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Effect of Inclusion of Masonry Infill on Fundamental Time Period

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Abstract Masonry infill walls in reinforced concrete (RC) frames significantly influence seismic response despite being classified as non-structural elements in design codes. This study investigates the impact of masonry infills on the fundamental time period (FTP) of RC buildings, particularly addressing structures with open ground floors and irregular configurations commonly found in modern high-rise construction. The research methodology employed STAAD Pro software to analyse 168 building frame models with varying parameters, including infill wall thicknesses (150 mm and 250 mm), beam sizes (300-550 mm), column sizes (450-800 mm), and building heights (6-30 stories). Dynamic modal analysis was performed to determine FTP values, which were subsequently compared with existing code-based empirical formulas, including IS 1893:2016, UBC 97, and Eurocode 8. The principal findings reveal that masonry infills reduce FTP by 26-80% compared to bare frame structures, with the reduction percentage directly correlated to the Young's modulus of infill materials. The study demonstrates that current code equations significantly overestimate FTP values, leading to conservative base shear calculations. Through nonlinear regression analysis of the compiled results, a new empirical equation was developed to accurately predict FTP for infilled RC frames. The proposed equation shows superior accuracy compared to existing code formulas when validated against dynamic analysis results. The research contributes to structural engineering by providing a more accurate method for FTP estimation that explicitly considers masonry infill contributions, thereby improving seismic design accuracy. The findings have practical implications for earthquake-resistant design, particularly in regions where infilled RC frames are prevalent. Research limitations include focus on specific infill materials and regular frame configurations, suggesting future work should explore diverse masonry types and irregular structural layouts.

Keywords Masonry Infill Walls, Reinforced Concrete Frames, Fundamental Period, Seismic Design, Dynamic Analysis, Structural Irregularities

1. Introduction

Buildings are intricate assemblies containing both structural and non-structural elements, and their performance under seismic loads is a topic of extensive research in structural engineering. Among the non-structural elements, masonry infill walls—usually present in reinforced concrete (RC) and steel frames—play a substantial yet often underestimated role in a building's seismic response. Despite their classification as non-structural components in numerous design codes, several research studies have emphasized the significant impact of masonry infills on a building's lateral stiffness, strength, and energy dissipation capacity [1,2,3,4,5]. Subsequently, overlooking their influence in seismic design and analysis could result in imprecise assessments of building performance and, eventually, structural failure under dynamic loads.

The mass and stiffness of infill walls modify the seismic response of the structure, particularly influencing the fundamental time period (FTP), a crucial dynamic feature that denotes the building's inherent vibration characteristics. Prior research indicates that the existence, configuration, and material characteristics of infill walls substantially affect the FTP, consequently altering the inertial forces encountered during seismic occurrences [6,7]. The incorporation of these infills can reduce the fundamental period owing to the increased lateral stiffness; however, this effect is contingent upon the distribution, continuity, and strength of the infill system [8]. In conventional dynamic analysis, such as Rayleigh's approach, the fundamental period is directly associated with mass and stiffness distribution, both of which are considerably influenced by the existence of infill walls.

Most seismic design rules, though, suggest empirical techniques for calculating the FTP, mostly depending on building height, hence ignoring the influence of infill walls and material anomalies [9,10,11,12]. Developed from small experimental datasets, these empirical equations lack general application and do not account for regional variations in building practices, materials, and design philosophies. For example, buildings built with hollow clay bricks or unreinforced masonry panels show different dynamic properties than those built with concrete block infills. Such differences require a more thorough analytical or numerical method in seismic design that clearly includes the impact of infill walls.

The approach to modelling masonry infills has advanced considerably, with the diagonal strut model becoming increasingly favoured due to its straightforwardness and efficiency. Stafford Smith [13] first proposed the idea of modelling infill panels as equivalent diagonal struts to mimic their compression response when subjected to lateral forces. This model posits that the lateral force exerted by the frame is conveyed to the masonry infill via a compression zone, functioning similarly to a diagonally positioned strut [14,15]. Despite its limitations—especially in cases involving large openings or considerable out-of-plane effects—this technique continues to be commonly employed in performance-based seismic assessments.

Masonry infill panels can lead to inconsistencies in the stiffness and mass distribution of a building, especially when their arrangement is not uniform throughout the height or layout of the structure. The identified irregularities contribute to torsional responses, stress concentrations, and soft-storey mechanisms, which may ultimately result in premature structural failure during seismic events [16,17]. Irregularities in design, including re-entrant corners or L-shaped configurations, result in an uneven distribution of stiffness, which increases the susceptibility of buildings to torsional forces. Vertical irregularities, including soft-storey configurations and abrupt stiffness changes, create discontinuities that are inadequately represented by code-based empirical expressions [16,17].

Advanced numerical modelling methods, such as finite element analysis (FEA), have demonstrated their efficacy in accurately representing the intricate interactions between infill walls and reinforced concrete frames. These approaches facilitate comprehensive simulations of material responses, mechanisms of load transfer, and the gradual deterioration under seismic conditions. For instance, employing software such as ETABS or SAP2000 allows researchers to assess stress concentrations, torsional responses, and collapse mechanisms in irregular infilled frames with enhanced precision [16,18]. The analyses conducted indicate that code-based approximations frequently fail to accurately account for the impact of infills, especially in the context of significant ground motion.

Furthermore, several factors other than geometry and material qualities affect the seismic behaviour of infilled structures. Defining structural response depends much on ground motion features, including frequency content, duration, and pulse-like effects. Particularly, near-field earthquakes can generate significant velocity pulses that disproportionately affect infilled frames [19]. The in-plane and out-of-plane interaction of infill walls must also be included in realistic seismic calculations. Neglecting out-of-plane behaviour leads to perilous assumptions and design deficiencies [20].

Experimental investigations and post-earthquake assessments have consistently revealed the dual nature of infill walls: they enhance the stiffness and strength of buildings, but their brittle behaviour and susceptibility to out-of-plane failure may also introduce vulnerabilities [21,22]. In the 2015 Gorkha earthquake, well-engineered reinforced concrete structures with uniformly distributed masonry infills demonstrated commendable performance. Conversely, structures marked by deficient construction methods and inconsistent infill configurations sustained significant damage [21].

Recent improvements in seismic retrofitting techniques have concentrated on mitigating the adverse effects of infill wall performance. Retrofitting techniques, including textile-reinforced mortar jacketing, substitution with autoclaved aerated concrete (AAC) blocks, and the integration of prefabricated reinforced concrete panels, have demonstrated improved seismic resilience [22,23,24]. These solutions not only improve structural integrity but also promote energy efficiency, along with sustainable construction objectives.

In spite of these advancements, significant research gaps continue to exist. The precise modelling of infill-frame interaction, particularly in the context of irregular layouts or configurations that involve openings or varying seismic intensities, is one of the significant challenges. Improving numerical models, checking them against experimental data, and creating generally applicable design rules that take structural and non-structural factors into consideration should be the priorities of future studies [25,26].

Masonry infills play a multi-faceted and intricate function in seismic analysis of building frames. Though stiffness and strength of masonry fills can be beneficial, overlooking their effects, specifically in irregular configurations, can lead to catastrophic consequences. A holistic understanding integrating dynamic analysis, refined modelling, and empirical validation is crucial for precise seismic performance assessment. This comprehensive approach will not only bridge the gap between design practice and real-world performance but also enhance the resilience and safety of built environments in earthquake-prone regions.

2. Structural Analysis and Modeling of Masonry Infills

Because of the interaction between the building structure and the masonry infill, the in-plane stiffness of a building with infill walls is greater than its lateral stiffness. This is due to the fact that in-plane stiffness is measured along the building's longitudinal axis. When lateral loads are applied to an infilled frame system in the plane of the structure, strong compression strains can be evident along the incline of an infilled wall. Tensile strains and stresses are on one side of a continuum, whereas compression strains and stresses are on the other. Fractures appear along the diagonal when the tensile loads on the masonry infill material are larger than the strain at which they would break. When the strain at which they will shatter is exceeded, this occurs. When a diagonal crack forms in a strut, it often signifies that the strut is starting to perform differently than it did previously. The third and final limiting scenario that often develops during the construction of a compression strut is known as "corner crushing."

To represent the masonry infill in the building frame, only the walls flanked by two beams and two columns were used. The infill wall contributed to the structural weight, increasing the overall weight of the structure. During an earthquake, this increased the building's inertia force. In the infill walls consideration, the tensile strength of the diagonal compression strut was not taken into account [27].

The FEMA 306 [28] standard was utilised to determine the strength and stability of a similar diagonal compression strut. These principles were developed based on research conducted by Mainstone and Weeks [29,30] at the turn of the twentieth century. The equivalent diagonal compression strut was calculated by the actual infill thickness, t_{inf}, which was in contact with the frame, and the diagonal length, r_{inf}. The equivalent width of a diagonal compression strut a, was given by:

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf}$$

Where h_{col} is Vertical Height of floor from centre to centre (m) and r_{inf} is Diagonal Length of infill walls (m)

$$\lambda_1 = \left[\frac{E_m t_{inf} \sin 2\theta}{4 E_f I_{cl} h_{inf}}\right]^{\frac{1}{4}}$$

and

$$\theta = \tan^{-1}\left(\frac{h_{inf}}{L_{inf}}\right)$$

where E_f, E_m, t_{inf}, h_{inf}, L_{inf} and I_{cl} denote the Expected modulus of elasticity of infill walls (MPa), Expected modulus of elasticity of frame elements (MPa) of masonry infill walls, Thickness of infill wall (m), Height of infill wall (m), and Length of infill wall (m) and moment of inertia of column (m^4), respectively.

Previous studies utilised the transverse length to approximate the compression strut [31]. However, under actual conditions, the scenarios alter, and the analytical program possesses the ability to simulate conditions akin to those in the real world for buildings. This research analyses infill walls as 4-noded plates, possessing features analogous to those of the infill walls. Beams and columns were modelled as typical two-noded frame elements, each possessing six degrees of freedom per node. Shear walls and slabs were modelled as conventional four-noded plate elements. Infill walls, additional dead load, and reduced live load were modelled as area mass elements.

3. Building Models Used in the Research Study

3.1. Details of Building Models Adopted

This research used three distinct types of building frames: a) Regular frames, b) Frames with 150mm infills and an open ground floor, and c) Frames with 250mm infills and an open ground floor. This combination was chosen to replicate the existing design of high-rise buildings featuring infills and an open ground floor for vehicle parking. The principal objective of this study was to investigate these three separate groups of structures. A total of 168 building frames were modelled and evaluated using STAAD Pro software. The square beam cross sections used were 300 mm, 350mm, 400mm, 450mm, 500mm and 550 mm. The square column cross sections used were 450 mm, 525 mm, 600 mm, 675 mm, 750 mm, and 800 mm. The detailed modelling parameters are present in Table 1. The combination of column to beam was taken as 1.5 times, indicating the column size to be greater than the beam size. The accuracy of the STAAD prof software has already been validated in previous research works [32,33]. The building frames were subjected to dynamic analysis using STAAD pro software, and the results were compiled, and a non – linear regression analysis was conducted on the compiled results to propose new equations to estimate the FTP. Finally, FTP was compared with code and previous research works to demonstrate the efficiency of the proposed approach.

Table 1. Details of Building models adopted for the analytical study

Models	Parameters	Values
Skeleton frame	Size of beam	300mm x 300mm, 350mm x 350mm, 400mm x 400mm, 450mm x 450mm, 500mm x 500mm, 550mm x 550mm
	Size of column	450mm x 450mm, 525mm x 525mm, 600mm x 600mm, 675mm x 675mm, 750mm x 750mm and 800mm x 800mm
	Concrete strength	27.57 Mpa
	No. of floors	6, 9, 12, 15, 18, 21, 24, 27 and 30
	Slab Thickness	150mm

3.2. Infill Wall Properties

The masonry infill wall is 150 mm thick with stretched bond, and the double-layer brick wall, which is commonly used in the construction of high-rise buildings, is 250 mm thick with English bond. The thickness of the joint mortar is considered part of the total thickness of the infill material. Bricks are manufactured from clay and have dimensions of 19 cm on each side and 13.5 cm in height. The parameters adopted are 19 kN/m^3, 20800 MPa, and 3.2721x10^{-7} in accordance with previous research work [34].

4. Framed Building Models

The details of building model adopted in the present study have been shown in Table 1. The columns and beams have been modelled as two-noded frame elements, which resulted in an additional six degrees of freedom for each individual component. The infill walls were represented by area mass elements modelled using 3D mesh elements and 4-noded plates with masonry infill properties shown in Table 2. Figures 1 and 2 show the 3D view of the building model created using STAAD pro software, which was used in the analytical study.

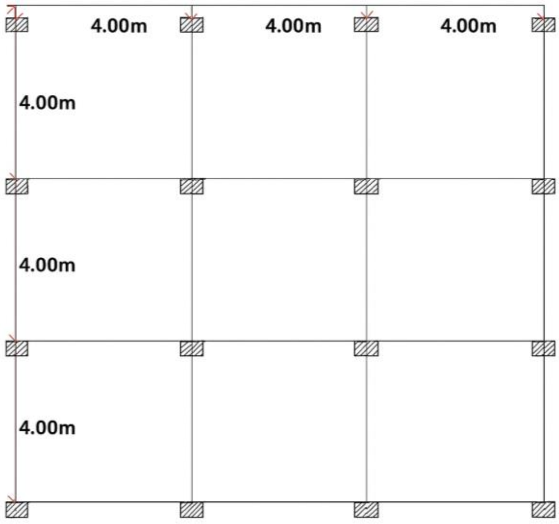


Figure 1. Plan for the building model

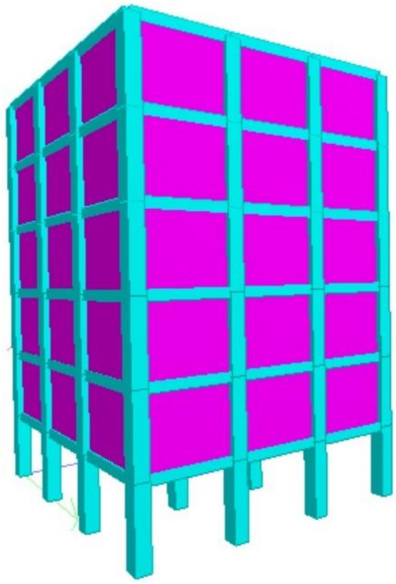


Figure 2. 3D model frame with masonry infill

Table 2. Material Properties of concrete adopted for analytical study

Material Properties of Concrete	Values
Young's Modulus (E)	32,721 MPa
Poisson's Ratio (nu)	0.25
Density	19 kN/m^3
Shear Modulus (G)	13,000 MPa
Compressive strength (F_{cu})	20.8 MPa

4.1. Existing Code Equations Available for Estimation of FTP of Buildings

The expressions to estimate FTP by different seismic design codes are shown in Table 3. The only thing that differs between UBC [35] and Eurocode 8 [36] is the conversion coefficient C_c from feet to meters. Rayleigh's technique assumes linearly distributed lateral loads along a structure's height and decreasing stiffness with height to move each story in the same direction. The research work by Goel and Chopra [11] led to a modification of the UBC formula [35], which was subsequently incorporated into the FEMA 450 document [37].

Table 3. FTP proposed by different building codes

Code	FTP (sec.)
UBC 97	$0.075\ H^{0.75}$
ASCE 7:2005	$0.075\ H^{0.75}$
NBCC 2005	$0.01\ N$
EC8	$0.075\ H^{0.75}$
IS 1893	$0.09H/D^{0.5}$

Where H is the building's total height in meters, D is the building's least lateral dimension, and N is the number of stories of the building for the European Region.

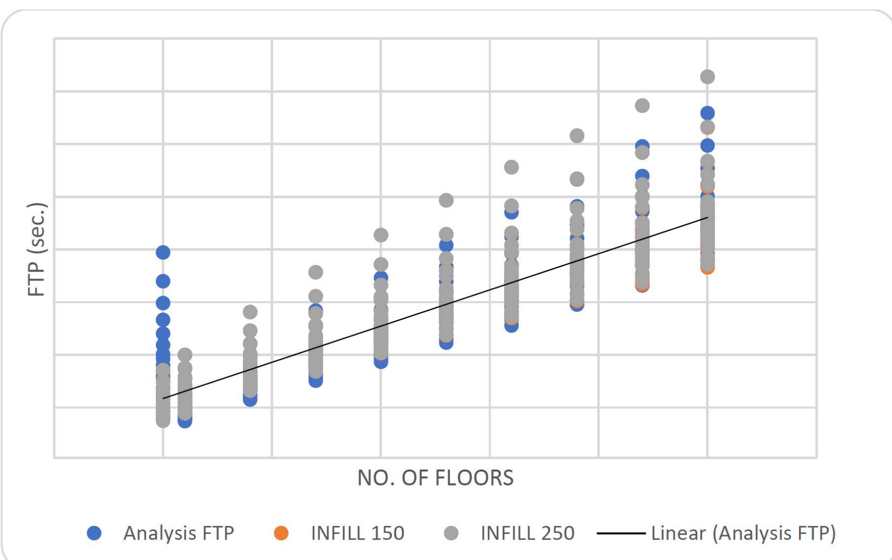
It is worth emphasizing that the FTP plays a crucial role in seismic design, as it directly influences the estimation of the seismic base shear force (*Fb*) for each horizontal direction of analysis. According to Eurocode 8 (EC8: 4.3.3.2.2 – Base Shear Force), the base shear force (*Fb*) is considered directly proportional to the product of the total mass of the building (*m*)—above the foundation or the top of a rigid basement mass—and the spectral acceleration ordinate $Sd(T_1)$, corresponding to the design spectrum (EC8: 3.2.2.5) at the fundamental period T_1 (the fundamental period of vibration of the building for lateral motion in the direction considered). This relationship is further affected by a correction factor λ, which equals unity (λ = 1.0) for buildings with more than two storeys, as is the case for the buildings considered in the present study.

Due to variations in materials and construction practices, the universal applicability of existing code-based equations is questionable. Researchers such as Crowley [10], Hong, and Hwang [12] found that structures in Taiwan were significantly more durable than those in California, allowing for more accurate predictions of vibration periods. This highlights the necessity of developing a new equation for estimating the FTP.

It has been observed that code-based equations tend to underestimate the FTP by about 10–20%, leading to overly conservative estimates of base shear. Research conducted by the University of British Columbia recommends multiplying the seismic intensity of high-seismic zones by 1.3 and that of other seismic zones by 1.4. According to Eurocode 8: 2004, there is no restriction on the number of times such a factor may be applied. However, the contribution of masonry infills has often been neglected in the computation of FTP, which can lead to lower base shear and higher FTP values—resulting in more flexible structures. Previous earthquake records have shown that a large number of such flexible structures have failed; therefore, avoiding excessively flexible configurations is essential for seismic safety.

5. Results and Discussions

As can be seen in Figure 3 and Table 4, the FTP is observed to decrease due to the inclusion of masonry infill, which could be attributed to a subsequent increase in stiffness, which in turn reduces FTP. Dynamic analysis has been utilised in order to calculate the FTP of regular frames, models with open GF and 150mm of infill, and models with 250mm of masonry infill. In addition, the IS1893: 2016 equation was utilised to determine the FTPs of the models, and the findings were compared to the proposed equation. The disparity between the FTPs determined by modal analysis and those determined by the IS1893:2016 equation widens whenever there is a change in the proportional sizes of the column, beam, and infill wall in comparison to one another. The greater the ratio, the less significant this difference becomes. The FTP decreases whenever there is an increase in the size of the structural members.



Series1- Skeleton frame's FTP, Series2- Infill 150mm FTP and Series3- Infill 250mm FTP

Figure 3. Variation of FTP due to the addition of infill

Table 4. Results of the analytical study

Beam Size (mm)	Column Size (mm)	No. of Storey	IS Code FTP (sec)	Analytical Models FTP (sec)		
				Without Infill	Infill 150mm	Infill 250mm
300X300	**450X450**	6	0.467653718	0.05587	0.07017	0.07017
		9	0.701480577	0.07991	0.10023	0.10024
		12	0.935307436	0.10297	0.12847	0.12848
		15	1.169134295	0.12559	0.15513	0.15514
		18	1.402961154	0.14807	0.18047	0.18048
		21	1.636788013	0.17061	0.20468	0.20466
		24	1.870614872	0.19322	0.22793	0.22786
		27	2.104441731	0.21592	0.25034	0.25018
		30	2.33826859	0.23868	0.27202	0.27172
350X350	**525X525**	6	0.467653718	0.05048	0.06249	0.06249
		9	0.701480577	0.07212	0.08886	0.08887
		12	0.935307436	0.09312	0.11352	0.11354
		15	1.169134295	0.11397	0.13681	0.13682
		18	1.402961154	0.13487	0.15898	0.15896
		21	1.636788013	0.15588	0.18021	0.18013
		24	1.870614872	0.17695	0.20067	0.20045
		27	2.104441731	0.19810	0.22049	0.22007
		30	2.33826859	0.21930	0.23980	0.23910
400X400	**600X600**	6	0.467653718	0.04664	0.05692	0.05693
		9	0.701480577	0.06670	0.08066	0.08067
		12	0.935307436	0.08643	0.10285	0.10287
		15	1.169134295	0.10619	0.12386	0.12386
		18	1.402961154	0.12604	0.14393	0.14386
		21	1.636788013	0.14599	0.16326	0.16303
		24	1.870614872	0.16599	0.18199	0.18152
		27	2.104441731	0.18605	0.20028	0.19947
		30	2.33826859	0.20614	0.21824	0.21699
450X450	**675X675**	6	0.467653718	0.04384	0.05275	0.05276
		9	0.701480577	0.06285	0.07456	0.07458
		12	0.935307436	0.08175	0.09502	0.09504
		15	1.169134295	0.10076	0.11447	0.11443
		18	1.402961154	0.11986	0.13315	0.13297
		21	1.636788013	0.13904	0.15126	0.15082
		24	1.870614872	0.15825	0.16897	0.16815
		27	2.104441731	0.17751	0.18638	0.18508
		30	2.33826859	0.19679	0.20361	0.20172

Table 4 continued

500X500	**750X750**	6	0.467653718	0.04173	0.04954	0.04956
		9	0.701480577	0.06004	0.06994	0.06997
		12	0.935307436	0.07838	0.08916	0.08917
		15	1.169134295	0.09684	0.10753	0.10742
		18	1.402961154	0.11538	0.12530	0.12495
		21	1.636788013	0.13397	0.14267	0.14195
		24	1.870614872	0.15260	0.15977	0.15856
		27	2.104441731	0.17125	0.17671	0.17490
		30	2.33826859	0.18992	0.19355	0.19105
550X550	**825X825**	6	0.467653718	0.04014	0.04702	0.04704
		9	0.701480577	0.05796	0.06637	0.06641
		12	0.935307436	0.07590	0.08470	0.08467
		15	1.169134295	0.09392	0.10234	0.10213
		18	1.402961154	0.11203	0.11953	0.11900
		21	1.636788013	0.12818	0.13646	0.13545
		24	1.870614872	0.14833	0.15323	0.15162
		27	2.104441731	0.16652	0.16991	0.16762
		30	2.33826859	0.18472	0.18655	0.18351

6. Proposed Equation to Estimate FTP

The following equation was proposed as a means of determining the FTP of RC moment-resistant frames. This equation was derived by the application of a multiple linear regression model analysis [12]. The presentation of this equation can be attributed to the findings of the parametric investigation.

$$T = 0.0022\text{h} + 0.0244 \quad (7)$$

where h = building height in meters,

It is important to note that this proposed equation was developed based on a specific parametric investigation that considered clay brick masonry infill walls, specifically, 150 mm thick clay brick masonry infill walls with stretched bond and 250 mm thick double-layer clay brick walls with English bond. Therefore, the proposed equation is especially suitable for buildings utilizing clay brick infill walls of these configurations. Its application to other types of infill materials or regional variations in construction practices (such as those employing hollow clay bricks or unreinforced masonry panels) should be approached with caution and may require appropriate correction factors to maintain accuracy.

Figure 4 shows a comparison of the proposed equation with various codes along with IS1893: 2016 [38]. The code proposed equations to estimate the fundamental time period are mainly based on the height of the building, as it has the most dominant impact on FTP. In Figure 4, a 45-line was made for comparing the values of the FTPs that were predicted by the code with the proposed equation based on iterative modal analysis. The comparison clearly shows that the code proposed equations overestimate the FTP, but the proposed approach yielded accurate results [11,12,26,40,41].

Figure 4 also clearly shows the accuracy of the proposed approach in comparison to codes like IS 1893:2016, the UBC 97 [35], Eurocode 8:2004 [36], and the NBC 1995 [39]. The proposed equations are not suitable for dwarf structures and are more accurate for buildings with a height greater than 6 m. Figure 6 shows the comparison of the proposed work with previous research approaches, demonstrating the efficiency of the proposed approach. The software used in this approach is STAAD Pro [42], which has been validated in previous research works [26,43,44,45,46,47].

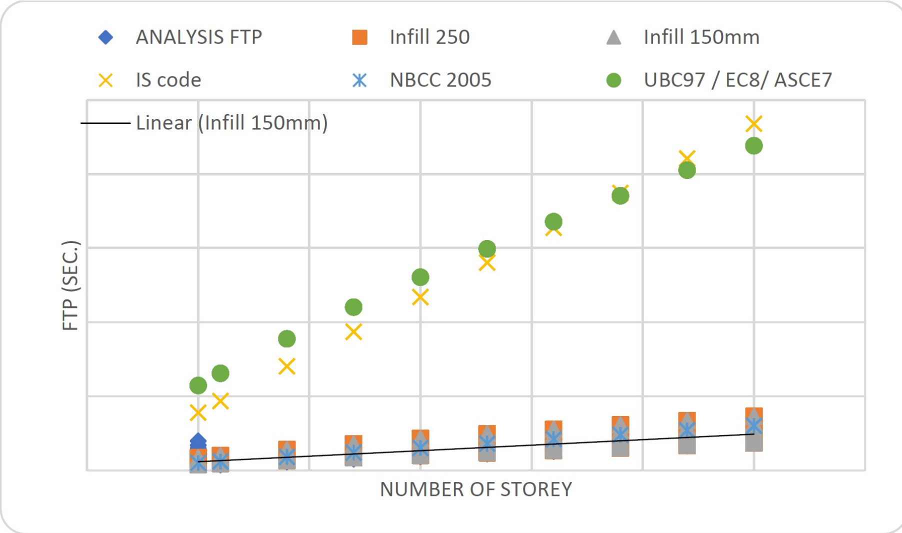


Figure 4. Comparison of FTP proposed by the dynamic analysis and design codes

7. Conclusions

The literature review of the research works pertaining to the estimation of the Fundamental Time Period (FTP) clearly shows that the previously proposed equations were derived from experimental datasets of a limited number of buildings located in specific regions. Consequently, the universal applicability of these equations is questionable due to variations in material properties and construction practices. Moreover, masonry infill considerably alters the dynamic properties of a structure, yet its effect has often been neglected in the estimation of FTP. Figure 5 shows the variation of the fundamental time period with respect to changes in column and beam sizes, highlighting the effect of member stiffness on the dynamic behavior of the building.

FTP is a critical parameter in seismic design, and its inaccurate estimation can result in erroneous design outcomes, potentially leading to catastrophic structural failures. The majority of previous studies have estimated FTP based on Rayleigh's method, which carries inherent limitations. The present research has addressed these deficiencies and proposed a new equation for estimating FTP that explicitly includes the contribution of masonry infill. In this study, the masonry infill was modeled using four-noded plate elements and wire mesh to overcome the limitations of the conventional diagonal strut model. An analytical study was conducted on 168 different building models, and the resulting data were subjected to regression analysis to derive a new empirical equation for estimating FTP.

The comparison of the proposed approach with dynamic analysis results clearly demonstrates its accuracy and reliability. In contrast, code-based and previously proposed empirical equations show poor correlation with dynamic analysis outcomes, as evident from Figure 4 and Figure 6.

Furthermore, it is recommended that future research should focus on developing more comprehensive analytical or numerical methods in seismic design that explicitly account for the effects of infill walls, regional variations in construction practices, and material properties. These future models should consider different types of unreinforced masonry infills—such as clay brick, concrete block, and hollow brick masonry—as well as potential material anomalies that influence the overall dynamic response of buildings. Such efforts will contribute to the formulation of more robust, regionally adaptable, and realistic equations for estimating the Fundamental Time Period in seismic design.

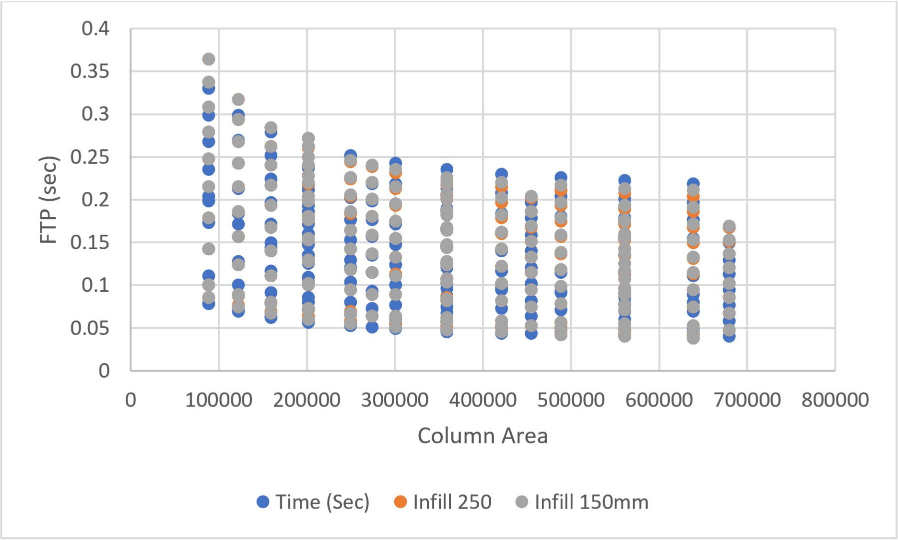


Figure 5. Variation of FTP of column and beam size

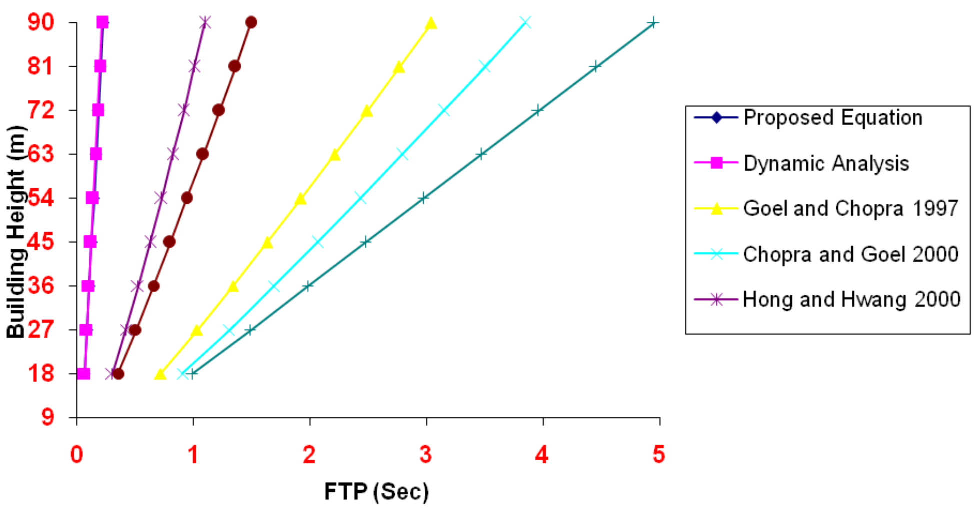


Figure 6. Comparison of the proposed equation with previous research studies

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