Coupled Systems Mechanics, *Vol. 4, No. 2 (2015) 173-189*
DOI: http://dx.doi.org/10.12989/csm.2015.4.2.173

Influence of masonry infill on reinforced concrete frame structures' seismic response

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(Received April 17, 2015, Revised June 9, 2015, Accepted June 12, 2015)

Abstract. In reality, masonry infill modifies the seismic response of reinforced concrete (r.c.) frame structures by increasing the overall rigidity of structure which results in: increasing of total seismic load value, decreasing of deformations and period of vibration, therefore masonry infill frame structures have larger capacity of absorbing and dissipating seismic energy. The aim of the paper is to explore and assess actual influence of masonry infill on seismic response of r.c. frame structures, to determine whether it's justified to disregard masonry infill influence and to determine appropriate way to consider infill influence by design. This was done by modeling different structures, bare frame structures as well as masonry infill frame structures, while varying masonry infill to r.c. frame stiffness ratio and seismic intensity. Further resistance envelope for those models were created and compared. Different structures analysis have shown that the seismic action on infilled r.c. frame structure is almost always twice as much as seismic action on the same structure with bare r.c. frames, regardless of the seismic intensity. Comparing different models resistance envelopes has shown that, in case of lower stiffness r.c. frame structure, masonry infill (both lower and higher stiffness) increased its lateral load capacity, in average, two times, but in case of higher stiffness r.c. frame structures, influence of masonry infill on lateral load capacity is insignificant. After all, it is to conclude that the optimal structure type depends on its exposure to seismic action and its masonry infill to r.c. frame stiffness ratio.

Keywords: masonry infill; reinforced concrete frame; stiffness ratio; lateral load capacity; storey resistance envelope

1. Introduction

With regard to carrying gravity loads, masonry infill walls are considered as non-structural elements and are not taken into account by design because their axial stiffness is insignificant compared to the frame columns axial stiffness. Meanwhile, when the building is subjected to seismic loads, influence of masonry infill on the behavior of the main structure depends on the connection between the infill and the frame, since infill walls rigidly connected with the frame impede deformation of the structure and represent a constituent part of the vibrating structural system. Masonry infill contribution to the overall stiffness and capacity of the structure is not

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http://www.techno-press.org/?journal=csm&subpage=7 ISSN: 2234-2184 (Print), 2234-2192 (Online)

simple. It is influenced by construction work quality and wide materials strength scattering.

1.1 Observed behavior

At low level of lateral load and small lateral deformations, the masonry infilled r.c. frame acts as a monolithic, composite structural element. Due to the initial masonry infill to r.c. frame stiffness ratio, the contribution of the flexible frame to the lateral load resistance is small and the largest amount of the lateral seismic load is carried by the rigid masonry infill. However, as the lateral deformations increase, the relatively weak masonry infill is no longer capable of carrying the increased lateral load. The cracks develop in the masonry and the infill wall starts to separate into two or more parts. This has an impact on r.c. frame and it can deform, depending on the type of separation of the infill wall and the length of the remaining contact zone between the infill and frame members. If the failure of the infill is brittle and the seismic actions are substantially increased just before the local collapse of the infill, severe damage to the main structural system, which had not been designed to resist the increased lateral loads, occurs. Basically, the failure mechanism is of shear type and depends on the masonry infill to r.c. frame stiffness ratio, the quality of materials, the contact between the infill and r.c. frame and load type. Typical mechanisms, according to Tomažević (1999), are characterized by:

- Diagonal tensile cracking of the infill wall usually occurs if the masonry is strong and the contact between the masonry and frame is good. A windward column, supported by the infill, fails in shear, whereas plastic hinging occurs at the bottom and top of the free to deform leeward column (see Fig. 1 and 2nd, 3rd and 4th floor in Fig. 3).
- Sliding shear failure of masonry infill and separation in two parts along mortar joints at the mid-height. As a result of slippage of the two separated parts of the infill, shear failure of the free parts of columns due to short column effect may take place, with plastic hinging at the bottom and top of the free parts of columns. For example, the column to the left of the central column in Fig. 2 was captured approximately 1 m above the floor by the residual infill wall. The shear cracks that were observed in this captive column formed at the top face of the remaining infill.
- Sliding shear failure of masonry infill along horizontal mortar joints and separation into several parts. Separated parts of masonry infill permit free deformations of columns, ultimately resulting in plastic hinging of columns at joints between columns and beams (see the first floor in Fig. 3).

However, if masonry infill walls are damaged before the development of high shear forces, which might possibly damage the main structural system, they dissipate seismic energy and prevent large deformations or r.c. frames, as shown in Fig. 4.

2. Structures analysis

In order to assess the stiffness increase due to masonry infill and relation between overall stiffness and infill to r.c. frame stiffness ratio, a finite element analysis of the three storey building, shown in Fig. 5, has been carried out using SAP software. Interstorey slabs are considered as rigid horizontal floor diaphragms, providing that distribution of the load to the vertical structural elements is proportional to their stiffnesses. Masonry infill walls are modeled as frame elements with corresponding dimensions (see Fig. 6) and their shear modulus and bending stiffness are modified by reduction coefficient, taking into account that infill walls, subjected to the seismic action, enter a non-linear range.

Fig. 1 Damaged structure, Turkey, 1998

Fig. 2 Damaged structure, Turkey, 1999

Fig. 3 Damaged structure, Mexico, 1995

Fig. 4 Damaged structure, Turkey, 1999

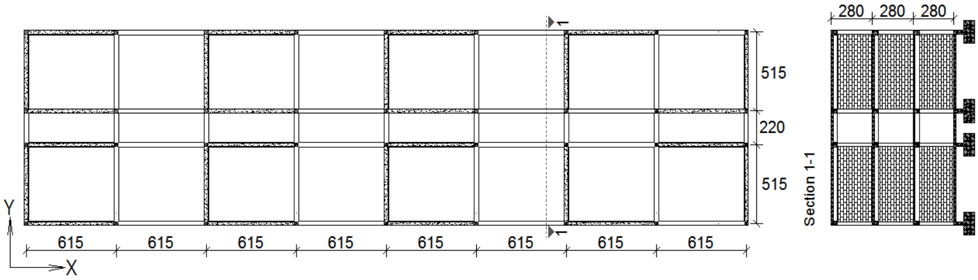
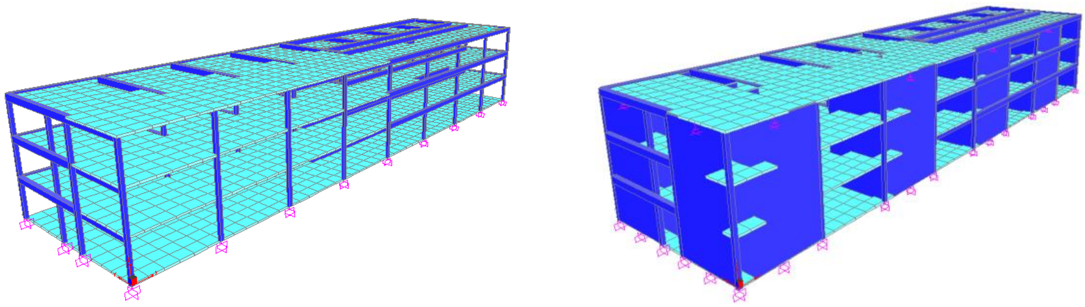


Fig. 5 Building layout and disposition of the masonry infill walls

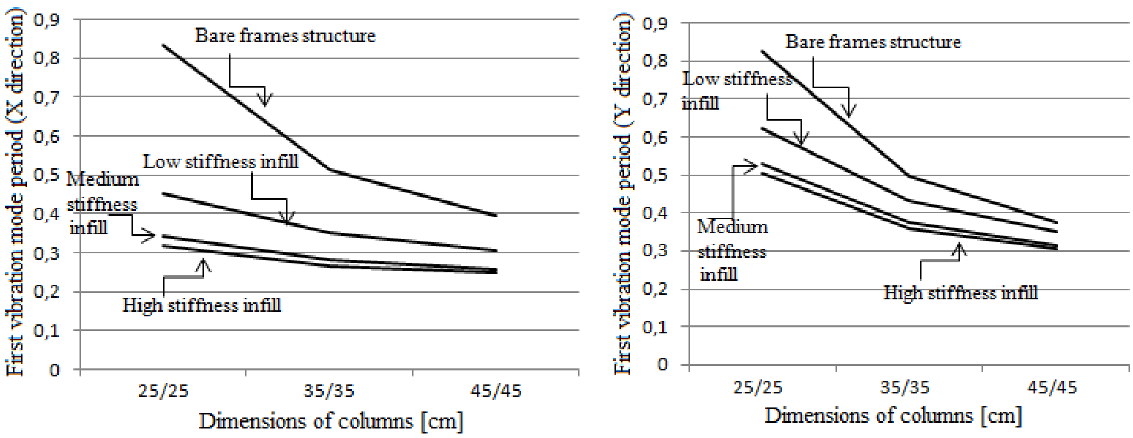
Table 1 Mechanical and deformational properties of analysed infill walls

		Dimensions of columns [cm]		
Infill stiffness levels		25/25	35/35	45/45
Low stiffness infill	f_b [N/mm^2]	12.15	17.25	16.65
	f_k [N/mm^2]	4.17	5.27	5.12
	E [N/mm^2]	2084	2618	2558
Medium stiffness infill	f_b [N/mm^2]	32.4	46.0	44.4
	f_k [N/mm^2]	8.60	10.81	10.56
	E [N/mm^2]	4302	5403	5280
High stiffness infill	f_b [N/mm^2]	44.55	63.25	61.05
	f_k [N/mm^2]	10.58	13.29	12.99
	E [N/mm^2]	5291	6645	6494



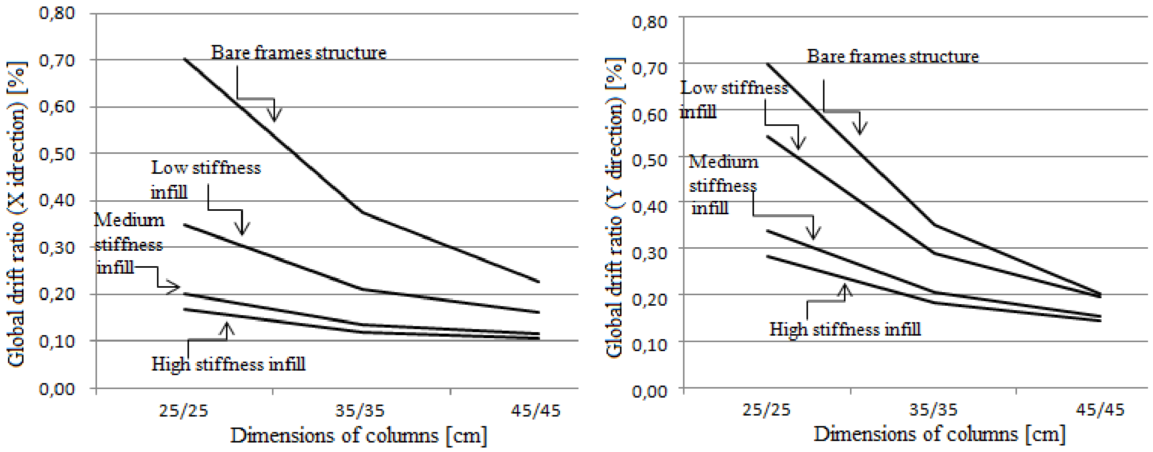
(a) Bare r.c. frames structure (b) Masonry infilled r.c. frames structure

Fig. 6 Structural models



(a) First vibration mode periods – X direction (b) First vibration mode periods – Y direction

Fig. 7 Comparison of the first vibration mode periods of frame structure with different column dimensions without infill and with different stiffness infills



(a) Global drift ratios – X direction (b) Global drift ratios – Y direction

Fig. 8 Comparison of the global drift ratios of frame structure with different column dimensions without infill and with different stiffness infills

In order to assess the influence of the masonry to r.c. frame stiffness ratio, dimension of the square columns has been varied between 25, 35 and 45 cm and each model has been combined with three different levels of masonry infill stiffness, as shown in Table 1.

Increase of overall stiffness is analysed through decrease of the first vibration mode period (see Fig. 7) and through decrease of global drift ratio (see Fig. 8).

It can be noted that lower stiffness frame structure's period of vibration and global drift ratio decrease is around two times higher than high stiffness frames structure's period of vibration andglobal drift ratio decrease. Also, as expected, decrease of vibration period and decrease of global drift ratio is higher in X direction, since there are more infill walls in X, than in Y, direction.

2.1 Lateral load capacity

As indicated by experiments (e.g., Arulselvan and Subramanian (2008), Arulselvan *et al*. (2007), Stradivaris (2009)), classical finite element models, based on the theory of elasticity, can be used for the prediction of the linear behavior of masonry infilled frame systems. In the non-linear range, however, the assumptions of the theory of elasticity are no longer valid. In order to predict the ultimate behavior of masonry infilled frames, according to Tomažević (1999), mathematical models should be developed on the basis of observed failure mechanisms. Mathematical model for simulation of inelastic response of infilled frame, developed by Žarnić (1994), was used for the assessment of resistance envelope of an infilled frame element. Model is based on experimental and analytical research of 34 one-bay, one-story models and is it reperents trilinear relationship between the displacements and base shear, as shown in Fig. 9. Expressions for calculation of the forces and stiffnesses, determining resistance envelope of an infilled frame, are derived from the basic assumptions and relations of strength of materials theory and calibrated by experimental results.

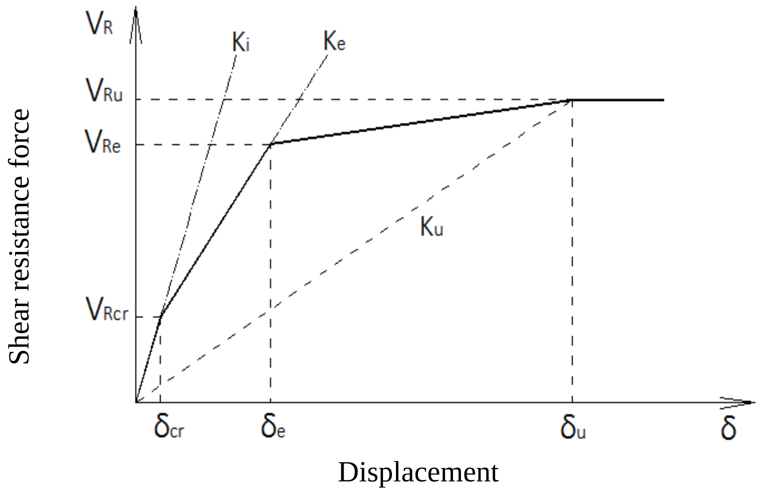


Fig. 9 Trilinear idealisation of an infilled frame behavior by Žarnić (1994)

The initial stiffness of an infilled frame, K_i, is determined by taking into account that r.c. frame and infill wall act monolithically

$$K_i = \frac{1}{\dfrac{h^3}{3 \cdot E \cdot I_e} + \dfrac{1{,}2 \cdot h}{G_i \cdot A_e}} \tag{1}$$

where h is height of the infill wall, E is modulus of elasticity of infill wall, G_i is initial shear modulus of infill wall. I_e is design moment of inertia of horizontal cross-section of frame along with infill wall

$$I_e = I + 2 \cdot C_E \cdot \frac{E_f}{E} \cdot \left(I_f + A_f \cdot \frac{(L_C + L)^2}{4} \right) \tag{2}$$

where I is the moment of inertia of masonry wall horizontal cross-section, C_E is coefficient of frame column's influence on masonry infill stiffness (in case of ideal adhesion between frame and infill $C_E = 1$ and if there is no connection between frame and infill whatsoever $C_E = 0$), A_f is the area of horizontal cross-section of the r.c. column, E_f is the modulus of elasticity of frame material, I_f is the moment of inertia of r.c. column, L_C is the length of the column's edge in direction of the infill wall and L is the length of the infill wall. A_e is the area of horizontal cross-section of the infill wall along with the frame

$$A_e = A_m + 2 \cdot C_E \cdot A_f \cdot \frac{G_f}{G_i} \tag{3}$$

where A_m is the area of horizontal cross-section of the infill wall, G_f is the shear modulus of frame material and t is thickness of the infill wall.

Stiffness of an infilled frame at the moment of separation of the infill from the frame, K_e, depends on the shear modulus of masonry infill at low level of damage, G_p, which is determined based on experimental research as follows

$$G_p = \frac{1{,}2}{\dfrac{A_m}{h \cdot K_m} - \dfrac{1}{E} \cdot \left(\dfrac{h}{L}\right)^2} \tag{4}$$

where K_m is the design stiffness of masonry infill given by

$$K_m = \frac{1}{\dfrac{h^3}{3 \cdot E \cdot I} + \dfrac{1{,}2 \cdot h}{G_i \cdot A_m}} \tag{5}$$

Stiffness of an infilled frame at the moment of separation of the infill from the frame, K_e, is determined by equation

$$K_e = \frac{1}{\dfrac{h^3}{3 \cdot E \cdot I_e} + \dfrac{1{,}2 \cdot h}{G_p \cdot A_e}} \tag{6}$$

Stiffness of an infilled frame with large crackings, i.e., the ultimate infilled frame stiffness, K_u, is calculated by considering the structure to be an equivalent diagonally-braced frame. K_u is determined as the stiffness of the uncracked frame which is supported by the 2/3 of it's height, i.e., height of the system taken into account, h_t, is 2/3 of the height h.

$$K_u = \frac{1}{\dfrac{5 \cdot h_t^3}{3 \cdot E \cdot I} + \dfrac{1{,}2 \cdot h_t}{G_p \cdot A_m}} \tag{7}$$

The shear resistance of an infilled frame, subjected simultaneously to the horizontal and vertical load, at the moment of separation of the infill, V_{Re}, is given by equation

$$V_{Re} = C_R \cdot \frac{A_m \cdot f_t}{C_I \cdot b} \cdot \left[1 + \sqrt{C_I^{\,2} \cdot \left(1 + \frac{\sigma_d}{f_t}\right) + 1}\right] \tag{8}$$

where C_R is coefficient of the quality of masonry construction work, b is the parameter of shear strengths in the wall (for infill wall with no openings, the value of b is 1,1, while value of b for infill wall with opening is at least 1,5). C_I is the coefficient of interaction between infill wall and frame and it is determined as follows

$$C_I = 2 \cdot \alpha \cdot b \cdot \frac{L}{h} \tag{9}$$

where α is coefficient of geometrical ratios, given by equation

$$\alpha = \frac{\left(x_1 - x_2\right) \cdot h}{y_1 \cdot L} \tag{10}$$

where x_1, x_2, and y_1 are the lengths shown in Fig. 10. Approximate values of these lengths can be calculated by following equations

$$x_1 = L - \frac{w}{6 \cdot sin\Phi} \tag{11}$$

$$x_2 = \frac{w}{6 \cdot sin\Phi} \tag{12}$$

$$y_1 = h - \frac{w}{6 \cdot cos\Phi} \tag{13}$$

where Φ is the slope angle of compression strut, w is the width of compression strut (see Fig. 10), calculated by following equation

$$w = \frac{A_d}{t} \tag{14}$$

where A_d is the area of the cross-section of compression strut in infill wall, determined by equation

$$A_d = K_u \cdot \frac{L_d}{E} \tag{15}$$

where L_d is the length of the compression strut in the infill wall (see Fig. 10).

Value of the compression stress in the infill wall, σ_d, is determined by next equation

$$\sigma_d = a \cdot \frac{N}{A_m} \tag{16}$$

where a is the coefficient of transmission of vertical load upon the infill wall (in case of undamaged wall, the value of coefficient a is 0,3, whereas, in case of damaged wall, a = 0), N is the vertical force carried by the infill wall.

Value of the wall's tension strength, f_t, can be calculated using the Eq. (17), if the actual vertical stress, σ_d, and shear stress, τ_u, at the moment of tension failure of freestanding infill wall, can be determined as

$$f_t = -\frac{\sigma_d}{2} + \sqrt{\left(1{,}5 \cdot \tau_u\right)^2 + \left(\frac{\sigma_d}{2}\right)^2} \tag{17}$$

The shear resistance of an infilled frame, at the moment of first cracks appearance, V_{Rcr}, can be approximately determined by next equation

$$V_{R,cr} \approx \frac{V_{Re}}{3} \tag{18}$$

The ultimate shear force, at the moment of masonry infilled frame failure, V_{Ru}, is given by equation

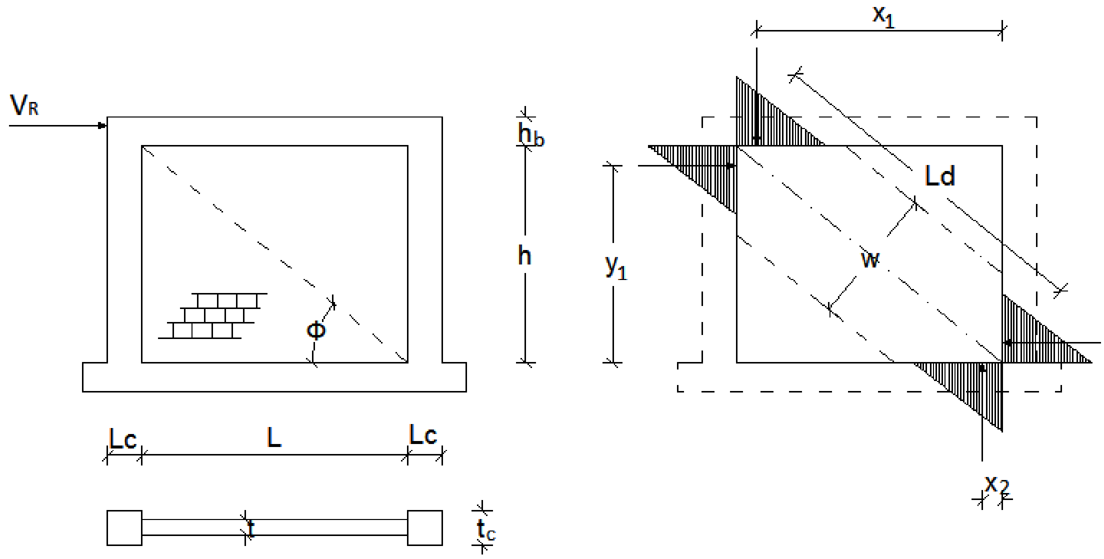


Fig. 10 model of masonry infilled frame and idealized stresses in the corners of compression strut

$$V_{Ru} = V_{Rf} + V_{Re} \tag{19}$$

where V_{Rf} is the ultimate shear resistance of frame without the masonty infill and it can be calculated by equation

$$V_{Rf} = \frac{3 \cdot M_R}{\left(h + \frac{h_b}{2}\right)} \tag{20}$$

where h_b is the total height of the beam above the infill wall and M_R is the frame's ultimate bending moment. It is considered that frame failure commences with the column' reinforcement failure.

$$M_R = A_s \cdot f_y \cdot \left(\frac{L_C}{2} - d_1\right) + f_{ck} \cdot t_c \cdot x \cdot \left(\frac{L_C}{2} - \frac{x}{2}\right) \tag{21}$$

where A_s is the area of horizontal cross-section of tensioned reinforcement in the frame's column, f_y is the yield stress of the reinforcement, f_{ck} is the characteristic compression strength of the concrete specimen in form of cylinder, L_C is the length of the column's edge in direction of the infill wall, t_c is the length of the column's edge perpendicular to the infill wall direction, d_1 is the distance from the tensioned reinforcement center of gravity to the nearest edge of the column's section, x is the height of the compression zone of the column's cross-section.

Having these values calculated, corresponding displacements can be determined. In such a way, displacement of an infilled frame at the moment of first cracks appearance, $\delta_{cr,}$ is given as

$$\delta_{cr} = \frac{V_{Rcr}}{K_i} \tag{22}$$

Displacement of an infilled frame at the moment of separation of the infill, $\delta_{e,}$ is given as

$$\delta_e = \frac{V_{Re}}{K_e} \tag{23}$$

Displacement of an infilled frame at the moment of ultimate shear force action, $\delta_{u,}$ is given as

$$\delta_u = \frac{V_{Ru}}{K_u} \tag{24}$$

2.2 Storey resistance envelope

The storey resistance envelope, which determines the relationship between the resistance and storey drift, is obtained as a superposition of resistance envelopes of all infilled frames in the storey under consideration. Either bilinear or trilinear idealization (which is the case in this paper) may be used to represent the resistance envelope of each contributing infilled frame. In the calculation of the storey resistance envelope, the structure is displaced by a small value, assuming the chosen shape of distribution of displacements along the height of the building. The infilled frames are deformed according to the assumed structural model and the resisting forces in structural members are calculated. The calculation is repeated step-by-step by increasing the imposed displacements. Once the infilled frames enter into the non-linear range, the structural system of the building and stiffness matrices are modified. The stiffness and resistance of individual infilled frames in each step of calculation are determined considering the calculated storey displacements and idealized resistance envelopes of infilled frames. As a result of calculation, the relationship between the resistance of the crititcal storey and storey drift, i.e. the resistance envelope, is obtained. At the given lateral displacement of the i-th frame δ_i, the resisting storey shear V_{tot} is determined as a sum of resistances of structural frames V_i in the storey under consideration, as shown in Fig. 11

$$V_{tot} = \sum_{i}^{n} V_i \tag{25}$$

where the resistance of the i-th frame V_i depends on the deformation of the frame δ_i.

In the calculation, the following assumptions are taken into account:

- Rigid horizontal floor diaphragm action. Frames are connected together with rigid floors and bond-beams, so that the displacements and action effects are distributed to the walls in proportion to their stiffness. Differential displacements and action effects due to torsional rotation are also distributed to the walls in proportion to their stiffness.

The contribution of an individual infilled frame to the lateral resistance of the storey depends on the lateral displacement attributed to that frame and the shape of the infilled frame's resistance envelope. Infilled frames resist the imposed displacements up to the attainment of their ductility capacity.

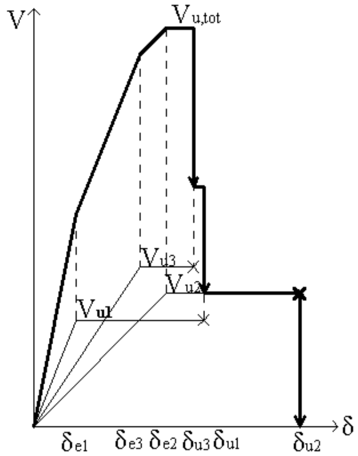


Fig. 11 Construction of the storey resistance envelope by Tomažević (1999)

Needless to mention, the resistance envelope of the critical storey is calculated and the seismic resistance is verified for the two orthogonal directions of the building under consideration.

For analysed building (see Fig. 5), critical storey resistance envelopes have been created (Muratović 2014) considering building as bare frames structure and as masonry infilled frames structure. In this case, critical storey is the first storey, since it is subjected to the highest shear forces. In order to analyse influence of the infill to frame stiffness ratio, lowest and highest rigid columns have been combined with lowest and highest stiffness infill walls. Later, these resistance envelopes have been overlapped with required resistances of corresponding structures in case of seismic action with PGA = 0,1 g and with PGA = 0,4g. Obtained results are shown in Figs. 13-19.

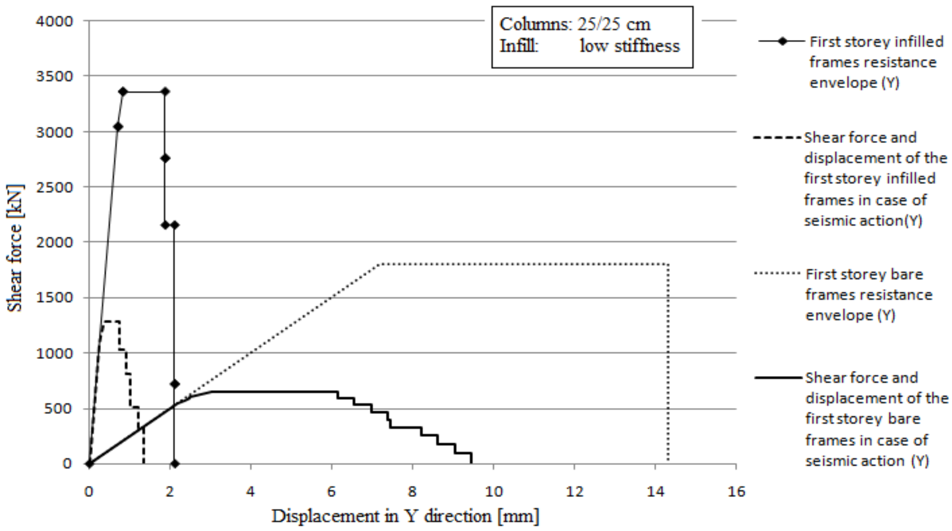


Fig. 12 Comparison of the resistance envelope and required resistance in case of seismic action with PGA = 0,1 g of the first storey frames in Y direction with and without the infill

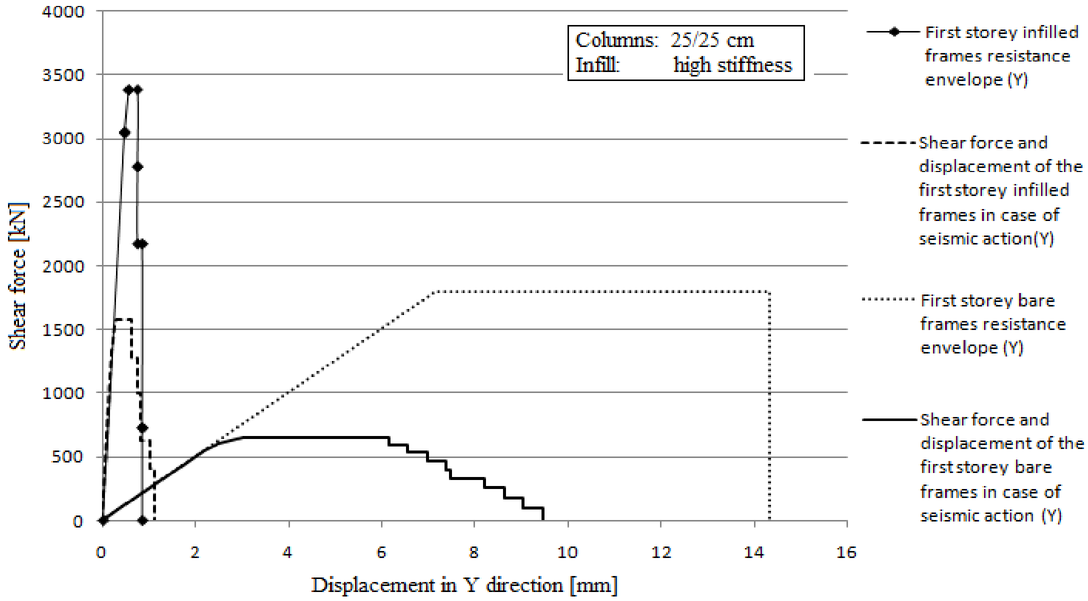


Fig. 13 Comparison of the resistance envelope and required resistance in case of seismic action with PGA = 0,1 g of the first storey frames in Y direction with and without the infill

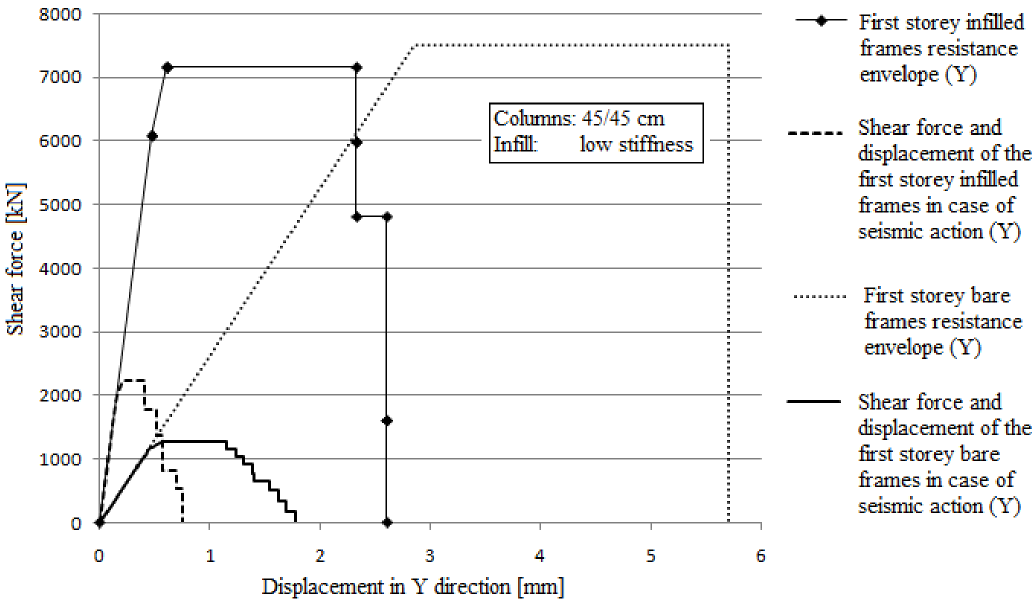


Fig. 14 Comparison of the resistance envelope and required resistance in case of seismic action with PGA = 0,1 g of the first storey frames in Y direction with and without the infill

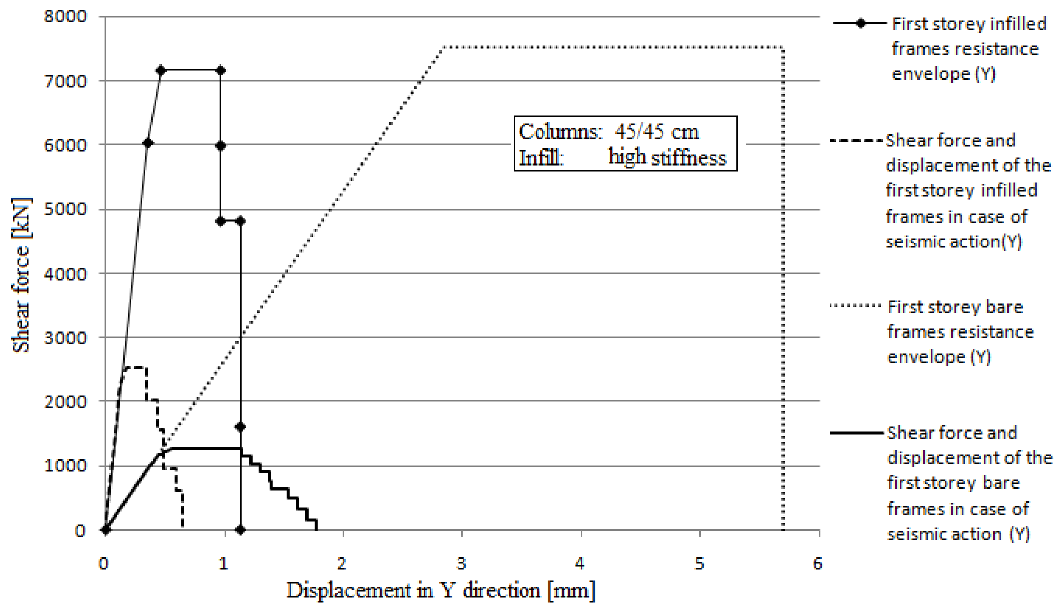


Fig. 15 Comparison of the resistance envelope and required resistance in case of seismic action with PGA = 0,1 g of the first storey frames in Y direction with and without the infill

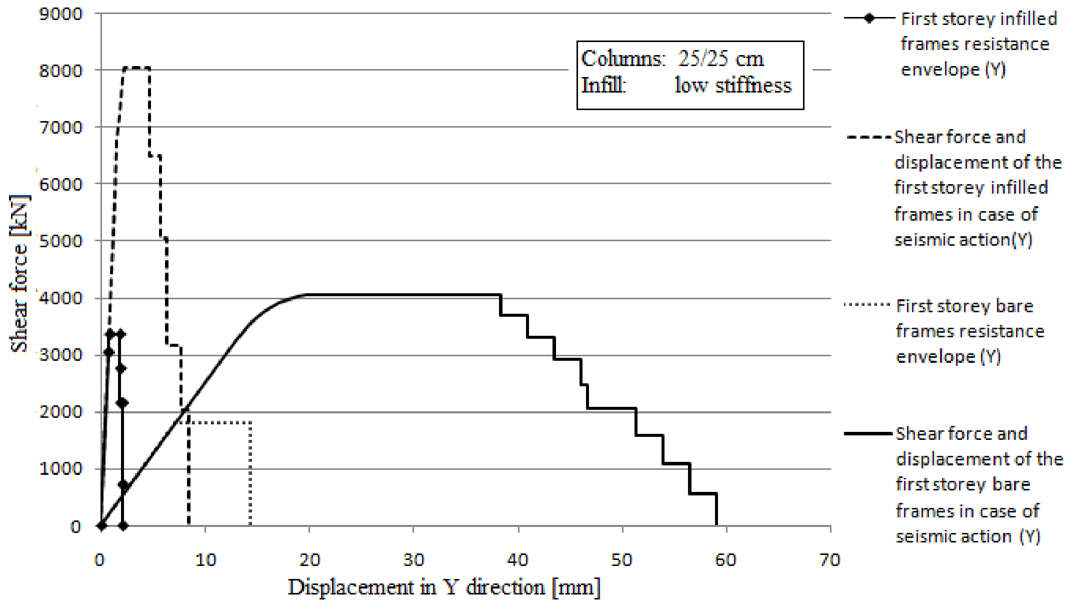


Fig. 16 Comparison of the resistance envelope and required resistance in case of seismic action with PGA = 0,4 g of the first storey frames in Y direction with and without the infill

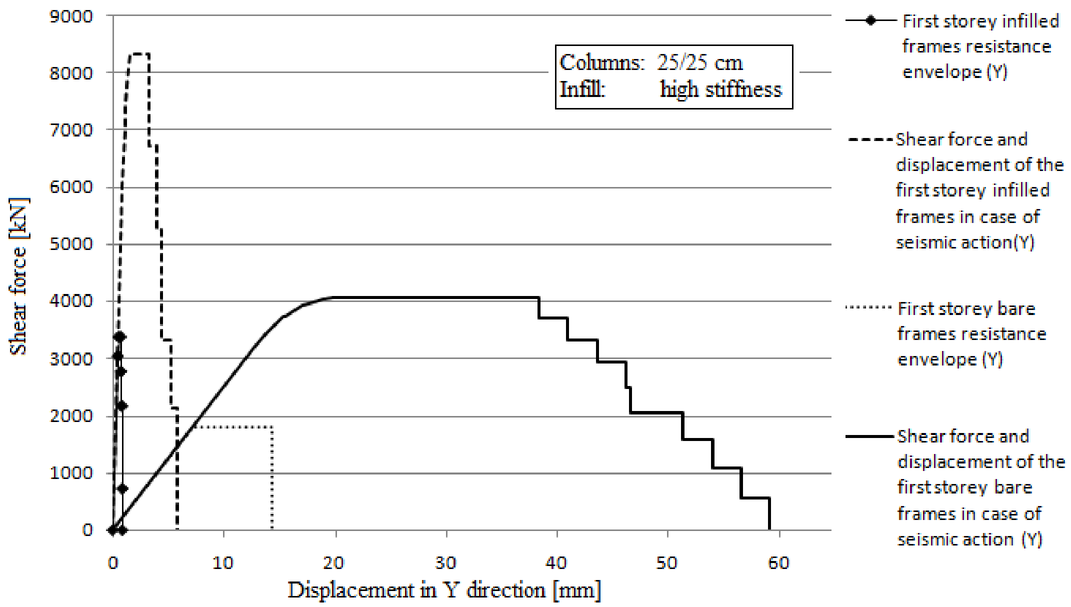


Fig. 17 Comparison of the resistance envelope and required resistance in case of seismic action with PGA = 0,4 g of the first storey frames in Y direction with and without the infill

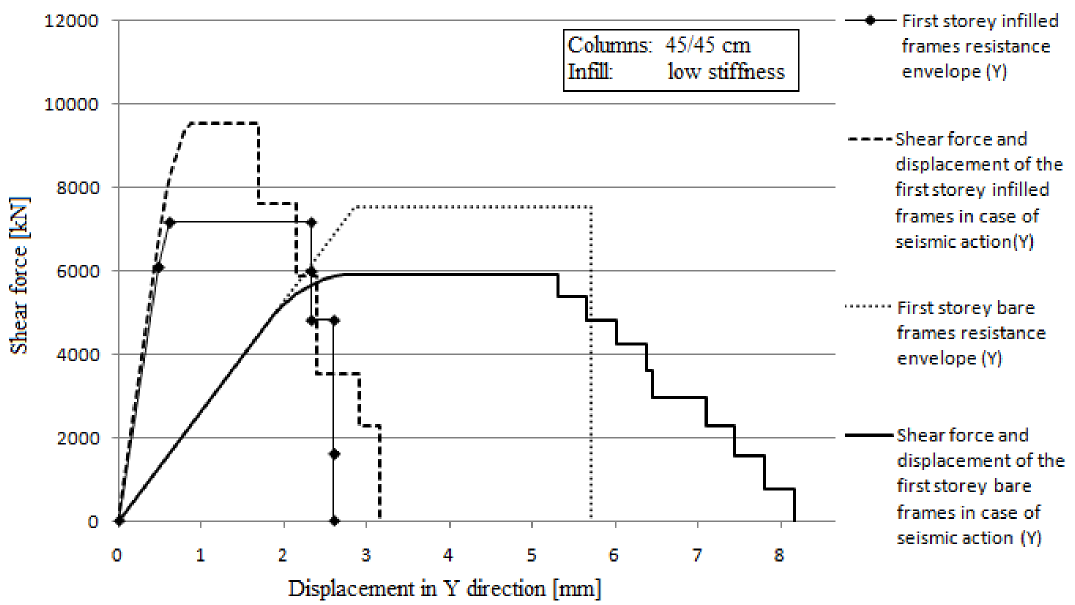


Fig. 18 Comparison of the resistance envelope and required resistance in case of seismic action with PGA = 0,4 g of the first storey frames in Y direction with and without the infill

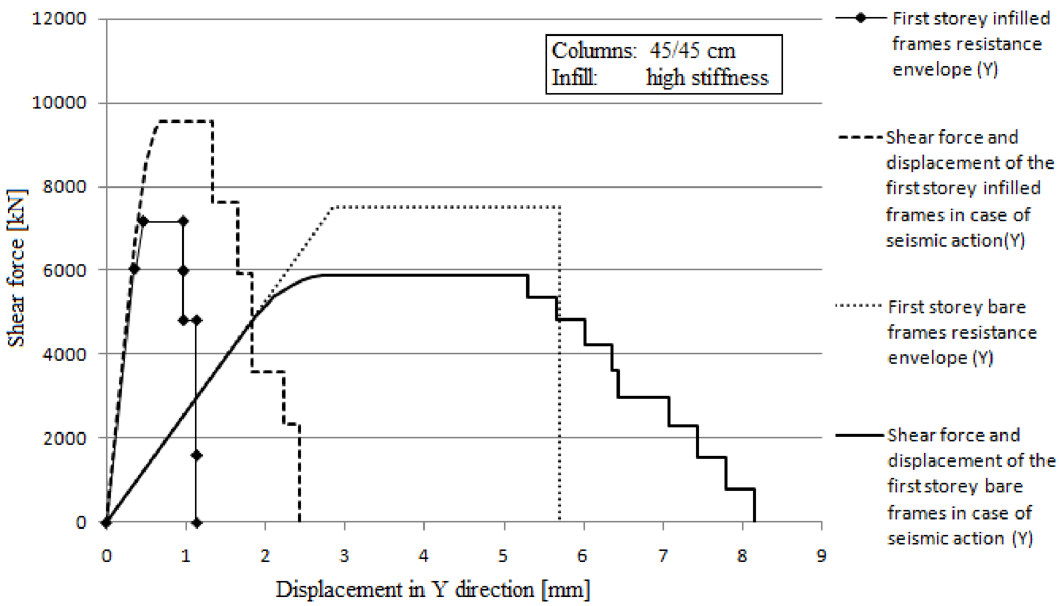


Fig. 19 Comparison of the resistance envelope and required resistance in case of seismic action with PGA = 0,4 g of the first storey frames in Y direction with and without the infill

3. Conclusions

It has been found that infill influences structure's rigidity in a way that vibration period decrease is between 7% and 60%, noting that the impact is greater if the frame stiffness is lower and infill stiffness is higher.

Analysis of different structures have shown that the seismic action on infilled frame structure is almost always twice as much as seismic action on the same structure with bare frames, regardless of the seismic intensity. However, infill increases lower stiffness frame structure's lateral load capacity, in average, two times, while infill's influence on higher stiffness frame structure is insignificant. This is the main reason why infill's influence on the seismic response cannot be ignored, since it is not in advance familiar if increased lateral load capacity will be higher than increased seismic action.

It can be noted that, optimal structure type depends on its exposure to seismic action: in case of lower seismic intensity, stiffer systems are preferred, because they provide higher lateral load capacity and smaller displacements. However, this applies only if infill stiffness is not greater that frame stiffness, because otherwise high stiffness infill can cause undesired inelastic structure's response, formation of cracks and significant damage (Fig. 5(b)). In case of higher seismic intensity, lower stiffness infill is also preferred, because high - rigid infill can lead to diagonal wall cracking, leading to columns damage, while lower stiffness infill crashes during strong seismic action and does not cause significant structure damage. It is to conclude that structure's seismic response is defined by its infill to frame stiffness ratio and its exposure to seismic action, which is mostly determined by the quality of the ground soil and type of the acting earthquake.

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