INFLUENCE DE LA MÉTHODE DE DIMENSIONNEMENT EN CAPACITÉ PAR RAPPORT À LA MÉTHODE CONVENTIONNELLE SUR LA RÉPONSE SISMIQUE DES VOILES EN BA DANS LES STRUCTURES MIXTES

INFLUENCE OF CAPACITY DESIGN METHOD IN COMPARISON WITH CONVENTIONAL DESIGN METHOD ON THE SEISMIC RESPONSE OF RC WALLS IN DUAL STRUCTURES

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Résumé- Le présent article vise à évaluer l'influence de la procédure de dimensionnement suivie pour le dimensionnement des voiles des bâtiments mixtes en béton armé (BA) sur la performance des voiles, ainsi que celle de la structure dans son ensemble, lorsque celle-ci est soumise à des charges sismiques. À cette fin, des structures mixtes en BA de 4, 8 et 12 étages ont été dimensionnées selon le code parasismique algérien, dans le cas de la méthode de dimensionnement conventionnelle, et selon les dispositions de l'Eurocode 8, dans le cas de la méthode de dimensionnement en capacité. Des analyses statiques non linéaires (Pushover) en utilisant cinq modèles de charge latérale ont été effectuées pour représenter la distribution probable des forces d'inertie imposées aux structures et pour identifier leurs modes de ruine dominants. Des critères de ruine tant au niveau local qu'au niveau global ont été adoptés pour détecter les mécanismes plastiques et les états limites d'effondrement des structures considérées. Les résultats obtenus indiquent que la méthode de dimensionnement en capacité des voiles crée des marges de sécurité adéquates contre la rupture par cisaillement par rapport à la méthode conventionnelle. D'autre part, les avantages de la méthode de dimensionnement en capacité sont clairement évidents. En tenant compte des dispositions de l'EC8, il est possible d'assurer une résistance et une ductilité adéquates. Ceci suggère des améliorations dans les dispositions de dimensionnement du code sismique algérien.

Mots - clés : Méthode conventionnelle, Méthode en capacité, Voiles en béton armé, Codes de dimensionnement sismique, Analyse statique non linéaire

Abstract-The present paper aims at assessing the influence of the design procedure followed in designing the walls of RC dual frame-wall building, on the performance of the walls, as well as the structure as a whole, when subjected to seismic loading. For this purpose, 4-, 8-, and 12 storey's RC dual structures were designed according to Algerian seismic design code, in case of conventional design method, and to Eurocode 8 provisions, in case of capacity design method. Nonlinear static pushover analyses using five different invariant lateral load patterns were carried out to represent the likely distribution of inertia forces imposed on the structures and to identify their dominant failure modes and failure paths. Failure criteria at both member and structural levels have been adopted to detect plastic mechanisms and collapse limit states of structures. The results obtained indicate that capacity design of walls results in adequate safety margins against shear failure in comparison with conventional design of walls. On the other hand, the advantages of capacity design method are clearly apparent. By taking into account the provisions of EC8 it is possible to ensure adequate strength and ductility. This suggests improvements in the design provisions of the Algerian seismic code.

Keywords: Conventional design method, Capacity design method, Reinforced concrete walls, Seismic design codes, Nonlinear static pushover analysis

1-Introduction

Reinforced concrete (RC) structural walls are effective for resisting lateral loads imposed by wind or earthquakes. They provide substantial strength and stiffness as well as the deformation capacity needed to meet the demands of strong earthquake ground motions [1]. Their importance has long been recognized and a higher degree of protection is sought in the design of these critical structural elements [2]. Thus, the design procedure which should be chosen in the structural *design* process is of great importance. Generally, two possibilities are offered for designers: conventional design method and capacity design method. The concepts and the application of the capacity design philosophy, relevant to the sismic design of structures, were developped over the past 40 years in New Zeland, mainly by Professor T. Paulay and colleagues and collaborators [3,4], where, after incorporation into relevant building codes, it has been widely accepted and used for many years. Gradually appreciation of this approach has spread and it was incorporated in the seismic provisions of other codes, for example in Canada, Japan and Eurocode 8 [5]. Instead of this, in Algerian seismic design code [6], the design approach is still based on conventional design method, especially in case of reinforced concrete (RC) structural wall element.

The main aim of this paper is to study the effect of capacity design method followed in designing the walls of RC dual frame-wall building according to EC8 on the seismic performance of the walls, as well as the structure as a whole, in comparison with the conventional design approach. EC8 code has been chosen as the code of reference, in this study, because among all current seismic design codes, it is the code that makes the most systematic and extensive use of capacity design to control the inelastic response mechanism. For this purpose, 4-storey, 8-storey and 12 storey RC dual frame-wall structures were designed according to RPA 99/Version 2003, in case of conventional design method, and to EC8 provisions, related to the capacity design of wall in flexure and shear, in case of capacity design method. Nonlinear static pushover analyses using five different invariant lateral load patterns (*uniform lateral load pattern, elastic first mode load pattern, code lateral load pattern, FEMA-356 lateral load pattern and Multi-modal lateral load pattern*) were carried out to represent the likely distribution of inertia forces imposed on the structures during an earthquake and to identify their dominant failure modes and failure paths. Failure criteria at both member and structural levels have been adopted to detect plastic mechanisms and collapse limit states of structures.

2- Conventional design and capacity design

The concepts and the methodology of working of these two methods, as described in Hugo Bachman et al. [4], are as follows:

- Conventional design method:
  Design and detail the structure for sectional forces derived by analyses for the appropriate combination of gravity loads and earthquake induced forces.
- Capacity design method:
  Design and detail the structure following a strategy that addresses the special nature of inelastic structural response to seismic excitations.

Procedures for the two design methods are summarized in table 1. The first two steps in this table are the same in both approaches. A preliminary design needs to be made. Subsequently the sectional forces for the chosen structural model resulting from gravity loads and earthquake forces need to be determined. The effects of earthquake can be derived using equivalent lateral static forces or a multi-modal response spectrum analyses. These techniques generally imply elastic structural response. The difference between a conventional and a capacity design technique appears only in the third step, i.e. the design of structural components. In conventional design dimensions of components are definitely chosen and the verification and detailing is carried out to meet the requirements of the design forces derived by

Tableau 1 : Procédures de dimensionnement dans les méthodes conventionnelles et en capacité.

Table 1: Procedures in conventional and capacity design methods

Procedures	
Conventional design method	Capacity design method
• preliminary design of the structure • derivation of the sectional forces using a structural model and appropriate - gravity loads - earthquake forces • design of structural components - dimensions - verifications - detailing	• preliminary design of the structure • derivation of the sectional forces using a structural model and appropriate - gravity loads - earthquake forces • design of structural components - choose a suitable mechanism - determine critical sections after inelastic redistribution - proportion and detail plastic hinge regions - proportion and detail parts of the structure intended to remain elastic considering the overstrength of plastic hinge regions

the analyses. The approach is the same as that used when designing for the combination of gravity and wind induced actions.

In capacity design a different approach is used in the third step. Firstly, a complete and admissible plastic mechanism must be chosen. Some engineering judgment is required to choose rational, advantageous and practical locations for the plastic hinges. By recognizing predominantly inelastic seismic response and to achieve optimum solutions, an inelastic redistribution of design actions, within certain limits, may be carried out. Subsequently, by considering the critical actions in members selected for the eventual development of plastic hinges, adequate member dimensions are derived and the potential plastic hinge regions are appropriately detailed. Finally other members or regions of members are designed to resist within the elastic domain actions generated at overstrength in adjacent potential plastic hinges.

3-Capacity design of RC walls according to EC8 approach

The design of walls in flexure and shear are according to the capacity design principles and their calculation is explained below according to EC8.

3.1- Capacity design of RC walls in flexure

The design bending moment diagram along the height of slender walls should be given by an envelope of the bending moment diagram from analysis, with a tension drift (Fig. 1). Slender walls are defined as walls having a height to length ratio greater than 2.0. The envelope is assumed to be linear since there are no discontinuities over the height of the building. It takes into account potential development of moments due to higher mode inelastic response after the formation of plastic hinge at the base of the wall, thus the region above this critical height is designed to remain elastic.

The wall critical region height, h_{cr}, is estimated using the following relationship:

$$h_{cr} = \max\left[l_w, \frac{h_w}{6}\right] \leq \begin{cases} 2\,l_w \\ h_s \;\; for: n \leq 6\, storeys \\ 2\,h_s \;\; for: n \geq 7\, storeys \end{cases} \qquad (1)$$

where n is the number of storeys, h_w, is the wall height, h_s, is the clear storey height, and l_w is the length of the cross section of the wall.

3.2- Capacity design shear of RC walls

The design envelope of shear forces – in Fig. 2 – takes into account the uncertainties of higher modes. The flexural capacity at the base of the wall M_{Rd} exceeds the seismic design bending moment derived from the analysis, M_{Sd}. Thus the design shear found from the analysis, V'_{Sd}, is magnified by the magnification factor ε; i.e. the ratio of M_{Rd}/M_{Sd}. The magnification factor depends on the ductility class of the structure. The design base shear is thus computed by:

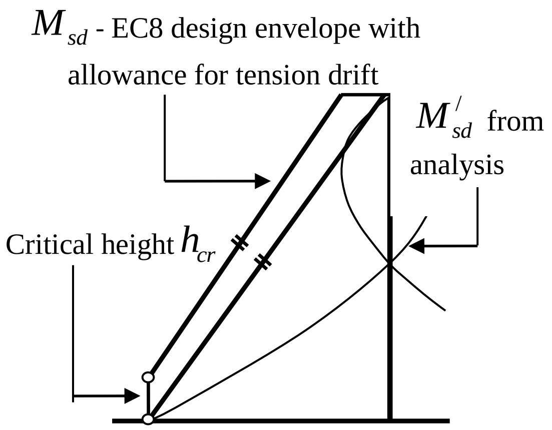


Figure 1 : Diagramme typique des moments dans les voiles en BA des structures mixtes a partir de l'analyse et de l'enveloppe linéaire pour leur dimensionnement selon l EC8

Figure 1 : Typical bending moment diagram in RC walls of dual systems from the analysis and linear envelope for its design according to EC8

$$V_{Sd} = \varepsilon V'_{Sd} \tag{2}$$

where,

- For walls in Ductility Class High buildings the magnification factor, ε, is taken as:

$$\varepsilon = q\sqrt{\left(\frac{\gamma_{Rd}}{q}\frac{M_{Rd}}{M_{Sd}}\right)^2 + 0.1\left(\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \le q \tag{3}$$

- For walls in Ductility Class Medium buildings the magnification factor, ε, is taken as:

$$\varepsilon = q\sqrt{\left(\frac{\gamma_{Rd}}{q}\frac{M_{Rd}}{M_{Sd}}\right)^2 + 0.1\left(\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \le \frac{1+q}{2} \tag{4}$$

where q is the seismic behavior factor, γ_{Rd} is the steel overstrength factor, S_e (T_1) is the value of the elastic spectral acceleration at the period of the fundamental mode, and S_e (T_C) is the spectral acceleration at the corner period, T_C, of the elastic spectrum.

4-Description of structures

4.1- Geometry and structural configuration

In this study 4-storey, 8-storey and 12-storey RC frame-wall dual building structures are considered. These are typical number of storeys used by some other investigators to cover low- to medium-rise framed dual buildings. The buildings are regular in plan and in elevation having storey height of H_{st}=3.0m, where all storeys are of the same height. The buildings consist of five bays along the two horizontal directions with the central bays braced by R/C walls over the whole building height as shown in Fig. 3.

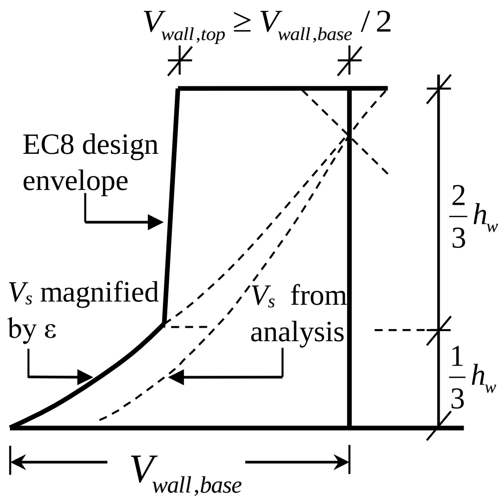


Figure 2 : Enveloppe de dimensionnement en cisaillement des voiles en BA des structures mixtes selon l EC8

Figure 2: Design envelope of shear in RC walls of dual systems according to Eurocode 8

4.2- Data assumed for the studied structures

The total dead and live loads on the floor slabs are assumed to be 5.1 and 2.5 kN/m^2, respectively and for roof slab, they are assumed to be 5.8 and 1.0 kN/m^2. The beam, column and wall elements of structure were designed according to reinforced concrete code BAEL 91 [7] and seismic code RPA 99/version2003 with the following parameters: zone of high seismicity, *zone III*, importance class *groupe 2*, soil type S_3 (soft soil), quality factor $Q=1$ (value which denotes that all the criteria related to the table 4.4 of the code are observed) and viscous damping ration ξ = *10%***.** The analysis will be performed for the zone acceleration factor *A*= *0.25*. A seismic behavior factor of *R* = 5 was taken into account for *dual system composed by walls and frames*. At this purpose, in the design process of studied structures, an attempt was made for moment members to tolerate 25% of earthquake forces in addition to bearing gravity load in order to fulfill the requirement of RPA 99/version2003 which stipulates «*the walls carry less than 20% of vertical loads. The horizontal loads are jointly carried by the walls and the frames in accordance to their relative rigidities. The frames shall have the capacity to resist no less than 25% of the storey shear force in addition to the forces due to the vertical loads*».

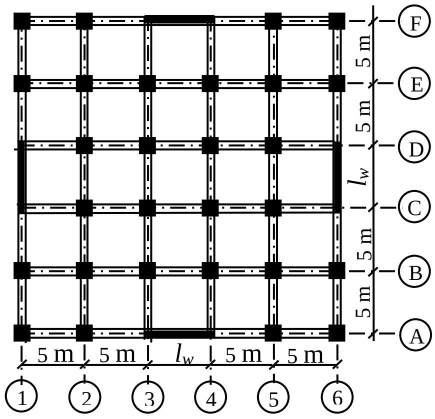
As the main purpose of the study was to evaluate the effect of wall design procedure, the flexural and shear design of walls was also carried out using capacity design procedure according to EC8 approach, as described in paragraph 3.Concrete characteristics cubic strength equal to 25 N/mm^2 and steel characteristics yielding strength equal to 500 N/mm^2 are adopted.

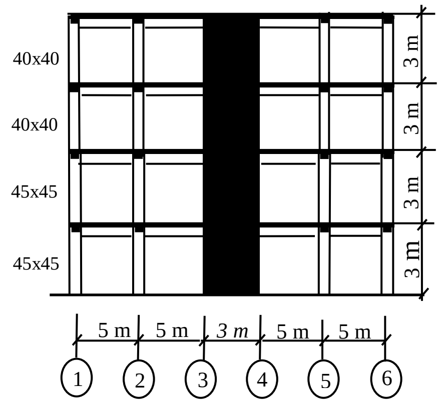
5-Modeling approach for inelastic analyses

Analyses have been performed using SAP2000 [8], which is a general-purpose structural analysis program for static and dynamic analyses of structures. In this study, SAP2000 Nonlinear Version 14 has been used. A two-dimensional representation is selected in the light of the symmetry of buildings and the limited significance of torsional effects. Thus, the model of each structure is created in SAP2000 to carry out nonlinear static pushover analysis. A description of structural members modeling is provided in the following.

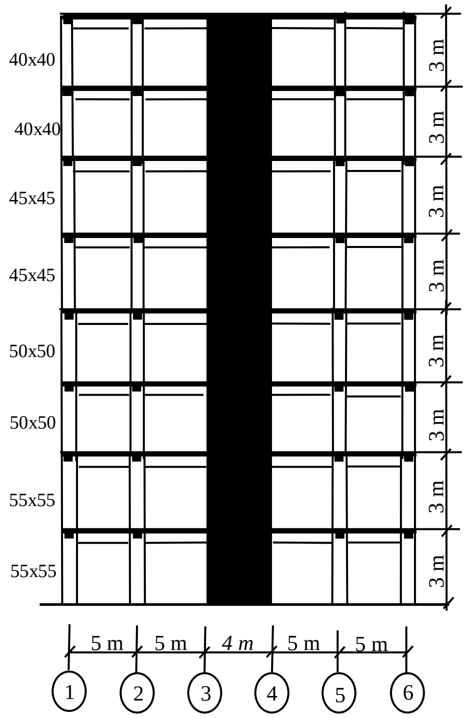
5.1-Modeling of frames

Beam and column flexural behavior was modeled by one-component lumped plasticity elements, composed of an elastic beam and two plastic hinges (defined by a moment-rotation relationship) located at both ends of the beams and columns. The element formulation was based on the assumption of an inflexion point at the midpoint of the element [9]. For beams, the plastic hinge was used for major axis bending.

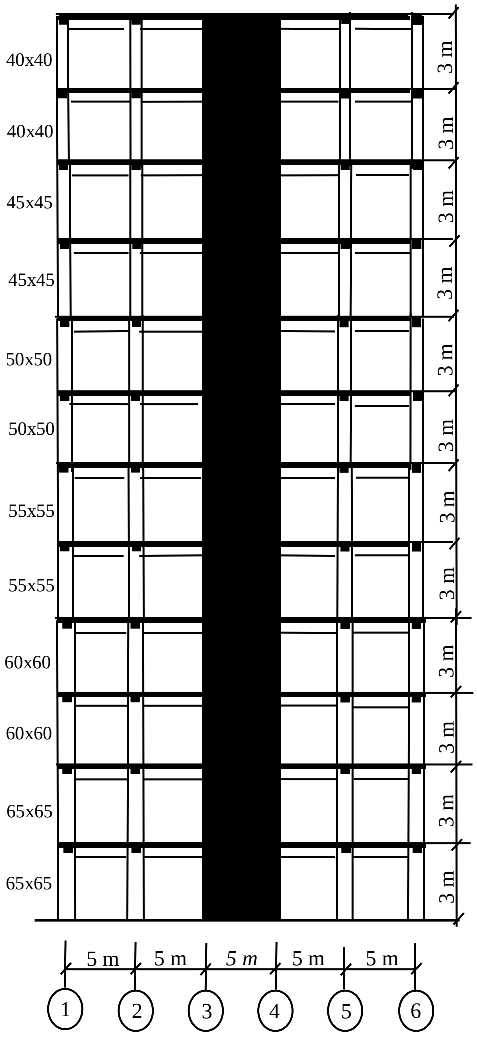




4 storey RC dual building



8 storey RC dual building



12 storey RC dual building

Figure 3 : Vues en plan et en élévation des bâtiments étudiés

Figure 3 : Plan and elevation views of studied buildings

For columns, the plastic hinge for bending about the principal axis perpendicular to the direction of the seismic loading was used. The interaction between axial force and bending moment was not considered, as in [10].

For plastic hinges, the moment-rotation relationship shown in Fig. 4 was used. It was assumed an elastic-perfectly-plastic nonlinear flexural response, where θ_y and θ_u are respectively the yield and ultimate rotations, θ_p is the plastic rotation capacity and M_p is the plastic moment capacity of concrete members. The calculation of these parameters requires moment-curvature characteristics of the plastic hinge section and the length of the plastic hinge. The moment-curvature $(M-\phi)$ characteristics of various RC sections are developed using Mander model for unconfined and confined concrete [11] and Park model for steel [12], which are implemented in moment-curvature analysis. For this study, the moment-curvature analysis is obtained from SAP2000 (SD-Section). The $M-\phi$ curve is converted into equivalent bilinear elastic-perfectly-plastic curve using Caltrans Idealized Model [13], as shown in Fig. 5. The plastic portion of the idealized curve should pass through the point marking the first reinforcing bar yield. The idealized plastic moment capacity, M_p, is obtained by balancing the areas between the actual and the idealized $M-\phi$ curves beyond the first reinforcing bar yield point. The yield rotation θ_y is determined using the following expression:

$$\theta_y = \phi_Y \frac{L}{6} \qquad (5)$$

where ϕ_Y is the yield curvature and L is the element length.

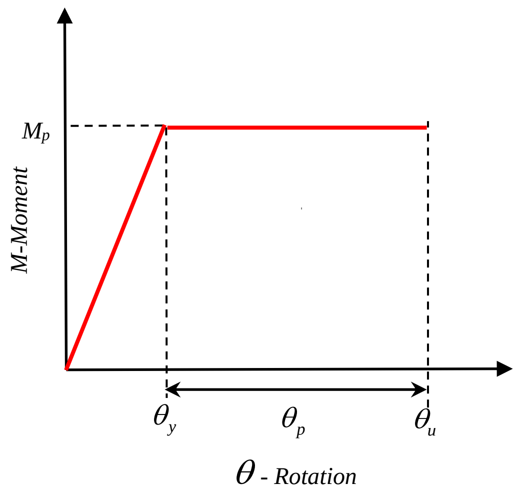


Figure 4: Relation Moment – Rotation

Figure 4: Moment – Rotation relationship

The plastic rotation capacity (θ_p) in reinforced concrete member depends on the ultimate curvature (ϕ_u) and the yield curvature (ϕ_Y) of the section and the length of the plastic hinge region (L_p):

$$\theta_p = (\phi_u - \phi_Y) L_p \qquad (6)$$

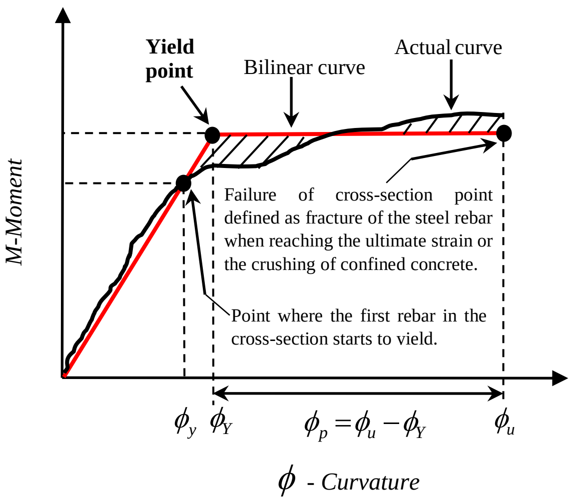


Figure 5: Relation Moment – Courbure

Figure 5: Moment –Curvature relationship

ATC-40 [14] suggests that the length of the plastic hinge equals to half of the section depth in the direction of loading is an acceptable value which generally gives conservative results. This suggestion was adapted to calculate plastic hinge length in this study. Thus,

$$L_p = 0.5h \qquad (7)$$

Where h is the section depth of the member.

Shear hinges are introduced for beams and columns. Because of the brittle failure of concrete in shear, no ductility is considered for this type of hinge. Shear hinge properties are defined such that, when the shear force in the member reaches its strength, the member fails immediately. The shear strength of each member V_r is calculated according to UBC 97 [15], also found in [16], as follows:

$$V_r = V_c + V_s \qquad (8)$$

where V_c and V_s are shear strength provided by concrete and shear reinforcement in accordance with Equations (8) and (9), respectively:

$$V_c = 0.182bd\sqrt{f_c}\left(1 + 0.07\frac{N}{A_c}\right) \qquad (9)$$

$$V_s = \frac{A_{sh}\, f_{yh}\, d}{s} \qquad (10)$$

where b is the section width, d is the effective depth, f_c is the unconfined concrete compressive strength, N is the axial load on the section, A_c is the concrete area, and A_{sh} , f_{yh} , and s are the area, yield strength, and spacing of transverse reinforcement, respectively.

5.2- Modeling of structural walls

In this paper, the numerical modeling of structural walls is carried out with macro element model, consisting of an equivalent beam-column element (lumped plasticity) at the wall centroidal axis with rigid links on beam girders, as seen in Fig. 6 [17]. This model consists of an elastic flexural element with a nonlinear rotational spring at each end to account for the inelastic flexural behavior of critical regions and with a nonlinear horizontal spring at the mid-height of the wall to account for the inelastic shear behavior.

Wall flexural behavior was modeled as per column member modeling described in Paragpraph5.1. However, the yield rotation θ_y is defined using Equation (11), taken from FEMA-356 [18], instead of Equation (5).

$$\theta_y = \left(\frac{M_p}{E_c I}\right) L_p \qquad (11)$$

where M_p is the plastic moment capacity of the RC wall, E_c is the concrete modulus, I is the member moment of inertia, and L_p is the plastic hinge length.

Wall shear behavior was modeled by using a uniaxial shear spring with a prescribed shear force-deformation behavior. For this study, the shear force-deformation relationship provided in FEMA 356 was utilized, as depicted in Fig. 7.

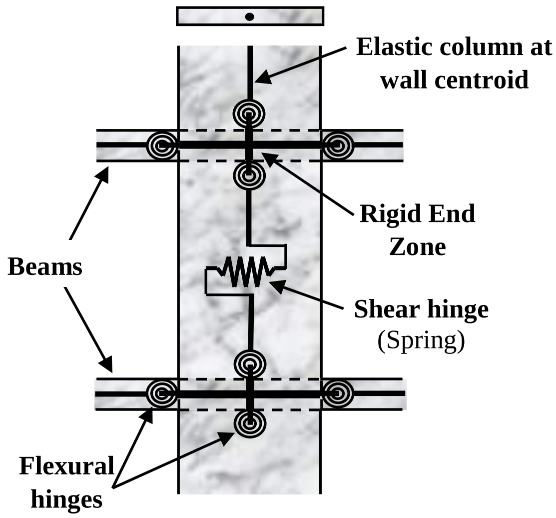


Figure 6: Représentation équivalente de l' élément poutre –poteau du voile en BA

Figure 6: Equivalent beam-column element representation of RC wall

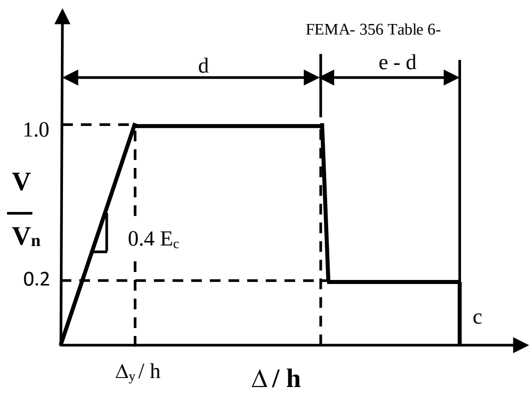


Figure 7: Courbe effort tranchant-déformation selon FEMA-356

Figure 7: Shear force-deformation curve based on FEMA-356

The nominal shear strength V_n of walls is typically defined using Equation (12), taken from ACI 318-08 [19]:

$$V_n = A_{cv}\left(\alpha_c \lambda \sqrt{f_c'} + \rho_t f_y\right) \quad (12)$$

where α_c = 3.0 for a height-to-length ratio, $h_w/l_w \le 1.5$, α_c = 2.0 for $h_w/l_w \ge 2.0$, and varies linearly for $1.5 \le h_w/l_w \le 2.0$. In this equation, λ is 0.75 for lightweight concrete and 1.0 for normal weight concrete, A_{cv} represents the cross-sectional web area of the wall, f_c' is the compressive strength of concrete, ρ_t is transverse reinforcement ratio, and f_y is the yield strength of transverse reinforcement.

The effective shear stiffness is typically taken as:

$$G_c A = \frac{E_c}{2(1+\nu)} A_{cv} = 0.4 E_c A_{cv} \quad (13)$$

where ν is Poisson's ratio, and A_{cv} is the cross-sectional area of the web. Based on the assumption that Poisson's ratio for uncracked concrete is approximately 0.2, the effective shear stiffness defined in [20] is $G_cA = 0.4\ E_c\ A_w$.

6- Nonlinear static pushover analyses

Nonlinear static pushover analyses of the three studied RC dual structures are performed to identify their dominant failure modes and failure paths. The analysis consist of applying gradually increasing the lateral loads appropriately distributed over the storeys, to obtain the relationship between the base shear and the top storey displacement, which is generally called pushover curve or capacity curve. There can be many alternatives for the distribution pattern of the lateral loads, and it may be expected that different patterns of lateral loads result in pushover curves with different characteristics and different sequence of plastic hinge formation [21]. That is to say, different failure modes will occur in different load patterns [22]. Thus, multiple lateral load patterns should be used to improve the accuracy of identification of failure modes and failure paths. Five lateral load patterns are used in this study and are described as follows [23]:

- **'Uniform' lateral load pattern**

The lateral force at any storey is proportional to the mass at that storey, i.e.,

$$F_i = m_i / \sum m_i \quad (14)$$

where F_i is the lateral force at i-th storey, and m_i is the mass of i-th storey.

- **'Elastic First Mode' lateral load pattern**

The lateral force at any storey is proportional to the product of the amplitude of the elastic first mode and the mass at that storey, i.e.,

$$F_i = m_i Ø_i / \sum m_i Ø_i \quad (15)$$

where $Ø_i$ is the amplitude of the elastic first mode at i-th storey.

- **'Code' lateral load pattern**

The lateral load pattern is defined in Algerian seismic design code (RPA 99/Version 2003) and the lateral force at any storey is calculated from the following formula:

$$F_i = (V_b - F_t)\frac{m_i h_i}{\sum_{j=1}^{N}(m_j h_j)} \quad (16)$$

where V_b is the base shear, h is the height of i-th storey above the base, N is the total number of storeys, and F_t is the additional earthquake load added at the N-th storey when $T > 0.7$sec (For T ≤ 0.7s, $F_t = 0$ otherwise; $F_t = 0.07TV_b \leq 0.25V_b$ where T is the fundamental period of the structure).

- **'FEMA-356' lateral load pattern**

The lateral load pattern defined in FEMA-356 is given by the following formula that is used to calculate the internal force at any storey:

$$F_i = m_i h^k_i / \sum m_i h^k_i \quad (17)$$

where h is height of i-th storey above the base, and k is the factor to account for the higher mode effects (k = 1 for $T \leq 0.5$ sec and k = 2 for $T \geq 2.5$ sec and varies linearly in between).

- **'Multi-Modal (or SRSS)' lateral load pattern**

The lateral load pattern considers the effects of elastic higher modes of vibration for long period and irregular structures and the lateral force at any storey is calculated as Square Root of Sum of Squares (SRSS) combinations of the load distributions obtained from the modal analyses of the structures as follows:

1. Calculate the lateral force at i-th storey for n-th mode from Equation (18).

$$F_{in} = \Gamma_n m_i Ø_{in} A_n \quad (18)$$

where Γ_n is the modal participation factor for the n-th mode, $Ø_{in}$ is the amplitude of n-th mode at i-th storey, and A_n is the pseudo-acceleration of the n-th mode of the SDOF elastic system.

2. Calculate the storey shears, $V_{in} = \sum_{j\geq i}^{N} F_{jn}$

where N is the total number of storeys.

3. Combine the modal storey shears using SRSS rule, $V_i = \sqrt{\sum_n (V_{in})^2}$.

4. Back calculate the lateral storey forces, F_i, at storey levels from the combined storey shears, V_i starting from the top storey.

5. Normalize the lateral storey forces by base shear for convenience such that $F_i' = F_i / \sum F_i$.

The contribution of first three elastic modes of vibration was considered to calculate the 'Multi-Modal (or SRSS)' lateral load pattern in this study, as in [23].

7- Performance criteria

Performance criteria must be defined for structures or structural member elements to monitor response during analysis. These criteria also help to detect plastic mechanisms and collapse limit states of structures. In this research, the following performance levels were used to identify the limiting conditions.

a) The inter-story drift ratio is limited to 3% in nonlinear static pushover analysis. This is consistent with the limit specified in [24,25] and close to the limits adopted by seismic design codes EC 8 and UBC 97 which vary between 2% and 3%.

b) Structural instability is based either on plastic hinge formation or conversion of

c) Structure to a mechanism (i.e. storey mechanisms).
d) The stability index is limited to 0.2, as per the RPA99/version2003 seismic design code.
e) Rotation is limited to the ultimate rotation of structural member elements.
f) Shear is limited to the shear strength of structural member elements.

The structure is assumed to have failed when the structure meets one or more of the above criteria. Table 2 provides a summary of the limiting performance criteria outlined above.

Tableau 2 : Critères de réponse des structures.

Table 2 : Response criteria for structures.

Parameter	Description	Limitation
Δ	Inter-story drift ratio	= 3%
-	Stability	Mechanism
$\theta_{p\text{-}\Delta}$	Stability Index	= 0.2
θ/θ_u	Rotation control	= 1
V/V_n	Shear control	= 1

8- Pushover analysis results

Inelastic static pushover analyses up to collapse are carried out on the three RC dual frame-wall structures investigated here. The performances of the conventional and capacity designed structures, in other words the causes of structural failure, are examined in the light of collapse parameters explained earlier.

The main results obtained from nonlinear static analyses were: (a) capacity curves (base shear force *vs* top storey displacement), (b) plastic hinge mechanisms corresponding to the collapse limit state, (c) maximum inter-storey drift (Δ_{max}), and the corresponding maximum stability index ($\theta_{p\text{-}\Delta,\ max}$) in relation to the collapse limit states observed in each structure, (d) maximum storey shear (V_{max}) of structural wall elements, (e) maximum storey rotation (θ_{max}) of structural wall elements.

8.1- Capacity curves

The capacity curves of conventional and capacity designed structures in the five lateral load patterns are shown in Figs. 8, 9 and 10, respectively for 4-, 8-, and 12- storey RC dual frame-wall buildings. These figures show also the mean ultimate base shear, $V_{u,\ mean}$, and the mean top storey displacement, $d_{u,\ mean}$, in relation to the collapse limit states observed in each building.

It can be seen that the capacity designed structures can sustain greater lateral load and displacement. Strength (plastic reserve forces) and ductility (plastic reserve displacements) are obviously improved through capacity design method, particularly for 4-storey building. Thus, mean ultimate base shear, $V_{u,\ mean}$, of the capacity designed 4-storey structure equals 6803.74 kN, increasing by 32.97% compared with that of conventional designed 4-storey structure that is 5116.48 kN, and the mean ultimate top storey displacement, $d_{u,\ mean}$, of the capacity designed 4-storey structure equals 32.91 cm, which increases by 208.78% compared with that of conventional designed 4-storey structure. This demonstrates the excellent behavior of the capacity designed structures. As mentioned in James Fox et al. [26], the capacity design method aims to ensure controlled ductile response of structures when subjected to earthquake.

8.2- Collapse mechanisms and distribution of plastic hinges at collapse limit state

The collapse mechanisms and the distribution of plastic hinges at collapse limit states of conventional and capacity designed structures in the five lateral load patterns are shown in Fig. 11. It should be mentioned that for the sake of brevity only collapse mechanisms of frames containing wall elements of the 4-storey RC dual frame-wall building are reported in this figure. Plastic hinge patterns permit to provide information about local and global failure mechanisms in the structure (i.e. rotation and shear of plastic hinges, and storey mechanisms).

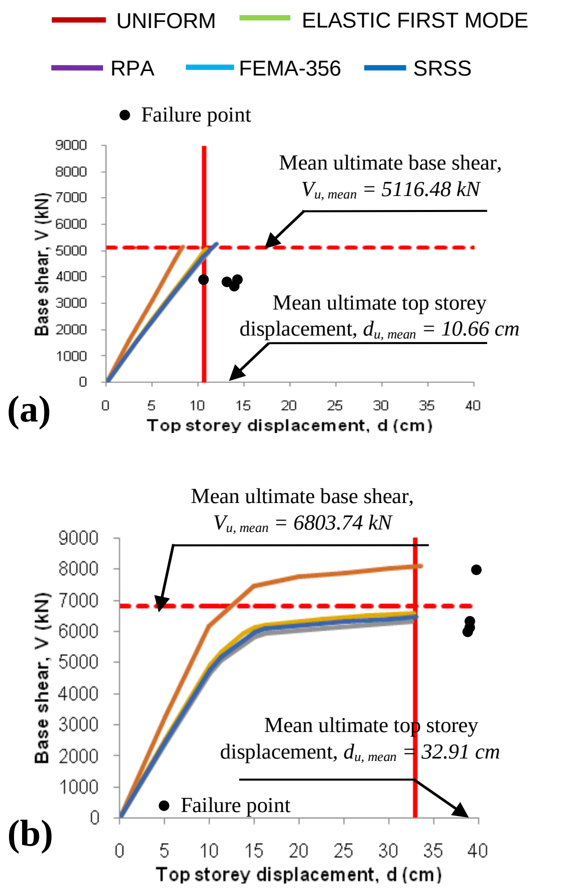
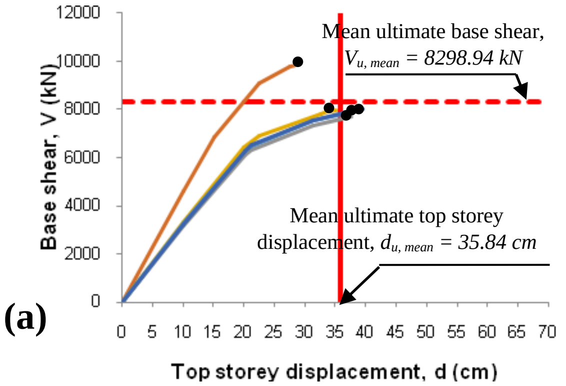


Figure 8 : Courbes de capacité du bâtiment mixte en BA de 4 étages dans les cinq modèles de charge pour: **(a)** méthode conventionnelle, **(b)** méthode en capacité

Figure 8 : Capacity curves of the 4-storey RC dual building in the five lateral load patterns for: **(a)** conventional design method, **(b)** capacity design method



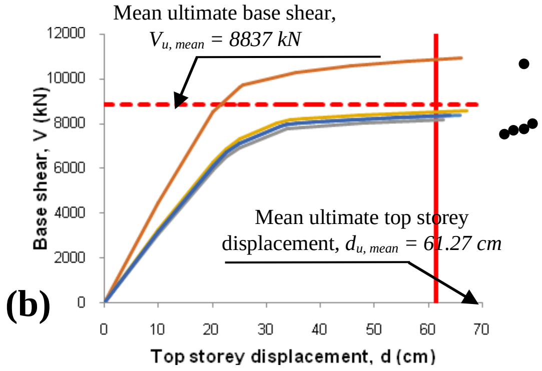


Figure 9 : Courbes de capacité du bâtiment mixte en BA de 8 étages dans les cinq modèles de charge pour: **(a)** méthode conventionnelle, **(b)** méthode en capacité

Figure 9: Capacity curves of the 8-storey RC dual building in the five lateral load patterns for: **(a)** conventional design method, **(b)** capacity design method

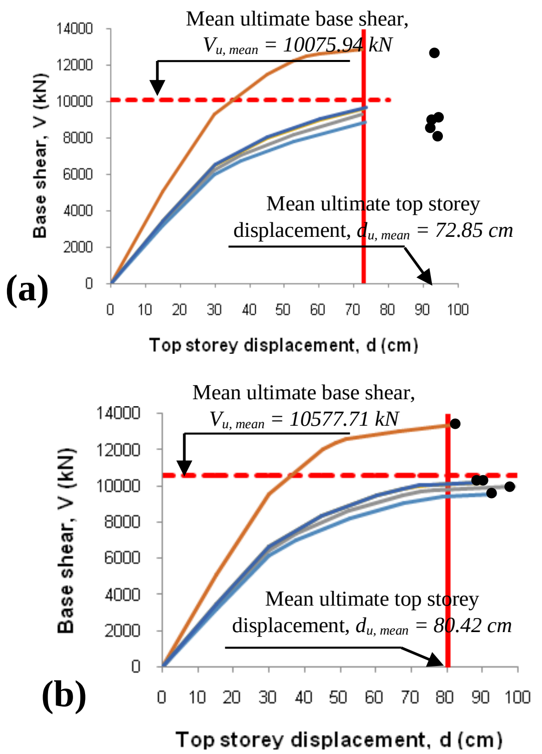


Figure 10 : Courbes de capacité du bâtiment mixte en BA de 12 étages dans les cinq modèles de charge pour: **(a)** méthode conventionnelle, **(b)** méthode en capacité

Figure 10 : Courbes de capacité du bâtiment mixte en BA de 12 étages dans les cinq modèles de charge pour: **(a)** méthode conventionnelle, **(b)** méthode en capacité

No shear failures of RC beam and column members were observed in any case of pushover analyses. Even in the case of conventional designed structures, the shear strength of members was sufficient to carry the shear forces that developed. Thus, the behavior of these members is dominated by flexure. Note that the shear strength of beam and column members is identical in the two methods of dimensioning since the capacity design method affects only the wall members. Also, the local criterion which consists on the limitation of plastic hinge rotation of different member elements, beams and columns, to the ultimate rotation, θ_u, has not been observed both in the conventional and capacity design methods. Moreover, as shown in the hinging pattern of conventional and capacity designed structures in the five lateral load patterns, no storey mechanism is detected. This is mainly due to the presence of wall members which prevent the formation of a column sidesway mechanism that gives rise to storey mechanism.

Comparison between the two methods of dimensioning of wall members in seismic failure modes of walls and the level of plastic rotations in beam and column elements shows clearly the deficiency of conventional design method, particularly in the case of 4-storey and 8-storey structures; and the former, i.e. 4-storey structure, appears to be the one which is strongly affected by design procedure followed in designing the walls. In fact, in the Fig. 11, shear failure of RC structural walls, detected using conventional design method, are observed at plastic rotation levels in beam and column elements notably smaller than what is obtained using the capacity design method, the maximum values not exceeding 14 mrad. It can be seen in several cases of lateral load distribution that the flexural hinges occur only in few second and third floor beams. No flexural hinges are observed in columns even at their fixed base. Also, the Fig. 11 reveals that the shear failure inhibits the development of the plastic rotation at the wall base. Whereas flexural hinges, utilizing the capacity design method, are observed in all floor beams at high rotation level and at the fixed base of columns, the maximum values exceeding 40 mrad. Moreover, since the undesirable shear failure modes are prevented by the application of the capacity design principles, full flexural plastic rotation of the wall is developed at its fixed base. The behavior of 8-storey structure is quite similar to what it was observed in the case of 4-storey structure, since the shear failure of RC structural walls is also observed when utilizing the conventional design method. Nevertheless, flexural hinges are developed at almost floor beams with rotation level higher than the 4-storey structure, the maximum values exceeding 25 mrad. Furthermore, flexural hinges in RC walls are depicted in the first and second storey even if the level of rotation is only at the beginning of yielding in comparison with what is observed in the case of capacity design method.

The above mentioned observations, especially those related to the RC walls, are not pointed out in the case of the 12- storey structure. In fact, no shear failure is observed in walls up to collapse limit state even using the conventional design method. However, it is found that the maximum shear forces (shear demand) appear to be closer to the shear strength (shear capacity) of walls in the second to the fifth storey, particularly in the case of the uniform lateral load pattern, as it will be seen in the paragraph 8.4 of this present paper. This denotes that even in this case, the conventional design method do not ensure a high safety margin against shear failure of walls.

The results indicate that shear failure is the controlling member failure criterion for the 4-, and 8-storey structures designed according to the conventional method, contrary to the 12-storey structure that presents nevertheless near shear failure of walls, as mentioned above.

The results clearly show that the capacity design provisions of Eurocode 8 have succeeded in protecting the walls from the undesirable shear failure mode and ensure in the same time a favourable global plastic mechanism, where most of the beams yield, as well as the columns and walls at their fixed base. In the light of the observations mentioned above, it is clear that in the case of the conventional design method the shear failure criterion is practically the controlling local collapse parameter. Whereas in the case of the capacity design method the shear failure criterion will not be a controlling parameter. Thus, hereinafter, other failure criteria in combination with the one observed in the former case will be detected, if they exist; and failure criteria governing the collapse state in the later case will be found.

8.3- Inter-storey drifts

The distribution of the maximum observed inter-storey drift (Δ_{max}) at the collapse limit state for the three studied structures from each method of design when subjected to five lateral load patterns are presented in Figs. 12, 13 and 14. The recorded maximum stability indexes ($\theta_{p\text{-}\Delta,\ max}$) at the same limit state are also shown.

It can be seen that in the case of the conventional design method, the average values of the maximum inter-storey drift, Δ_{max}, of the 4- and 8-storey structures are well below the collapse inter-storey drift (Δ = 3%), especially in the 4-storey structure, which value is only equal to 1.12% (Figure 12.a). For the 12-storey structure, its value is equal to 3% (Figure 14.a), which is the upper limit considered in this study (Δ = 3%).

In the case of the capacity design method, it can be observed that the average values of the maximum inter-storey drift, Δ_{max}, of the 4- and 12-storey structure are equal to 3% (Figures 12.b and 14.b). For the 8-storey structure, the evaluated value is below the limiting value adopted here ($\Delta_{max,\ average}$ = 2.32%, Fig. 13.b).

The observed values of the stability index ($\theta_{p\text{-}\Delta,\ max}$), which place a further limitation on *P*-Δ effects, up to collapse limit state are bellow the limiting value adopted here (0.2). This forgoing observation is noted in all structures designed with the two methods of dimensioning and under all lateral load patterns employed (Figs. 12, 13 and 14). This implies that second order effects are not significant. However, the values of the stability index obtained from the capacity design method are higher than those obtained from the conventional design method. Except in the case of the 12-storey structure where the values are near (Fig. 14). Also, it can be found that the stability index increases as the number of storey increases; this highlighted the sensitivity of the high-rise structure to the P-Δ effects. However, the values of the stability index obtained from the capacity design method are higher than those obtained from the conventional design method. Except in the case of the 12-storey structure where the values are near (Fig. 14). Also, it can be found that the stability index increases as the number of storey increases; this highlighted the sensitivity of the high-rise structure to the P-Δ effects.

8.4- Storey shear in RC walls

The distribution of the maximum storey shear (V_{max}) at the collapse limit state in RC wall members for the three studied structures from each method of design when subjected to five lateral load patterns are presented in Figs. 15, 16 and 17. The calculated nominal shear strengths (V_n) of walls are also shown.

As shown in these figures, in the case of the conventional design method, shear failure of RC walls is observed in the 4-, and 8-storey structures. For the 12-storey structure, as mentioned above (Paragraph. 8.2), even if there is no shear failure of RC wall, the maximum storey shear forces (V_{max}) appear to be closer to the storey shear strengths (V_n) of walls in the second to the fifth storey, particularly in the case of the uniform lateral load pattern (Fig. 17.a). This implies that the conventional design method do not ensure a high safety margin against shear failure of walls. In the case of the capacity design method, it can be seen that the storey shear strengths (V_n) of walls are much greater than the observed maximum storey shear forces (V_{max}), which leads to high safety margin against shear failure.

8.5- Flexural rotation in RC walls

The distribution of the maximum flexural rotation (θ_{max}) at the collapse limit state in RC wall members for the three studied structures from each method of design when subjected to

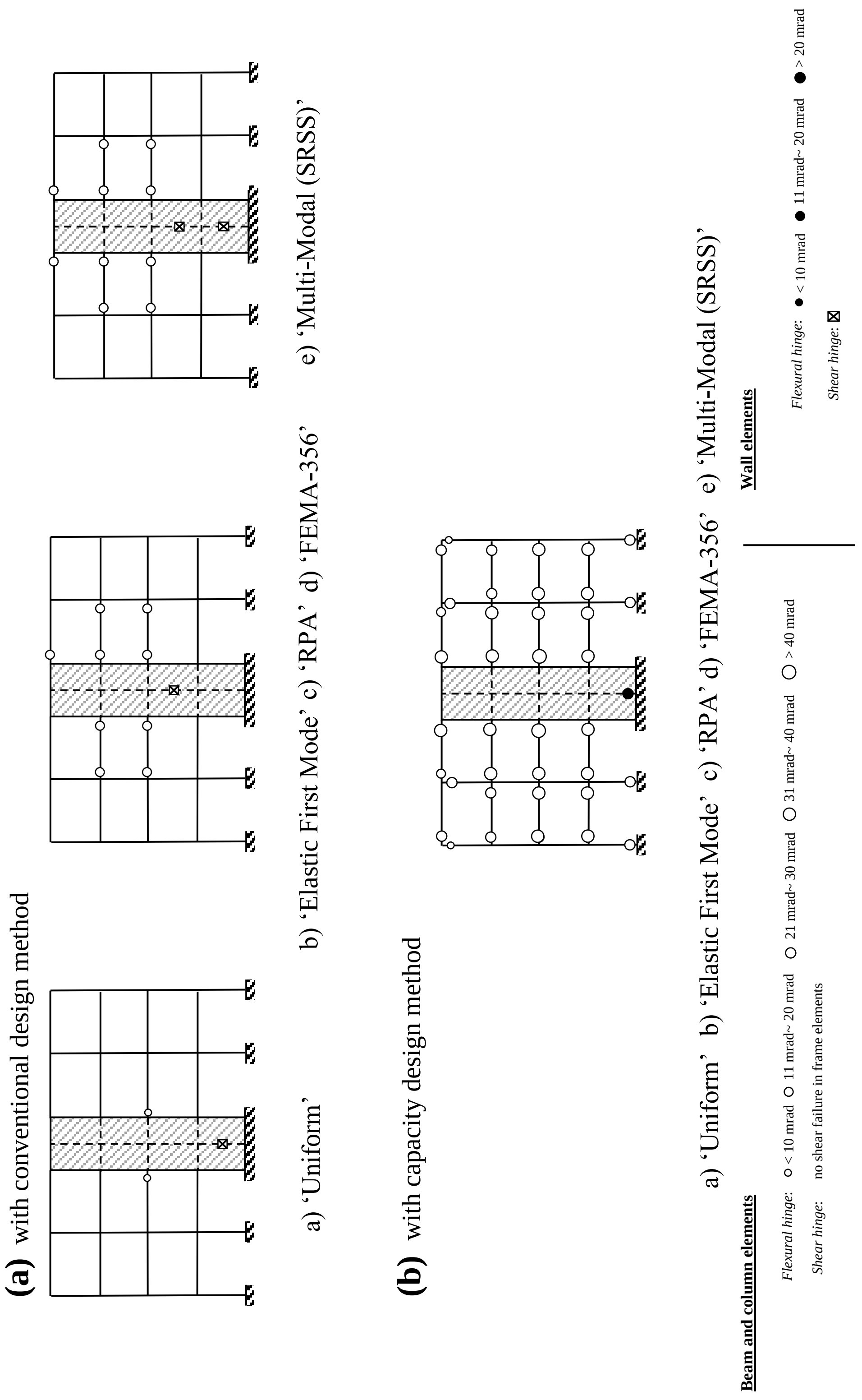


Figure 11 : Mécanismes de ruine du système mixte portique –voile de 4étages pour (a) méthode conventionnelle et (b) méthode en capacité

Figure 11: Collapse mechanisms of the 4-storey dual frame-wall for (a) conventional design method and (b) capacity design method

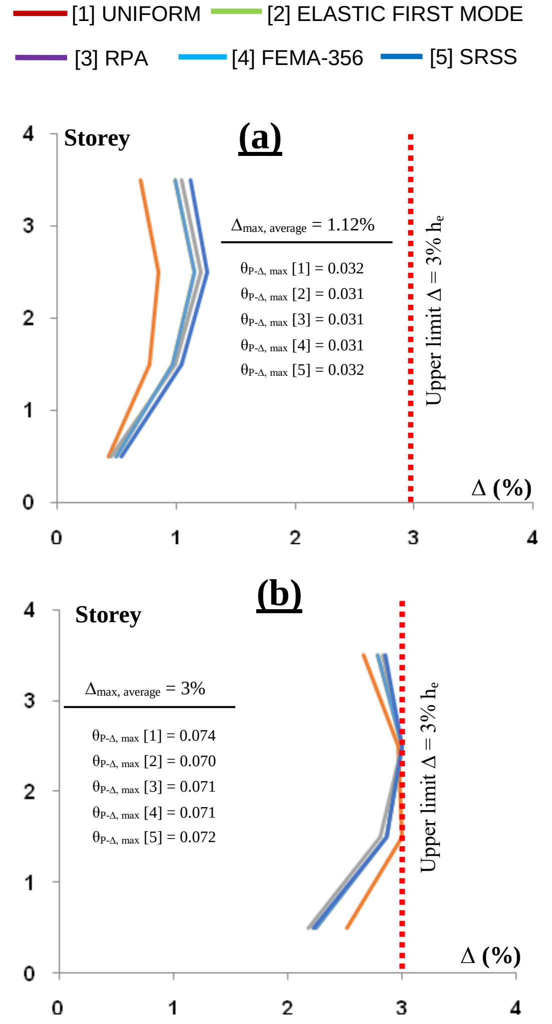


Figure 12: Déplacement inter-étages et coefficient de stabilité du bâtiment mixte de 4 étages pour **(a)** méthode conventionnelle et **(b)** méthode en capacité

Figure 12: Inter-storey drift and stability index for the 4-storey RC dual frame-wall for **(a)** conventional design method and **(b)** capacity design method

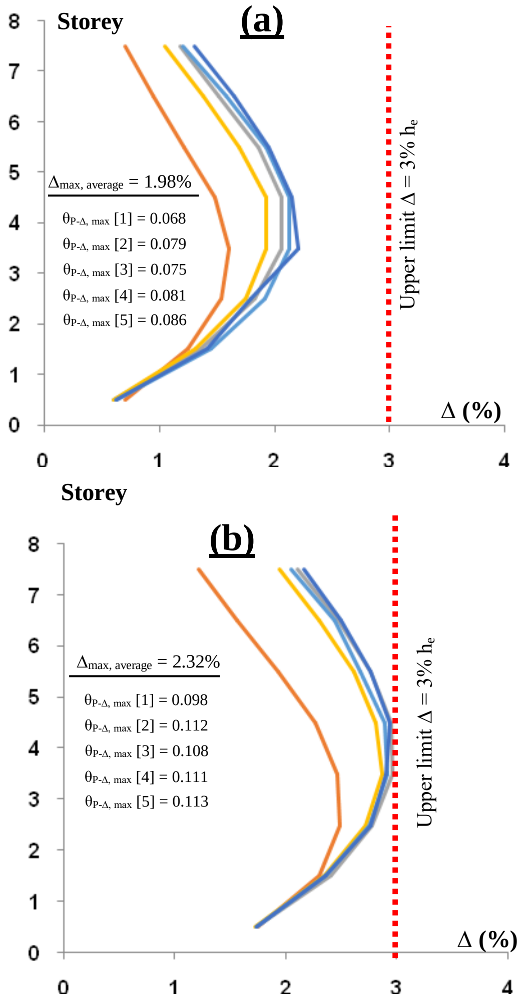


Figure 13: Déplacement inter-étages et coefficient de stabilité du bâtiment mixte de 8 étages pour **(a)** méthode conventionnelle et **(b)** méthode en capacité

Figure 13: Inter-storey drift and stability index for the 8-storey RC dual frame-wall for **(a)** conventional design method and **(b)** capacity design method

five lateral load patterns are presented in Figs. 18, 19 and 20. The calculated of the corresponding ultimate rotation capacities (θ_u) of walls are also shown. It can be observed that in the case of the conventional design method, the flexural rotation at the fixed base of the wall is inhibited by the premature shear failure. This phenomenon is accentuated in the 4-storey structure, as shown in Figure 18.a, where the ultimate rotation capacity (θ_u) is much greater than the maximum demand flexural rotation (θ_{max}). In the case of the capacity design method, it can be noted that the walls can develop a full flexural rotation at their fixed base, since no shear failure is occurred. Also, local flexural failure of wall is detected in the 8-storey structure (Fig. 19.b), and near local flexural failure of wall in uniform load case is found in the 12-storey structure.

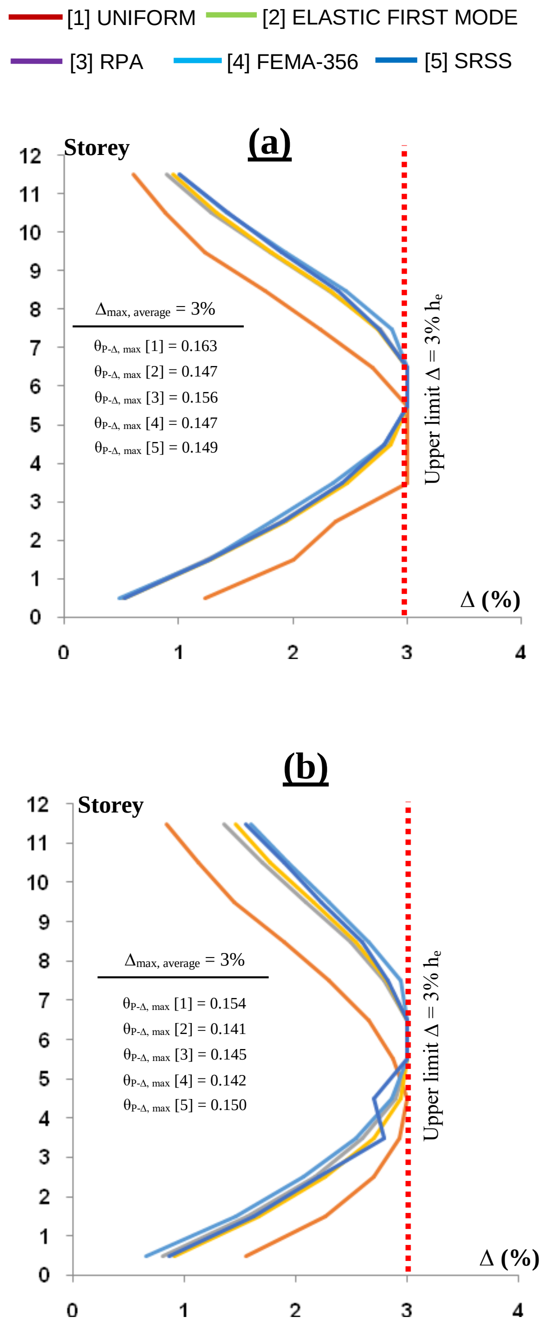


Figure 14: Déplacement inter-étages et coefficient de stabilité du bâtiment mixte de 12 étages pour **(a)** méthode conventionnelle et **(b)** méthode en capacité

Figure 14: Inter-storey drift and stability index for the 12-storey RC dual frame-wall for **(a)** conventional design method and **(b)** capacity design method

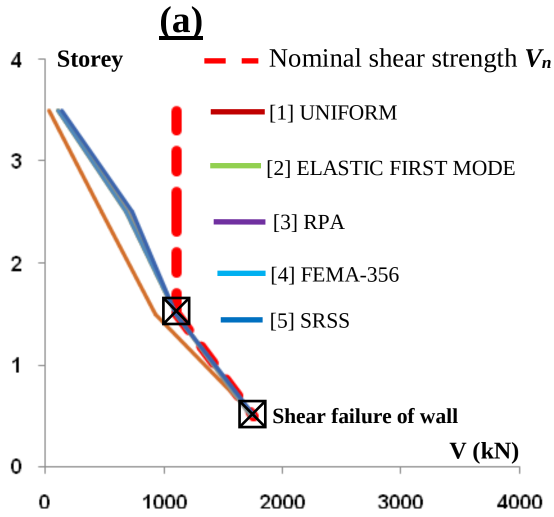


Figure 15 (a) Effort tranchant d'étage (V_{max}) et résistance en cisaillement d'étage (V_n) dans les éléments voiles en BA du bâtiment mixte de 4 étages pour la méthode conventionnelle

Figure 15 (a) Maximum storey shear (V_{max}) and nominal storey shear strength (V_n) in RC wall members for the 4-storey RC dual frame-wall for conventional design method

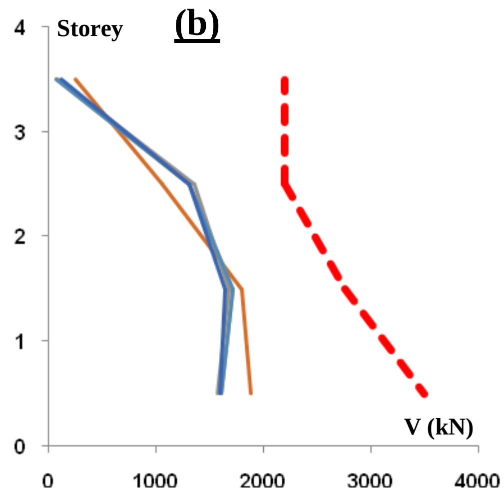


Figure 15 (b) Effort tranchant d'étage (V_{max}) et résistance en cisaillement d'étage (V_n) dans les éléments voiles en BA du bâtiment mixte de 4 étages pour la méthode en capacité

Figure 15 (b) Maximum storey shear (V_{max}) and nominal storey shear strength (V_n) in RC wall members for the 4-storey RC dual frame-wall for capacity design method

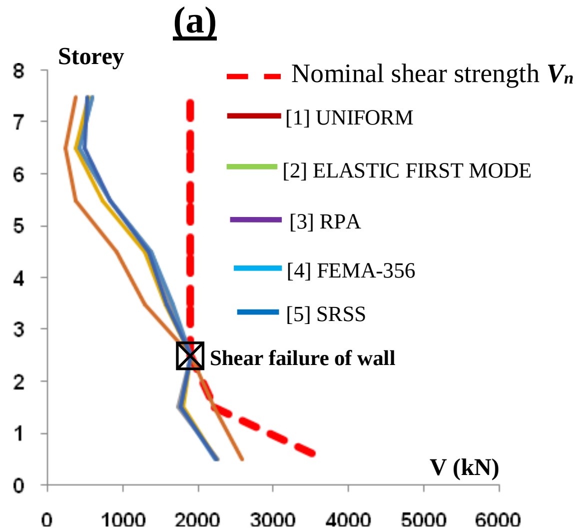


Figure 16 (a) Effort tranchant d'étage (V_{max}) et résistance en cisaillement d'étage (V_n) dans les éléments voiles en BA du bâtiment mixte de 8 étages pour la méthode conventionnelle

Figure 16 (a) Maximum storey shear (V_{max}) and nominal storey shear strength (V_n) in RC wall members for the 8-storey RC dual frame-wall for conventional design method

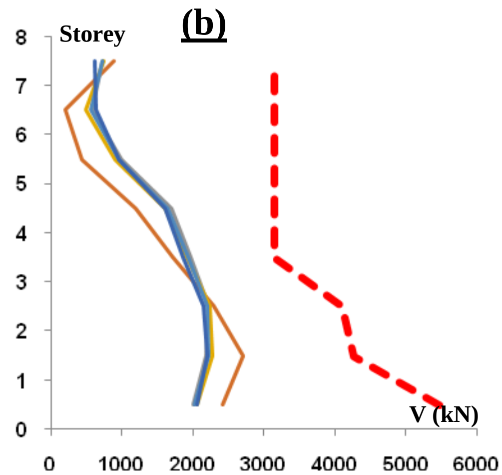


Figure 16 (b) Effort tranchant d'étage (V_{max}) et résistance en cisaillement d'étage (V_n) dans les éléments voiles en BA du bâtiment mixte de 8 étages pour la méthode en capacité

Figure 16 (b) Maximum storey shear (V_{max}) and nominal storey shear strength (V_n) in RC wall members for the 8-storey RC dual frame-wall for capacity design method

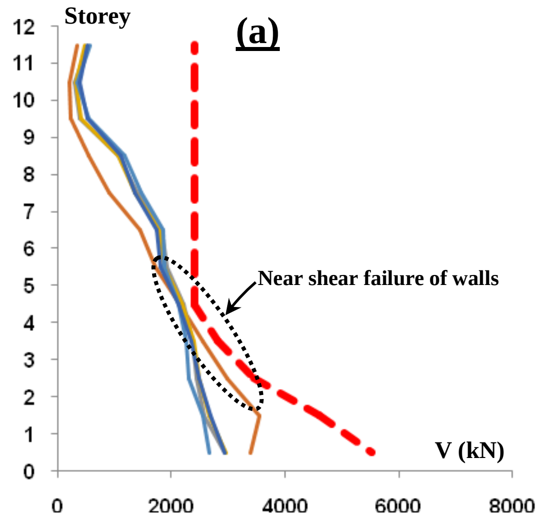


Figure 17 (a) Effort tranchant d'étage (V_{max}) et résistance en cisaillement d'étage (V_n) dans les éléments voiles en BA du bâtiment mixte de 12 étages pour la méthode conventionnelle

Figure 17 (a) Maximum storey shear (V_{max}) and nominal storey shear strength (V_n) in RC wall members for the 12-storey RC dual frame-wall for conventional design method

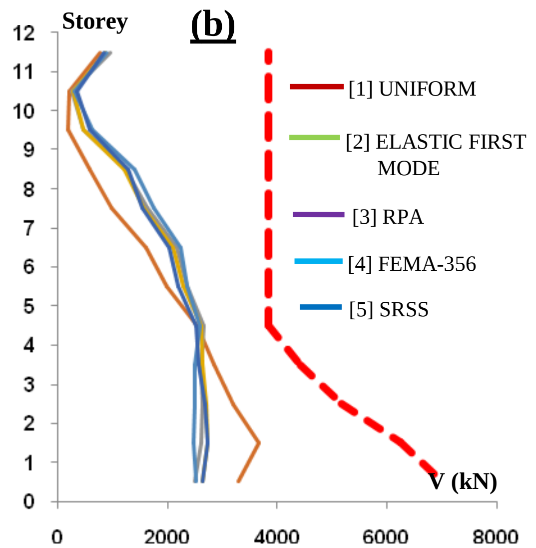


Figure 17 (b) Effort tranchant d'étage (V_{max}) et résistance en cisaillement d'étage (V_n) dans les éléments voiles en BA du bâtiment mixte de 12 étages pour la méthode en capacité

Figure 17 (b) Maximum storey shear (V_{max}) and nominal storey shear strength (V_n) in RC wall members for the 12-storey RC dual frame-wall for capacity design method

In view of the aforementioned observations presented in paragraphs 8.2 to 8.5 of the present paper, it is clearly shown that in the case of the conventional design method, the shear failure criterion is the controlling local collapse for the 4-, and 8-storey structures; the collapse of the 12- storey structure is controlled by the inter-storey drift global failure criterion. In the case of the capacity design method, the inter-storey drift criterion is the controlling global collapse for the 4-, and 12-storey structures; the collapse of the 8- storey structure is controlled by the flexural local failure criterion.

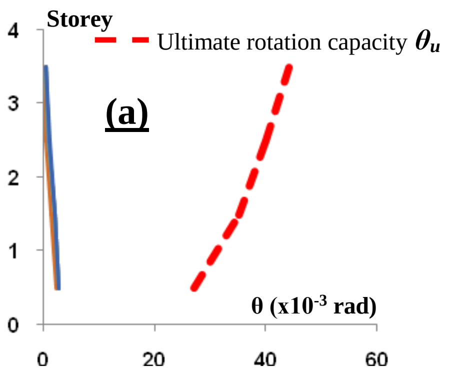


Figure 18 (a) Rotation maximale (θ_{max}) et rotation ultime (θ_u) dans les éléments voiles en BA du bâtiment de 4 étages pour la méthode conventionnelle

Figure 18 (a) Maximum flexural rotation (θ_{max}) and ultimate rotation (θ_u) in RC wall members for the 4-storey RC dual frame-wall for conventional design method

9- Conclusions

In this study the influence of capacity design method in comparison with conventional design method on the seismic performance of the walls, as well as the structure as a whole, when subjected to seismic loading, has been investigated.

For this purpose, 4-storey, 8-storey and 12 storey RC dual frame-wall structures were designed according to Algerian seismic design code RPA 99/Version 2003, in case of conventional design method, and to EC8 provisions, related to the capacity design of wall in flexure and shear, in case of capacity design method.

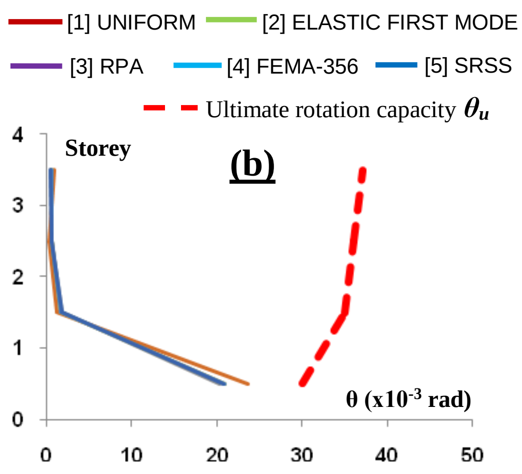


Figure 18 (b) Rotation maximale (θ_{max}) et rotation ultime (θ_u) dans les éléments voiles en BA du bâtiment de 4 étages pour la méthode en capacité

Figure 18 (b) Maximum flexural rotation (θ_{max}) and ultimate rotation (θ_u) in RC wall members for the 4-storey RC dual frame-wall for capacity design method

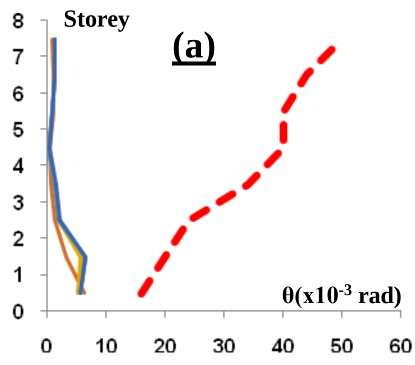


Figure 19 (a) Rotation maximale (θ_{max}) et rotation ultime (θ_u) dans les éléments voiles en BA du bâtiment de 8 étages pour la méthode conventionnelle

Figure 19 (a) Maximum flexural rotation (θ_{max}) and ultimate rotation (θ_u) in RC wall members for the 8-storey RC dual frame-wall for conventional design method

Nonlinear static pushover analyses using five different invariant lateral load patterns were carried out to represent the likely distribution of inertia forces imposed on the structures during an earthquake and to identify their dominant failure modes and failure paths.

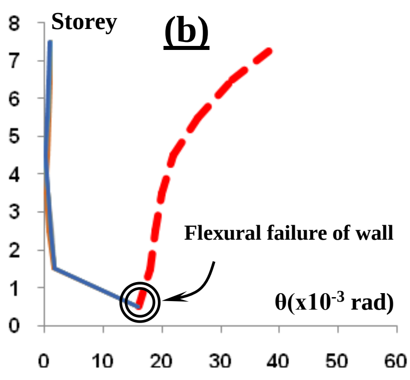


Figure 19 (b) Rotation maximale (θ_{max}) et rotation ultime (θ_u) dans les éléments voiles en BA du bâtiment de 8 étages pour la méthode en capacité

Figure 19 (b) Maximum flexural rotation (θ_{max}) and ultimate rotation (θ_u) in RC wall members for the 8-storey RC dual frame-wall for capacity design method

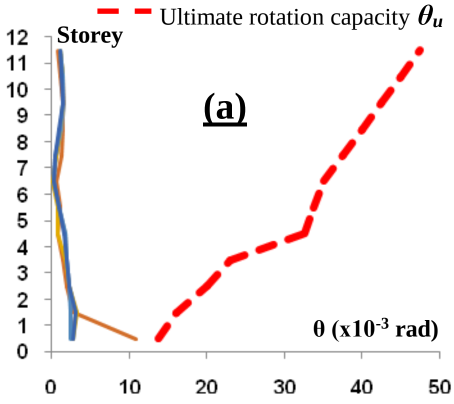


Figure 20 (a) Rotation maximale (θ_{max}) et rotation ultime (θ_u) dans les éléments voiles en BA du bâtiment de 12 étages pour la méthode conventionnelle

Figure 20 (a) Maximum flexural rotation (θ_{max}) and ultimate rotation (θ_u) in RC wall members for the 12-storey RC dual frame-wall for conventional design method

Failure criteria at both member and structural levels have been also adopted to detect plastic mechanisms and collapse limit states of structures.

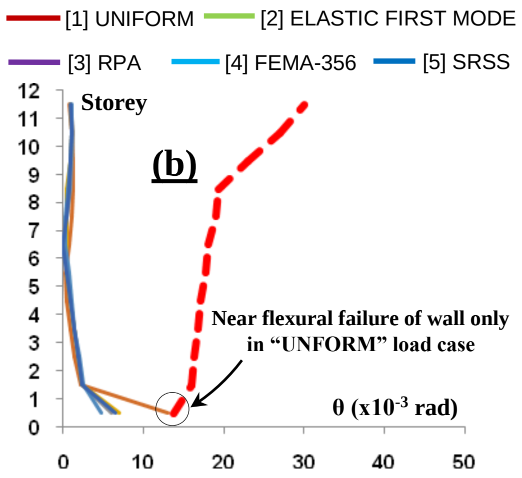


Figure 20 (b) Rotation maximale (θ_{max}) et rotation ultime (θ_u) dans les éléments voiles en BA du bâtiment de 12 étages pour la méthode en capacité

Figure 20 (b) Maximum flexural rotation (θ_{max}) and ultimate rotation (θ_u) in RC wall members for the 12-storey RC dual frame-wall for capacity design method

All of the comparisons lead to the expected conclusions that the structures designed according to the capacity design method are much safer than buildings belonging to the conventional design method, particularly in low-rise structures represented in this study by 4-, and 8-storey buildings. Due to larger seismic forces they have a higher lateral load-carrying capacity, and due to better design provisions and the ensuring of a suitable plastic mechanism they demonstrate much greater displacement ductility.

The study confirmed that the capacity design procedure can be revealed as an appreciate tool to improve seismic performance of the structures and to avoid any undesirable seismic failure mode, such as shear failure in RC structural walls. This type of failure leads to substantial loss of strength and ductility, and is primarily responsible for the collapse of buildings. This suggests improvements in the design provisions of the Algerian seismic design code, particularly for low-rise structures represented here by 4-, and 8-storey RC dual buildings.

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