NON LINEAR STATIC ANALYSIS OF A REINFORCED CONCRETE BUILDING WITH AND WITHOUT INFILL WALLS

ANALYSES STATIQUE NON LINEAIRE D'UN BATIMENT EN BETON ARME AVEC ET SANS MURS EN MAÇONNERIE DE REMPLISSAGE

Youcef Mehani[1], Abderrahmane Kibboua[1], Benazouz Chikh[1], Mustapha Remki[1]

Réception : 08/05/2017 Acceptation : 10/08/2017 Publication : 31/01/2018

[1]Centre National de Recherche Appliquée en génie Parasismique (CGS), Alger, e-mail : ymehani@cgs-dz.org

[1]Centre National de Recherche Appliquée en génie Parasismique (CGS), Alger, e-mail : akibboua@cgs-dz.org

[1]Centre National de Recherche Appliquée en génie Parasismique (CGS), Alger, e-mail : bchikh@cgs-dz.org

1National Earthquake Engineering Research Center, mremki@cgs-dz.org, ALGÉRIE

ABSTRACT. Many post earthquakes investigations have shown the important role that infill walls play in seismic response of structures and their effects in some situations susceptible to provoke high level of damage including collapse. Despite their importance to define a performance level of the whole building, they are often neglected in numerical models and analyses, because they are generally considered to be non structural elements and not considered as a part of load bearing system. When a reinforced concrete frame with infill walls is subjected to lateral deformations, the infill wall acts as a diagonal strut, while the separation of the infill occurs on the opposite side. This study investigates the performance levels of a building with and without infill walls, using macro-modelling elements for infill walls and nonlinear static analysis (pushover). The results are carried out in accordance with the Algerian Seismic Design Code in force RPA99/version 2003 and ETABS 2015 program, using N2 method proposed by Eurocode 8.

.

Keywords: Reinforced concrete frames, Infill walls, Equivalent strut model, Pushover analysis, Capacity curve, Lateral displacements.

RESUME. Beaucoup d'investigations post-sismiques ont prouvé que les murs de remplissage en maçonnerie jouent un rôle important dans la réponse sismique des structures et leurs effets dans certaines situations peuvent provoquer de grands dommages allant jusqu'à l'effondrement. En dépit de leur importance pour définir un niveau de performances du bâtiment en entier, ils sont souvent négligés dans les modèles numériques et les analyses parce qu'ils sont généralement considérés comme éléments non structurels et non pas comme partie du système porteur. Lorsqu'un portique en béton armé avec mur de remplissage en maçonnerie est soumis aux déformations latérales, le mur de remplissage agit en tant qu'élément bielle (diagonale), alors que la séparation du remplissage se produit du côté opposé. Cette étude traite des niveaux de performances d'un bâtiment avec et sans murs de remplissage en utilisant les éléments de macro modélisation pour ces derniers ainsi que l'analyse statique non linéaire (Pushover). Les résultats sont obtenus selon les Règles Parasismiques Algériennes RPA99/version 2003 via le programme ETABS 2015, et selon la méthode N2 proposée par l' Eurocode 8.

Mots-clés : Portiques en béton armé, maçonnerie de remplissage, Modèle de bielle équivalent, Analyse par poussée progressive, Courbe de capacité, déplacement transversal.

1- Introduction

In many countries, reinforced concrete frames are filled by brick masonry panels. Although the infill walls could enhance the stiffness and the strength of the structure, their contribution is often not considered in the analyses. The main problem is that behavior of infill walls is difficult to predict because of significant variations in material properties and failure modes that are brittle in nature. The 2003 Boumerdes major earthquake, magnitude 6.9 showed some shortcomings and negative effects on the local and global behavior of buildings. This paper compares analyses related to reinforced concrete structures with and without infill walls in terms of capacity curves according to the new Algerian seismic code RPA99/version 2003 [1].

2- Non linear analysis

The evaluation of seismic performance of any structure requires the assessment of its dynamic characteristics and the prediction of its response to a probable earthquake motion that could take place in the future during the building service life. The deterministic approach of the seismic performance is derived by using the nonlinear static analysis which determines the lateral load resisting capacity of a structure and the maximum level of damage in the structure at the ultimate load in terms of a capacity curve.

2.1- Non linear static analysis

Linear static analysis assumes that the relationship between loads and the induced response is linear. For instance, if you double the magnitude of loads, the response (displacements, strains, stresses, reaction forces, etc.), will also double. All real structures behave nonlinearly at some level of loading. In some cases, linear analysis may be adequate. In many other cases, the linear solution can produce erroneous results because the conservatism assumptions. Nonlinear analysis methods are best applied when either geometric or material nonlinearity is considered [2, 3].

- ✓ ***Geometric nonlinearity***: This is a type of nonlinearity where the structure is still elastic, but the effects of large deflections cause the geometry of the structure to change, so that linear elastic theory breaks down. Typical problems that lie in this category are the elastic instability of structures, such as in the Euler bulking of struts and the large deflection analysis of a beam-column member. In general, it can be said that for geometrical non-linearity, an axially applied compressive force in a member decreases its bending stiffness, but an axially applied tensile force increases its bending stiffness. In addition, P-Delta effect is also included in this concept.
- ✓ ***Material nonlinearity***: In this type of nonlinearity, material undergoes plastic deformation. Material nonlinearity can be modeled as discrete hinges at a number of locations along the length of a frame (beam or column) element and a discrete hinge for a brace element as discrete material fibers distributed over the cross-section of the element, or as a series of material points throughout the element.

2.2- Static Pushover Analysis

It is a static nonlinear procedure in which a structural system is subjected to a constant gravity loading and a monotonic lateral load which increases iteratively, through elastic and inelastic behavior until an ultimate condition is reached to indicate a range of performance level. As a function of both strength and deformation, the resultant nonlinear force-deformation (F- δ) relationship defines the base shear versus the roof displacement and may be proportional to the distribution of mass along the building height, mode shapes, or fundamental lateral loads mode, to define a capacity curve. The capacity curve defines in general four structural performance levels:

- **Fully Operational**: No significant damages have occurred to structural and nonstructural components. Building is suitable for normal occupancy and use.
- **Operational:** no significant damage has occurred to structure, which retains nearly all of its pre-earthquake strength and stiffness. Non structural components are secure and most would function.

- **Life Safety**: significant damage to structural elements, with substantial reduction in stiffness, however, margin remains against collapse. Nonstructural elements are secured but may not function. Occupancy may be prevented until repair can be instituted.
- **Near Collapse**: substantial structural and nonstructural damage. Structural strength and stiffness substantially degraded. Little margin against collapse. Some falling debris hazards may have occurred.

The capacity curve can then be combined with a demand curve, typically in the form of an Acceleration Displacement Response Spectrum (ADRS). This combination essentially reduces the problem to an equivalent single degree of freedom system. Static pushover analysis is most suitable for systems in which the fundamental mode dominates the behavior of the structure. Results provide insight into the ductile capacity of the structural system, and indicate the mechanism, load level, and deflection at which failure occurs.

There are two nonlinear procedures using pushover methods:

- Capacity Spectrum Method.
- Displacement Coefficient Method.

2.3- Pushover Analysis according to EC8

Pushover analysis is performed under two lateral load patterns. A load distribution corresponding to the fundamental mode shape and a uniform distribution proportional to masses [4, 5].

Classical steps of the Eurocode 8 approach are then [6]:

- Cantilever model of the structure with concentrated masses with elastic behavior (uncracked cross sections).
- Determination of the fundamental period of vibration.
- Determination of the fundamental mode shape (Eigen vectors) normalized in such a way that ϕ_n=1.
- Determination of the modal mass coefficient for the first natural mode.
- Determination of the lateral displacements at each level for the first natural mode.
- Determination of seismic forces at each level for the first natural mode.
- Transformation of the multi degree of freedom system to an equivalent single degree of freedom with an equivalent mass m* determined as:

$$m^* = \sum m_j \phi_{j,1} = \sum \overline{F}_j \qquad (1)$$

Where:

m^*: mass of the equivalent single degree of freedom.

$\phi_{j,1}$: normalized eigen vector of the fundamental mode.

$\overline{F}_j$: seismic forces at level j of the fundamental mode.

- Determination of the modal participation factor of the first natural mode:

$$\Gamma = \frac{m^*}{\sum m_j \phi_{j,1}^2} = \frac{\sum \overline{F}_j}{\sum \left(\frac{\overline{F}_j^2}{m_j} \right)} \qquad (2)$$

The force F* and displacement d*of the equivalent single degree of freedom are computed as:

$$F^* = \frac{F_b}{\Gamma} \quad et \quad d^* = \frac{d_n}{\Gamma} \tag{3}$$

Where F_b and d_n are, respectively, the base shear force and the control node displacement of the multi degree of freedom system.

- Determination of the idealized elasto perfectly plastic force displacement relationship.

The yield force F_y^*, which represents also the ultimate strength of the equivalent single degree of freedom system, is equal to the base shear force at the formation of the plastic mechanism. The initial stiffness of the equivalent single degree of freedom system is determined in such a way that the areas under the actual and the equivalent single degree of freedom system force displacement curves are equal. Based on this assumption, the yield displacement of the equivalent single degree of freedom system d_y^* is given by:

$$d_y^* = 2\left(d_m^* - \frac{E_m^*}{F_y^*}\right) \tag{4}$$

Where E_m^* is the actual deformation energy up to the formation of the plastic mechanism and d_m^* is the estimated target displacement. Figure 1 shows the principle of energies idealization.

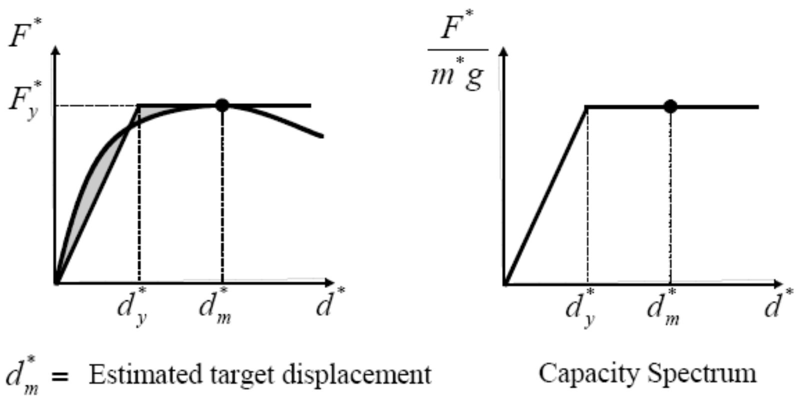

Figure 1: Linearization of capacity curve (Linéarisation de la courbe de capacité)

- Determination of the period of the equivalent single degree of freedom system T* as:

$$T^* = 2\pi\sqrt{\frac{m^* d_y^*}{F_y^*}} \tag{5}$$

- Determination of the target displacement for the equivalent single degree of freedom system as:

$$d_{et}^{*} = S_e(T^{*})\left(\frac{T^{*}}{2\pi}\right)^2 \quad (6)$$

Where S_e (T*) is the elastic acceleration response spectrum at the period T*. For the determination of the target displacement d_t^{*} for structures in the short period range and for structures in the medium and long period ranges, different expressions should be used:

a) $T^{*} < T_C$ (short period range)

- If $F_y^{*}/m^{*} \geq S_e(T^{*})$, the response is elastic and thus:

$$d_t^{*} = d_{et}^{*} \quad (7)$$

- If $F_y^{*}/m^{*} < S_e(T^{*})$, the response is non linear and:

$$d_t^{*} = \frac{d_{et}^{*}}{q_u}\left(1 + (q_u - 1)\frac{T_C}{T^{*}}\right) \geq d_{et}^{*} \quad \text{with} \quad q_u = \frac{S_e(T^{*})m^{*}}{F_y^{*}} \quad (8)$$

Where d_{et}^{*} is the target displacement for the equivalent single degree of freedom system and q_u is the ratio between the acceleration in the structure with unlimited elastic behavior S_e (T*) and in the structure with limited strength F_y^{*}/m^{*}.

b) $T^{*} \geq T_C$ (medium and long period range), the response is nonlinear and:

$$d_t^{*} = d_{et}^{*} \quad with \quad d_t^{*} \leq 3d_{et}^{*} \quad (9)$$

- The target displacement of the multi degree of freedom system is then:

$$d_t = \Gamma d_t^{*} \quad (10)$$

3- Modeling approaches of infill walls with macro-models

To consider the effect of masonry infill walls to carry horizontal loadings due to wind or seismic action, various approximate methods have been proposed by researchers. Significant experimental and analytical research is reported in the literature since decades, which attempts to understand the behavior of infill panels. The available analytical models are categorized in macro and micro models. The proposed analytical development assumes that the contribution of the masonry infill panels to the global response of the structure can be modeled by replacing the brick infill panel by an equivalent diagonal strut. Originally, Polyakov [7] suggested the possibility of considering the effect of modeling the masonry infill panels as equivalent to one diagonal strut, which was later modified by Holmes [8] that replaced the masonry infill panel with an equivalent pin-jointed diagonal strut made of the same material and having the same thickness of the masonry infill wall and subsequently developed by Stafford- Smith, as shown in Fig. 1. Since the tensile strength of masonry is negligible, the strut is to be ineffective in tension and, then, is activated only in compression [9]. This approach appears to be very attractive due to the obvious advantage in terms of computation simplicity [10].

The effective characteristics of equivalent strut are defined as a rectangular section as shown in Fig. 2, and given by the following expressions [11-13]:

$$A_{ef} = W_{ef} \times B_t \quad with \quad W_{ef} = 0.10 \; to \; 0.20 \; L_d \tag{11}$$

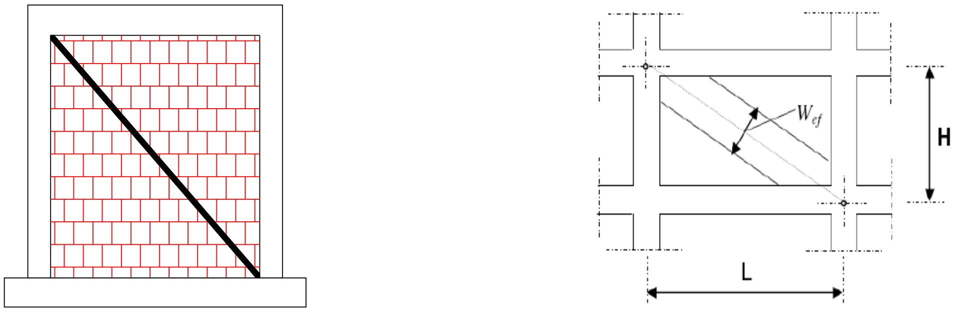

Figure 2: Single diagonal strut model of masonry infill walls (Modèle d'une bielle diagonale d'un mur de remplissage en maçonnerie)

Where:

A_{ef}: effective strut cross section.
W_{ef}: effective strut depth.
B_t: thickness of the infill panel.
L_d: effective strut length.
L: beam length.
H: column height.

Thus the reinforced concrete frames with unreinforced masonry walls can be modeled as equivalent braced frames with infill walls replaced by equivalent diagonal struts [14]. To include material nonlinear behavior, frame hinges need to be defined and assigned to both extremities of the concrete elements (M2 hinges for beams and PMM hinges for columns). For the diagonal struts, hinges activated by compression axial forces were also assigned to the extremities of the elements (P hinges) [15-17].

4- Case study

With the aim of evaluating the influence of the masonry infill walls in the structural response when subjected to seismic loadings, two case studies without and with infill walls will be discussed. The transversal direction (YY) will be the interest of our study. The building is five (05) stories with regularity in plan and elevation. The structure is reinforced concrete resisting moment frames with hollow clay bricks infill walls. Typical bay width and columns height in this study are selected as 4 m and 3 m respectively. A configuration of four (04) longitudinal bays, three (03) transversal bays is considered in this study. The building is located in a high seismic zone (Zone III) with housing usage [1]. The details of beams and columns are shown in Table1.

- The grade of concrete is fc28 = 25 MPa.
- The grade of steel is fe = 400 MPa.
- The roof dead load is GT = 6.54 KN/m².
- The roof live load is QT =1KN/m².
- The current level dead load is GE = 6.19 KN/m².
- The current level live load is QE = 1.5 KN/m².

Figures 3 and 4 show a 3-D model of the structure without and with masonry infill walls, using ETABS program [18].

Table 1: Dimensions of beams and columns (Dimensions des poutres et poteaux)

Story	Beam (cm²)	Longitudinal Reinforcement	Column (cm²)	Longitudinal Reinforcement
5	1.22	3HA16	30x30	8HA12
4	3.60	3HA16	35x35	4HA16 + 4HA14
3	1.22	3HA16	35x35	4HA16 + 4HA14
2	3.60	3HA16	40x40	8HA16
1	3.60	3HA16	40x40	8HA16

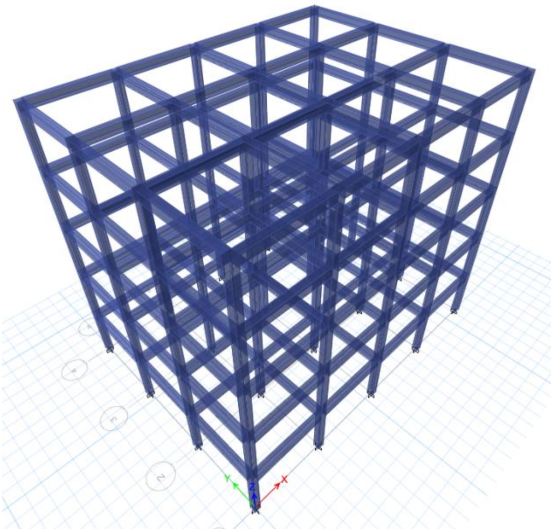

Figure 3: Three-dimensional model of the structure without infill walls

(Modèle tridimensionnel de la structure sans murs de remplissage)

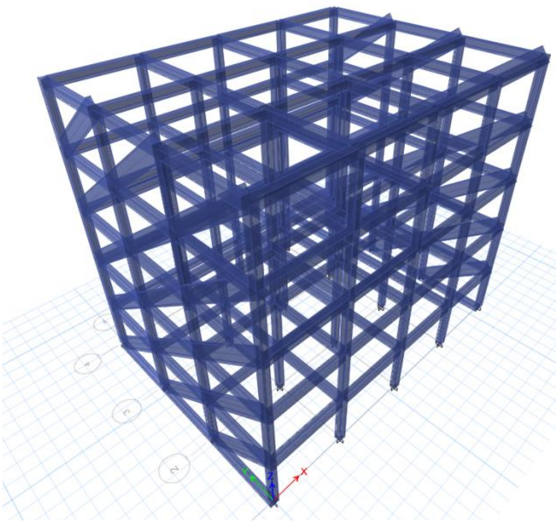

Figure 4: Three-dimensional model of the structure with infill walls

(Modèle tridimensionnel de la structure avec murs de remplissage)

4.1- Linear and Nonlinear Analysis

4.1.1- Linear and Nonlinear Analysis

The main intrinsic characteristic of the structure in terms of natural periods and mode shapes are summarized in tables 2 and 3.

Table 2: Natural periods and modal participating mass for the transverse direction YY (Périodes propres et participation de la masse vibrante de chaque mode dans la direction transversale YY)

Case	T_{1yy} (s)	α_{1yy} (%)
Without infill walls	0.949	80.012
With infill walls	0.705	82.890

Table 3: Mode shapes for the transverse direction YY (Déformées modales dans la direction transversale YY)

Story	Without infill walls	With infill walls
5	1.000	1.000
4	0.951	0.892
3	0.792	0.702
2	0.528	0.424
1	0.237	0.165

4.1.2- Non Linear Analysis: Non linear static analysis using pushover

The non linear static analysis using pushover method gave the capacity curves for the two considered cases in the main transversal (YY) direction as shown in Fig. 5.

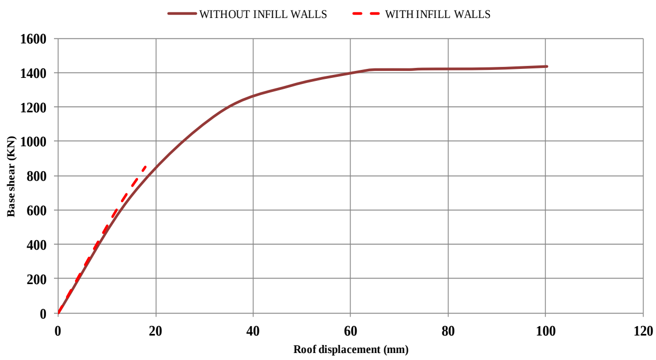

Figure 5: Capacity Curve for the transverse direction YY

(Courbe de capacité dans la direction transversale YY)

The modal participation factors of the first natural modes for the main transverse direction (YY) are shown in Table 4.

Table 4: Modal participation factors
(Facteurs de participation modale)

Direction	Without infill walls	With infill walls
Γ_1	1.253	1.317

The main parameters of the equivalent single degree of freedom for the main transverse direction (YY) are shown in Table 5.

Table 5: Parameters for an equivalent SDOF
(Paramètres équivalents pour un SSDL)

	Long (XX)	Trans (YY)
d_m^*	0.080	0.013
d_y^*	0.033	0.013
T_C	0.500	0.500
T^*	1.172	0.926
$S_e(T^*)$	4.600	5.405
$d_{et}^*(m)$	0.160	0.117
$d_t(m$	0.201	0.154

The lateral displacement profiles of the various models for the two analysis performed in this study obtained from the nonlinear static analysis using pushover method are shown in Figs. 6-9. They resume the main results in terms of absolute and inter story displacements for the transversal direction (YY). The limit states are given according to the recommendations of response limits given in the American Technology Council, ATC 40.

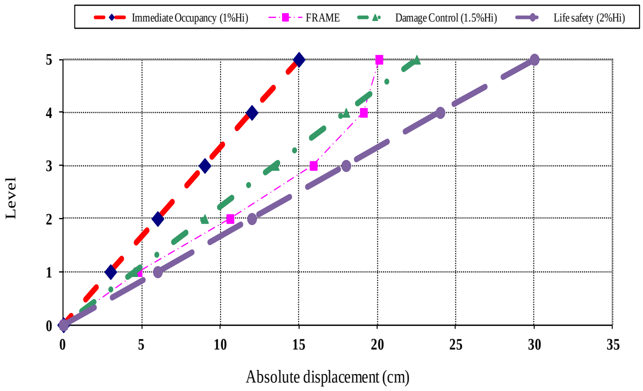

Figure 6: Absolute displacements for the transverse direction YY without infill walls

(Déplacements absolus dans la direction transversale YY sans murs de remplissage)

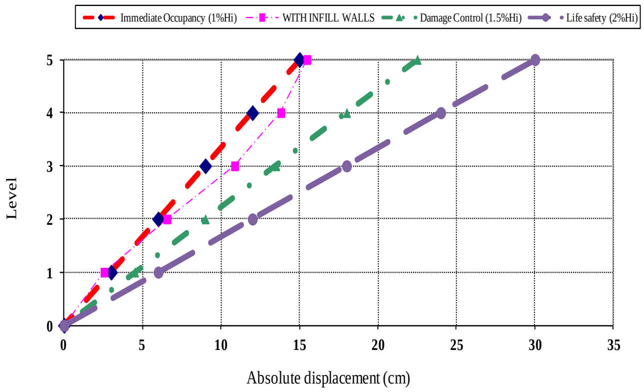

Figure 7: Absolute displacements for the transverse direction YY with infill walls

(Déplacements absolus dans la direction transversale YY avec murs de remplissage)

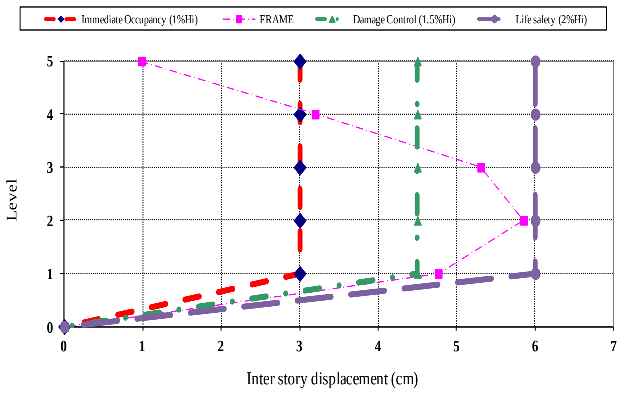

Figure 8: Interstory displacements for the transverse direction YY without infill walls

(Déplacements inter-étages dans la direction transversale YY sans murs de remplissage)

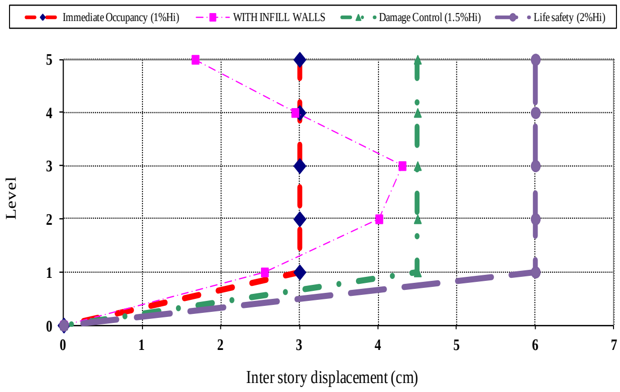

Figure 9: Interstory displacements for the transverse direction YY with infill walls

(Déplacements inter-étages dans la direction transversale YY avec murs de remplissage)

5- Results and discussion

- The analysis of the main results for the transversal (YY) direction showed that the nonlinear static analysis gave the followings:

- The fundamental natural period of the structure with infill walls is lower about 25% than the one of the structure without infill walls.

- The absolute displacements of the structure with infill walls are lower about 23% than the ones of the structure without infill walls.

- In terms of inter story displacements, considering the effect of infill walls in the analysis showed that le performance level is at most in the range of damage control (1.5% Hi), while neglecting the effect of infill walls in the analysis showed that the performance level is in the range of life safety (2% Hi).

- For adjacent buildings, the dimension of the seismic gap is a critical choice. The fact of taking into account the effect of infill walls in the analysis will give a small one. Otherwise, we would get large gap and it is not economic.

6- Conclusions

The analysis of a reinforced concrete frame with and without infill walls was investigated in order to make in evidence the role of masonry infill panels with the surrounding frames and their nonlinear behavior during an earthquake. The nonlinear static analysis according to the Eurocode 8, known as N2 method was been performed. It was observed that the presence of masonry infill walls both strengthens and stiffens the system. The macro-model can reproduce, with a good agreement, the real behavior of these non-structural elements with less computational requirement and time.

The results showed a large difference between the two cases in terms of natural periods, strength and displacements. The question that remains is if we do not take into account the effect of infill walls in the rigidity, we will get a flexible structure, and the seismic forces will be underestimated. If the effect of infill walls is considered in the analysis, the physical behavior of the structure will be more realistic, but any future remove of any infill masonry panel could change totally the global behavior of the structure and provokes heavy damage until collapse in case of major earthquakes.

References

[1] DTR BC 48 (2003) Seismic Code for Building Design and Construction, RPA 99/ Version 2003. National Earthquake Engineering Research Centre (CGS), Technical document, Algiers, Algeria.

[2] Suharwardy MI (1999) Performance Based Design Modeling for Pushover Analysis Use of the Pushover Curve. Computers and Structures, University of Berkley, California, USA.

[3] ATC (2011) Applied Technology Council, Seismic Performance Assessment of Buildings. ATC-58-1, Volume 1, Methodology; California, USA.

[4] Dautaj A, Kabashi N and Sadiku H (2015) Method N2 according to FAJFAR, Proceeding of the Second International Conference on Advances in Civil, Structural and Construction Engineering – CSCE, Rome, Italy.

[5] Menachery D. D and Manjula N. K (2014) Application of Extended N2 Method to Reinforced Concrete Frames with Asymmetric Setbacks. International Journal of Civil Engineering and Technology (IJCIET) 5(12): 143-154.

[6] Eurocode 8 (2004) Design of structures for earthquake resistance-Part 1: General rules, seismic actions and rules for buildings, European Standard.

[7] Polyakov S.V (1963) Masonry in Framed Buildings, Gosudalst-Vennoe Izdatel' stvo Literature Straitd'stvu i Arkitecture, Moskva, 1956, Trans. G.L. Caims, Building Research Station, Watford, Herts.

[8] Stafford-Smith B (1996) Behavior of Square Infilled Frames, Journal of the Structural Division. Proceedings of ASCE; Vol. 91, No. STI: 381-403.

[9] Motovali Emami S.M and Mohammadi M (2016) Influence of vertical load on in-plane behavior of masonry infilled steel frames. Earthquakes and Structures 11(4): 609-627.

[10] Onat O, Lourenco P.B and Kocak A (2016) Nonlinear analysis of RC structure with massive infill wall exposed to shake table. Earthquakes and Structures 10(4): 811-828.

[11] Longo F (2016) Numerical Modeling of Unreinforced Masonry Infill Walls under Seismic Load Considering In-Plane/Out-Of-Plane Interaction, Ph.D. Dissertation, University of Trento, University of Padova, McMaster University.

[12] Adukadukam A and Sengupta A (2013) Equivalent strut method for the Modeling of Masonry Infill Walls in the Nonlinear Static Analysis of Buildings. Journal of the Institutions of Engineers, Series A 94(2): 99-108.

[13] Le Dinh Q.K, Cong Bui T and Van Nguyen Y (2016) A macro-element model for masonry-infilled structural frames. Proceedings of the Institution of Civil Engineers - Structures and Buildings 169(2): 108-120.

[14] Fenerci A, Binici B, Ezzatfar P, Canbay E, Ozcebe G (2016) The effect of infill walls on the seismic behavior of boundary columns in RC frames. Earthquakes and Structures 10 (3): 539-562.

[15] Shekhappa D and Dyavanal S. S (2015) Nonlinear Static Analysis of G+6 RC Building with Infill Walls and User Defined Hinges. International Research Journal of Engineering and Technology (IRJET) 2(9): 2060-2065.

[16] Futado A, Rodrigues H and Arede A (2015) Modeling of Masonry Infill Walls Participation in the Seismic Behavior of RC Buildings using OpenSees. International Journal of Advanced Structural Engineering (IJASE) 7(2): 117-127.

[17] Swarnkari S and Datta D (2015) Analysis of Building with Infill Walls. Journal of Civil Engineering and Environmental Technology (JCEET) 2 (9): 71-76.

Etabs version 15 (2015) CSI Analysis Reference Manual, Computers and Structure, University of Berkeley, California, USA.