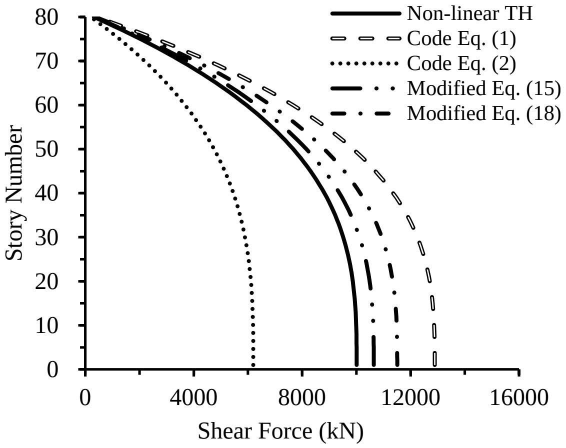


Fig. 17 Vertical distribution of seismic shear forces into each story of the 80-story model using different periods

The shear forces exerted at the each story obtained based on period from code Eq. (1) is higher than these obtained from non-linear time history analysis. In this case of study, the percent of increase in shear force ranges from (+42.2%) in the 1st story to (+61.3%) in the 20th story, whereas that percent of increase for the 80-story building is approximately constant and equal to (+28.8%) in any story.

Similarly, the shear forces exerted at each story obtained for both of the 20- and 80-story buildings based on period from code Eq. (2) are found alternating from these obtained by non-linear TH analysis. For this case of study on the 20-story building, these shear forces are found varying from decrease at low-to-intermediate levels to an increase at much higher levels. The percent of decrease in shear force in the 20-story building ranges from (-7.9%) in the 1st story to (-0.6%) in the 14th story, whereas an increase occurred with percent ranges from (+0.4%) in the 15th story to (+5.5%) in the 20th story. On the contrary, the code Eq. (2) led to approximately constant increase equal to (+28.8%) in any of story shear forces compared to the TH procedure.

The modified Eqs. (15) and (18) for the 20-story building led to an increase in the shear forces in 1st to 20th story ranges from (+16%) to (+23.1%) and (+9.3%) to (+13.5%) for each equation, respectively. However, these percentages become constants in the 80-story building with ratios of (+6.3%) and (+15%) for each of the two equations,

Table 8 Maximum lateral displacements at roof level in *X* and *Y* directions using different periods

Case Study for Different Periods	Maximum Lateral Displacement (mm)			
	20-Story Model		80-Story Model	
	X	*Y*	*X*	*Y*
3D Non-linear time history analysis (ETABS)	48.7	28.0	106.4	185.5
Code Eq. (1)	74.6	42.9	172.2	299.4
Code Eq. (2)	47.2	27.1	103.0	179.4
Modified Eq. (15)	58.5	33.7	118.8	206.5
Modified Eq. (18)	54.5	31.3	123.6	214.5

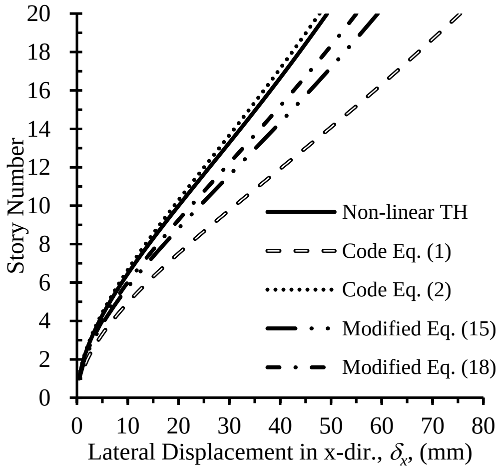


Fig. 18 Lateral displacement in *X* direction for the 20-story model using different periods

respectively. Thus, it is obvious that using the modified Eqs. (15) and (18) produces more reliable shears and accordingly more economical design. However, Eq. (15) seems to produce more consistent shears for high-rise buildings, while Eq. (18) appears to be proper for low-to-intermediate rise buildings.

6.4.3 Story lateral displacement and story drift

Table 8 represents the maximum lateral displacement at the roof level in both *X* and *Y* directions for all cases of study. Moreover, Figs. 18-21 successively show the lateral displacements obtained for each story in both directions for the two tested 20- and 80-story buildings. It clearly appears from Table 8 and Fig. 18-21 that the period of the code Eq. (1) introduces high and very conservative values for lateral displacement in both *X*- and *Y*-direction with approximated average increase of (+55%) and (+62%), respectively, compared with these obtained from the periods of non-linear TH analysis. Thus, code Eq. (1) led to lateral displacement in both directions for both of the 20- and 80 story buildings equal to these obtained by using the period of non-linear TH analysis multiplied by factors of 1.55 and 1.62, respectively.

On the other hand, the lateral displacements in *X*- and *Y*-direction obtained based on the period of the code Eq. (2) appeared not to to be sufficient to achieve adequate and reliable design, since they are less than rational values. The percentage of decrease in the lateral displacement in both

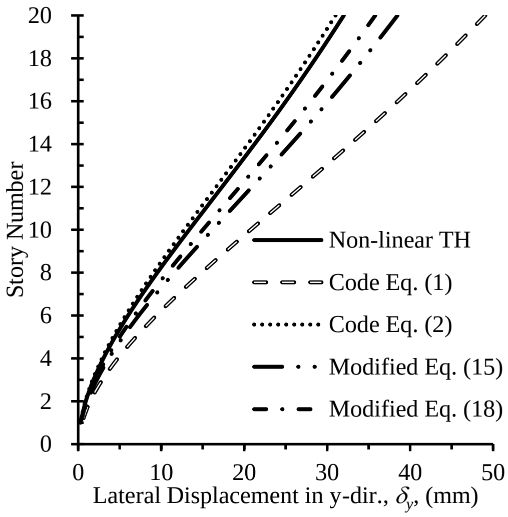


Fig. 19 Lateral displacement in Y direction for the 20-story model using different periods

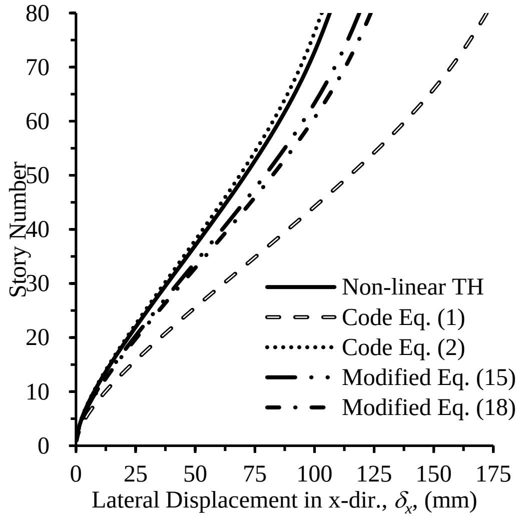


Fig. 20 Lateral displacement in X direction for the 80-story model using different periods

directions for the 20- and 80-story building are found about (-4.5%) and (-3.2%), respectively.

Conversely, the modified Eqs. (15) and (18) show reasonable values of lateral displacements in both X- and Y-direction. The modified Eq. (15) produces lateral displacements in both directions higher than the TH values with percent of increase in the 20- and 80-story building of (+21%) and (+11.6%), respectively. Similarly, Eq. (18) produces lateral displacements in both directions higher than the TH values with percent of increase in the 20- and 80-story building of (+12.3%) and (16%), respectively.

Table 9 shows that all estimated periods introduce conservative maximum drift, except the case of using the period of code Eq. (2), which introduces design values less than the demand, with a maximum percent of decreases equal to (-2.5%) and (-38%) for the 20- and 80-story buildings, respectively. This may affect the human comfort, and the stability of the structure. It clearly appears that the period of the code Eq. (1) introduces very conservative drift

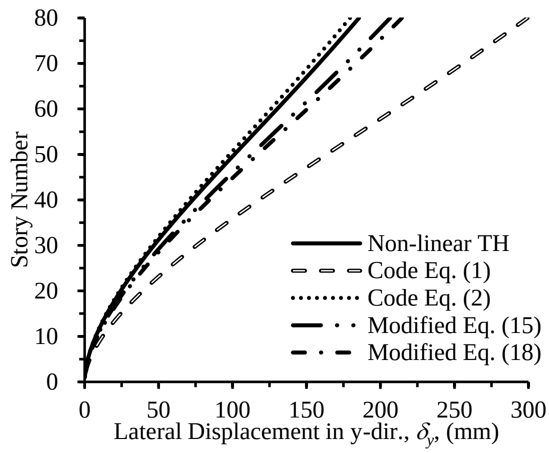


Fig. 21 Lateral displacement in Y direction for the 80-story model using different periods

Table 9 Maximum story drift in X and Y directions using different periods

Case Study for Different Periods	Maximum Story Drift (mm)					
	20-Story Model			80-Story Model		
	X	Y	Max. Error%	X	Y	Max. Error %
3D Non-linear time history analysis (ETABS)	3.1	2.0	0	2.0	5.29	0
Code Eq. (1)	4.7	3.0	+53	2.6	6.8	+29
Code Eq. (2)	3.0	1.9	-2.5	1.2	3.3	-38
Modified Eq. (15)	3.7	2.4	+20	2.1	5.4	+4.5
Modified Eq. (18)	3.4	2.2	+12	2.1	5.5	+6

values, with a maximum percent of increases equal to (+53.0%) and (+29%) for the 20- and 80-story buildings, respectively.

The percent of increase in the maximum drift values obtained by Eqs. (15) and (18) are respectively equal to (+20.0%) and (+12..0%) for the 20-story building and (+4.5%) and (+6%) for the 80-story building. It is clearly noticed that these modified equations will lead to more economical design for drifts with better accuracy. However, it is suggested to employ the modified Eq. (18) for drift calculations of RCSW buildings, where they give much closer values to the these obtained from non-linear TH analysis.

According to IBC 2012/ASCE 7-10, the bearing RCSW systems which are located in risk category II, are assigned an allowable story drift ratio of $0.02h_{sx}$, where, h_{sx}, is the story height below level x. Figs. 22 -25 successively show the story drift ratios, in X and Y directions, for both 20- and 80-story test model buildings. It appears that the maximum drift ratios in both directions for both buildings using all predicted periods do not exceed the code limit of $2.0\%\ h_{sx}$. As shown in these figures, code Eq. (1) yields drift ratios that are less than what is predicted by the rational 3D TH analysis and in contrast the code Eq. (2) produces a very conservative (i.e., over-estimated) ratios which will be burdened with during design. However, each of the modified Eqs. (15) and (18) approves similarly that they

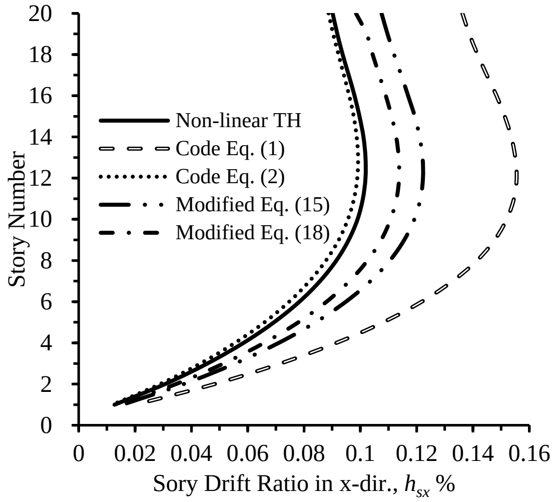


Fig. 22 Story drift ratio in X direction, $h_{sx}\%$ for the 20-story model using different periods

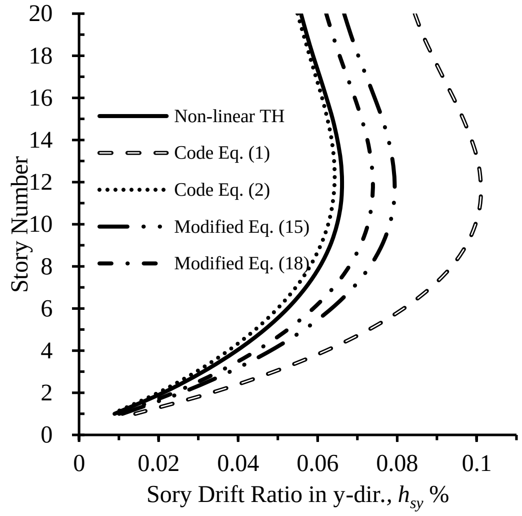


Fig. 23 Story drift ratio in Y direction, $h_{sy}\%$ for the 20-story model using different periods

yield to the most appropriate story drift ratios, where economical, safe, and comfortable design will be achieved.

7. Conclusions

Theoretically modified approximate equations to estimate the fundamental period of vibration, T, have been established for bearing wall systems with reinforced concrete especial shear walls. The proposed equations were derived based on unconstraint regression analyses of 110 selected models using results of 3D non-linear response TH analysis for building models with and without middle corridors and presence of core SWs. The proposed formulas considered many effecting parameters including structural height, number, height and length of SWs, and ratio of SWs areas to the base area of the structure, configuration of horizontal plan including building width, number and width of bays and presence of middle corridors. The seismic performance of selected model structures designed using

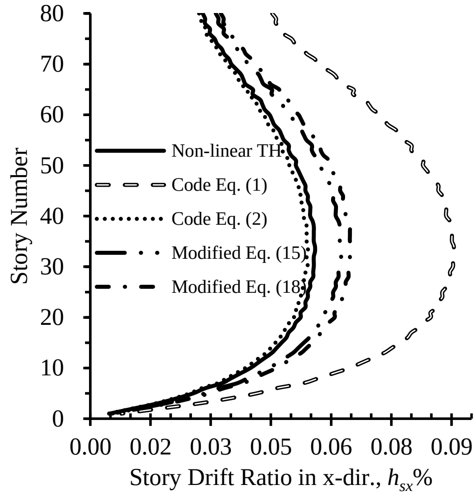


Fig. 24 Story drift ratio in X direction, $h_{sx}\%$ for the 80-story model using different periods

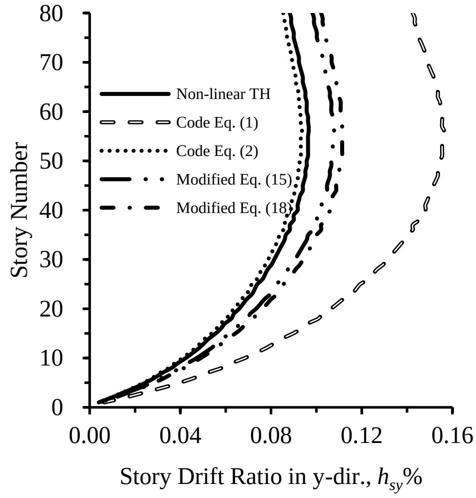


Fig. 25 Story drift ratio in Y direction, $h_{sx}\%$ for the 80-story model using different periods

the proposed formulas was investigated to verify the validity of these formulas, and results were compared with these obtained from formulas of IBC 2012 and ASCE/SEI 7-10. The results showed that the performance of the tested structural system, when using the present proposed equations for estimating T, yields to reasonable, economic, and safe design compared to the approximate equations suggested by codes. Thus, the results of this study demonstrate the following detailed findings:

• Considering building height alone may not ensure a valid period for the bearing RCSW systems.

• Variation of the number of bays does not have significant effect on the fundamental natural period of vibration for bearing RCSW buildings at same height.

• Variation of the building width for bearing RCSW buildings has a significant effect on the fundamental natural period of vibration where if the building width increases, the fundamental natural period of vibration decreases.

• Variation of the bay width for bearing RCSW buildings has a pointed effect on the fundamental natural period of vibration, that is when the bay width increases, the fundamental natural period of vibration increases.

• The ratio of the SW area to the building area and the ratio of the SW height to the SW length, have a significant effect on the fundamental natural period of vibration for bearing RCSW buildings.

• The equations of lower bound, T_L, which are obtained from regression analysis, introduces conservative design and thus are the best fit to estimate the fundamental natural period of vibration for the considered systems.

• The available approximate formulas in codes may be incompatible and seemed unsound through appearance of considerable inaccuracies through underestimations or unjustified overestimations of anticipated structural design responses. Thus, realistic enhancement is found essential to improve estimation of T towards enriched, accurate and reliable analysis, design, and evaluation of bearing RCSW structural systems.

• Code Eq. (1) does not consider the effects of number of bays, width of bays, width of building, and presence of middle corridor or core SW. Thus, it probably yields to a highly conservative estimation of T for multi-story bearing RCSW structures with various architectural configurations. However, compared with code Eq. (1), the proposed theoretically established Eq. (15), with similar form as of code Eq. (1), introduces more justifiable values of T in terms of C_t and power of h_n since it is allowing for the effects of extensive additional range of feasibly effective parameters altogether.

• In estimation T of multi-story bearing RCSW structures, code Eq. (2) does not reflect the effects of number of bays, width of bays, width of building, and presence of middle corridors or core SWs. Therefore, it apparently introduces non-conservative values of T, especially when the architectural configurations include middle corridor or core SWs. Though, code Eq. (2) is so perfect to estimate T for multi-story bearing RCSW structures, particularly when architectural configurations do not contain middle corridors or core SWs. Alternatively, in addition to the effects of all parameters that are considered by code Eq. (2), the proposed theoretically resulting Eq. (18) in this study contains the effect of presence of middle corridors or core SWs, and thus may be utilized as apposite alternate for estimation of T for multi-story bearing RCSW structures with relevant accuracy, particularly when such systems include middle corridors or core SWs.

• The recommended Eqs. (15) and (18) presented in this study are developed to be applicable for the estimation of T for multi-story bearing RCSW structures with various architectural configurations along with many other possible effective parameters on T. However, the alternative form of Eq. (18) introduces more reasonable responses than Eq. (15) for most cases of study of bearing RCSW systems. This is due to the fact that of containing more and detailed explicit and implicit active parameters which may influence values of T and consequently other responses.

• The results of the proposed equations in this study agree well with the results of 3D non-linear response TH analysis, and are consistent with the expectation levels with many affecting parameters; therefore, they can be preferred in order to calculate well-grounded seismic responses from design spectra.

• More enhancements are required to improve T formulas of bearing RCSW systems with more consistent and cost-effective responses.

• Regression analysis should be frequently carried out based on more analytical and larger experimental data to include more types of structural systems and taking into account other affecting factors on T such as foundation system, soil type, and seismic hazard parameters and zones.

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