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Research Article

EFFECTS OF INFILL WALLS ON FREE VIBRATION CHARACTERISTICS OF MULTI-STOREY FRAMES USING DYNAMIC STIFFNESS METHOD

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ABSTRACT

This study aims to obtain exact natural frequencies and mode shapes of infilled multi-storey frames using single variable shear deformation theory (SVSDT) which considers parabolic transverse shear stress distribution across the cross-section. The effects of infill walls on free vibration characteristics are investigated for different frame models such as one storey infilled, soft storey and fully infilled. The infill walls are modeled using equivalent diagonal strut approach. Natural frequencies are calculated via dynamic stiffness formulations for different wall thickness values. The results of SVSDT are tabulated with Euler-Bernoulli beam theory (EBT) and Timoshenko beam theory (TBT) results. Additionally, finite element solutions are presented to verify the natural frequencies that obtained from dynamic stiffness formulations. The results show that SVSDT can be used effectively for free vibration analysis of infilled frame structures by using dynamic stiffness formulations. The numerical analyses show that the effects of shear deformation and rotation inertia become observable for higher modes of infilled frame structures. It is seen from the results that ignoring effects of infill walls may cause significant errors on calculation of natural frequencies of frames.
Keywords: Infilled frame, mode shape, natural frequency, single variable shear deformation theory.

1. INTRODUCTION

The calculation of natural frequencies and plotting mode shapes of frames are significant parts of dynamic analysis for safe design against earthquakes or other dynamic excitations. Most of the buildings are constructed with infill walls because of architectural needs or concerns. However, the behavior of infilled frames are complicated when compared with bare ones and effects of infill walls on static and dynamic analysis are generally omitted. Thus, mathematical modeling of infill walls has been an important research area for recent decades. The first study about effects of infill walls was performed by Polyakov [1]. Holmes [2] modeled the infill using an equivalent pin-jointed diagonal strut which has a width of one-third of the infill with the assumption of strut material is same as infill. The variations of equivalent strut method were proposed by several researchers [3-6]. The equivalent diagonal strut approach (macro-model) is specified in codes and standards [7-9]. Besides macro-modeling of infill walls, various studies about micro-modeling of infill walls based on finite element method (FEM) can be found in open literatures [10-15].

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The infill walls with openings for windows and doors can be used due to architectural reasons. Asteris et al.[16] reviewed in detail different macro-models of infill walls with different opening percentages and locations. Asteris et al. [17] studied response of infilled RC frames by macro-modeling approach considering openings of infill. Asteris et al. [18] presented the effects of infill walls including openings on frame behavior with detailed advantages and disadvantages of micro-modeling and macro-modeling approaches.

Most of the studies about infilled frames aimed to perform linear or nonlinear static analysis. The natural frequencies of infilled frames were calculated in the limited studies. Thiruvengadam [19] investigated first three natural frequencies of different plane frames using finite element solutions. It was stated that infill walls decrease natural periods of frames. Chaker and Cherifati [20] measured vibration frequencies of an infilled frame building and a bare frame building. It was mentioned that fundamental period of infilled frame building is much smaller than bare frame building. Thambiratnam [21] calculated the fundamental period of an one-bay one-storey frame via FEM considering openings of infill. Tamboli and Karadi [22] performed seismic analysis of 3D frames using equivalent diagonal strut method via ETABS. Manju [23] tabulated natural periods of infilled multi-storey buildings using ETABS. Beiraghi [24] presented fundamental period of moment resisting steel frame buildings using equivalent diagonal strut approach via finite element software ETABS.

Crowley and Pinho [25] investigated period-height relationship of European buildings. It was stated that the fundamental period of infilled structures can be computed by multiplying number of storeys by coefficients. Crowley and Pinho [26] presented analytical period-height relationship for uncracked infilled RC buildings. Amanat and Hoque [27] performed a computational analysis on fundamental period of infilled frames by means of 3D finite element analysis. Asteris et al. [28] studied fundamental period of multi-storey RC frames using finite element solutions. Asteris et al. [29] presented effects of soil-structure interaction (SSI), span length, number of spans, infill walls with openings and soft storey on fundamental period of multi-storey frames. Asteris et al. [30] investigated infill effects on first natural period of multi-storey frames with vertical irregularities that can be arised because of architectural designs. Asteris et al. [31] obtained fundamental period of multi-bay multi-storey RC frames considering different opening percentages. Asteris et al. [32] reviewed and discussed in-plane/out-of-plane macromodels of infilled structures. A fiber based macromodel was presented.

The exact natural frequency values can only be obtained using distributed parameter model. However, analysis procedure of FEM that provides approximate solutions is very simple in accordance with distributed parameter model solutions. Various studies about vibration analysis of frames using FEM were presented [33-38].

Limited number of studies about exact vibrations of frames were performed. Mei [39] obtained natural frequencies of single-storey frames using EBT and wave vibration method. Labib et al. [40] investigated free vibrations of multiple cracked frames via dynamic stiffness formulation using Euler-Bernoulli beams. Lien and Hao [41] determined the shape functions of cracked beams and applied to free vibrations of cracked frames via EBT. The wave propagation approach was applied to vibrations of a spatial frame via EBT by Mei and Sha [42]. Bozyigit and Yesilce [43] performed free vibration analysis of three different bare frames using SVSDT.

The dynamic stiffness method (DSM) is used effectively for vibrations of beam-like structures, plates and shells. The DSM provides exact results by using exact mode shapes [44]. In recent years, the DSM was applied to vibration analysis of different type of structures in many studies [45-61].

It is a known issue that natural frequencies which calculated using EBT are overestimated. Therefore, rotational inertia and shear deformation effects are considered by using TBT for more accurate results. However, a shear coefficient factor is mandatory for TBT to decrease the error of assuming constant shear stress distribution along cross-section [62]. From this point of view, the researchers studied on high order shear deformation theories based on realistic shear stress distribution of cross-sections [63-66]. Li et al. [67] investigated free vibrations of laminated beams under different boundary conditions using various high-order beam theories. Ghugal and Shimpi [68] presented results of TBT and high order beam theories comparatively for isotropic and anisotropic laminated beams. A refined plate theory that consists of shearing and bending components of deflections was presented by Shimpi [69]. Shimpi et al. [70] studied on novel displacement based first-order shear deformation theories for plate bending. The use of high-order beam theories are not effectively applicable for vibration analysis of frame structures as formulations become complicated and time-consuming. Shimpi et al. [71] presented a new SVSDT that based on unconstant shear stress distribution on the cross-section of beams.

In this study, the first three natural frequencies of a three-storey fixed-supported frame model are calculated for various infill wall locations using SVSDT as well as EBT and TBT for comparison. The infill walls are modeled using pin-jointed diagonal struts that vibrate only in axial directions. The mode shapes are plotted. The results of DSM are compared with FEM solutions of commercial software ANSYS. The effects of locations of infill walls on dynamic characteristics of multi-storey frames are revealed.

2. RESEARCH SIGNIFICANCE

The novelty of this study is based on showing the effects of infill wall location on the natural frequencies of multi-storey plane frames. For the one storey infilled frames, three different wall locations are considered. Moreover, fully infilled and soft-storey frame models are investigated. The effectiveness of SVSDT with dynamic stiffness formulations on natural frequency calculation of infilled frames is proved. One of the important characteristic of SVSDT is providing EBT results by ignoring the shear deformation related terms of governing equation of motion unlike TBT.

3. MODEL, THEORY AND FORMULATION

The model of this study is a single-bay three-storey planar frame. Several combinations of infill wall locations are considered to reflect the effects of infilled storeys. The infill walls are modeled by means of equivalent diagonal strut approach. The elastic modulus and thickness of strut are considered to be equal to infill's. The width of the equivalent strut is obtained using Eq. (1) [8].

$$\lambda = \sqrt[4]{\frac{E_{inf} t_{inf} \ sin(2\varphi)}{4E_{fr} I_c h_{inf}}} \tag{1}$$

where λ is a coefficient to calculate the equivalent width of diagonal strut, E_{inf} is elastic modulus of infill material, E_{fr} is elastic modulus of frame elements, t_{inf} and h_{inf} are thickness and height of infill wall, φ is angle between equivalent diagonal strut and horizontal plane, I_c is moment of inertia of columns. The equivalent width of the strut is defined in Eq.(2) by using λ.

$$\alpha = 0.175(\lambda H_c)^{-0.4} \, r_{inf} \tag{2}$$

where α is width of the strut and r_{inf} is length of the strut.

The parameters about infill wall given in Eqs.(1) – (2) are presented in Figure 1.

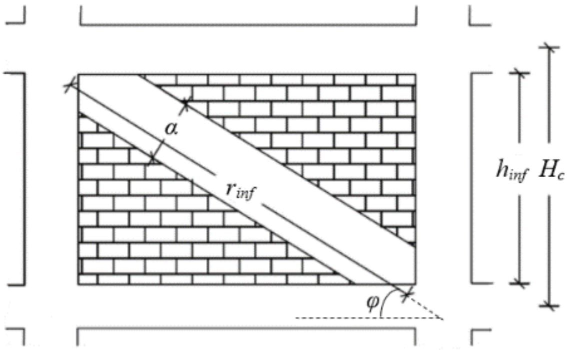

Figure 1. Parameters of equivalent strut technique for an infill wall

The schematic view and degrees of freedom of bare frame are presented in Figure 2 where X and Y represent global axes of frame, x and y represent local axes of frame members, H_c and L_b represent height of the columns and length of the beams, respectively.

One storey infilled frame models are used to reflect the effects of infills on different storeys (Figures.3 (a)-(b)-(c)). Figures.3(d) represents the mathematical model of first storey infilled frame. Figure.3(e) and Figure.3(f) are mathematical models of second storey infilled and third storey infilled frames, respectively. Beside one storey infilled frame models, soft storey and fully infilled frame models are considered in this study (Figures.4 (a)-(b)). Figure.4(c) and Figure.4(d) are mathematical models of soft storey and fully infilled frames, respectively.

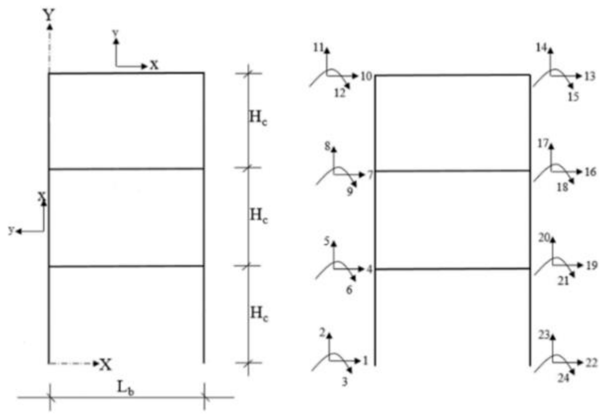

Figure 2. Schematic view of bare frame and degrees of freedom

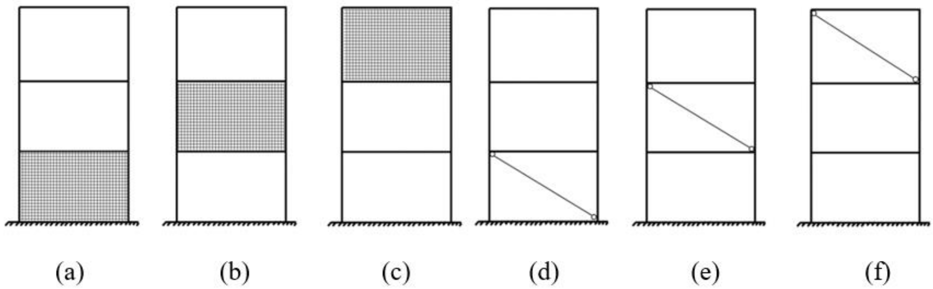

Figure 3. One storey infilled frames and mathematical models

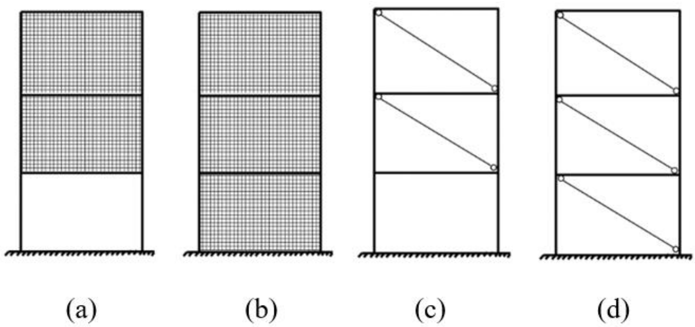

Figure 4. Soft storey and fully infilled frames and mathematical models

Some assumptions are considered for convenience as follows:

1) The cross-sections of frame members and equivalent diagonal struts are uniform.
2) The materials of frame members and infill walls are isotropic.
3) The frame members and equivalent diagonal struts behave linear elastic.
4) The damping is neglected.

The transverse displacement function of a member is defined according to SVSDT by Shimpi et al. [71] as:

$$y^S = y_b + y_s \tag{3}$$

where y^S is total transverse displacement, y_b and y_s are bending displacement component and shearing displacement components, respectively. The governing equation of motion of a beam according to SVSDT is given in Eq.(4) [71]:

$$EI\frac{\partial^4 y_b}{\partial x^4} - \frac{\bar{m}I}{A}\left(1+\frac{12(1+\mu)}{5}\right)\frac{\partial^4 y_b}{\partial x^2 \partial t^2} + \bar{m}\frac{\partial^2 y_b}{\partial t^2} + \frac{\bar{m}^2 I}{A^2 E}\frac{12(1+\mu)}{5}\frac{\partial^4 y_b}{\partial t^4} = 0 \tag{4}$$

where A and μ represent cross-sectional area and Poisson's ratio, respectively. y_b is derived from the solution Eq. (4). Assuming $y_b(x,t)= y_b(x)e^{i\omega t}$ and applying separation of variables method, Eq.(5) is obtained where ω is angular natural frequency.

$$A_0\frac{d^4y_b}{dz^4}+B_0\omega^2\frac{d^2y_b}{dz^2}-C_0\omega^2y_b(z)+D_0\omega^4y_b(z)=0$$

$$A_0=\frac{EI}{L^4};B_0=-\frac{\bar{m}I}{AL^2}\left(1+\frac{12(1+\mu)}{5}\right);C_0=\bar{m};D_0=\frac{\bar{m}^2I\left(\frac{12(1+\mu)}{5}\right)}{A^2E};z=\frac{x}{L} \tag{5}$$

Eq.(6) is assumed as solution of $y_b(z)$:

$$y_b(z)=\{D\}e^{ikz} \tag{6}$$

By substituting Eq. (6) into Eq. (5), $y_b(z)$ is obtained as follows:

$$y_b(z)=(D_1e^{ik_1z}+D_2e^{ik_2z}+D_3e^{ik_3z}+D_4e^{ik_4z}) \tag{7}$$

The bending component of slope function is given in Eq. (8):

$$\frac{dy_b}{dz}=(ik_1D_1e^{ik_1z}+ik_2D_2e^{ik_2z}+ik_3D_3e^{ik_3z}+ik_4D_4e^{ik_4z}) \tag{8}$$

According to SVSDT, the internal force functions are given in Eqs. (9)-(10) [71].

$$M^S(z)=-\frac{EI}{L^2}\frac{d^2y_b}{dz^2} \tag{9}$$

$$Q^S(z)=-\frac{EI}{L^3}\frac{d^3y_b}{dz^3}-\frac{\bar{m}I\omega^2}{AL}\frac{dy_b}{dz} \tag{10}$$

where $M^S(z)$ and $Q^S(z)$ are bending moment function and shear force function, respectively. Eqs. (9)-(10) can be rewritten as Eqs.(11)-(12) using Eq. (7):

$$M^S(z)=(Hk_1^2D_1e^{ik_1z}+Hk_2^2D_2e^{ik_2z}+Hk_3^2D_3e^{ik_3z}+Hk_4^2D_4e^{ik_4z}) \tag{11}$$

$$Q^S(z)=(Jik_1^3-Kik_1)D_1e^{ik_1z}+(Jik_2^3-Kik_2)D_2e^{ik_2z}+(Jik_3^3-Kik_3)D_3e^{ik_3z}+(Jik_4^3-Kik_4)D_4e^{ik_4z} \tag{12}$$

where

$$K=\left(\bar{m}I\omega^2\right)/(AL),H=EI/L^2,J=EI/L^3$$

The displacement function due to shearing $y_s(z)$ and total displacement $y^S(z)$ function are written in Eqs. (13) and (14), respectively.

$$y_s=T\left(-H\frac{d^2y_b}{dz^2}-Py_b(z)\right) \tag{13}$$

$$y^S=\left(THk_1^2-TP+1\right)D_1e^{ik_1z}+\left(THk_2^2-TP+1\right)D_2e^{ik_2z}+\left(THk_3^2-TP+1\right)D_3e^{ik_3z}+\left(NHk_4^2-TP+1\right)D_4e^{ik_4z} \tag{14}$$

where

$$T=\frac{12(1+\mu)}{5AE};P=\left(\bar{m}I\omega^2/A\right)$$

The total slope function is obtained as assembly of $\frac{dy_b}{dz}$ and $\frac{dy_s}{dz}$:

$$\frac{dy^S}{dz}=\left(ik_1+TJik_1^3-TRik_1\right)D_1e^{ik_1z}+\left(ik_2+TJik_2^3-TRik_2\right)D_2e^{ik_2z}+\left(ik_3+TJik_3^3-TRik_3\right)D_3e^{ik_3z}+\left(ik_4+TJik_4^3-TRik_4\right)D_4e^{ik_4z} \tag{15}$$

where $R=P/L$

The axial displacement function and axial force function are obtained by solving the equation of motion of a beam in free axial vibration [72] given in Eq. (16):

$$AE\frac{\partial^2 u(x,t)}{\partial x^2} - \bar{m}\frac{\partial^2 u(x,t)}{\partial t^2} = 0 \tag{16}$$

where *u(x,t)* represents axial displacement function. Assuming $u(x,t)=u(x)e^{i\omega t}$ and applying separation of variables technique, Eq.(17) is obtained.

$$\frac{d^2u(z)}{dz^2} + \frac{\bar{m}\omega^2 L^2}{AE}u(z) = 0 \tag{17}$$

where $z = x / L$.

The axial force function *N(z)* and *u(z)* are obtained by substituting Eq.(18) into Eq.(17) as Eqs.(19)-(20), respectively.

$$u(z) = \{D\}e^{ikz} \tag{18}$$

$$u(z) = (D_5 e^{ik_5 z} + D_6 e^{ik_6 z}) \tag{19}$$

$$N(z) = V(ik_5 D_5 e^{ik_5 z} + ik_6 D_6 e^{ik_6 z}) \tag{20}$$

where $V = AE / L$

4. APPLICATION OF DYNAMIC STIFFNESS METHOD (DSM)

The dynamic stiffness matrices of beams and columns are constructed using end forces and displacements. The global dynamic stiffness matrix of whole vibrating system is formed by assembly of global stiffness matrices of members using a standard coding technique. Using SVSDT formulations, the end displacement vector of a member and the coefficient vector can be written as Eqs. (21) - (22), respectively.

$$\delta^S = [u_0 \quad y_0^S \quad \theta_0^S \quad u_1 \quad y_1^S \quad \theta_1^S]^T \tag{21}$$

$$D = [D_1 \quad D_2 \quad D_3 \quad D_4 \quad D_5 \quad D_6]^T \tag{22}$$

where

$$u_0 = u(z=0), y_0^S = y^S(z=0), \theta_0^S = \theta^S(z=0), u_1 = u(z=1), y_1^S = y_1(z=1), \theta_1^S = \theta^S(z=1)$$

Eqs. (14) - (15) and Eq.(19) are used to construct following matrix form:

$$\begin{bmatrix} u_0 \\ y_0^S \\ \theta_0^S \\ u_1 \\ y_1^S \\ \theta_1^S \end{bmatrix} = \begin{bmatrix} 0 & 0 & 0 & 0 & 1 & 1 \\ \lambda_1^S & \lambda_2^S & \lambda_3^S & \lambda_4^S & 0 & 0 \\ \eta_1^S & \eta_2^S & \eta_3^S & \eta_4^S & 0 & 0 \\ 0 & 0 & 0 & 0 & e^{ik_5} & e^{ik_6} \\ \lambda_1^S e^{ik_1} & \lambda_2^S e^{ik_2} & \lambda_3^S e^{ik_3} & \lambda_4^S e^{ik_4} & 0 & 0 \\ \eta_1^S e^{ik_1} & \eta_2^S e^{ik_2} & \eta_3^S e^{ik_3} & \eta_4^S e^{ik_4} & 0 & 0 \end{bmatrix} \begin{bmatrix} D_1 \\ D_2 \\ D_3 \\ D_4 \\ D_5 \\ D_6 \end{bmatrix} \tag{23}$$

where

$$\lambda_n^S = \left(THk_n^2 - TP + 1\right), \eta_n^S = \left(ik_n + TJik_n^3 - TRik_n\right), (n = 1,2,3,4)$$

Eq. (23) can be rewritten in simple form as Eq. (24):

$$\delta^S = \Delta^S D \tag{24}$$

where

$$\Delta^S = \begin{bmatrix} 0 & 0 & 0 & 0 & 1 & 1 \\ \lambda_1^S & \lambda_2^S & \lambda_3^S & \lambda_4^S & 0 & 0 \\ \eta_1^S & \eta_2^S & \eta_3^S & \eta_4^S & 0 & 0 \\ 0 & 0 & 0 & 0 & e^{ik_5} & e^{ik_6} \\ \lambda_1^S e^{ik_1} & \lambda_2^S e^{ik_2} & \lambda_3^S e^{ik_3} & \lambda_4^S e^{ik_4} & 0 & 0 \\ \eta_1^S e^{ik_1} & \eta_2^S e^{ik_2} & \eta_3^S e^{ik_3} & \eta_4^S e^{ik_4} & 0 & 0 \end{bmatrix}$$

The end force vector of the frame members (F^S) is given in Eq.(25):

$$F^S = [\, N_0 \quad Q_0^S \quad M_0^S \quad N_1 \quad Q_1^S \quad M_1^S \,]^T \tag{25}$$

where

$$N_0 = N(z=0), Q_0^S = Q^S(z=0), M_0^S = M^S(z=0), N_1 = N(z=1), Q_1^S = Q^S(z=1), M_1^S = M^S(z=1)$$

The sign convention in Eq.(26) is valid for end force relations.

$$N_0 = -N_1, \quad Q_0^S = -Q_1^S, \quad M_0^S = -M_1^S \tag{26}$$

The matrix form of end force functions given in Eq.(25) is presented using Eqs. (11) - (12) and Eq.(20) as:

$$\begin{bmatrix} N_0 & Q_0^S & M_0^S & N_1 & Q_1^S & M_1^S \end{bmatrix}^T = \kappa^S \begin{bmatrix} D_1 & D_2 & D_3 & D_4 & D_5 & D_6 \end{bmatrix}^T \tag{27}$$

where

$$\kappa^S = \begin{bmatrix} 0 & 0 & 0 & 0 & Vik_5 & Vik_6 \\ \Lambda_1^S & \Lambda_2^S & \Lambda_3^S & \Lambda_4^S & 0 & 0 \\ \Psi_1^S & \Psi_2^S & \Psi_3^S & \Psi_4^S & 0 & 0 \\ 0 & 0 & 0 & 0 & -Vik_5 e^{ik_5} & -Vik_6 e^{ik_6} \\ -\Lambda_1^S e^{ik_1} & -\Lambda_2^S e^{ik_2} & -\Lambda_3^S e^{ik_3} & -\Lambda_4^S e^{ik_4} & 0 & 0 \\ -\Psi_1^S e^{ik_1} & -\Psi_2^S e^{ik_2} & -\Psi_3^S e^{ik_3} & -\Psi_4^S e^{ik_4} & 0 & 0 \end{bmatrix}$$

$$\Lambda_n^S = (Jik_n^3 - Kik_n), \Psi_n^S = Hk_n^2, (n = 1,2,3,4)$$

The closed form of Eq. (27) is rewritten as:

$$F^S = \kappa^S \{D\} \tag{28}$$

The dynamic stiffness matrices of beams and columns are obtained using relation between Eqs. (24) and (28) as

$$F^S = \kappa^S (\Delta^S)^{-1} \delta^S \tag{29}$$

$$K^{*S} = \kappa^S (\Delta^S)^{-1} \tag{30}$$

where K^{*S} represents local dynamic stiffness matrix. A transformation procedure is needed to obtain the global dynamic stiffness matrix of members. The transformation matrix and global dynamic stiffness matrix of a member are given in Eqs. (31) - (32), respectively [73].

$$TM = \begin{bmatrix} cos(\Omega) & sin(\Omega) & 0 & 0 & 0 & 0 \\ -sin(\Omega) & cos(\Omega) & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & cos(\Omega) & sin(\Omega) & 0 \\ 0 & 0 & 0 & -sin(\Omega) & cos(\Omega) & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \tag{31}$$

$$\bar{K}^{*S} = (TM)^{-1}(K^{*S})(TM) \tag{32}$$

In Eq. (32), Ω represents the angle between local axes of the frame member and global axes of the frame.

The equivalent struts vibrate only in axial direction. Thus, local dynamic stiffness matrix of diagonal struts K_{inf} is given in Eq. (33) [73]:

$$K_{inf} = E_{inf} A_{inf} \beta \begin{bmatrix} cot\, \beta r_{inf} & -cos\, ec \beta r_{inf} \\ -cos\, ec \beta r_{inf} & cot\, \beta r_{inf} \end{bmatrix} \tag{33}$$

where A_{inf} is cross-sectional area of diagonal strut, β equals to $\sqrt{m_{inf}\omega^2 / E_{inf} A_{inf}}$ considering m_{inf} as mass per unit length of diagonal strut.

The diagonal struts are pin-jointed at nodes that have 2 degrees of freedom in vertical and horizontal directions. Eq.(33) is used to obtain global dynamic stiffness matrix of diagonal strut K^*_{inf} as:

$$K^*_{inf} = \begin{bmatrix} cot\, \beta r_{inf}\, cos\, \varphi & 0 & -cs\, c\beta r_{inf}\, cos\, \varphi & 0 \\ 0 & cot\, \beta r_{inf}\, sin\, \varphi & 0 & -cs\, c\beta r_{inf}\, sin\, \varphi \\ -cs\, c\beta r_{inf}\, cos\, \varphi & 0 & cot\, \beta r_{inf}\, cos\, \varphi & 0 \\ 0 & -cs\, c\beta r_{inf}\, sin\, \varphi & 0 & cot\, \beta r_{inf}\, sin\, \varphi \end{bmatrix} \tag{34}$$

The global dynamic stiffness matrix of frame models are obtained by using Eqs.(32) and (34) for SVSDT using a standard coding method. It is important to note that the dimension of global dynamic stiffness matrices are reduced by applying boundary condition. The natural frequencies ara detected by equating the determinant of the reduced global dynamic stiffness matrix to zero. The Wittrick-Williams algorithm can be an useful tool for root finding algorithm. Besides, a trial and error technique is an alternative procedure for calculating roots. When there is a sign change between trial natural frequencies, there must be a root lying in this interval. After some iterations, the natural frequencies can be obtained [60].

The same procedure can be repeated for EBT and TBT. The details about derivation of dynamic stiffness matrix of well known Euler-Bernoulli beams and Timoshenko beams are not presented to shorten the paper. The dynamic stiffness matrix formulations of various type of Timoshenko beams [51,74-76] and Euler-Bernoulli beams [77-78] can be found in many studies for further information.

5. NUMERICAL ANALYSIS AND DISCUSSIONS

The first three natural frequencies of frame models are calculated by using computer programs prepared in Matlab by the authors. Three different infill wall thickness values are considered which are 0.10 m, 0.15 m and 0.20 m. The numerical analyses are performed using the following properties: Elastic modulus of beams and columns: $2.94\text{x}10^7$ kN/m^2, elastic modulus of infill walls: $9.81\text{x}10^5$ kN/m^2, unit weight of beams and columns: 24.525 kN/m^3, unit weight of infill walls: 7.85 kN/m^3, Poisson ratio of beams and columns: 0.2, height of columns: 3.5 m, length of beams: 5 m, beam dimensions (width x height): 0.25 x 0.50 m, column dimensions (width x height): 0.30 x 0.50 m. The first three natural frequencies that calculated via DSM and ANSYS are presented in Table 1 and Tables 2-4 for bare frame and infilled frame models, respectively.

Table 1. First three natural frequencies of bare frame

	EBT	TBT	SVSDT	ANSYS
f_1 (Hz)	4.5754	4.9777	4.5199	5.0312
f_2 (Hz)	16.3742	17.1424	16.1248	17.2460
f_3 (Hz)	32.8128	32.6671	32.1193	32.6940

Tables 1-4 shows that SVSDT provides lower natural frequencies when compared to EBT and TBT as a result of considering inconstant shear stress distribution along cross-section of frame members. According to Tables 1-4, natural frequencies calculated using EBT are highest for the third mode for all frame models considered in numerical analysis. This result indicates that effects of shear deformations are observable starting from third mode for frame models. It is seen from Tables 2-4 that, natural frequencies of first three modes are increased when the frame is fully infilled. The dynamic behavior of frame becomes complicated for single storey infilled models. Tables 2-4 implies that, for the first storey infilled frame and third storey infilled frame, the first three natural frequencies are increased in comparison with bare frame. However, second storey infilled frame provides an increment of first and third natural frequencies and a slight decrease of second natural frequency. The natural frequencies of soft storey frame model (first and second storey infilled) are higher than bare frame for all wall thickness values. Tables 2-4 reveal that an augmentation of natural frequencies is observed by increasing wall thickness for all infilled frame models. Tables 2-4 shows that DSM solutions have a good agreement with FEM solutions of ANSYS for free vibration analysis of infilled frames. It should be noted that FEM results that obtained from ANSYS are based on 0.1 m long meshed frame members and diagonal struts. According to Tables 2-4, the relative error between ANSYS and DSM results is increased by augmentation of wall thickness. The ANSYS solutions are based on using Timoshenko beams and the differences between ANSYS and TBT solutions in Tables 1-4 are expected results by means of FEM and analytical based solutions.

The effects of different infill combinations on first natural frequency using EBT, TBT and SVSDT are presented in Figures 5-7, respectively. The mode shapes of frames are plotted by equating a nonzero nodal displacement to an arbitrary value. For instance, the first three mode shapes of bare frame and fully infilled frame are presented in Figure 8. According to Figure 8, the mode shapes of frames are not effected significantly by infill walls that modeled as equivalent diagonal struts.

Table 2. First three natural frequencies of infilled frame models (t_{inf} = 0.10 m)

Theory	f_n(Hz)	First Storey Infilled Frame	Second Storey Infilled Frame	Third Storey Infilled Frame	Soft Storey Frame	Fully Infilled Frame
EBT	f_1	5.1068	5.6044	4.7917	5.7324	6.4856
TBT		5.5281	5.8798	5.1107	5.9505	6.7161
SVSDT		4.9797	5.4145	4.7255	5.5528	6.1800
ANSYS		5.4349	5.6713	5.0875	5.6902	6.1943
EBT	f_2	17.5328	16.2598	19.6409	19.8713	20.8590
TBT		18.3007	17.0799	20.0425	20.3606	21.3554
SVSDT		17.1081	16.0397	18.9503	19.1200	19.9781
ANSYS		17.9910	17.1200	19.3210	19.3550	20.0000
EBT	f_3	33.2702	35.5620	35.1321	37.4409	37.6626
TBT		33.1078	35.4291	35.0866	37.3670	37.5798
SVSDT		32.5152	34.4368	34.0441	36.0411	36.2612
ANSYS		32.8450	34.4600	34.1770	35.7160	35.7630

Table 3. First three natural frequencies of infilled frame models (t_{inf} = 0.15 m)

Theory	f_n(Hz)	First Storey Infilled Frame	Second Storey Infilled Frame	Third Storey Infilled Frame	Soft Storey Frame	Fully Infilled Frame
EBT	f_1	5.2653	5.8758	4.7987	5.9609	7.0422
TBT		5.6939	6.1168	5.1674	6.1436	7.2407
SVSDT		5.1240	5.6676	4.7426	5.7821	6.6876
ANSYS		5.5711	5.8629	5.0751	5.8443	6.5685
EBT	f_2	17.9703	16.2070	20.5249	21.0205	22.3146
TBT		18.7389	17.0421	20.8192	21.4403	22.7434
SVSDT		17.4882	15.9991	19.7714	20.1436	21.2820
ANSYS		18.2860	17.0640	19.9700	20.1000	20.9550
EBT	f_3	33.4689	44.9569	36.2271	39.2288	39.4553
TBT		33.3006	36.5412	36.2137	39.1668	39.3852
SVSDT		32.6882	35.3823	34.9547	37.5886	37.8326
ANSYS		32.9100	35.1870	34.8630	36.9100	36.9300

Table 4. First three natural frequencies of infilled frame models (t_{inf} = 0.20 m)

Theory	f_n(Hz)	First Storey Infilled Frame	Second Storey Infilled Frame	Third Storey Infilled Frame	Soft Storey Frame	Fully Infilled Frame
EBT	f_1	5.3912	6.0779	4.7873	6.1066	7.4917
TBT		5.8262	6.2917	5.2484	6.2643	7.6680
SVSDT		5.2411	5.8623	4.7420	5.9379	7.1051
ANSYS		5.5986	5.9028	4.9847	5.9480	6.8789
EBT	f_2	18.3567	16.1560	21.1787	21.9852	23.5257
TBT		19.1261	17.0031	21.3908	22.3543	23.9043
SVSDT		17.8283	15.9592	20.4003	21.0171	22.3870
ANSYS		18.3529	16.8638	19.8929	20.7430	21.7690
EBT	f_3	33.6587	37.6806	37.2923	40.8285	41.0307
TBT		33.4855	37.5606	37.3020	40.7712	40.9674
SVSDT		32.8542	36.2559	35.8480	38.9897	39.2320
ANSYS		32.7724	34.5647	34.0784	37.9910	37.9690

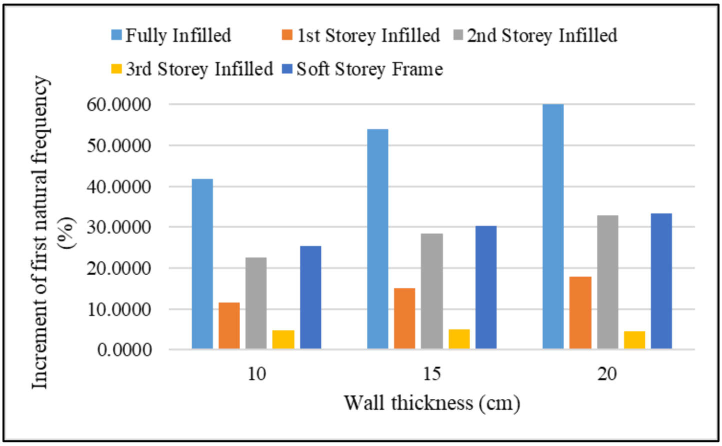

Figure 5. Increment percentage of first natural frequency using EBT

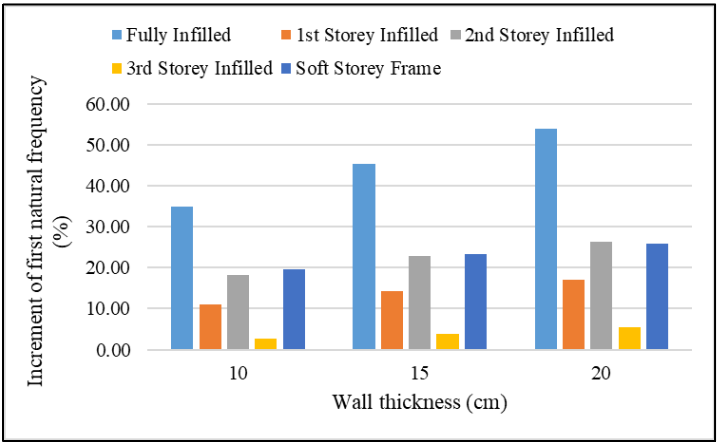

Figure 6. Increment percentage of first natural frequency using TBT

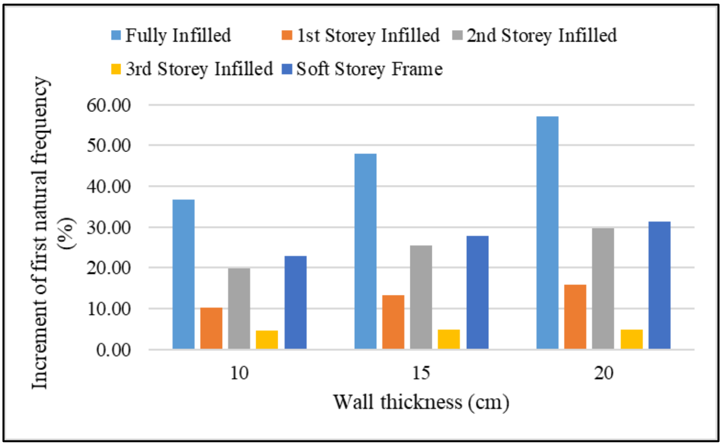

Figure 7. Increment percentage of first natural frequency using SVSDT

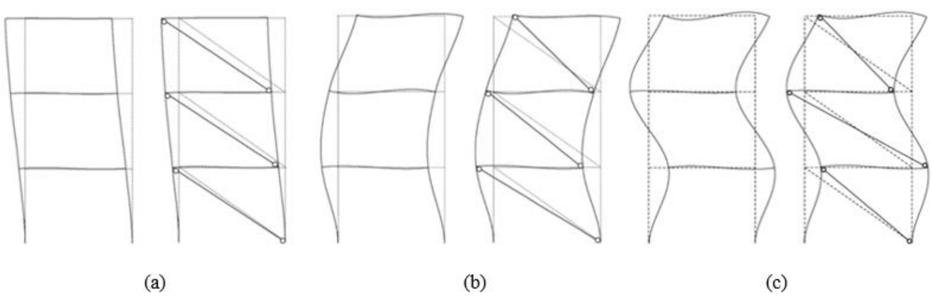

Figure 8. a) First mode shape of bare frame and fully infilled frame b) Second mode shape of bare frame and fully infilled frame c) Third mode shape of bare frame and fully infilled frame

It should be noted that the width of equivalent diagonal strut is multiplied by a stiffness reduction factor that ranges from zero (bare frame) to one (fully infilled frame) for frames having openings. Therefore, the proposed approach can be applied to free vibrations of partially infilled frame structures without any difficulty as only difference would be a decrement on value of α [15]. The calculated fundamental frequencies of numerical examples can be used for developing an artificial neural network (ANN) to predict dynamic characteristics of structures in future studies as ANN has been an attractive research area on obtaining fundamental frequencies of infilled frames in recent years [79-80].

6. CONCLUSIONS

The infill walls are considered on calculation of natural frequencies and mode shapes of frame structures using dynamic stiffness formulations that based on analytical solutions. SVSDT which considers parabolic shear stress distribution along cross-section is used as well as Euler and Timoshenko beams for frame member modeling. DSM results are verified with results of FEM. Effects of different location combinations of infill walls are revealed. It is seen that ignoring infill walls on free vibration analysis of frames may cause unacceptable errors. The location of infills plays an important role on dynamic characteristics of frames especially for one storey infilled models. Effectiveness of SVSDT, which can also provide EBT results by ignoring shear deformation related terms from governing equation of motion, is proved for free vibration analysis of infilled frames for various infill locations. EBT overestimates natural frequencies of the third mode for all infilled frame combinations as well as bare frame. This result indicates that effects of shear deformation and rotational inertia become more observable for higher modes of infilled frames. This study can be extended for frames having partial openings such as windows or doors without any difficulty as only difference is cross-sections of equivalent diagonal strut due to stiffness reduction of the wall. The mode shapes of frames are not affected significantly from infill walls by using pin jointed diagonal strut approach used in the study. The computer programs that prepared for calculation of natural frequencies and plotting mode shapes are working fast.

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