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Full Length Research Paper

A method for lateral static and dynamic analyses of wall-frame buildings using one dimensional finite element

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This study presents an approximate method which is based on the continuum approach and one dimensional finite element method to be used for lateral static and dynamic analyses of wall-frame buildings. In this method, the whole structure is idealized as an equivalent sandwich beam which includes all deformations. The effect of shear deformations of walls is considered and incorporated in the formulation of the governing equations. Initially the differential equations of this equivalent sandwich beam are written; then shape functions and stiffness matrix can be obtained by solving the differential equations. For static and dynamic analysis the lateral forces and masses were applied on the storey levels. Angular frequency and modes were obtained by using system mass and system stiffness matrices. Finally, numerical examples have been solved using MATLAB to verify the presented method. The results of these examples display the agreement between the proposed method existing methods.

Key words: One dimensional finite element, wall-frame buildings, dynamic analysis.

INTRODUCTION

Global analysis, as structural design itself, can be carried out at two levels. An “exact” analysis called exact-relies on a mathematical model as exact as possible and uses a static model which takes into account as many structural elements, material properties geometrical and stiffness characteristics as possible. Taking everything into consideration, however, can result in computational problems. Even using a powerful computers, the problem can be to handled. However, because of the complexity of results for certain problems, they can be difficult to interpret (Zalka, 2000). The lengthy and time-consuming procedure of handling large data can always be a source of inaccuracy and errors. Another disadvantage of this approach may be that the importance of the key structural elements is sometimes hidden inside the great number of input and output data (Zalka, 2000). Therefore, approximate methods have been developed for static and dynamic analyses of buildings. The most widely used approximate methods are those based on the “continuum method”. There are numerous studies in the literature dealing with the continuum method e.g (Basu et al., 1979; Bilyap, 1979; Balendra et al., 1984; Stafford and Crowe, 1986; Nollet and Stafford, 1993; Zalka, 1994; Li and Choo, 1996; Toutanji, 1997; Miranda, 1999; Mancini and Savassi, 1999; Hoenderkamp, 2000; Wang et al., 2000; Hoenderkamp, 2001; Swaddiwudhipong et al., 2001; Hoenderkamp, 2002; Miranda and Reyes, 2002; Zalka, 2002; Potzta and Kollar, 2003; Zalka, 2003; Tarjan and Kollar, 2004; Savassi and Mancini, 2004; Civalek, 2004; Boutin et al., 2005; Reinoso and Miranda, 2005; Georgoussis, 2006; Michel et al., 2006; Rafezy et al., 2007; Civalek, 2007; Kaviani et al, 2008; Laier, 2008; Meftah and Tounsi, 2008; Lee at al., 2008; Bozdogan, 2009; Zalka, 2009; Savassi and Mancini, 2009) .

Rosman (1964) proposed a continuum method for a pair of high rise coupled shear walls. Heidebrecht and Stafford (1973) derived the differential equations of system for buildings with uniform stiffness along the height and obtained closed-form solutions under a uniform and triangular static lateral load distributions. Zalka (2001) derived simple expressions for the circular natural frequencies of wall-frame buildings. Kuang and Ng (2000) considered the problem of doubly asymmetric structures, in which the motion is dominated by shear

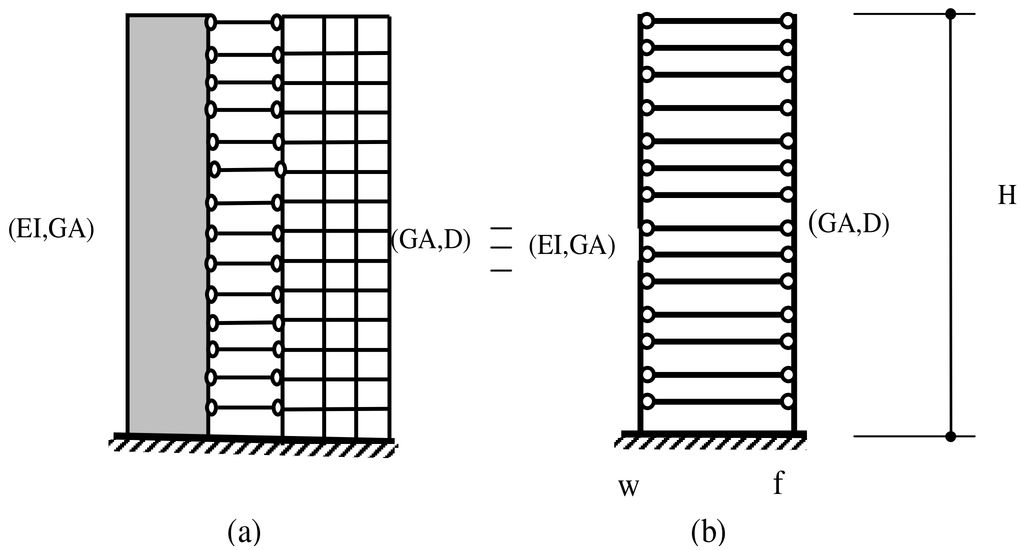

Figure 1. Sandwich beam: (a) Wall-frame (b) Physical model.

walls. For the analysis, the structure is replaced with an equivalent uniform cantilever whose deformation is coupled in flexure and warping torsion. In a recent study by Miranda and Taghavi (2005), an approximate method for estimating the floor acceleration demands in multi-story buildings subjected to earthquake ground motions has been developed. In their paper, the dynamic properties of multistory buildings are approximated by using equivalent continuum model consisting of a flexural cantilever beam and a shear cantilever beam that are assumed to be connected by an infinite number of axially rigid members. The dimensionless parameter is presented which controls the degree of overall flexural and overall shear deformations in the simplified model of the building. In a different paper by Taghavi and Miranda (2005), the accuracy of the methodology is evaluated by comparing the results of the approximate method with the response computed using detailed finite element analysis in the case of the two generic buildings and compared to recorded accelerations in the case of the four instrumented buildings.

Rafezy and Howson (2008) proposed a global approach for the calculation of the natural frequencies of doubly asymmetric, three dimensional, multi bay, multi-storey, wall-frame structures. It is assumed that the primary walls and frames of the original structure run in two original directions and that their properties may vary in a step-wise fashion at one or more storey levels. The structure is therefore divided naturally into uniform segments between changes of section properties. A typical segment is then replaced by an equivalent shear-flexure-torsion coupled beam whose governing differential equations are formulated using continuum approach and posed in the form of a dynamic member stiffness matrix. Bozdogan (2009) proposed the transfer matrix method for static and dynamic analysis of wall-frame buildings. However, this study neglects shear deformation of the shear walls.

With the exception of Savassi and Mancini's (2009), Rafezy and Howson's (2008) and Bozdogan papers (2009), none of the studies allows for step changes of properties along the height of the structure.

In this study, a one dimensional finite element method is suggested for lateral static and dynamic analysis of wall-frame buildings. The following assumptions are made in this study: (1) the behavior of the material is linear elastic, (2) small displacement theory is valid, (3) P-delta effects are negligible, (4) torsional effects are ignored, and (5) masses and lateral loads act at the storey level for static and dynamic analyses.

ANALYSIS

Physical model

High rise buildings, demonstrate neither Timoshenko beam behavior, nor Euler-Bernouilli beam behavior under the horizontal loads (Potzta and Kollar, 2003). The sandwich beam which consists of two Timoshenko beams representing the behavior of the high rise buildings may demonstrate both of the above mentioned behaviors (Figure 1).

Initially, the differential equation of this equivalent shear-flexural beam can be written. The flexural rigidity of the w beam is the sum of the flexural rigidity of shear walls and columns. The shear rigidity of the w beam is the sum of shear rigidity of walls. Meanwhile, the shear rigidity of the f cantilever beam is equal to the sum of shear rigidities of frames and sum of shear rigidities of the connecting beams. The global flexural rigidity of the f cantilever beam structural system can be calculated with

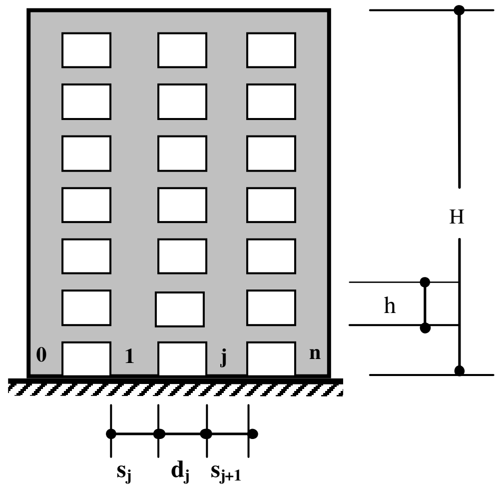

Figure 2. Coupled shear wall.

the help of axial deformation of shear walls and columns.

Storey stiffness matrix for the wall-frame

Under the lateral loads acting at the storey levels, the equation of i.th storey can be written as (Lee et al., 2008) :

$$\frac{d}{dz_i}[(GA)_{wi}(\frac{dy_i}{dz_i}-\psi_{wi})+\frac{d}{dz_i}[(GA)_{fi}(\frac{dy_i}{dz_i}-\psi_{fi})=0 \quad (1)$$

$$\frac{d}{dz_i}[(EI)_{wi}\frac{d\psi_{wi}}{dz_i}]+(GA)_{wi}(\frac{dy_i}{dz_i}-\psi_{wi})=0 \quad (2)$$

$$\frac{d}{dz_i}[(D)_i\frac{d\psi_{fi}}{dz_i}]+(GA)_{fi}(\frac{dy_i}{dz_i}-\psi_{fi})=0 \quad (3)$$

where y_i is the total displacement function, z_i is the vertical axis of the ith storey, ψ_{wi} denotes the rotation of w beam , ψ_{fi} denote rotations of a f beam, EI_i is the total bending rigiditiy of shear the wall and columns and D_i is the global bending rigidity of frame and can be calculated using the equation below:

$$D_i=\sum_{j=1}^{n}EA_j r_j^{\,2} \quad (4)$$

where A_j is the cross sectional area of the j-th shear wall/column, (*n*) is the number of columns and r_j is the distance of the j-th shear wall/column from the center of the cross sections. (GA_{wi}) are the equivalent shear rigidity of walls, (GA_{fi}) are the equivalent shear rigidity of the storey for framework. For frame elements which consists of n columns and n-1 beams, GA_{fi} can be calculated as follows (Bilyap, 1979; Murashev at al., 1972):

$$GA_{fi}=\frac{12\,E}{h_i[1/\sum_1^n I_c/h_i+1/\sum_1^{n-1} I_g/l)} \quad (5)$$

where $\sum I_c/h_i$ represents the sum of moments of inertia of the columns per unit height in i.th storey of the ith frame ~~j~~, and $\sum I_g/l$ represents the sum of moments of inertia of each beam per unit span across one floor of frame j . For coupled shear walls which consist of n walls and n-1 connecting beams, GA_{fi} can be calculated from Equation (6) (Potzta and Kollar, 2003):

$$GA_{fi}=\sum_{j=1}^{n-1}\frac{6\,EI_{bj}[(d_j+s_j)^2+(d_j+s_{j+1})^2]}{d_j^{\,3}(h_i+\dfrac{12\,k\,EI_{bj}}{GA_{bj}\,d_j^{\,2}})} \quad (6)$$

where, h_i is the height of the storey, d_j is the clear span length~~s~~ of the coupling beam, s_j is the jth wall length (Figure 2), EI_{bj} and GA_{bj} represent the flexural rigidity of connecting beam and the shear rigidity of connecting beams, respectively and k is a constant depending on the shape of cross-section of the beams (k =1.2 for rectangular cross-sections).

In the operator notation Equations (1), (2) and (3) can be written as:

$$m^2[(GA)_{wi}+(GA)_{fi}]y_i-m(GA)_{wi}\psi_{wi}-m(GA)_{fi}\psi_{fi}=0 \quad (7)$$

$$m(GA)_{wi} y_i + [m^2 (EI)_{wi} - (GA)_{wi}]\psi_{wi} = 0 \tag{8}$$

$$m(GA)_{fi} y_i + [m^2 (D)_i - (GA)_{fi}]\psi_{fi} = 0 \tag{9}$$

where $m = \frac{d}{dz_i}$

Since

$$\left|L(m)\right| = m^6 (EI)_{wi} D_i [(GA)_{wi} + (GA)_{fi}] - m^4 (GA)_{fi} (GA)_{wi} [D_i + (EI)_{wi}] = 0$$

by Cramer's rule $y_i, \psi_{wi}, \psi_{fi}$ satisfy

$$L(m) y_i = 0 \tag{10}$$

$$L(m)\psi_{wi} = 0 \tag{11}$$

$$L(m)\psi_{fi} = 0 \tag{12}$$

Solving these equations, we get

$$y_i(z_i) = c_1 + c_2 z_i + c_3 z_i^2 + c_4 z_i^3 + c_5 \cosh(a_i z_i) + c_6 \sinh(a_i z_i) \tag{13}$$

$$\psi_{wi}(z_i) = c_7 + c_8 z_i + c_9 z_i^2 + c_{10} z_i^3 + c_{11} \cosh(a_i z_i) + c_{12} \sinh(a_i z_i) \tag{14}$$

$$\psi_{fi}(z_i) = c_{13} + c_{14} z_i + c_{15} z_i^2 + c_{16} z_i^3 + c_{17} \cosh(a_i z_i) + c_{18} \sinh(a_i z_i) \tag{15}$$

where a_i can be calculated as follows

$$a_i = \frac{[D_i + (EI_{wi})](GA)_{wi}(GA)_{fi}}{[(GA)_{wi} + (GA)_{fi}](EI)_{wi} D_i} \tag{16}$$

After rearrangements, the constants c_1, c_2 c_3, c_4, c_5 and c_6,, can be taken as independent constants, thus total displacement function and rotation angle can be obtained as follows:

$$y_i(z_i) = c_1 + c_2 z_i + c_3 z_i^2 + c_4 z_i^3 + c_5 \cosh(a_i z_i) + c_6 \sinh(a_i z_i) \tag{17}$$

$$\psi_{wi}(z_i) = c_2 + 2c_3 z_i + (3z_i^2 + \frac{6EI_{wi}}{GA_{wi}})c_4 + R_{wi} c_5 \sinh(a_i z_i) + R_{wi} c_6 \cosh(a_i z_i) \tag{18}$$

$$\psi_{fi}(z_i) = c_2 + 2c_3 z_i + (3z_i^2 + \frac{6D_i}{GA_{fi}})c_4 + R_{fi} c_5 \sinh(a_i z_i) + R_{fi} c_6 \cosh(a_i z_i) \tag{19}$$

R_{wi}, and R_{fi}, can be calculated from Equations (20) and (21) as shown below:

$$R_{wi} = \frac{(GA)_{wi} a_i}{[(GA)_{wi} - a_i^2 (EI)_{wi}]} \tag{20}$$

$$\left|L(m)\right| = m^6 (EI)_{wi} D_i [(GA)_{wi} + (GA)_{fi}] - m^4 (GA)_{fi} (GA)_{wi} [D_i + (EI)_{wi}] = 0 \tag{21}$$

At the initial point of the storey for z_i=0, Equations (17), (18) and (19) can be re-written as:

$$y_i(0) = c_1 + c_5 \tag{22}$$

$$\psi_{wi}(0) = c_2 + \frac{6EI_{wi}}{GA_{wi}} c_4 + R_{wi} c_6 \tag{23}$$

$$\psi_{fi}(0) = c_2 + \frac{6D_i}{GA_{fi}} c_4 + R_{fi} c_6 \tag{24}$$

At the end point of the storey for z_i=h_i Equations (17), (18) and (19) can be written as:

$$y_i(h_i) = c_1 + c_2 h_i + c_3 h_i^2 + c_4 h_i^3 + c_5 \cosh(a_i h_i) + c_6 \sinh(a_i h_i) \tag{25}$$

$$\psi_{wi}(h_i) = c_2 + 2c_3 h_i + (3h_i^2 + \frac{6EI_{wi}}{GA_{wi}})c_4 + R_{wi} c_5 \sinh(a_i h_i) + R_{wi} c_6 \cosh(a_i h_i) \tag{26}$$

$$\psi_{fi}(h_i) = c_2 + 2c_3 h_i + (3h_i^2 + \frac{6D_i}{GA_{fi}})c_4 + R_{fi} c_5 \sinh(a_i h_i) + R_{fi} c_6 \cosh(a_i h_i) \tag{27}$$

Equation (28) shows the matrix form of Equations (22), (23), (24), (25), (26) and (27).

$$\begin{bmatrix} y_i(0) \\ \psi_{wi}(0) \\ \psi_{fi}(0) \\ y_i(h_i) \\ \psi_{wi}(h_i) \\ \psi_{fi}(h_i) \end{bmatrix} = B_i \begin{bmatrix} c_1 \\ c_2 \\ c_3 \\ c_4 \\ c_5 \\ c_6 \end{bmatrix} \tag{28}$$

With the help of Equations (17), (18) and (19), bending moment of the w beam (M_{wi}), bending moment of the f beam (M_{fi}) and the total shear force (V) can be obtained as follows:

$$M_{wi}(z_i)=(EI)_{wi}\frac{d\psi_w}{dz_i}=(EI)_w[2c_3+6c_4z_i+c_5a_iR_{wi}\cosh(a_iz_i)+c_6a_iR_{wi}\sinh(a_iz_i)] \quad (29)$$

$$M_{fi}(z_i)=D_i\frac{d\psi_{fi}}{dz_i}=D_i[2c_3+6c_4z_i+c_5a_iR_{fi}\cosh(a_iz_i)+c_6a_iR_{fi}\sinh(a_iz_i)] \quad (30)$$

$$V_i(z_i)=(EI)_{wi}\frac{d^2\psi_{wi}}{dz_i^2}+D_i\frac{d^2\psi_{fi}}{dz_i^2}$$
$$=[6(EI)_{wi}+6D_i]c_4+[R_{wi}(EI)_{wi}+R_{fi}D_i]a_i^2\sinh(a_iz_i)c_5+[R_{wi}(EI)_{wi}+R_{fi}D_i]a_i^2\cosh(a_iz_i)c_6 \quad (31)$$

At the initial point of the storey for $z_i=0$ Equations (29), (30) and (31) can be written as:

$$M_{wi}(0)=(EI)_{wi}\frac{d\psi_w}{dz_i}=(EI)_w[2c_3+c_5a_iR_{wi}] \quad (32)$$

$$M_{fi}(0)=D_i\frac{d\psi_{fi}}{dz_i}=D_i[2c_3+c_5a_iR_{fi}] \quad (33)$$

$$V_i(0)=(EI)_{wi}\frac{d^2\psi_{wi}}{dz_i^2}+D_i\frac{d^2\psi_{fi}}{dz_i^2}=[6(EI)_{wi}+6D_i]c_4+[R_{wi}(EI)_{wi}+R_{fi}D_i]a_i^2c_6 \quad (34)$$

At the end point of the storey for $z_i=h_i$ Equations (29), (30) and (31) can be written as:

$$M_{wi}(h_i)=(EI)_{wi}\frac{d\psi_w}{dz_i}$$
$$=(EI)_w[2c_3+6c_4h_i+c_5a_iR_{wi}\cosh(a_ih_i)+c_6a_iR_{wi}\sinh(a_ih_i)] \quad (35)$$

$$M_{fi}(h_i)=D_i\frac{d\psi_{fi}}{dz_i}$$
$$=D_i[2c_3+6c_4h_i+c_5a_iR_{fi}\cosh(a_ih_i)+c_6a_iR_{fi}\sinh(a_ih_i)] \quad (36)$$

$$V_i(h_i)=(EI)_{wi}\frac{d^2\psi_{wi}}{dz_i^2}+D_i\frac{d^2\psi_{fi}}{dz_i^2}$$
$$=[6(EI)_{wi}+6D_i]c_4+[R_{wi}(EI)_{wi}+R_{fi}D_i]a_i^2\sinh(a_ih_i)c_5+[R_{wi}(EI)_{wi}+R_{fi}D_i]a_i^2\cosh(a_ih_i)c_6 \quad (37)$$

Equation (38) represents the matrix form of equations (32), (33), (34), (35),(36) and (37) as given below.

$$\begin{bmatrix} V_i(0) \\ M_{wi}(0) \\ M_{fi}(0) \\ -V_i(h_i) \\ -M_{wi}(h_i) \\ -M_{fi}(h_i) \end{bmatrix} = A_i \begin{bmatrix} c_1 \\ c_2 \\ c_3 \\ c_4 \\ c_5 \\ c_6 \end{bmatrix} \quad (38)$$

When vector c is solved by implementing Equation (28) and substituted in Equation (38) and (39) would be obtained:

$$\begin{bmatrix} V_i(0) \\ M_{wi}(0) \\ M_{fi}(0) \\ -V_i(h_i) \\ -M_{wi}(h_i) \\ -M_{fi}(h_i) \end{bmatrix} = A_i * B_i^{-1} \begin{bmatrix} y_i(0) \\ \psi_{wi}(0)) \\ \psi_{fi}(0) \\ y_i(h_i) \\ \psi_{wi}(h_i) \\ \psi_{fi}(h_i) \end{bmatrix} = k_i \begin{bmatrix} y_i(0) \\ \psi_{wi}(0)) \\ \psi_{fi}(0) \\ y_i(h_i) \\ \psi_{wi}(h_i) \\ \psi_{fi}(h_i) \end{bmatrix} \quad (39)$$

where k_i represents the storey stiffness matrix. For static and dynamic analysis the n- storey structure is discretized into n storey.

Dynamic analysis

The system stiffness matrix obtained from storey stiffness matrices (32) can be used for the dynamic analysis of wall-frame structures. The mass system matrix is formed by using lumped mass model as:

$$M=\begin{bmatrix} 0 & & & & & & & & & & & & & \\ & 0 & & & & & & & & & & & & \\ & & 0 & & & & & & & & & & & \\ & & & m_1 & & & & & & & & & & \\ & & & & 0 & & & & & & & & & \\ & & & & & 0 & & & & & & & & \\ & & & & & & . & & & & & & & \\ & & & & & & & . & & & & & & \\ & & & & & & & & . & & & & & \\ & & & & & & & & & . & & & & \\ & & & & & & & & & & . & & & \\ & & & & & & & & & & & m_n & & \\ & & & & & & & & & & & & 0 & \\ & & & & & & & & & & & & & 0 \end{bmatrix}_{(n+1)*(n+1)}$$

where, m_i is the mass of the i. th storey .

By using K and M matrices, the frequency equation can be written as:

$$[K-\omega^2M]\{\phi\}=0 \quad (41)$$

The boundary conditions of the wall-frame system are:

1. $y_{base}=0$ (42)

2. $\psi_{wbase}=0$ (43)

3. $\psi_{fbase}=0$ (44)

The values of ω, which set the frequency equation to zero, are the natural frequencies of the wall-frame structure. The angular frequencies and relevant modes

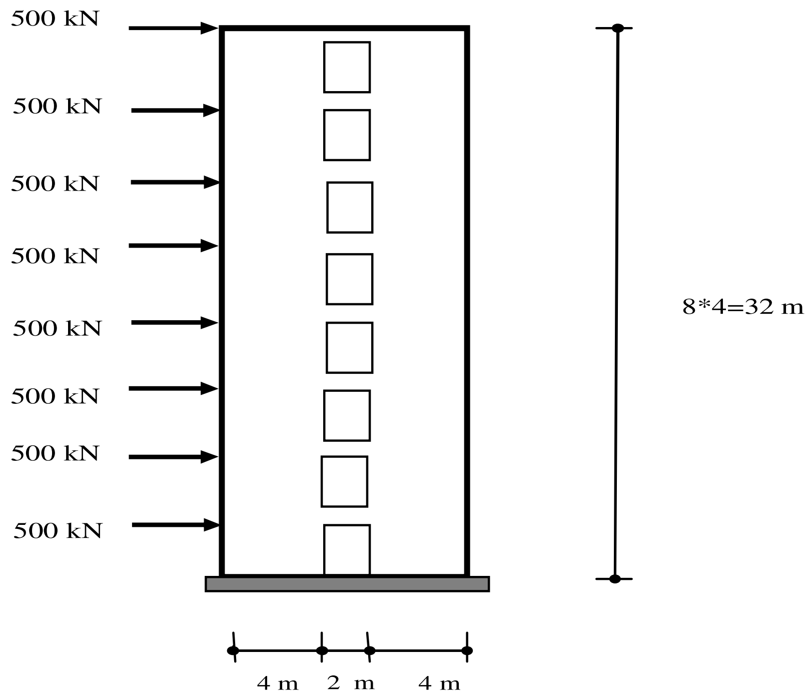

Figure 3. 8 Storey coupled shear wall (example 1).

are found with the help of the frequency Equation (41). The effective mass (M_r) and participation factor (Γ) can be found as:

$$M_{ri} = \frac{(\phi_i^T * M * I)^2}{(\phi_i^T * M * \phi_i)} \quad (45)$$

$$\Gamma_i = \frac{\phi_i^T * M * I}{(\phi_i^T * M * \phi_i)} \quad (46)$$

With the help of the acceleration and the displacement spectrums, obtained from an earthquake record or design spectrum from codes, the displacement and internal forces are found by using the effective mass and the participation factor.

PROCEDURE OF COMPUTATION

A program that considers the method presented in this study as basis, has been implemented in MATLAB and the coding/ programming steps are presented below:

1. The equivalent rigidities of each storey are calculated by using the geometric and material properties of the structure.
2. Stiffness matrices are calculated for each storey by using equivalent rigidities.
3. System stiffness matrix is obtained with the help of storey stiffness matrices.
4. For static analysis the lateral displacements can be found by using the well known equation: F=KxD
5. For dynamic analysis, system mass matrix is obtained using Equation (40).
6. The angular frequencies and mode shapes are found from using (Equation 41).
7. With the help of the acceleration and the displacement spectrums, obtained from an earthquake record or design spectrum from codes, the displacement and internal forces are found by using the effective mass (Equation 45) and the participation factor (Equation 46).

Numerical examples

Four numerical examples are considered to validate the proposed method. The results are compared with those given in the literature.

Example 1

A two- dimensional coupled shear wall having eight storey with 4.0 m height (Figure 3) subjected to lateral loads is analyzed by the proposed method and compared with the literature (Paknahad et al., 2007). The shear walls with 4.0 m width are connected by rigid beams. The height of the connecting beams is equal to 0.8 m and the width (thickness) of the system is equal to 0.4 m. The modulus of elasticity and Poisson's ratio are 20 kN/mm^2 and 0.25, respectively. The lateral displacements are calculated and compared with those in literature (Paknahad at al., 2007) (Table 1 and Figure 4).

Example 2

A 8-storey wall-frame system (Figure 5) is analyzed as an example. The section properties of columns, shear walls and beams are

Table 1. Comparison of Lateral displacements in example 1.

Floor no.	Paknahad et al. (2007)	Presented method	Diff. (%)
2	0.74	0.68	-8.1
4	1.98	1.85	-6.57
6	3.28	3.10	-5.49
8	4.51	4.28	-5.10

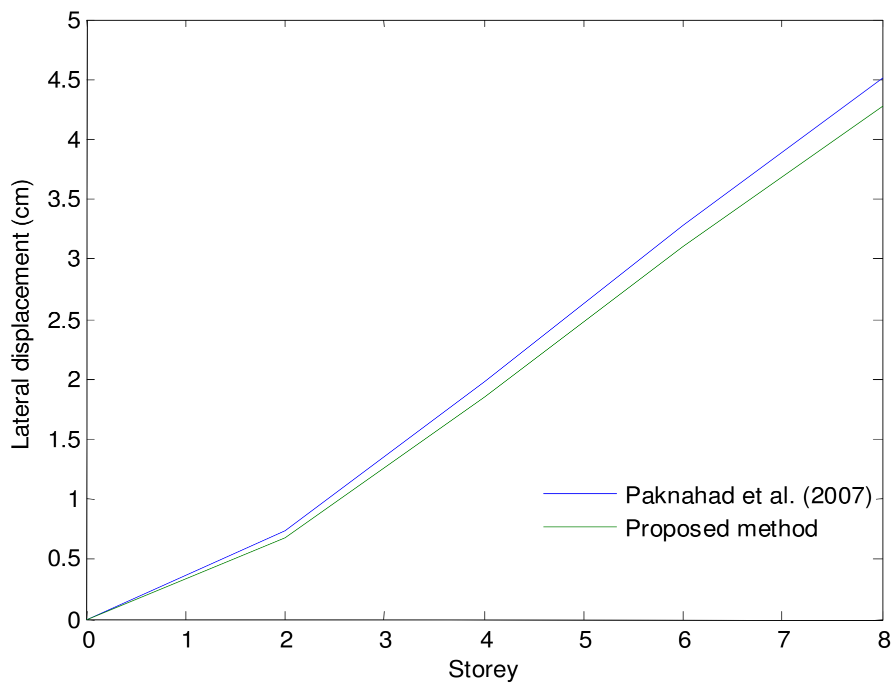

Figure 4. Lateral displacements (example 1).

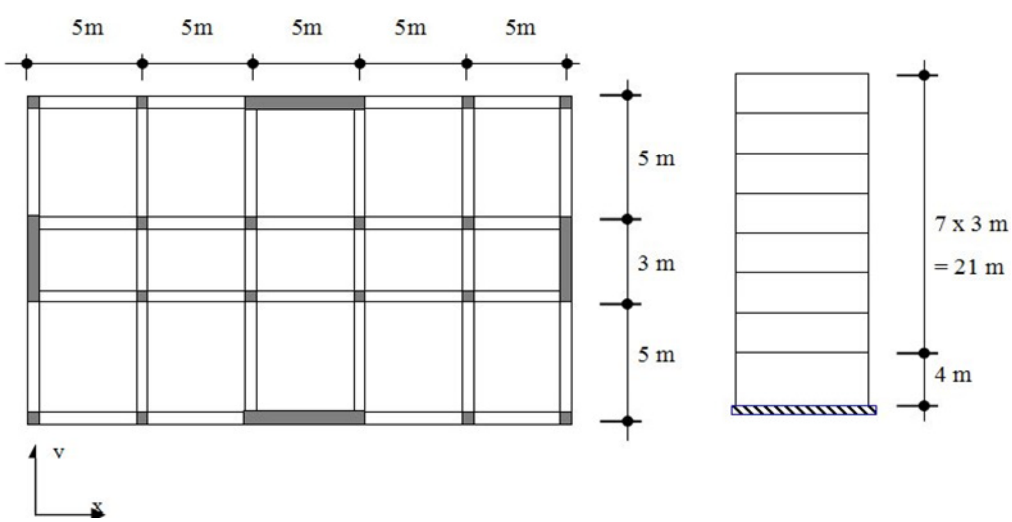

Figure 5. Wall-frame structure.

given in Tables 2 and 3. The young modulus is E=2.85*10^7 kN/m^2. The natural periods in y direction are calculated by this method and compared with those found in the literature (Ozmen et al., 2005) in Table 4.

Table 2. The section properties of shear wall and columns.

Floor	Shear wall thicknes (m)	Columns
6-8	0.25	0.40 m*0.40 m
3-5	0.25	0.45 m*0.45 m
1-2	0.35	0.50 m*0.50 m

Table 3. The section properties of beams.

Beam	b_w (m)	b (m)	t (m)	h (m)
Outer beam	0.25	0.70	0.14	0.60
Interior beam	0.25	1.00	0.14	0.60

Table 4. Natural periods for example 2 (s).

Mode	Özmen et al. (2005)	Proposed method	Difference (%)
1	0.696	0.702	0.86
2	0.211	0.214	1.42
3	0.102	0.109	6.86
4	0.061	0.068	11.48
5	0.041	0.048	17.07
6	0.031	0.038	22.58
7	0.025	0.032	28.00
8	0.022	0.030	36.36

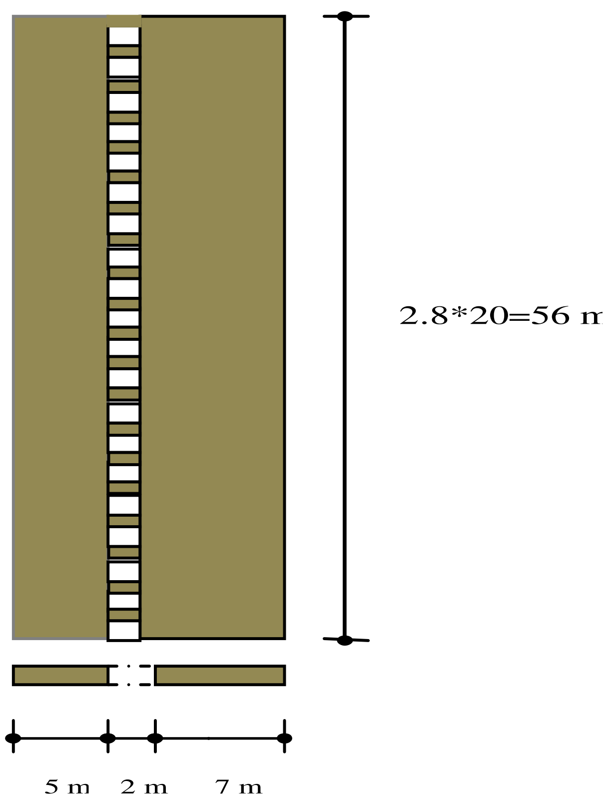

Figure 6. 20 storey coupled shear wall.

Example 3

A 20-storey coupled shear wall system (Figure 6) is analyzed as an example. The numerical data for the coupled shear wall are given in Table 5. The first four natural frequencies are calculated by this method and compared with those found in the literature (Takabatake, 2010) in Table 6. In the literature Takabatake (2010) used the finite element program NASTRAN which use the shell element in analysis.

Example 4

A doubly symmetrical system (Figure 7) is analyzed as an example. The building is stiffened in the y direction by solid walls while in the z direction by two solid walls at the symmetry plane and by two coupled shear walls arranged symmetrically. The geometric and material characteristics are given in Table 7. The first natural periods and the seismic base shear force are calculated according to Eurocode 8 by this method and compared with those found by using continuum model in the literature (Tarjan and Kollar, 2004) (Tables 8 and 9).

RESULTS

In this study four numerical examples are considered to validate the proposed method. Example 1 results compared with the work of (Paknahad et al., 2007) show differences greater than 5%. Example 2 shows good agreement with those of Ozmen et al. (2005) for the first

Table 5. The numerical data for the coupled shear wall (example 3).

Geometric and structural properties of coupled shear wall	
Young modulus	$3.6*10^{10}$ N/m^2
Mass density	$2.4*10^3$ kg/m^3
Poisson ratio	0.15
Total storeys	20
Total height	56
Thickness of left side wall	0.3
Width of left side wall	5.0
Thickness of Right Side Wall	0.3
Width of Right Side Wall	7.0
Thickness of Lintel	0.3
Height of Lintel	0.4
Length of Lintel	2.0

Table 6. Natural frequencies for example 3 (rd/s).

Mode	NASTRANTakabatake (2010)	Proposed method	Difference (%)
1	13.09	13.15	0.46
2	55.55	56.66	2.00
3	129.00	133.03	3.12
4	224.90	231.40	2.89

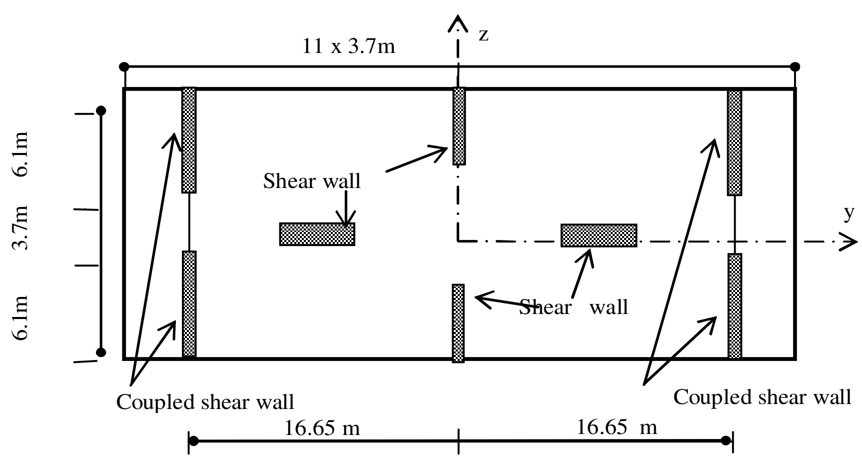

Figure 7. Floor plan of a 28-storey building (example 4).

and second modes but for the remaining modes comparison shows high disagreement. On the other hand, the third and fourth examples show good agreement with literature. When the number of storey increases, the suitability of the method also increasing due to the assumptions of the method.

Conclusions

In this study, an approximate method based on the continuum approach and one dimensional finite element method for lateral static and dyamic analysis of buildings is presented. In this method, the whole structure is idealized as a sandwich cantilever beam, which includes all the deformations. The numerical examples presented show that results obtained from the proposed method are in a good agreement with the classical finite element and the analytical solution developed in literature. The validity of the procedure is dependent on the height of building. The proposed method is simple and accurate enough to be used both at the concept design stage and for final

Table 7. Geometric and material characteristics of shear walls (example 1).

Number of stories	28
Story height	2.97 m
Total height	83.2 m
Mass/unit height	280640 kg/m
Young's modulus of walls	$1.95*10^{7}$ kN/m ^{2}
Young's modulus of beams	$2.3*10^{7}$ kN/m^{2}
Shear modulus of beams	$9.58*10^{6}$ kN/m^{2}
Area of beams	0.07 m^{2}
Moment of inertia of beams	$5.79*10^{-4}$ m^{4}
Area of walls	1.4640 m^{2}
Moment inertia of walls	4.5396 m^{4}

Table 8. Natural periods for example 4.

	Natural periods (sec.)		
	ETABS	**Tarjan and Kollar**	**Presented method**
1.Period of z direction	6.888	7.197	6.991

Table 9. Base Shear forces for example 1.

	Base shear force (kN)		
	ETABS (Tarjan and Kollar)	**Tarjan and Kollar**	**Presented method**
Base shear force in z direction	1481.45 kN	1310.30 kN	1438.57 kN

analyses of high rise buildings and takes less computational time than the classical Finite Element method.

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