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Influence of masonry infills in torsional irregular RC buildings Part 2: Analysis and results according to the Eurocodes

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ABSTRACT

The contribution of masonry infill walls to the seismic response of in-plan torsional irregular reinforced concrete buildings was examined. A one-storey and a four-storey-building with the same layout were initially designed according to Eurocodes 2 and 8 for non-seismic and seismic actions. The seismic behavior of these buildings was consecutively assessed for a given performance level through non-linear static and non-linear time-history analyses. Both the presence and the absence of all infills, as well as different in-plan layouts of infill walls were examined. The masonry infills were modeled as proposed in Part 1, the companion of this paper. The effect of torsional irregularity and of the layout of masonry infills on the seismic behavior, in terms of resistance and displacements, are discussed for all the cases that have been considered.

1. INTRODUCTION

The presence of masonry infills enhances the capacity of a building to resist lateral forces. Many existing buildings that have been designed according to older codes that did not include specific regulations for ductile design, behave nonetheless satisfactorily during earthquakes for which they are supposed to fail according to modern concepts. This is due to an available margin of safety for lateral strength owing the presence of infills that were not taken into account in the initial calculations. In general it is on the safe side to omit the presence of infills in the design of a building. Irregular distribution of masonry infills, though, may result in increased demand for a building (in terms of resistance or displacement) as compared to the design for bare structure with no infills.

The asymmetric layout of infills in plan is in general considered less serious than the irregularity in height. Nevertheless, Eurocode 8 requires doubling the accidental eccentricity in the structural analysis of a building with planwise irregular infills that are not included in a spatial model.

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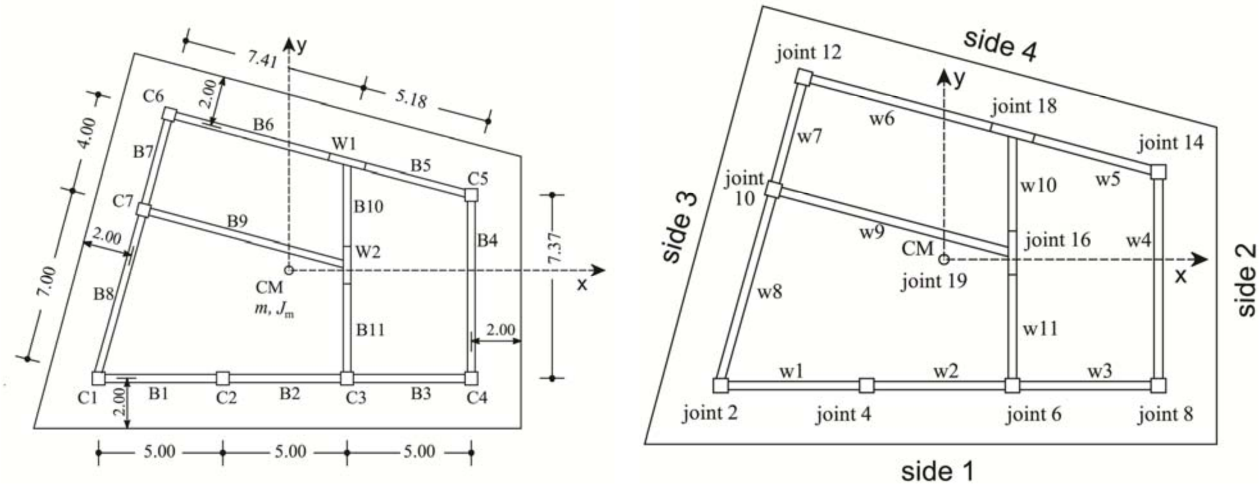
In order to investigate the influence of masonry infills on the seismic behavior of torsional in-plan irregular building, different layouts of infills were selected: Apart from the total absence of infills and the presence of all infills, two alternative cases have been examined: Omission of infills along two adjacent sides of the perimeter, first of the two flexible sides, and then of the two other sides of the perimeter. This was selected in order to enhance the torsional behavior of the structures.

2. CHARACTERISTICS OF BUILDINGS

The typical layout of the reinforced concrete (RC) buildings that were modeled is shown in Fig. 1. The buildings were chosen so as to be irregular and torsionally flexible. The structural system consists of seven rectangular columns (C1 to C7) and two RC walls (W1 and W2) connected through beams (B1 to B11). For the sake of uniformity, the same cross-section was selected in all elements of each type, and kept constant in all storeys. An uninterrupted RC slab (height of 23 cm) with a perimetric cantilever of 2.00 m ensured the diaphragmatic action around vertical axis. Storey height (between axes of beams) is 3.50 m.



vertical bearing elements and beams

joints and infill walls

Fig. 1 Typical layout of the modeled buildings

Gravity loads consisted of self-weight, uniform permanent load equal to 1.5 kN/m^2 in all slabs, and live load q=2 kN/m^2 in all interior slabs and q=5 kN/m^2 for the slab cantilevers along the perimeter. Infill walls at the perimeter were assumed to be double (g=3.56 kN per square meter of surface) and single in the interior (g=1.78 kN/m^2). The loads of the slabs were distributed to the beams and all alternative cases of gravity loads were considered with the appropriate γ-factors as designated in the Eurocodes. For the seismic action gravity loads were considered to be q + 0.3q, constant in all beams.

The single-storey building has translational mass *m=230 t*. In the 4-storey building equal masses *m=251.6 t* are assumed for the diaphragms of all storeys, so the total mass is *4m=1006.4 t*. In the model the masses are assumed to be concentrated at the geometrical center of the layout, indicated as CM in Fig. 1. The mass moments of inertia J_m around a vertical axis passing through the center of mass are $J_m=9{,}404\ t\ m^2$ and $J_m=10{,}295\ t\ m^2$ for the single- and the 4-storey buildings, respectively.

Ground type D and DCH (High Ductility Class) have been adopted. Design Importance Factor has been assumed γI=1.0 (building of "important class II" in EC8). Note that the importance factor does not enter in the calculations of storey drifts.

The design acceleration spectrum of Eurocode 8 part 1 was used, with A=0.24g (seismicity zone II) and behavior factor q=3.0 (for torsionally flexible systems).

The materials assumed are: concrete C30/37 (concrete cylinder compressive strength f_c=30 MPa). Steel grade B500C. Exposure class XD1 related to environmental conditions was assumed, i.e. clear concrete cover equal to 3.5 cm. For masonry average compressive strength f_{wc}=2.269 MPa was assumed, with modulus of elasticity E_{wc}=1702 MPa.

3. PROCEDURE

One-storey and four-storey buildings with the above described characteristics were analyzed through SAP2000.

Initially the 1-storey and 4-storey buildings were designed through SAP2000 without taking into account the contribution of infills (although their load was included in gravity loads). Design was performed according to Eurocode 2 part 1-1 and Eurocode 8 part 1 with response spectrum analysis for each principal horizontal direction, x, y, separately, applying the design acceleration spectrum of Eurocode 8 part 1. All vertical bearing members were assumed to be fixed at the foundation.

In order to account for uncertainties (in the location of masses and in the spatial variation of the seismic motion) accidental eccentricities e_{ax} = 0.05 L_{ay} and e_{ay} = 0.05 L_{ax}, where L_{ay}, L_{ax} the average dimensions of the storey layout at directions *y* and *x*, respectively, are used to calculate torsional effects of eccentricities. Eurocode 8 part 1 suggests that the calculated center of mass at each floor *i* is considered as being displayed from its nominal location in each direction by the above defined accidental eccentricities. In this work, rather than displaying the center of mass, an alternative method has been applied (Makarios and Asteris, 2013) which is simpler and leads to similar results. This method is described in the following:

The external floor static moments M_x and M_y around a vertical axis with the same sign at all floors are calculated according to the following expressions:

$$M_{x,i} = \pm F_{x,i} \cdot e_{ax} \quad (1)$$

$$M_{y,i} = \pm F_{y,i} \cdot e_{ay} \quad (2)$$

where $F_{x,i}$ and $F_{y,i}$ are the external static forces of storey *i* along the principal horizontal axes x and y of the building, and are calculated along the respective directions by the following simplified expression of Eurocode 8 part 1 (allowed when the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height)

$$F_i = F_b \cdot (z_i \cdot m_i)/(\Sigma\, z_j \cdot m_j) \tag{3}$$

where z_i and z_i are the heights of the masses m_i and m_i above the level of application of the seismic action (foundation or top of a rigid basement), and F_b is the seismic base shear calculated with expression (4)

$$F_b = m_{tot} \cdot S_a(T_1) / q \tag{4}$$

where m_{tot} is the total mass of the building, $S_a(T_1)$ is the elastic spectral acceleration for T_1, the fundamental period of vibration for lateral motion in the direction considered.

In the analyses for design the effective flexural stiffness $E_c\ I_{eff}$ was taken as 50% of the elastic one.

In all elements the minimum dimensions were selected, so as to fulfill all the prerequisites of the codes for DCH. The dimensions of the cross sections of the beams were finalized so to guarantee the required flexural resistance in combination to the fulfillment of the maximum amount of longitudinal reinforcement in beams. The dimensions of all columns were calculated as 40 cm x40 cm for the 1-storey building and 50 cm x50 cm for the 4-storey building. The shear walls are 150 cm x 30 cm and 350 cm x 35 for the one-storey and for the four-storey building, respectively. For the 1-storey building these dimensions were determined by the requirements for beam-column joints and for anchorage of longitudinal reinforcement of beams. For the 4-storey building they were determined by the capacity design of the joint. It is noted that the columns of the 1-storey building have smaller dimensions than those of the 4-storey buildings, since no capacity design of joints was performed due to the exemption of 1-storey buildings from this requirement. In both buildings the dimensions of the beam cross section is b x h = 30 cm x 60 cm and b_{eff}=150 cm.

4. ASSESSMENT

Following the design of the buildings without the contribution of infills, assessment of the buildings was performed through a) non-linear static (pushover) analysis and b) non-linear time history analysis. Analyses were also performed with the structures as planar frames (without floor rotational Degree Of Freedom around vertical axis) in order to exclude the torsional effects.

Different layouts of infills were examined:

- model with all infills (masonry infills were placed under all the beams)
- model with no infills
- model with no infills along sides 1 and 2 (Fig. 1)
- model with no infills along sides 3 and 4 (Fig. 1)

In both types of analysis all the prescriptive rules required in EC8-3 were applied for the modeling of the structure, e.g. confined characteristics of compressive concrete strength, mean strength values for materials. In all critical sections the moment-chord rotation (M - θ) diagrams were calculated, assuming an ideal perfectly elastic-plastic relationship. Inelastic springs with the derived (M - θ) characteristics were added to all

RC end members. In the analyses the effective flexural stiffness the mean value of $M_y L_v / 3\theta_y$ at the two ends of the elements, where M_y is the yield moment, L_v is the shear span taken to be equal to half the element length, and θ_y is the chord rotation at yielding, was used for all RC structural members (see Part 1, companion paper). The infills were modeled as described in the companion paper, Part 1 (Fotakopoulos et al, 2013, Makarios 2013). It is noted that the bending moments in vertical members due to gravity loads have been taken into account (although EC8 allows them to be neglected in non-linear analysis unless they are significant with respect to the yield moment.) by starting the non-linear seismic response analysis from a non-zero initial force state.

In time history analysis three pairs of horizontal artificial seismic accelerograms have been used (Makarios and Asteris, 2013) which are compatible with the design elastic response spectrum that is proposed by Eurocode 8 for soil category D and equivalent viscous ratio damping 0.05. The accelerograms of each pair are practically uncorrelated and act simultaneously. All accelerograms are digitized every 0.005 sec, have total duration 25 sec and the strong motion duration is more than 18 sec.

5. RESULTS

The vibration periods of the one-storey and the four-storey buildings are shown in Tables 1 and 2 for three different values of stiffness of the frame members: elastic, 50% elastic, and effective stiffness according to Eurocode 8 part 3.

In Figures 2 and 3 the maximum top horizontal displacements at the –x axis for the one-storey building are depicted for various joints, for structure with no infills and for structure with infills along all beams (Fig. 3). In the analysis with no infills the larger displacements occur in joint 8 which is at the corner of the two “flexible” sides (1 and 2) of the building, while the smaller displacements occur at joints 10 and 12, around the point of intersection of the rigid sides (3 and 4).

Table 1 Vibration periods of the one-storey building with and without infills

With no infills			
Mode	Reduced elastic stiffness 50%	Effective stiffness EN1998-3	Elastic stiffness 100%
1	0.34168	0.55601	0.24392
2	0.23206	0.4579	0.16895
3	0.17072	0.34065	0.12512
With all infills			
Mode	Reduced elastic stiffness 50%	Effective stiffness EN1998-3	Elastic stiffness 100%
1	0.17738	0.21311	0.15737
2	0.16161	0.19184	0.13537
3	0.12927	0.17032	0.10597

Table 2 Vibration periods of the four-storey building with and without infills

With no infills			
Mode	Reduced elastic stiffness 50%	Effective stiffness EN1998-3	Elastic stiffness 100%
1	0.87122	1.26457	0.62638
2	0.58082	0.82783	0.42338
3	0.43551	0.55992	0.31627
With all infills			
Mode	Reduced elastic stiffness 50%	Effective stiffness EN1998-3	Elastic stiffness 100%
1	0.55682	0.608514	0.489651
2	0.421452	0.498817	0.359544
3	0.311809	0.383599	0.257388

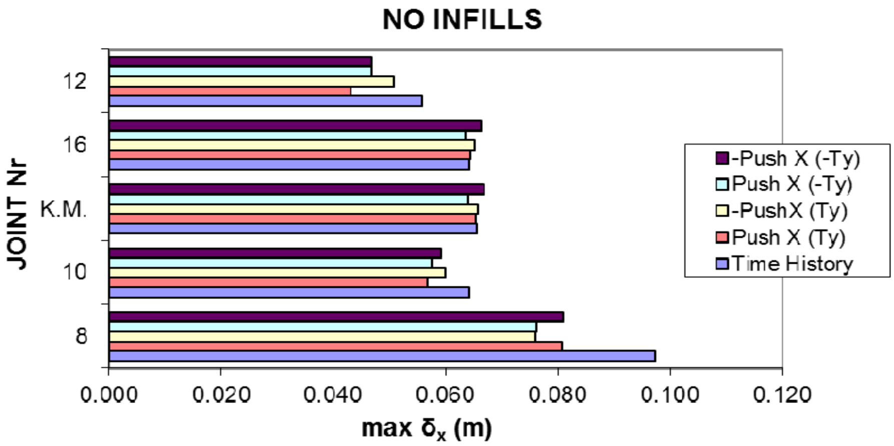


Fig. 2 Maximum joint displacements along x-axis in case of no infills

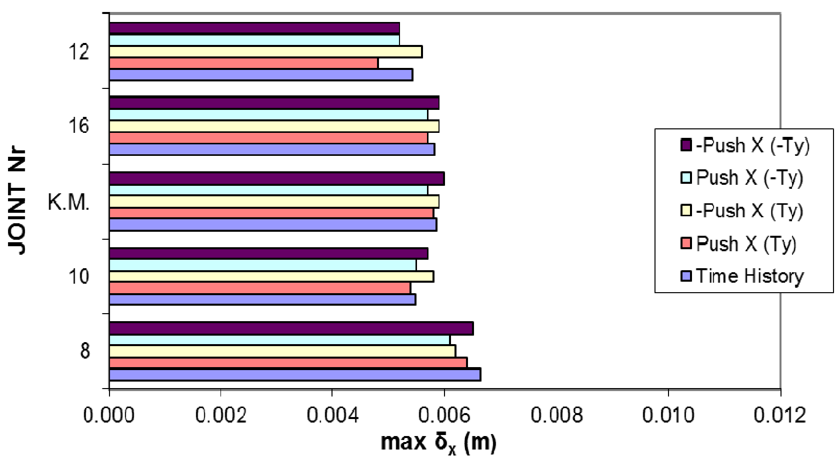


Fig. 3 Maximum joint displacements along x-axis in case of infills along all beams

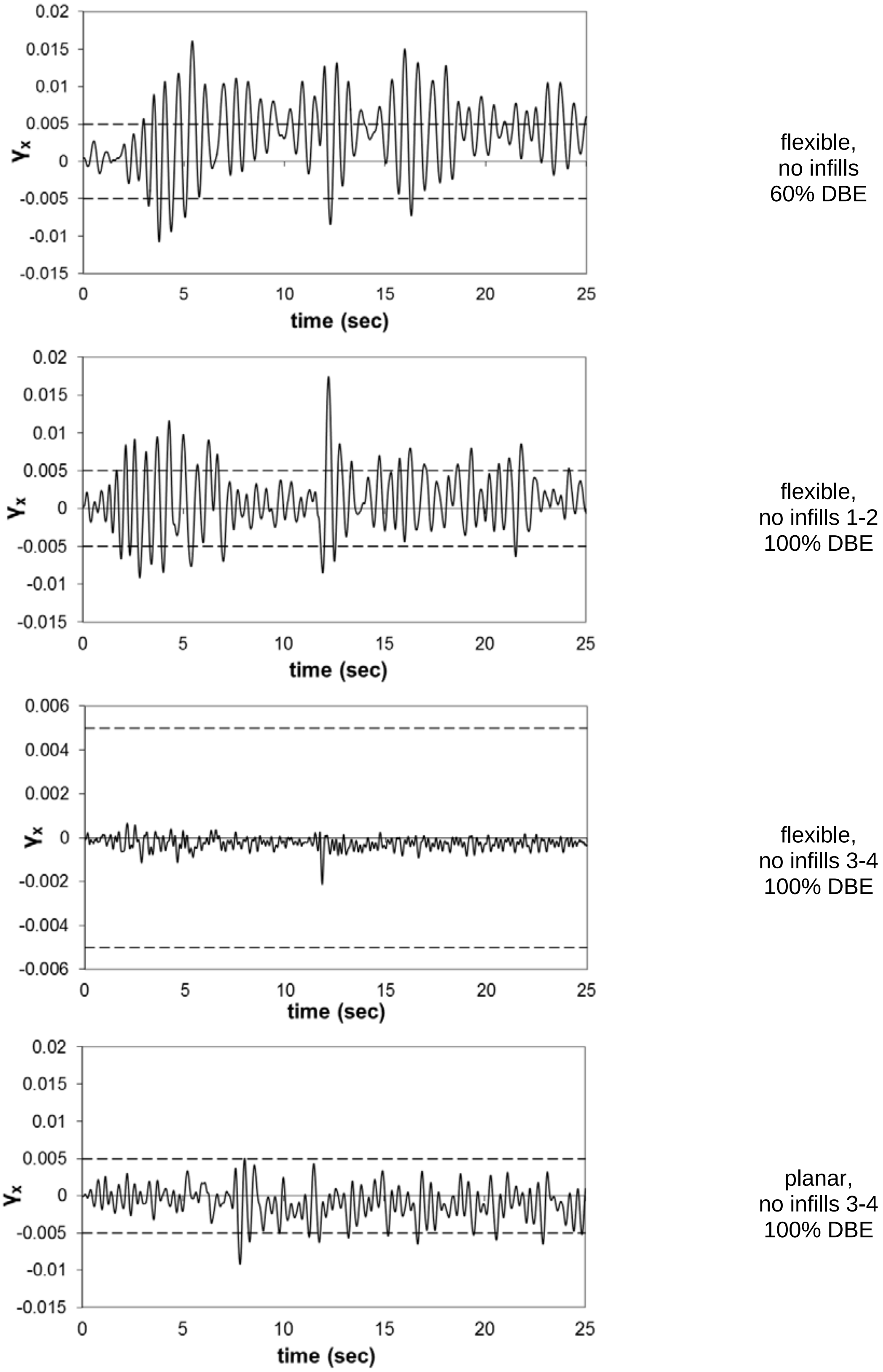


Fig. 4 Variation of drifts along x-axis of joint 8 for the one-storey building

A comparison between the displacements calculated by time history analysis (TH) and the pushover analysis along the x-axis for the four possible combinations with the moment due to accidental eccentricity shows that in this case pushover analysis may reliably estimate the maximum displacements of the joints with the exception of joint 8 for the case of no infills.

It is pointed out that all results of displacements refer to the “Significant Damage” state, with corresponding seismic action with mean return period of 475 years, and a 10% exceedance probability in 50 years. This earthquake is considered as the Design Basis Earthquake (DBA) (CEN, 20004) and it is the earthquake level for which the design in new structures is performed.

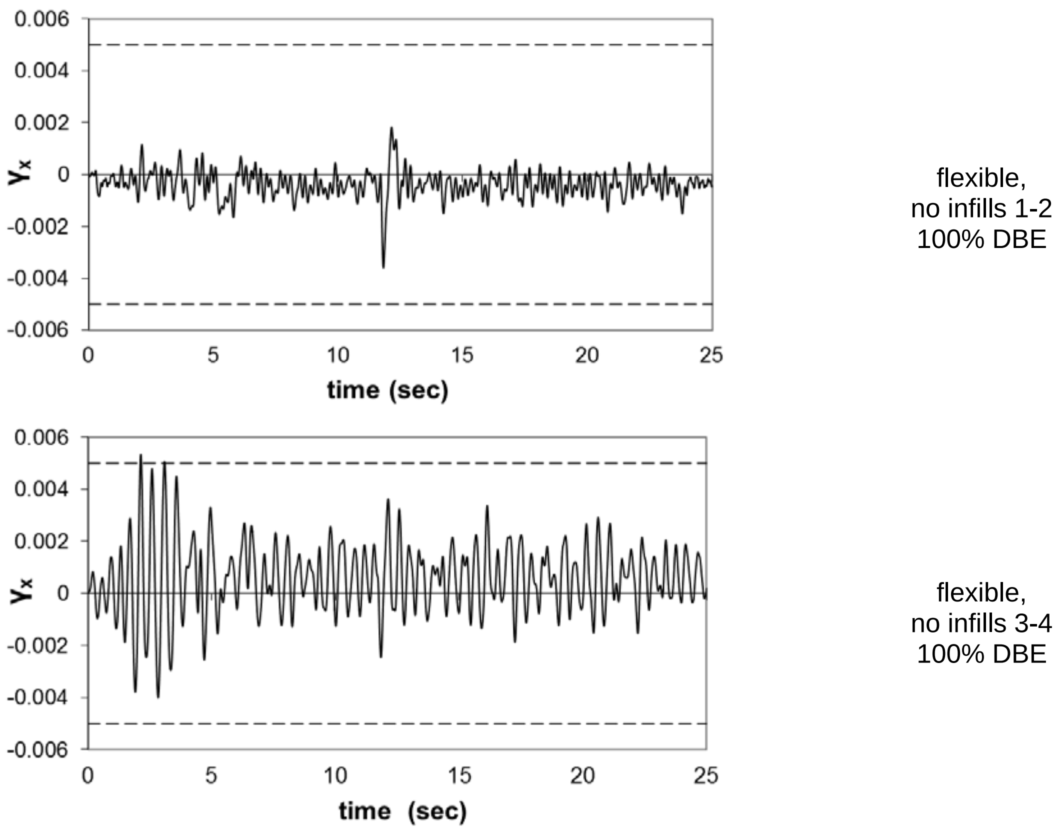


Fig. 5 Variation of drifts along x-axis of joint 12 for the one-storey building

In Fig. 4 are shown the drifts, γ_x, (displacements along axis x divided by storey height) of joint 8 in case of no infills at all, no infills along sides 1 and 2, no infills along sides 3 and 4, and with no infills at all but for structural analysis as planar frames. The absence of infills along the “rigid” sides 3 and 4 does not seem to increase the displacements compared to the case of presence of all infills. In case of planar frames without infills the displacements are considerably reduced. It is noted that for no contribution of infills for torsional flexible structure the displacements shown correspond to only 60% of DBA, since it is the highest earthquake resisted by the building.

In case of joint 12 (in the rigid corner of the building) the absence of infills along sides 3 and 4 increase significantly the displacements of the joint as compared to the absence of the infills along the sides 1 and 2 as shown in Fig. 5.

In Fig. 4 and Fig. 5 the limit of drift γ=0.005 is indicated for the sake of comparison. This is an interstorey drift limit and its validity should be verified in buildings with non-structural elements of brittle materials attached to the structure. In case of assessment, this verification corresponds to the limit state of design Damage Limitation (DL), with a corresponding seismic action with mean return period of 225 years, and a 20% exceedance probability in 50 years. It is noted that verification against the exceedance of this limit state is not required in Eurocode 8 part 3. Nevertheless in order to perform this check, the effective flexural stiffness of RC elements in the analysis should be 50% of that corresponding to the geometric cross-sections and not the reduced stiffness assumed for the DL limit state for the earthquake with mean return period of 475 years.

In Figs. 6 and 7 are shown the diagrams of base shear against displacement from pushover analysis of the four-storey building. All the cases considered regarding the presence of infills are shown.

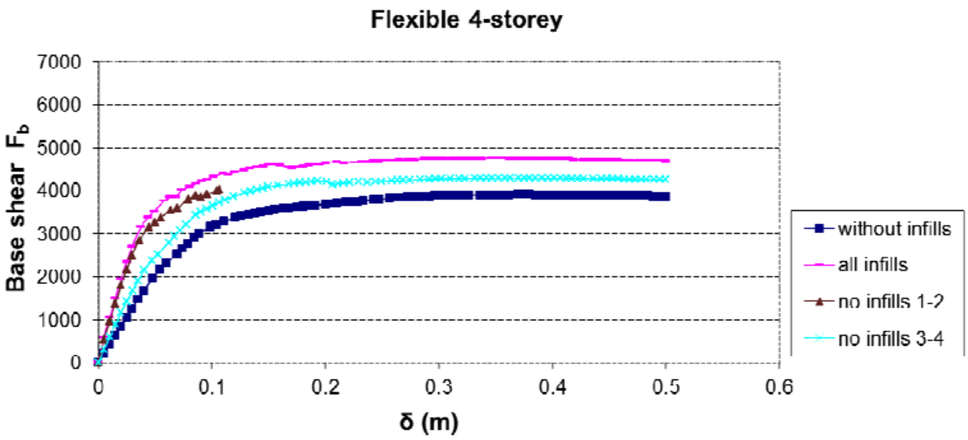


Fig. 6 Drifts along x-axis of joint 8 for flexible four-storey building

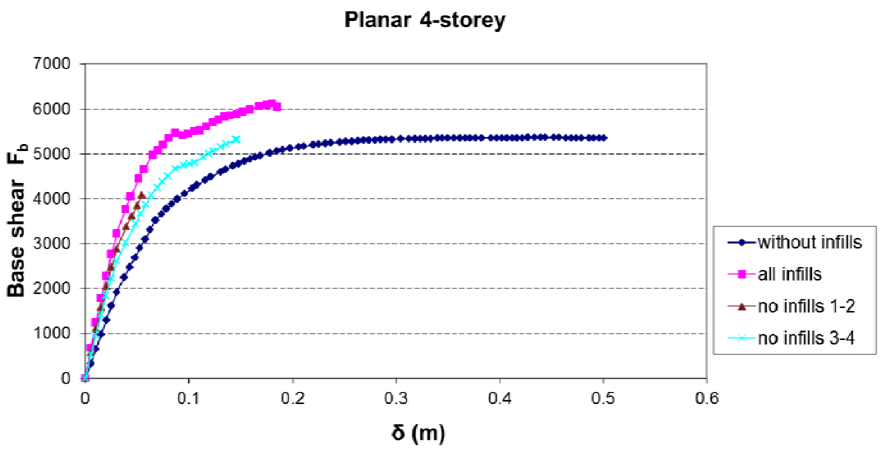


Fig. 7 Drifts along x-axis of joint 8 for planar four-storey building

6. CONCLUSIONS

The participation of infills in the analytical models examined increased the stiffness and strength of the structure as anticipated. In-plan irregulartiy in the layout of infills leads to increased displacement of the joints close to areas with no infills. More affected from the absence of infills are the joints near the flexible sides of the buildings, particularly in case of torsional irregular buildings in which certain joints are subjected to increased displacements, regardless of the irregularity of infills.

In case in-plan torsional irregular buildings for the joints that are at the flexible sides of the building the estimation of storey drifts and displacements should be performed through t
ime-history analysis so as not to be underestimated.

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