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# Effect of Building Height and Vertical Seismic Load Patterns on the Behavior Factor of Reinforced Concrete Frames with Rigid Masonry Infill

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**ABSTRACT**

In the seismic design of reinforced concrete (RC) frame structures, the interaction between the structural frame and masonry infill walls is generally neglected or treated in a simplified manner. Masonry infill walls, typically constructed from hollow clay bricks or concrete blocks bonded with low-strength mortar, may be either fully connected to or detached from the surrounding RC frame. Numerous experimental and analytical studies have demonstrated that infill walls can significantly influence the lateral strength, stiffness, and overall seismic response of RC frames.

In the present study, the nonlinear behavior of RC frame–infill systems is modeled using the equivalent diagonal compression strut approach, with the objective of quantifying the contribution of rigid masonry infill walls to the seismic performance of such structures. First, linear modal response spectrum analyses are performed to highlight the limitations of the Algerian seismic code RPA99/Version 2003 in adequately capturing frame infill interaction effects. Subsequently, the impact of masonry infill on the nonlinear seismic response of RC frames is thoroughly investigated through nonlinear static (pushover) analyses.

Based on the pushover results, the influence of building height on the bidirectional evaluation of the global seismic behavior factor (R) of RC frames with masonry infill is examined, considering several vertical distributions of lateral seismic forces. The computed values of the behavior factor are then compared with those prescribed by the Algerian seismic code RPA99/Version 2003 for fully infilled frames and frames with a soft or open story.

The results obtained from the different analyses confirm the necessity of explicitly accounting for masonry infill effects in seismic design. In particular, they underline the relevance of incorporating equivalent diagonal strut models into the Algerian seismic code to achieve a more realistic assessment of the seismic performance and behavior factor of RC frame structures with masonry infill.

**Keywords:** Reinforced concrete frames, Masonry infill walls, Seismic behavior factor (R), Pushover analysis, Frame–infill interaction, Open (soft) story, Seismic design codes.

## INTRODUCTION

The main objective of this study is to perform a bidirectional assessment of the seismic behavior factor (R) of reinforced concrete (RC) frame structures with rigid masonry infill, considering both fully infilled configurations along the entire height and frames incorporating an open (soft) story. This evaluation is conducted on the basis of predefined global and local failure criteria using nonlinear static incremental analysis (pushover analysis), while accounting for different vertical distributions of lateral seismic forces.

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Furthermore, the values of the behavior factor (R) derived from the analyses are compared with those recommended by the Algerian seismic code RPA 99/Version 2003, namely R = 3.5 for fully infilled frames and R = 2.0 for frames with an open story. Accordingly, the study aims to assess the adequacy and validity of the seismic design provisions for reinforced concrete frame structures with rigid masonry infill (full infill or open story) prescribed by RPA 99/Version 2003, which rely on an elastic response spectrum in conjunction with an assumed behavior factor representative of the structural system.

## MODELING OF MASONRY INFILL WALLS

Experimental investigations conducted by Mainstone [8], as well as by Klingner and Bertero [12], on masonry-infilled reinforced concrete frames subjected to lateral loading have shown that diagonal cracking typically initiates at the center of the infill panel. Simultaneously, separation gaps develop between the frame and the infill along the unloaded diagonal, while full contact is maintained at the two corners along the loaded diagonal, as illustrated in Figure 1 [1].

Based on these observations, Polyakov [23] introduced the macro-modeling approach in 1966, commonly referred to as the equivalent diagonal strut method. In this method, the masonry infill panel is replaced by an equivalent compression strut that simulates the load transfer mechanism between the frame and the infill, allowing the global structural response of infilled frame systems to be effectively evaluated (Figure 2).

Despite its simplicity and wide acceptance, a major limitation of the equivalent diagonal strut model lies in its inability to accurately represent the influence of openings within the infill walls [22]. Nevertheless, several improvements have been proposed in the literature, where multiple diagonal struts are introduced to account for the presence and effects of openings in masonry infill panels [10].

In the present study, only exterior masonry infill walls without openings are considered. These walls are modeled using equivalent infill panel elements in order to focus on their contribution to the global seismic response of the reinforced concrete frame system.

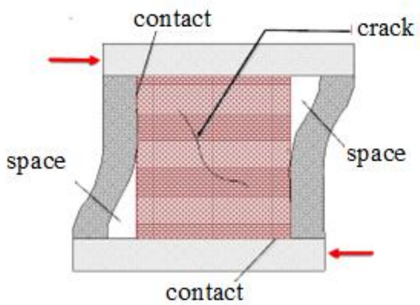


**Fig.1 Masonry-Infilled Frame under Seismic Loading.**

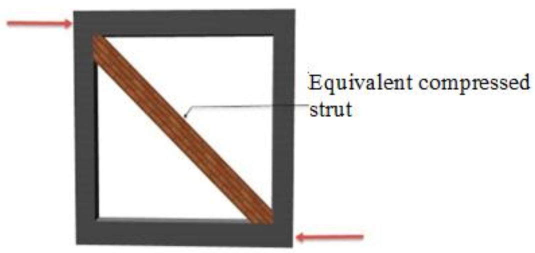


**Fig. 2 Equivalent Diagonal Strut Model for Masonry Infill under Seismic Loading.**

Polyakov [23] in 1966 introduced the macro-model method. Also, it was called the equivalent diagonal strut method, replacing the masonry infill with an equivalent masonry compressed strut to study the overall response of frame buildings with infill as shown in Figure 2. The main disadvantage of this method is the deficiency in the precise modeling of the openings [22]. However, some progress has been made in the opening of infill walls where a number of struts can be used to account for the effect of openings [10]. In the present study, only exterior infill walls are modeled as infill panel elements without any opening.

### 1. *Geometric and Mechanical Properties of the Equivalent Strut*

Different formulations have been proposed in the literature to estimate the width of the equivalent diagonal strut and the resistance of masonry infill panels. Some of these approaches have been incorporated into national design codes; however, a unified and generally accepted methodology has not

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yet been established [2]. In the present study, the recommendations of FEMA 356, which adopt the Mainstone formulation, are employed to model masonry infill walls [10].

a. *Width of the Equivalent Strut*

According to FEMA 356, masonry infill walls in the pre-cracking stage are idealized by an equivalent diagonal compression strut of width a. The thickness and modulus of elasticity of the strut are assumed to be identical to those of the infill panel, as illustrated in Figure 3.The width of the equivalent diagonal strut, $a$, based on the Mainstone formulation [17], is expressed as a function of the clear column height between beam and column axes, $h_{col}$, the diagonal length of the infill panel, $r_{inf}$, and the coefficient $\lambda_1$, as follows:

$$a = 0.175\,(\lambda_1 h_{col})^{-0.4} r_{inf} \tag{1}$$

where the diagonal length of the infill panel, $r_{inf}$, is given by:

$$r_{inf} = \sqrt{(L_{inf})^2 + (h_{inf})^2} \tag{2}$$

The coefficient $\lambda_1$ is defined as a function of the infill panel height where $h_{inf}$, the modulus of elasticity of the frame material ($E_{fe}$) , the modulus of elasticity of the masonry infill ($E_{me}$ ), , the moment of inertia of the columns ( $I_{col}$), , the thickness of the infill panel $t_{inf}$, and the angle $\Phi$ between the diagonal strut and the horizontal axis. It is calculated using the following expression:

$$\lambda_1 = \left[\frac{E_{me}\, t_{inf} \sin 2\Phi}{4\, E_{fe}\, I_{col} h_{inf}}\right]^{1/4} \tag{3}$$

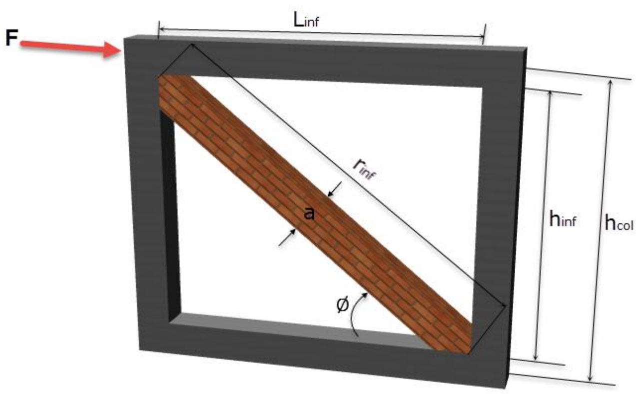


**Fig. 3 Parameters Characterizing the Equivalent Diagonal Compression Strut**

## NONLINEAR STATIC (PUSHOVER) ANALYSIS.

Using pushover analysis, the nonlinear force–displacement relationships of beams and columns can be explicitly determined. A critical step of the procedure is the selection of an appropriate lateral load distribution, generally proportional to the fundamental mode shape, which governs the development of plastic mechanisms along the height.

Once the pushover analysis is performed, the global structural response is represented by a characteristic capacity curve expressing the relationship between base shear and control-node displacement. This curve can be idealized by a bilinear single-degree-of-freedom (SDOF) system and is characterized by key parameters such as yield point, ultimate capacity, and post-yield stiffness (Figure 4).

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The pushover curve provides a direct representation of stiffness degradation, ductility demand, and energy dissipation capacity, allowing identification of yielding sequences and potential failure mechanisms without resorting to nonlinear dynamic analysis.

### 1. *Capacity Spectrum Method (CSM)*

The seismic demand corresponding to the nonlinear capacity is evaluated using the Capacity Spectrum Method (CSM) defined in ATC-40 (1996). This method estimates earthquake-induced deformations by comparing structural capacity with seismic demand in the acceleration–displacement response spectrum (ADRS) format.

ATC-40 proposes three procedures for determining the performance point. Procedures A and B are analytical and suitable for numerical implementation, whereas Procedure C is graphical and mainly intended for manual applications. Procedure A is generally recommended due to its simplicity and directness.

The CSM procedure involves :

- determination of the base shear–roof displacement capacity curve,
- evaluation of modal dynamic characteristics,
- transformation of the capacity curve into a spectral acceleration–spectral displacement capacity spectrum,
- definition of response spectra for different damping levels,
- identification of the performance point as the intersection between capacity and demand spectra.

The performance point represents the expected seismic displacement demand and provides a rational basis for evaluating ductility, force reduction, and the adequacy of the adopted behavior factor.

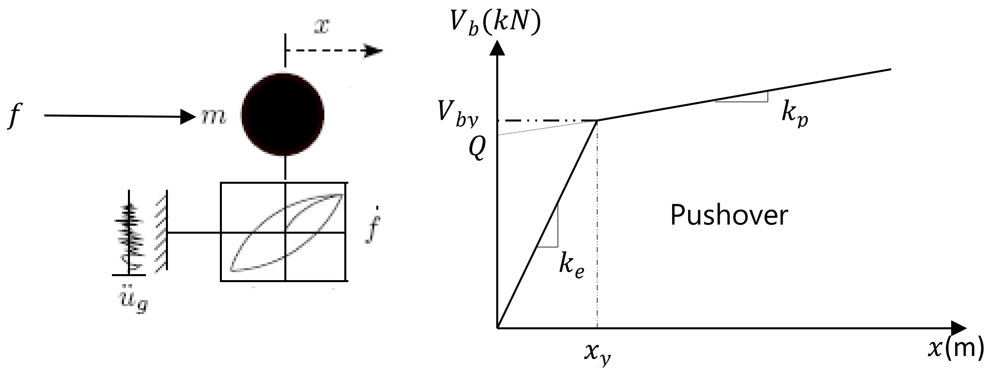


**Fig. 4 Capacity curve of an equivalent bilinear single-degree-of-freedom (SDOF) system, illustrating yield point, ultimate capacity, and post-yield stiffness.**

## STUDIED STRUCTURES

To achieve the objectives of this study, a set of structural models with the following characteristics is considered.

### 1. *Geometry and Structural Configuration*

Seven three-dimensional structures with the same number of bays in plan (Figure5 ) but with different numbers of storeys (Figures 5(a) and 5(b)) are investigated. The buildings range from 2 to 8 storeys in height. Starting from the slenderest structure (R+7), the remaining configurations (R+6 down to R+1) are successively derived by progressively removing the top storey. The reinforcement detailing of all structures is obtained using the same design procedure. All seven structures are assumed to be fully fixed at their bases.

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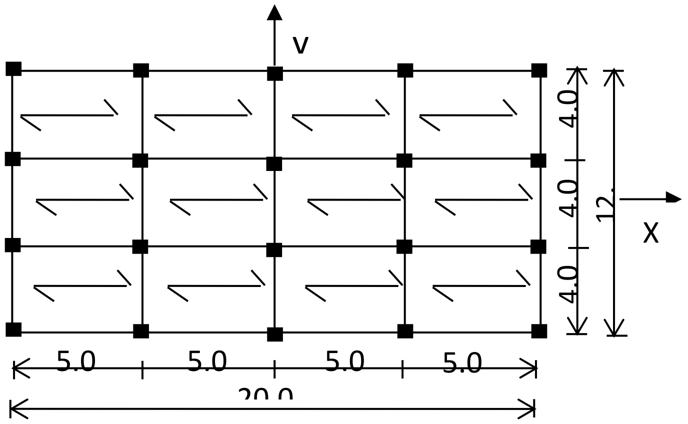



**Fig. 5. Floor plan of the studied buildings**

**a- Direction XZ XZ**

**b- Direction YZ**

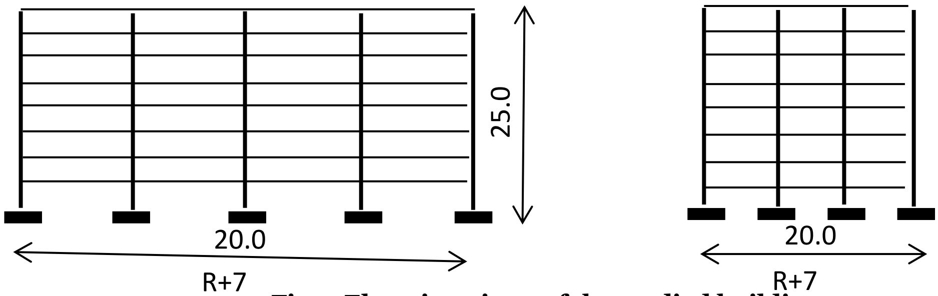
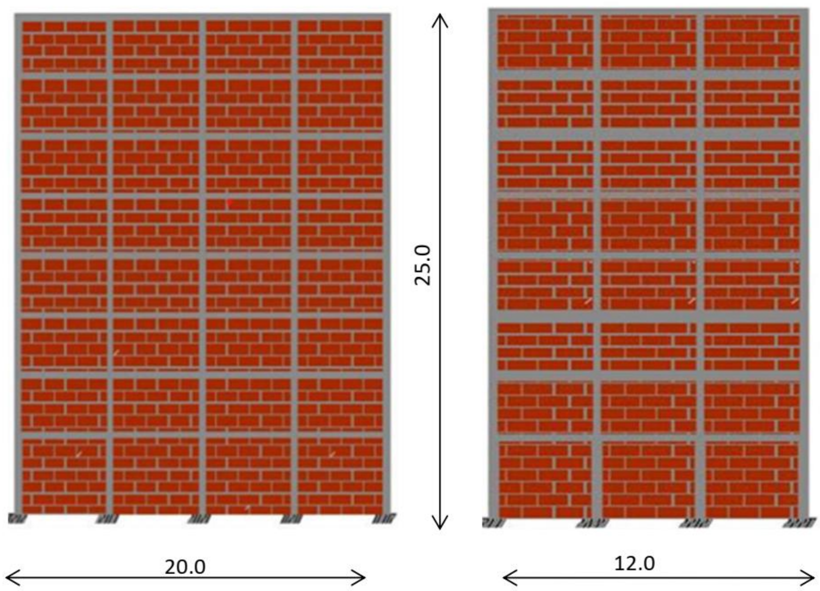



**Fig. 5 Elevation views of the studied buildings.**

### 2. *Design Data of the Studied Structures*

The reference structure considered is a reinforced concrete moment-resisting frame building with a ground floor plus seven storeys (R+7), infilled over its entire height with masonry made of hollow fired-clay bricks (Figure.6).




(a) XZ direction
(b) YZ direction.

**Fig. 6 R+7 structure with full-height masonry infill: (a) XZ direction; (b) YZ direction.**

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The main properties of the infill material (hollow fired-clay brick) are summarized in Table1.

|  | Units | Fired clay brick |
|---|---|---|
| **Density** | Kg/m³ | 1000 à 1600 |
| **Elastic modulus** | Mpa | 3550 |
| **Characteristic compressive strength** | Mpa | 10 |

**Table1. Main properties of the hollow fired-clay brick infill material [71]**

The seven studied structures (R+1 to R+7) are office buildings with hollow-core slab floors of type (16+4) and identical plan layouts. The permanent loads (R) are taken as 6.2 kN/m² for the roof slab and 5.6 kN/m² for typical floors. The imposed (live) loads (Q) are assumed equal to 1.0 kN/m² for the roof slab and 1.5 kN/m² for typical floors.

All buildings are assumed to be located in a high seismicity region (Zone III according to RPA99/Version 2003) and classified as use group 2. The site conditions correspond to soil type S3 (soft soil). The design behaviour factor is taken as R = 3.5, and the damping ratio is assumed equal to ξ = 7% (dense infill), in accordance with RPA99/Version 2003. A quality factor Q = 1 is adopted.

The design of beams and columns (concrete sections and reinforcement) is carried out considering the bare-frame configuration. Masonry infill walls are taken into account only as vertical loads acting on the frame, while their contribution to stiffness and strength is neglected. The thickness of the external infill walls is assumed equal to 30 cm.

Structural design is performed in accordance with the reinforced concrete limit state design code BAEL91 and the Algerian seismic code RPA99/Version 2003. The mechanical properties of the materials are as follows: for concrete, the characteristic compressive strength at 28 days is 25 MPa; for steel reinforcement, both longitudinal and transverse bars are of grade FeE400, with a yield strength of 400 MPa.

It should be noted that for the five tallest buildings (R+3 to R+7), although RPA99/Version 2003 practically limits the height of reinforced concrete frame buildings with or without masonry infill to three storeys in high seismic zones, this limitation is intentionally disregarded in the present study. This assumption is justified by the objectives of the research, which focus on investigating the influence of structural slenderness on the value of the behaviour factor. The design of longitudinal and transverse reinforcement in beams and columns is performed by considering the effects of gravity loads (permanent and imposed) and seismic actions, combined according to the load combinations prescribed by both BAEL91 and RPA99/Version 2003

### *3. Formwork and Reinforcement of Beams and Columns of the Studied Structures*

The cross-sectional dimensions of the beams and columns, along with their corresponding longitudinal reinforcement areas, are summarized in **Table2** It should be noted that the steel sections presented in this table correspond to the beam and column ends. These regions are subjected to the highest seismic demands and are therefore the critical zones where plastic hinges are likely to form.

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| | Beam | | | Column | |
|---|---|---|---|---|---|
| Floor | Concrete section | Steel section | | Beams (Dimensions) | Colums (Dimensions ) |
| | | Top steel layer | Bottom steel layer | | |
| 8 | 30×50 | 3Ø16+3Ø14 | 4Ø12 | 40×40 | 8Ø16 |
| 7 | 30×50 | 3Ø16+3Ø14 | 4Ø12 | 45×45 | 10Ø16 |
| 6 | 30×50 | 3Ø20+3Ø12 | 4Ø14 | 50×50 | 8Ø20 |
| 5 | 30×50 | 3Ø20+3Ø12 | 4Ø14 | 50×50 | 8Ø20 |
| 4 | 30×50 | 3Ø20+3Ø12 | 4Ø14 | 55×55 | 10Ø20 |
| 3 | 30×50 | 3Ø20+3Ø14 | 4Ø14+3Ø12 | 55×55 | 10Ø20 |
| 2 | 30×50 | 3Ø20+3Ø14 | 4Ø14+3Ø12 | 60×60 | 12Ø20 |
| 1 | 30×50 | 3Ø20+3Ø14 | 4Ø14+3Ø12 | 60×60 | 12Ø20 |

**Table2: Reinforcement at the ends of beams and columns of the studied structures.**

For each considered structure, the study was conducted using a bidirectional pushover analysis implemented in SAP2000 (version 18). This analysis yields, for each principal direction, a capacity curve (also referred to as a pushover curve). From these curves, the parameters required to evaluate the structural behavior factor in both planar directions are extracted, as will be detailed in the following sections.

## PARAMETRES DU FACTEUR DE COMPORTEMENT, R

The behavior factor R is defined as the product of the ductility factor (Rμ) and the overstrength factor Rs:

$$R = R_s R_\mu \qquad (4)$$

This formulation provides a clear and practical representation of the structural capacity to dissipate seismic energy through both ductility and inherent overstrength.

### *1. Components of the Global Behavior Factor, R*

The choice of the value of the behavior factor R is not straightforward; most regulatory codes adopt a single fixed value for this factor. In reality, the behavior factor is a complex function of a large number of parameters, and its expression cannot be reduced to a simple constant [5]. Various researchers [26–29] have highlighted the empirical nature and lack of rationality of the behavior factor values assigned by different seismic codes [15].

The philosophy of seismic design is that a structure should withstand an earthquake without collapsing, but with some damage. In accordance with this philosophy, the structure is designed for shear forces much lower than would be required if the building were to remain elastic during a strong ground motion at a site. To justify this reduction in seismic forces, seismic codes rely on the reserve strength and ductility, which enhance the structure's capacity to absorb and dissipate energy.

Thus, the role of the global behavior factor and the parameters influencing its assessment and control are essential elements of seismic design according to the codes. The values assigned to the response modification factor (R) in American codes (FEMA, 1997; UBC, 1997 [4]) are intended to account for both reserve strength and ductility (ATC, 1995). Some documents also mention redundancy in the structure as a separate parameter (ATC-19, 1995).

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The Uang (1991) method is adopted in this research for the determination of the R factor. The response reduction factor R is expressed as a function of various structural system parameters, such as overstrength, ductility, damping, and redundancy [19–20], which are defined based on the capacity curves obtained from the static pushover analysis.

Where (also denoted $R_s$) is the design reserve strength factor (overstrength), Rμ is the ductility factor, $R_\xi$ is the damping factor, and $R_R$ is the redundancy factor.

The global behavior factor R can be expressed as the product of its individual components [19–20]:

$$R = R_s R_\mu R_\xi R_R \qquad (5)$$

Where:

Where Rs (also denoted $R_\Omega$) is the design reserve strength factor (overstrength),
Rμ is the ductility factor,
$R_\xi$ is the damping factor, and
$R_R$ is the redundancy factor.

Thus, the overall response reduction factor R integrates the contributions of overstrength, ductility, damping, and redundancy to quantify the actual capacity of the structure to withstand seismic forces while sustaining limited damage. This approach ensures that the design reflects both the strength reserve and the energy dissipation capacity of the structural system, in line with modern seismic design philosophies [5–19].

### 2. *Ductility Factor, $R_\mu$*

The ductility factor ($R_\mu$) is a measure of the global nonlinear response of a structural system in terms of its plastic deformation capacity. It is defined as the ratio of the ultimate base shear assuming elastic response ($V_e$) to the ultimate base shear considering inelastic response ($V_u$). The different levels of base shear used to define these two components (Rs and Rμ).

$$R_\mu = \frac{V_e}{V_u} \qquad (6)$$

Over the past three decades, significant research has been conducted to establish the ductility factor based on SDOF systems subjected to various types of ground motion. Among these, the works of Newmark and Hall [23], Riddell and Newmark, Vidic et al. and Krawinkler and Nassar [24] are particularly notable and frequently cited. For a comprehensive review, the reader is referred to Miranda and Bertero [16].

The ductility reduction factor (Rμ) reflects the energy dissipation capacity of adequately designed and well-calculated structures. It primarily depends on the global ductility demand (μ\muμ) of the structure defined as the ratio of the maximum roof displacement to the elastic roof displacement its damping, fundamental period of vibration (T), and characteristics of the ground motion, among other parameters [26]. Several approaches for calculating Rμ exist in the literature. In this study, the formulations of Newmark and Hall [26], Krawinkler and Nassar, Fajfar, and Priestley are employed for computing the ductility reduction factor Rμ.

### 3. *Overstrength Factor, Rs*

The overstrength factor (Rs) accounts for the reserve lateral capacity of structures, which typically exceeds the design seismic base shear ($V_d$) [21]. This reserve, also referred to as structural overstrength, is a key parameter influencing the response of structures under seismic loading. Rs is defined as the ratio of the ultimate base shear ($V_u$) to the design base shear ($V_d$):

$$R_S = \frac{V_u}{V_d} \qquad (7)$$

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### Vertical Distribution of Horizontal Seismic Loads

The selection of the vertical distribution of horizontal seismic loads for incremental pushover analysis is a critical aspect of the method. In general, the distribution depends on the seismic intensity (induced inelastic displacements) and time during the event.

If the structure is dominated by the fundamental mode and exhibits a single collapse mechanism, a single vertical load distribution may suffice. However, a single distribution cannot capture local displacement demands or all potential local failure mechanisms. Therefore, at least two load distributions are recommended. Lateral loads are commonly applied proportionally to the floor masses, increasing demand at lower stories. The distributions recommended by FEMA356 are widely used.

### SRSS Distribution (Square Root of the Sum of Squares)

The vertical load per level is obtained from the combination of modal responses from spectral modal analysis, using enough modes to capture at least 90% of the building mass. This accounts for higher-mode effects in long-period or irregular structures.

1. Lateral force at level i for the n mode:

$$F_s = \Gamma_n m_i \Phi_{in} A_n \qquad (8)$$

where $\Gamma_n$ is the modal participation factor, mi is the mass of level i, $\Phi_{in}$ is the modal amplitude at level i, and $A_n$ is the pseudo-acceleration of the elastic SDOF system for mode n.

2. Compute story shear :

$$V_{in} = \sum_{j=1}^{N} \mathrm{F}_{j,n} \qquad (9)$$

N is the total number of stories.

Combine the modal forces at each level using the SRSS method.

$$V_i = \sqrt{\sum_{i=0}^{n} (\mathrm{V}_{in})^2} \qquad (10)$$

3. Combine modal forces using SRSS
4. Determine lateral forces Fi at each level.

$$F_i' = \frac{F_i}{\sum_{j=1}^{N} \mathrm{F}_j} \qquad (11)$$

5. Normalize lateral forces by base shear for convenience.

### Definition of the Target Displacement of the Structure.

The displacement control point may be defined either at the center of mass of the structure or at its highest point. In the present study, the control point is selected at the top level of the frame.

### Failure Criteria

The evaluation of the behavior factor R is carried out herein on the basis of local and global failure criteria, defined respectively as follows.

**Local failure criteria:** Local failure is defined by limiting the value of the **elasto-plastic rotation** in a structural member, namely a **beam or a column**, to the **ultimate rotation** of the element, denoted by θu , as defined by equation 12:

$$\theta_u = \theta_u + \theta_p \qquad (12)$$

$\theta_p$ : Rotation plastique de la section en béton armé, calculée en utilisant l'équation proposée par l'AtC40

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$\theta_u$: Courbure ultime correspondant à la ruine de la section en béton arme, soit par traction des aciers tendus ou par écrasement du béton.

For masonry infill panels, the adopted local failure criterion is that defined in (Table 7-7 of FEMA 273). When the masonry reaches a relative drift equal to d (%), it is considered completely failed and no longer provides any resistance.

**Global failure criteria:** Global failure of the structure is governed by one or more of the following criteria:

- **Inter-story drift limitation**: The inter-story displacement Δ is limited to 2.5% of the story height he in the performed pushover analyses. This limit is also reported in references [12, 29] and is close to the limits recommended by several seismic design codes [18], which generally range between 2% and 3% of the story height.

According to Eurocode 8 . Part 1, the allowable inter-story drift $d_r$ for buildings incorporating brittle non-structural elements attached to the structure is defined as:

$d_r \nu \leq 0{,}005h$ ,where $\nu$ is a reduction factor accounting for the lower return period of the seismic action associated with damage limitation requirements, taken equal to 0.4 for importance classes III and IV.

- **Formation of a plastic mechanism**: The development of a plastic mechanism at a single story or involving several stories of the structure, leading to structural instability of the bare frame [30], and/or of the frame with rigid masonry infill, is considered as a global failure condition.

- **P–Δ instability**: Global failure may also occur due to **P–Δ effects**, characterized by exceeding the limiting value of the stability coefficient $\theta_{P-\Delta}$, taken equal to 0.2. Thus, the structure is considered unstable when P–Δ>0.2

According to RPA 99 / Version 2003, the coefficient $\theta_{P-\Delta}$is defined by Equation (13).

$$\theta_{\mathrm{K}} = \frac{P_K \Delta_K}{V_K \mathrm{h}_K} \leq 0{,}10 \qquad (13)$$

Pk : poids total de la structure et des charges d'exploitation associées au-dessus du niveau « k »,

$V_K$ : Effort tranchant d'étage au niveau "k"

Δk : déplacement relatif du niveau « k » par rapport au niveau « k-1 »

$\mathrm{h}_K$ : hauteur de l'étage « k »

**Local failure criteria**

Local failure is defined by limiting the value of the elastoplastic rotation in a structural member beam or column to the ultimate rotation, θu, of the element (as defined by Equation (12)).

For masonry infill, the adopted local failure criterion is that defined in Table (Table 7-7 of FEMA 273). When the masonry reaches a relative displacement equal to $d$ (%), it is considered to be completely failed and no longer provides any resistance.

**Global failure criteria**

Global failure of the structure is governed by one or more of the following criteria:

- **Interstory drift limitation:** the interstory drift, Δ, is limited to 2.5% of the story height (he) in the performed pushover analyses. This limit is also specified in [20, 18] and is close to the limits recommended by several seismic design codes [4], which range between 2% and 3% of the story height. The interstory drift limit, dr, prescribed by Eurocode 8 – Part 1 (Eurocode 8 Committee, 2003) for buildings with brittle non-structural elements attached to the structure is given by : $d_{\mathrm{r}} = \nu$

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where ν\nuν is a reduction factor accounting for the lower return period of the seismic action associated with damage limitation requirements; ν=0.4for importance classes 3 and 4.

- **Formation of a plastic mechanism:** the development of a plastic mechanism at a single story or involving several stories, leading to structural instability of the bare frame [30], and/or of the frame with rigid masonry infill.
- **P–Δ instability:** instability due to second-order (P–Δ) effects, characterized by exceeding the limiting value of the stability coefficient $\theta_{P-\Delta}$ , taken equal to 0.2; that is, when $\theta_{P-\Delta} > 0.2$. According to RPA 99/Version 2003, the coefficient $\theta_{P-\Delta}$ is defined by Equation (13).

**Characteristics and constitutive laws of struts**

The geometric and mechanical characteristics, as well as the most unfavorable constitutive behavior laws of the equivalent struts adopted for each structure (Table3), are governed by the infill panel (3 m × 5 m) located at the second story in the longitudinal direction and inserted between two columns with cross-sectional dimensions of 60 × 60 cm.

| | | | | **Elastic limits** | | **Performance levels** | | | **Ultimate limit** |
|---|---|---|---|---|---|---|---|---|---|
| Equivalent strut width, $a$ (m) | Material density | Elastic modulus $E_m$ (MPa) | $f'_m(Mpa$ | $Fe(KN)$ | $De(m)$ | $IO(m)$ | $LS(m)$ | $CP(m)$ | $C(m)$ |
| 0,66 | 1500,00 | 3550,00 | 6,46 | 810,44 | 0,00529 | 0,00595 | 0,00600 | 0,00840 | 0,00860 |

**Table3: Geometric and mechanical characteristics and constitutive behavior laws of the generalized compression strut.**

It should be noted that, in order to refine the interpretation, monitoring, and understanding of the plastic behavior of the strut, the Immediate Occupancy (IO) and Collapse Prevention (CP) performance levels located on either side of the Life Safety (LS) level were intentionally defined, although they are not provided in Table 7-7 of **FEMA 273**.

Figure 7: below illustrates the three-dimensional macro-modeling of the reference structure with rigid masonry infill

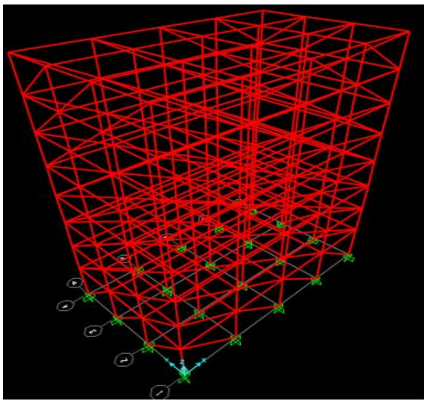

**Fig. 7: Three-dimensional macro-modeling of the reference (baseline) structure (R+7).**

## RESULTS OF THE ANALYSES

In the following, the results of the pushover analyses carried out on the seven studied structures, in both the X and Y directions, are presented and discussed. These structures range from 2 to 8 storeys and were previously described and designed.

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Subsequently, the ductility reduction factor Rμ and the overstrength factor Ω are evaluated. The parameters related to the global ductility and the two aforementioned reduction factors are extracted from the capacity curves obtained from the pushover analyses.

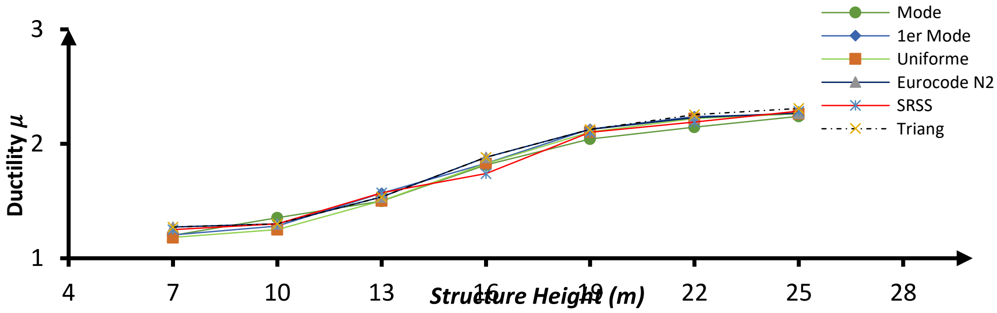
It should be noted that the pushover analyses were performed in both the X and Y directions using the different vertical distributions of lateral seismic loads described above.

### Comparative Analysis of Results for the Studied Structures

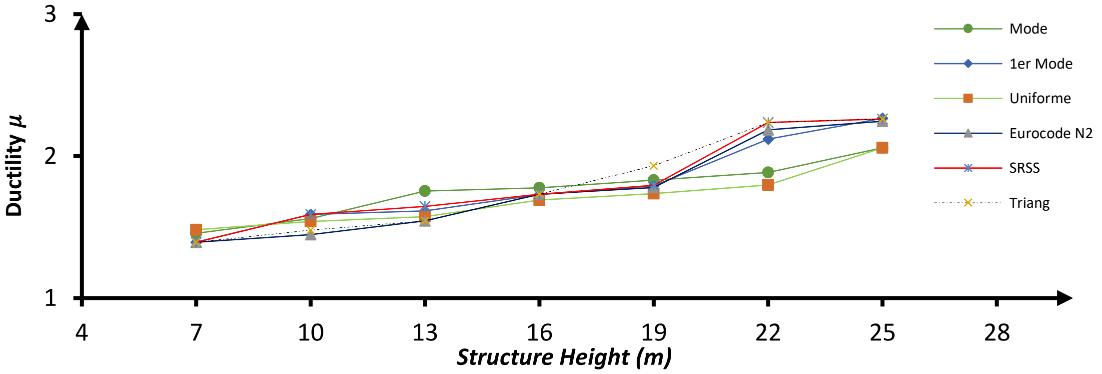
In this section, a comparison is carried out between the results of various parameters obtained from pushover analyses performed in both plan directions for the seven studied structures. In addition to global ductility and overstrength parameters, the behavior factor RRR represents the key parameter for comparison. It should be noted that these comparisons are made considering the frames at their ultimate limit states, corresponding to their respective collapse conditions.

### Global Ductility μ of the Studied Structures

The global ductility as a function of the height of the seven studied frames, in both plan directions and considering several vertical distributions of horizontal loads, is shown in Figure8. The global ductility μ\muμ is expressed as the ratio between the maximum top displacement of the frame and its elastic limit displacement dy, corresponding to the end of the elastic phase of the idealized pushover curve.




a) X-Direction




b) Y-Direction

**Fig. 8 Global ductility of the seven studied structures as a function of building height for different vertical distributions of lateral loads**

ductility μ increases substantially from the 2 story frame to the 8-story frame in both plan directions. Therefore, the global ductility of the frame increases with the number of stories.

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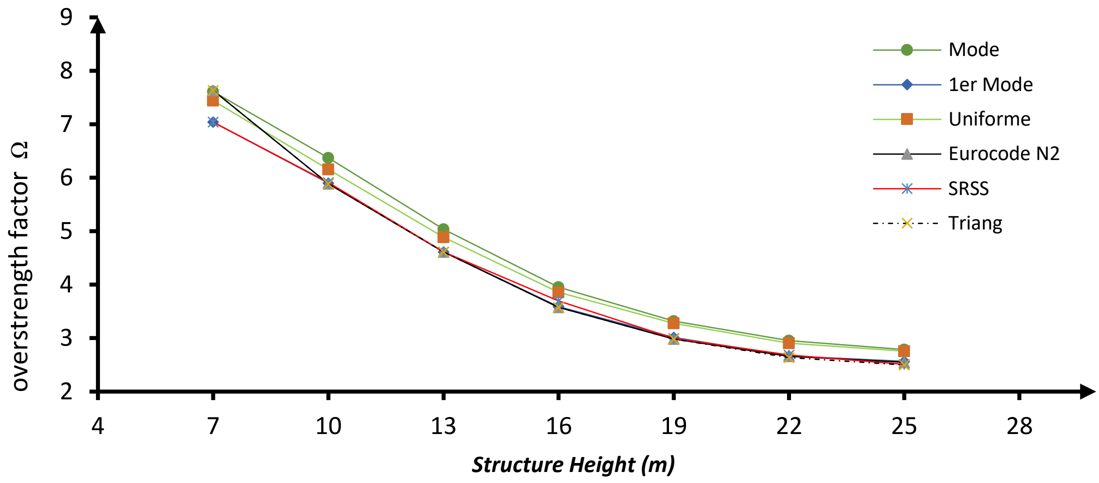
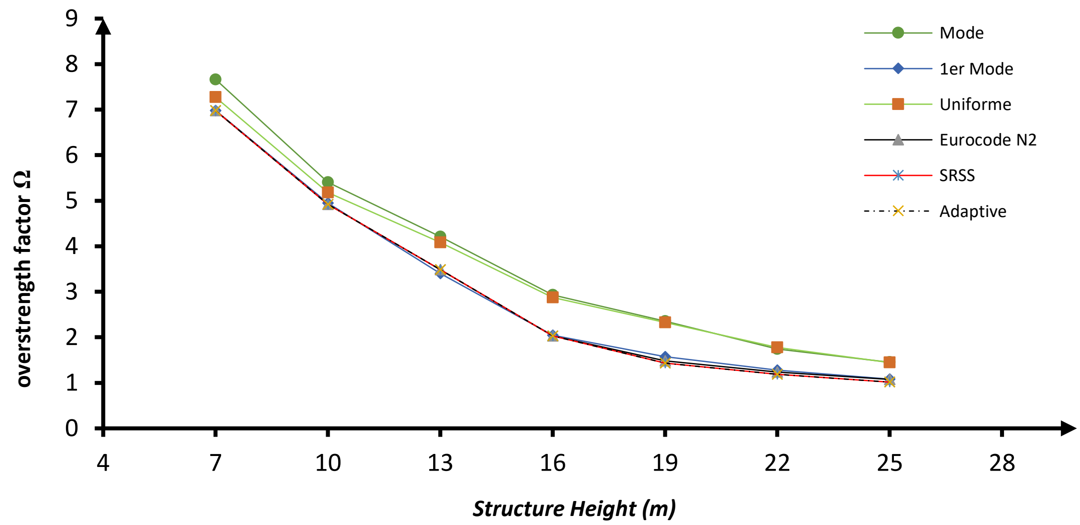
It is also observed that the values of global ductility μ in the longitudinal direction, resulting from the different vertical distributions of lateral loads, exhibit very little dispersion, in contrast to the transverse Y-axis, where some divergence is observed, particularly from the 7th story onward. Moreover, on average, the ductility in the transverse Y-direction is relatively close to that in the X-direction.

### Overstrength Factors Ω of the Studied Structures

The overstrength factors of the seven studied frames in both plan directions are presented in Figure9. This figure indicates that, regardless of the vertical distribution of lateral loads, the overstrength Ω decreases substantially from the 2-story frame to the 8-story frame in both plan directions.




a) X-Direction




b) Y-Direction

**Fig. 9 Overstrength Factors of the Studied Structures**

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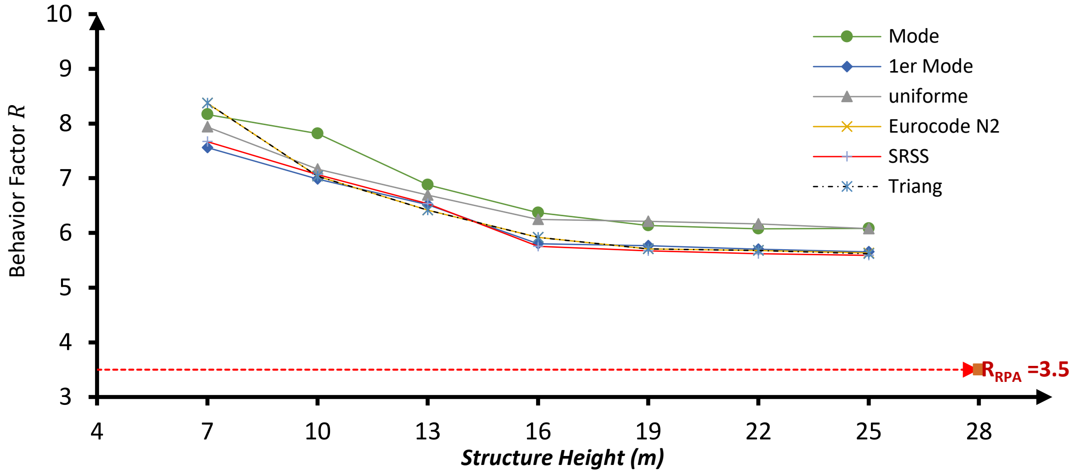
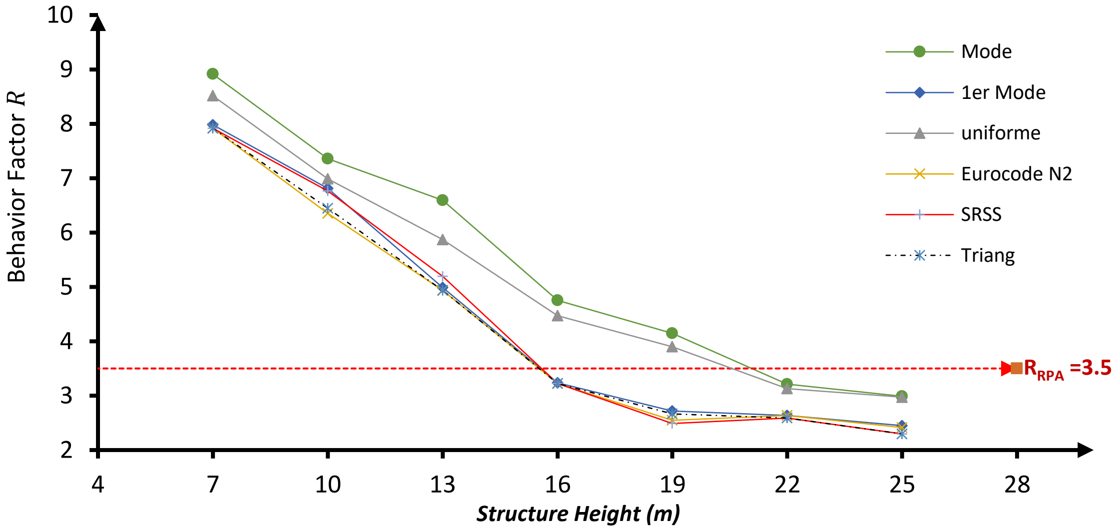
**Behavior Factor, R, of the Studied Structures**

Depending on the vertical distribution of horizontal seismic forces, the behavior factor R of the seven studied frames in each planar direction is estimated by considering them at their ultimate limit states corresponding to their respective failure states.

For each vertical distribution of seismic loads, the variation of the behavior factor R with the number of stories is shown in Figure10.




a) X-Direction




b) Y-Direction

**Fig. 10.: Influence of Building Height on the Behavior Factor R**

It can be observed that, regardless of the vertical distribution of seismic loads, the value of the behavior factor R decreases as the number of storeys increases in both directions of the plan, with higher values in the X direction than in the Y direction.

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This indicates that the behavior factor depends, among other parameters, on the slenderness of the structure and the calculation direction—parameters not accounted for in current seismic design codes. A comparison between the RRR values evaluated in this study and those recommended by the RPA 99 / 2003 version code is therefore highly valuable.

The RPA 99 code recommends a behavior factor of R=3.5R = 3.5R=3.5 in both directions, which was considered for the design of the studied structures. As shown in Table 7, the calculated RRR values in the longitudinal X direction are significantly higher than the code recommendation, with an average increase of more than double. In contrast, for frames in the transverse direction, structures with more than five storeys exhibit RRR values well below the code-specified value.

## CONCLUSIONS

This study presents an evaluation of the behavior factor R for reinforced concrete frame structures. Several failure criteria were considered to assess the collapse of the frame structures. Pushover analyses were performed in both plan directions under various vertical distributions of seismic loads on seven structures with identical plans but different numbers of storeys, ranging from 2 to 8. The main conclusions drawn from the analysis are summarized as follows:

- The overall ductility of the frames increases with the number of storeys.
- The overstrength of the frames decreases as the number of storeys increases.
- The behavior factor R decreases with the increase in the number of storeys.
- The behavior factor R decreases with the increase in overall ductility.
- The behavior factor R increases with the increase in overstrength.
- The values of R, μ and Ω are dependent on the calculation direction (X and Y).
- The behavior factor depends, among other parameters, on the slenderness of the structure and the calculation direction, which are not accounted for in the Algerian seismic design code.

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