Effect of the capacity ratio developed at column-beam nodes and evaluation of its random degradation impact on the formation of global ruin mechanisms

Amar Messas, Karim Benyahi, Arezki Adjrad and Youcef Bouafia
Department of Civil Engineering, Universite Mouloud Mammeri de Tizi Ouzou, Tizi Ouzou, Algeria, and
Sarah Benakli
Department of Civil Engineering, Laboratory LMSSC, CNAM, Paris, France

Abstract

Purpose – The purpose of this study, is to deals with capacity design (strong column – weak beam) in reinforced concrete frames, slightly slender, which depends on the determination of a capacity ratio necessary to reach a structural plastic mechanism. To find the capacity ratio allowing to achieve a fairly ductile behavior in reinforced concrete frames, it is necessary to validate this concept by a non-linear static analysis (push-over). However, this analysis is carried out by the use of the ETABS software, and by the introduction into the beams and columns of plastic hinges according to FEMA-356 code.

Design/methodology/approach – This approach makes it possible to assess seismic performance, which facilitates the establishment of a system for detecting the plasticization mechanisms of structures. It is also necessary to use a probabilistic method allowing to treat the dimensioning by the identification of the most probable mechanisms and to take only those that contribute the most to the probability of global failure of the structural system.

Findings – In this study, three reinforced concrete frame buildings with different numbers of floors were analyzed by varying the capacity ratio of the elements. The results obtained indicate that it is strongly recommended to increase the ratio of the resistant moments of the columns on those of the beams for the Algerian seismic regulation (RPA code), knowing that the frameworks in reinforced concrete are widespread in the country.

Originality/value – The main interest of this paper is to criticize the resistance condition required by RPA code, which must be the subject of particular attention to reach a mechanism of favorable collapse. This study recommends, on the basis of a reliability analysis, the use of a capacity dimensioning ratio greater than or equal to two, making it possible to have a sufficiently low probability of failure to ensure a level of security for users.

Keywords Resistance moment, Ductility, Ruin mechanism, Capacity ratio, Plastic hinges, Probability of failure

Paper type Research paper

Notations

- θ_y = Elastic limit rotation of the reinforced concrete section;
- θ_u = Ultimate limit rotation of the reinforced concrete section;
- θ_p = Plastic rotation of the reinforced concrete section;
- M_y = Elastic moment limit of the reinforced concrete section;
- M_p = Plastic moment of the reinforced concrete section;
- Φ_y = Elastic limit curvature;
- Φ_u = Ultimate curvature;
- L_p = Length of the plastic hinge;
- h = Element section height;
- V = Shear base of the structure;
- U = Displacement of the structure;
- β = Capacity ratio of resistant moments developed at column-beam nodes; and
- P_f = Probability of failure of a structural system.

1. Introduction

Framed structures constitute the most vulnerable structures during earthquakes; they presented during major earthquakes, numerous collapses, causing deaths of thousands of people. To address this vulnerability, many recommendations have been made by earthquake codes around the world to reduce the risk of collapse. Among these recommendations is the transition to

The current issue and full text archive of this journal is available on Emerald Insight at: **https://www.emerald.com/insight/1708-5284.htm**

World Journal of Engineering
19/5 (2022) 605–619
© Emerald Publishing Limited [ISSN 1708-5284]
[DOI 10.1108/WJE-12-2020-0659]

Conflict of interest: On behalf of all authors, the corresponding author states that there is no conflict of interest.

Received 24 December 2020
Revised 15 March 2021
Accepted 28 March 2021

capacity design. It constitutes one of the basic principles of earthquake-resistant construction, which consists of creating "fuses" in elements of the structure repairable after an earthquake and preventing vital elements from suffering damage, causing their ruin. Indeed, during major earthquakes, damage to the structure is almost inevitable, but by locating it in certain elements, the structure is allowed to develop sufficient ductility and energy dissipation capacity to avoid ruin of the latter during major earthquakes. The principle of "strong column-weak beam" is one of the foundations of capacity design and constitutes the most important criterion for the structure to avoid ruin during major earthquakes. By respecting this principle, the structure is allowed to dissipate energy by forming plastic hinges in the beams instead of the columns, which offers the possibility of reaching a structural mechanism, allowing the use of most of the capacity of the structure. Most earthquake codes around the world have introduced this principle, requiring a ratio (noted β in the literature) of the resistant moments of the columns to the resistant moments of the beams at the same node greater than the unit. The purpose of this recommendation was to limit the formation of plastic hinges in the columns, which constitute vertical load-bearing elements essential for the stability of the structure.

In the scientific literature, there are several works carried out on the value of the capacity to take ratio at the level of the nodes necessary to reach a favorable ruin mechanism. Here, only a few of them are reviewed. Kuntz and Browning (2003) have shown, using limit analysis, that the recommendations of the seismic codes are insufficient to reach a favorable mechanism. The authors demonstrated that the coefficient β necessary to reach a structural mechanism increased with the increase in the number of stages of the structure. Coefficients of 1.8 for slender structures up to 3.6 for very slender structures. The authors also recommend another method, which consists of a gradual decrease in the resistance of the beams along the height of the structure, a minimum threshold is established, which is the resistance necessary for the resumption of vertical loads. They have also proposed an equation for calculating this reduction, but admit that the use of this method is quite limited. Haselton *et al.* (2011) conducted a non-linear dynamic analysis to assess the risk of collapse of self-supporting reinforced concrete structures. The structures were designed according to ACI 318-02 – American Concrete Institute (2002), ASCE7-02 – American Society of Civil Engineers (2002) and ASCE7-05 – American Society of Civil Engineers (2005) code. In all 30 structures that were studied, heights were ranging from 1 to 20 floors. The coefficient b is part of the studied criteria influencing the behavior of a structure β. The authors base themselves on their study to assert that for slender structures, the risk of collapse can be reduced by increasing the coefficient β. This gave the possibility of spreading the damage over a greater number of stories, making it possible to make the most of the bearing capacity of the structure. Among the recommendations made is the variation of the β ratio along the structure, implying higher β coefficient at the level of the lower floors. Murty *et al.* (2012) carried out a study on the β ratio necessary for a structure to reach a favorable failure mechanism. The study consists of a push-over analysis on a frame structure with five floors in an area of high seismicity. The seismic load as well as the dimensioning are carried out according to the Indian regulation [IS 1893. (Part 1), 2007]. The coefficient β is varied from 1.2 to 3.6. The authors note in particular an increase in lateral resistance with the increase in the coefficient β, as well as an improvement in ductility. It was concluded that ratios ranging from 1.2 to 3.2 were not sufficient to achieve a favorable ruin mechanism, the latter case being reached only from the coefficient 3.6. The authors affirm the need to reach the latter, in order to allow a better distribution of the damage along the height of the structure. The posts being important elements for the transfer of the gravitational loads, their damage must be limited to the maximum and this to ensure their role even after earthquake. Sudarsana *et al.* (2014) conducted a study on the influence of the variation of the β coefficient (SCWB) on the seismic performance of self-supporting reinforced concrete structures. Fourteen five- and ten-story portal frames are studied, varying the coefficient β from 1.0 to 2.0 in each of the structures. The structures were dimensioned with the Indonesian regulation SNI 2847 (2013). The authors performed push-over analyses using SAP 2000 V.15 software, and concluded that increasing the β coefficient to 1.4 increased the level of ductility significantly. The study also looked at the influence of using the probable resistive moment instead of the nominal resistive moment; in the non-linear field, the reinforcement of the beams can undergo a work hardening, increasing thereafter the resistant moment of the beams, by taking into account this phenomenon, it was noted a clear improvement of the factor of ductility of the structures studied. Cagurangan (2015) studied the influence of the coefficient β on slender structures; it was found that increasing the ratio β leads to a decrease in the probability of failure of structures; nevertheless, the author demonstrates that the influence of the β ratio decreased with the increase in the height of the structures. Ning *et al.* (2016) demonstrated, through an experimental study, that the introduction of reinforced concrete slabs in reinforced concrete gantries modified the mechanism of formation of plastic hinges, ranging from beams before their introduction, to columns directly after their introduction. Karanjit (2017) conducted a study on the influence of the β coefficient (SCWB) on the seismic performance of structures. The study relates to five moment-resistant frame structures having different heights by varying the coefficient β, values from 1.0 to 2.0 were given. The study is based on a push-over analysis with SAP 2000 version 14.0.0 software; the frames structures were dimensioned according to the Indian regulations (IS 456, (2000), IS 1893 (2002) and IS 13920, (1993)). The results of the study showed an improvement in the ruin mechanism and the capacity curve with the increase in the β coefficient. The author also proposes a design model, in which he recommends provisions at the base of columns of the ground floor, to improve the resistant moments of the latter. Surana *et al.* (2018) carried out incremental dynamic analyses on several structures with different slenderness. Structural fragility curves have shown that the concept of SCWB allows a significant reduction in the probability of failure. Gökdemir and Günaydin (2018) conducted a study on the influence of the "strong column-weak beam" principle on the behavior of a self-stable structure, by comparing two types of structures. One respecting the strong post weak beam principle noted SCWB, the other the reverse principle of strong weak post beam noted WCSB, these principles are applied by varying the flexural stiffnesses of beams and columns. The structures were compared according to three loading cases; vertical loading, horizontal loading and combined loading, then compared in terms of displacements and internal forces at the level of the structural elements in each of the load cases. A state of the art of various studies concerning the influence of the capability ratio was carried out by Kadukar *et al.* (2018).

Studies have also been carried out on the seismic resistance of concrete and reinforced concrete elements. Zhang and Alam (2020) as well as Kurama *et al.* (2018) give a state of the art on the various works carried out so far. Saravanan *et al.* (2018) propose a state of the art on the various works concerning the seismic resistance of steel structures.

For a long time, calculation codes have been satisfied with methods based on simple mechanical calculations, ensuring that the resistance of the structure is greater than the various stresses to which it was subjected. However, the information concerning the various resistances of the various elements constituting a structure, as well as the loads requesting it are marred by many uncertainties, the degree of safety of the structures was then indeterminate, the reliability methods then appeared to answer this problem.

Mayer (1926) proposed to take into account the variability of resistances and loads, use the mean values and the variances. Streletskii (1947) introduced the notion of security index, Rzhanitzyn (1949) introduced the notion of reliability index. Many authors have subsequently helped to develop the different notions of structural reliability (Cornell, 1968; Ravindra *et al.*, 1974; Carvajal *et al.*, 2011; Calgaro, 1991).

The reliability methods are mainly classified into two categories: approximation methods and simulation methods.

The best known approximation methods are the FORM and SORM methods, which allow an approximation of the limit state surface. Many authors have carried out work on the FORM method (Xiaoping Du, 2008; Hohenbichler *et al.*, 1987; Maier *et al.*, 2001) as well as on the SORM method (Zhao *et al.*, 2017; Cai and Elishakoff, 1994; Der Kiureghian *et al.*, 1987; Huang *et al.*, 2018; Zhao *et al.*, 2002). The Monte Carle method is the best known of the simulation methods, it was invented in 1947 by Metropolis and Ulam (1949) and was widely used (Tarawneh and Majdalaweyh, 2020; Balomenos *et al.*, 2018; Brunesi and Parisi, 2017).

Several works have been carried out on taking into account the uncertainties concerning a physical system and studying the impact of these uncertainties on the response of the system studied. Here, only a few of them are reviewed. Soares *et al.* (2002) studied the efficiency of the response surface reliability method, by applying the latter to several examples of structures. By coupling with a mechanical model, this method makes it possible to obtain a reliability index in the vicinity of the design point, making it possible to deal with any errors generated by the mechanical model used. In addition, the authors were able to examine the partial safety factors recommended by the international codes of the time; by carrying out a parametric study, they conclude that the latter provides reasonable safety, but in most cases, these coefficients do not allow optimal economy. Altarejos-García *et al.* (2012) propose a reliability analysis method for concrete gravity dams, considering a given failure mode. The method is divided into five phases comprising steps in each one of them, deterministic models are coupled with reliability models. The method uses two deterministic models; a mathematical and a numerical one, as well as Levels 1 to 3 reliability methods. Benyahi *et al.* (2018) propose an analytical model taking into account mechanical and geometric non-linearities in the case of trusses structures. They propose a new approach to estimate the distribution laws of random variables, thus making it possible to estimate the reliability index of mechanical models by indirect coupling (response surface). Bouzid *et al.* (2020) conducted a study on the influence of the cover on the seismic performance of a self-stabilizing structure. The study consisted of push-over analyses on frame structures with low, medium and high slenderness. It demonstrated through a reliability study with the Monte Carlo method, by taking as random variables the resistance of concrete and the elastic limit of steels and as deterministic variable the value of the cover, that an error of 0.5 cm in the cover causes a structure to pass from the domain of safety to the domain of failure.

Many other reliable methods have been developed such as the Kriging method (Ni *et al.* (2020) or methods using an intelligent hybrid system (Santana *et al.* (2021); these methods have a definite advantage in terms of calculation precision, but present, on the other hand, the drawback of the complexity of the calculations (Zhao *et al.* (2017). One of the reliable methods present in the literature is the response surface method, widely used by researchers (Hammoudi *et al.*, 2019; Sofi *et al.*, 2020; Zhang *et al.*, 2017); this method is distinguished by the possibility of calculating the probability of failure in the absence of a limit state function; in certain mechanical problems, the limit state function cannot be explained, which is the case in this study, this method can be used.

In this article, we will show the effect of the variation of a capacity ratio from which a ruin mechanism is a structural plastic mechanism, which makes it possible to allow the most use of the capacity of the structure. This study will be carried out to issue, in particular, criticisms of the recommendations of Règlement Parasismique Algérien (RPA) (2003) on the modalities of a capacity design. This document is organized as follows. Section 2 deals with a comparison of the value of the coefficient β between different seismic codes and the different types of structural failure mechanisms. Section 3 presents the non-linear behavior of the constituent elements of reinforced concrete structures by the introduction of a model having a moment (M) – rotation (θ) diagram at their ends and over a length known as plastic hinges. The FEMA-356 code (FEMA-356. Federal Emergency Management Agency, 2000) will be used to describe the state of degradation of the sections, and therefore its level of penetration in the plastic field. Section 4 presents a reliable method for integrating the randomness of resistance. This approach makes it possible to study at best the value of the capacity ratio to be taken at the level of the column-beam nodes necessary to reach a global failure mechanism; for this, we have integrated a random degradation process and to evaluate its impact on the global ruin of a reinforced concrete structural system. Finally, Section 5 provides applications for the evaluation of seismic performance, making it easier to set up a system for detecting the plasticization mechanisms of structures. This involves using a non-linear static analysis (push-over) to study the effect of the increase in the capacity ratio of the resistant moment on the ductility and the lateral

World Journal of Engineering
Volume 19 · Number 5 · 2022 · 605–619

resistance of a reinforced concrete structure. It is also necessary to use a reliability model to study the variations in the probabilities of failure of the different structures studied, according to the criteria adopted in its design.

2. Formation of ruin mechanisms according to seismic codes

The origin of the coefficient β dates back to the 1970s with ACI 318–71 – American Concrete Institute Committee 318 (1971), which at the time required a value greater than or equal to the unit. Nowadays, the condition on the ratio of the resistant moments of the beams and columns of the same node is given in Table 1.

Many earthquake-resistant codes speak of the principle of "strong column-weak beam." Vijayanarayanan *et al.* (2017) summarized the different divergences between the different codes in the world, namely:

1 taking into account or not the axial force in the columns; taking into account the axial force, considerably reduces the moment of resistance in the columns;
2 effect of the floor on the resistant moment of the beams; the rigidity provided by the floor is added to that of the beams, thus increasing the resistance moment of the beams;
3 place where the resistant moment is considered; whether in the center of the node or at the face of the ends of the elements of the frame; and
4 limits of the value of β (Table 1, Figure 1).

The Algerian seismic code [Règlement Parasismique Algérien (RPA), 2003] Article 7.6.2 stipulates:

> It is necessary to verify for the frames participating in the bracing system and for each of the possible orientations of the seismic action that the sum of the ultimate resistant moments of the ends of columns or uprights leading to the node is at least equal in absolute value to the sum of the absolute values of the ultimate resistant moments of the ends of the beams or crosspieces affected by a coefficient of increase of 1.25. This arrangement tends to cause plastic hinges to form in the beams rather than in the columns.

That is to say the resistance condition required by the seismic regulations, which is expressed in the RPA (2003), Article 7.6.2 in the following form:

$$|M_n| + |M_s| \geq 1.25(|M_w| + |M_e|) \tag{1}$$

$$|M_n'| + |M_s'| \geq 1.25(|M_w'| + |M_e'|) \tag{2}$$

As the Algerian seismic code [Règlement Parasismique Algérien (RPA), 2003], Article 3.4/A.1.a stipulates that there are three level limitations for framed structures according to the different seismic zones:

1 Buildings must not exceed five stories or 17 m in Zone I.
2 Buildings must not exceed four stories or 14 m in Zone IIa.
3 Buildings must not exceed three stories or 11 m in Zones IIb and III.

The study will focus on a non-linear static analysis (push-over) of these structures to determine the coefficient β necessary to reach a structural plastic mechanism. For this, three structures will be designed according to regulation [Règlement Parasismique Algérien (RPA), 2003], keeping the sections of beams and columns constant; the different values of the coefficient β will be determined by varying the reinforcement of the columns, going from 1.2 until reaching the structural mechanism. The aim of the study is to find the coefficient from which a ruin mechanism is a structural plastic mechanism. There are three different types of plasticizing mechanisms for structures, which are as shown in Figure 2.

1 Story mechanism: where all of the plastic hinges are concentrated on one of the stories of the structure, in particular the columns. This mechanism being the less recommended, allowing a very limited energy dissipation capacity, this case is common in structures with flexible floor.
2 Intermediate mechanism: in this mechanism, the plastic hinges are formed in the columns and beams of some of the stories of the structure. In this type of mechanism, part of the energy dissipation capacity is used. The formation of plastic hinges in the columns of the lower floors prevents the collapse mechanism from spreading over the upper floors.
3 Structural mechanism: all of the hinges are formed in the beams of the structure, and at the level of the base of the columns of the first story. This mechanism is the most advantageous for a structure, because it allows the most optimal energy dissipation for the structure.

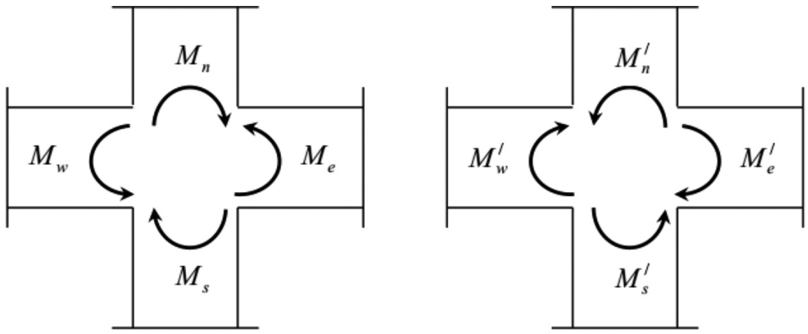
3. Nonlinear modeling of reinforced concrete frame elements

This study will be based on the non-linear analysis (push-over) of reinforced concrete frames. The beams and columns of the reinforced concrete frames are characterized by non-linear laws of behavior in bending; the shear behavior is assumed to be linear (no plasticization by shear). The beams and columns are modeled by elements having linear

Table 1 Comparison of the value of the coefficient β between different earthquake codes

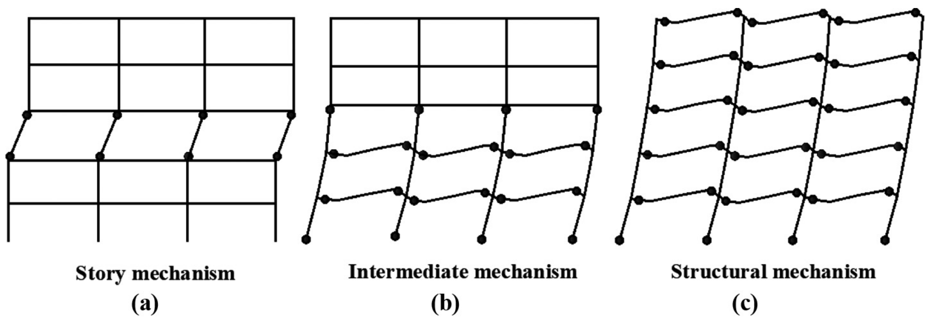
Code	Value of β
American : ACI 318-11. ACI Committee, American Concrete Institute, and International Organization for Standardization (2011)	$\sum M_c/\sum M_b \geq 1.2$
New Zealand: (NZS3101, 1995)	$\sum M_c/\sum M_b \geq 1.4$
European: EC98. Eurocode (2004)	$\sum M_c/\sum M_b \geq 1.3$
India: IS 13920 (2016)	$\sum M_c/\sum M_b \geq 1.4$
Turkish: TEC 2007. Turkish Earthquake Design Code (2009)	$\sum M_c/\sum M_b \geq 1.2$
Morocco: [Règlement Parasismique Marocain (RPS), 2002]	$\sum M_c/\sum M_b \geq 1.15$

Figure 1 Design of a beam-column node



Source: RPA. Règlement parasismique Algérien, 2003

Figure 2 Plastic mechanisms for framed structures



Source: Kuntz and Browning (2003)

elastic properties, the non-linear behavior of the elements is translated by the introduction of plastic hinges at the level of the sections likely to plasticize. The inelastic deformations are therefore concentrated at the two ends. The building modeling is generated in the ETABS 2017 building analysis and design software (CSI. Computers and Structures, 2017), which allows us to define different types of hinges and assign them in the areas likely to plasticize at the level of the structural elements. The non-linear behavior of the constituent elements of reinforced concrete structures is taken into account by the introduction of the appropriate models of the moment (M) – rotation (θ) diagrams at their ends and over a length known as plastic length. The diagram shown in Figure 3 illustrates the curve of the curvature moment relationship of a reinforced concrete section associated with bending plasticization for beams and columns.

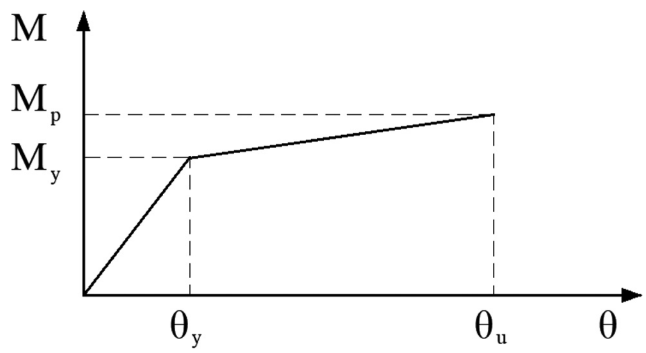
The moment-curvature law of a section depends on its geometrical characteristics, on the mechanical characteristics of the materials that compose it but also on the longitudinal, transverse reinforcement and on the normal force of the section.

The ultimate rotation is calculated using the following equation:

$$\theta_u = \theta_y + \theta_p \tag{3}$$

The plastic rotation of the reinforced concrete section is calculated by:

Figure 3 Moment-rotation diagram associated with flexural plasticization for elements



Source: FEMA-356. Federal Emergency Management Agency, 2000

$$\theta_p = (\phi_u - \phi_y)\,.\,L_p \tag{4}$$

The length of the plastic hinge is derived according to the ATC-40 code (ATC-40. Applied Technology Council, 1996):

$$L_p = 0.5h \tag{5}$$

The elastic limit rotation of the reinforced concrete section is calculated by:

$$\theta_y = \phi_y . L/6 \quad (6)$$

The ultimate curvature corresponds to the ruin of the reinforced concrete section, either by traction of the tensed steels or by crushing of the compressed concrete.

For our calculation models concerning beams, the hinges allocated are the "M3-type" bending hinges. For the columns, the hinges allocated are the "PMM" hinges (normal force coupling and bending moment). We will use the FEMA-356 code (FEMA-356. Federal Emergency Management Agency, 2000) defining the points (IO, LS, CP) to describe the state of degradation of the sections, and therefore its level of penetration in the plastic domain (Figure 4).

Figure 4 consists of several points adopted the calculation codes, thus allowing to appreciate static non-linear analysis, and which are:

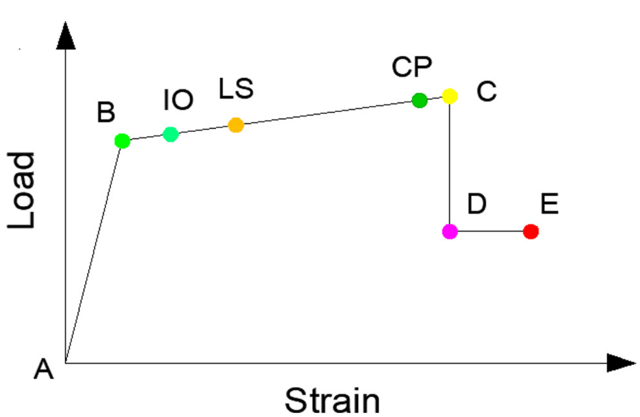
- Point A represents the origin point.
- Point B corresponds to the first plasticization, no deformation at the level of the hinges, all elastic deformations are ignored.
- Level IO (immediate occupancy): the damage is relatively limited; the section retains much of its initial stiffness.
- Level LS (life safety): the section suffered significant damage, which could lead to a significant loss of stiffness.
- CP level (collapse prevention): the section has undergone large post elastic deformations; beyond this level, the section is likely to break.
- Point C corresponds to the ultimate capacity of the push-over analysis.
- Point D represents the residual resistance of the analysis.
- Point E represents the total rupture of the element.

4. Probabilistic model

4.1 Introduction

The FORM and SORM methods consist in approaching the integration domain by its restriction to order one or two, and in reducing the probability calculus to simple formulas, using the properties of the Gaussian distribution. The response surface method is a complementary approach, rather than a reliability method in itself; it consists of replacing the initial physical model with a so-called response surface approximation, which is numerically fast to calculate. This response surface can be constructed on the basis of polynomials.

Figure 4 Behavior law (force – strain) and damage levels



Source: FEMA-356. Federal Emergency Management Agency, 2000

It is about using the Rackwitz–Fiessler algorithm (HLRF) developed in the reference (Benyahi *et al.*, 2018) [13], which is an adaptation of the method of the gradient projected to the problem of optimization in mechanical reliability (Rackwitz, 1976; Fiessler *et al.*, 1979), in which the computation of the Hasofer Lind reliability index is a constrained optimization problem.

4.2 Methodology

The transformation of the random vector x in physical space into a reduced centered Gaussian random vector u, whose mean is zero and the covariance matrix is the unit matrix, is necessary for the determination of the design point. If the variables are independent and if the distribution functions are known, the simplest transformation T consists in separately transforming each variable x_i into a normal centered variable u_i reduced by:

$$x_i \xrightarrow{T} u_i = \Phi^{-1}\left(F_{X_i}(x_i)\right) \quad (7)$$

where Φ is the distribution function of the reduced centered normal distribution (with mean 0 and standard deviation 1), and F_{X_i} is the distribution function of the variable x_i.

The design point (or most likely point of failure) is the point on the limit state surface, where the probability density of U is greatest. It is also defined as the point on the limit state surface closest to the origin. The index is obtained by solving the following minimization problem:

$$\beta_{HL} = \|u^*\| \quad (8)$$

$$\beta_{HL} = \min_{g\{x_i(u_j)\}\leq 0} \sqrt{\{u\}^T\{u\}} \quad (9)$$

Under stress $H(u) \leq 0$

P*: most probable point of failure, is the point in normalized space that achieves this minimum.

The function to be minimized is the Euclidean distance in standardized space. The Rackwitz and Fiessler algorithm iteratively solves the problem by generating a sequence of points that converge to an optimal solution. The most probable point of failure is obtained by successive iterations when the desired precisions on the limit state function ε_h and on two consecutive points of the algorithm ε_u are obtained, i.e. for two successive iterations k and $k + 1$, if:

$$H\left(\{u\}^{(k)}\right) \leq \varepsilon_H$$

$$\|\{u\}^{(k+1)} - \{u\}^{(k)}\| \leq \varepsilon_u$$

The index $\beta^{(k+1)}$ is the norm of the vector of random variables in the standardized space. It is deduced by:

$$\beta^{(k)} = -u^{(k)}\{\alpha\}^{(k)} + \frac{H(u^{(k)})}{\|\nabla H(u)\|_{u^{(k)}}} \quad (10)$$

$\{u\}^{(k+1)}$, deducted from $\{u\}^{(k)}$ by:

$$\{u\}^{(k+1)} = \left(\langle u\rangle^{(k)}\{\alpha\}^{(k)}\right)\{\alpha\}^{(k)} - \frac{H(u^{(k)})}{\|\nabla H(u)\|_{u^{(k)}}}.\{\alpha\}^{(k)} \quad (11)$$

From the reliability index β, the probability of failure is estimated by:

$$P_r = P(G(z) \leq 0) = \sum_{i=1}^{m} \Phi(-\beta_i) \quad (12)$$

with: β_i reliability index associated with u_i^*

5. Mechanical–reliability coupling

The coupling between the mechanical model and a reliability method is essential to carry out more realistic studies. The method used in this present work allows a mechanical–reliability coupling by the analytical response surface method. The interest of this coupling lies in the explicit form of the approximate limit state function, which makes it possible to apply the classical reliability methods (FORM, SORM) for the determination of the probability of failure of a studied structure.

First, the mechanical model consists in looking for a global failure mechanism, and this by varying the coefficient β. Failure equations (limit state functions) are calculated by a push-over analysis carried out by ETABS 2017 building analysis and design software (CSI. Computers and Structures, 2017). After having identified all the ruin mechanisms of the studied frames, we assume that the retained random variables are independent. The capacity curves obtained are extracted by taking the variables of the latter as random variables. Then, they are transformed in the normalized space by an iso-probabilistic method.

Finally, the reliability indices of the ruin equations (limit state functions) are calculated using the HLRF algorithm, and the probability of failure is estimated for the different structures studied. A summary diagram of the entire method used to calculate the probability of ruin of the studied structures is exposed in Figure 5.

6. Analysis and discussion of the results

In this study, three idealized doubly symmetrical structures with moment-resistant reinforced concrete frame, each having four longitudinal and transverse spans (Figure 6). The dimensions of the columns and beams, after checking the regulatory requirements are given in Table 2. These reinforced concrete frames are first designed according to the limit state concrete BAEL91 code (BAEL 91. Béton Armé aux Etats Limites, 1992) and the Algerian seismic code [Règlement Parasismique Algérien (RPA), 2003], and are calculated for the static loading on the basis of an elastic linear analysis carried out by ETABS 2017 building analysis and design software (CSI. Computers and Structures, 2017).

In the design of the columns, no change of section will be made along the height of the structure. For floors, they will be hollow bodies (16 + 4) and the permanent loads G (due to the dead weight of the elements) and operating overloads Q are deducted from the D.T.R.-B.C. 2.2. Document technique réglementaire, (1988): G = 5.23 KN/m^2 and Q = 1.5 KN/m^2.

Figure 5 Approach used in this research to quantify the failure probability

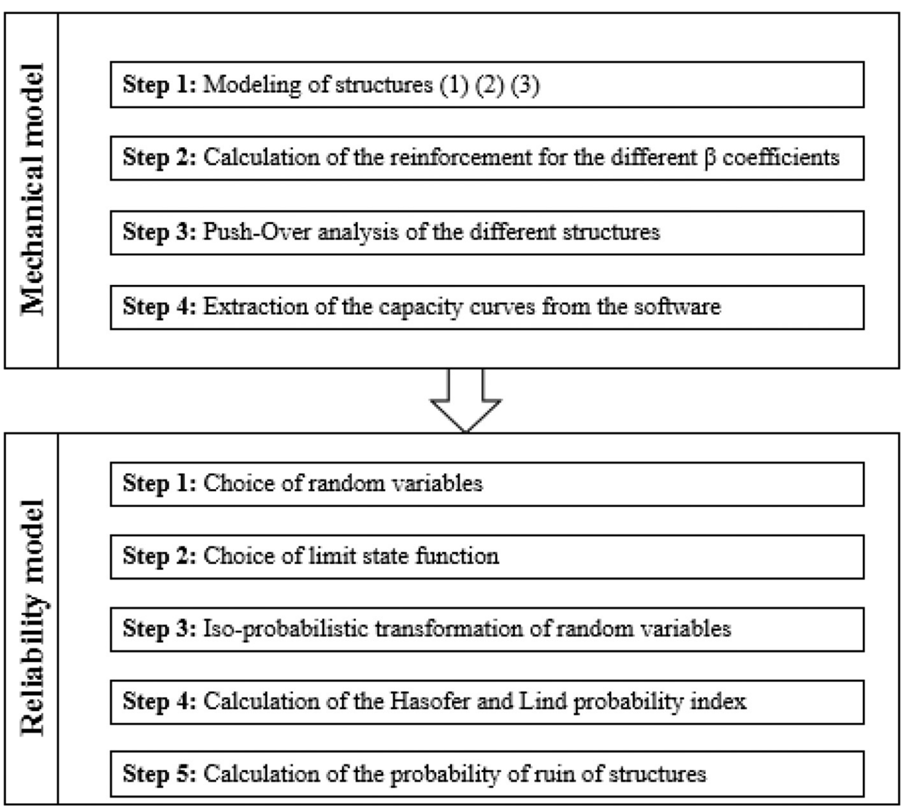
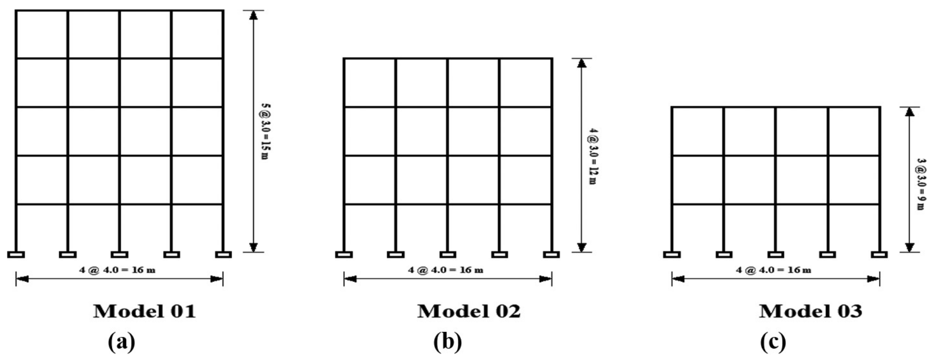


Figure 6 Details of the framework models chosen for this study



The mechanical characteristics of concrete and steels according to Articles A.2.1,21 and A.2.2,1, respectively, of BAEL91 code (BAEL 91. Béton Armé aux Etats Limites, 1992) are summarized in Table 3.

After analysis and verifications with regard to the seismic code [Règlement Parasismique Algérien (RPA), 2003], a calculation under gravity and seismic loads according to the combinations of actions is carried out to determine the longitudinal and transverse reinforcements in beams and columns. It is necessary to determine the resistant moments of the beams as well as those of the columns to check Article 7.6.2 of RPA (2003) at the level of the different frames. Finally, a push-over analysis is carried out to assess the seismic performance of the different reinforced concrete frames studied.

6.1 Comparison of the failure mechanisms on the different structures studied

After performing a non-linear static analysis (push-over), a comparison of the results obtained for the three structures studied is carried out. Figures 7–9 show the failure mechanisms of the three structures, as well as the plastic hinges that form at failure based on the defined failure criteria.

Table 2 Sections of the beams and columns

Structures	Columns	Beams
Model 01	40 × 40	40 × 35
Model 02	35 × 35	35 × 30
Model 03	30 × 30	30 × 30

Table 3 Mechanical characteristics of materials

Mechanical characteristics of concrete		
f_{cj} (MPa)	f_t (Mpa)	E_{bo} (Mpa)
25	2.1	32164.20
	Mechanical characteristics of steel	
Nature of steel	Elasticity limit σ_e (Mpa)	Elasticity module E_a (Mpa)
High adhesion	400	200,000

We note that as we increase the coefficient β, the ruin mechanism tends to spread over the upper stories, with a reduction in the appearance of plastic hinges in the columns, by promoting their appearance in the beams. When β = 1.2, the coefficient close to that recommended in the regulation [Règlement Parasismique Algérien (RPA), 2003], one can notice plastic hinges that develop in the columns of the base, but also in the columns of the higher stories. The plastic mechanism observed remains an intermediate and not a structural mechanism.

By increasing the coefficient β, we see that the plastic hinges tend first to disappear from the columns, then to extend on the upper floors in the beams. The mechanism is structural only from the ratio β = 2.8 for structures (01 and 02) on the other hand, it is only from the ratio β = 2.6 for structure (03). Below this value, the mechanisms that develop are intermediate mechanisms, with, in particular, plastic hinges in the columns, or in a reduced number of stories. From the figures (Figures 7–9), it can be seen that the ratio β = 1.25 recommended by RPA (2003) is not sufficient to achieve a structural mechanism allowing better distribution of damage along the height of the structure, and reducing hinges in the columns, which can cause the collapse of the reinforced concrete frame.

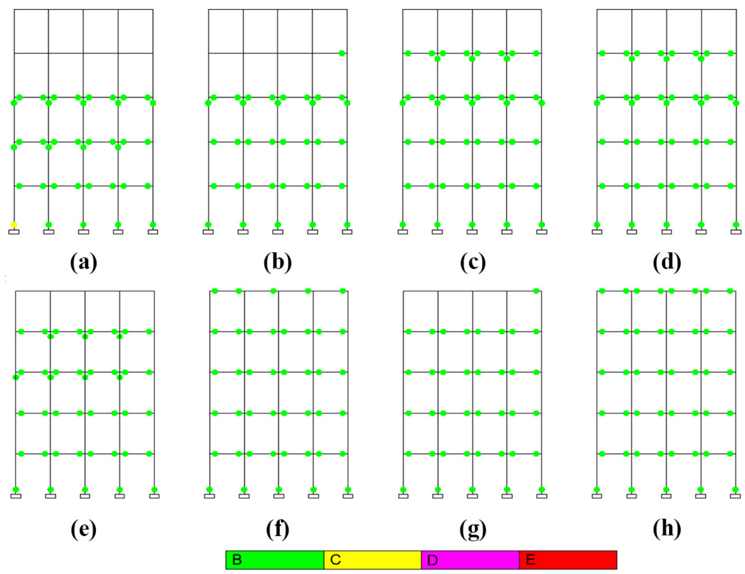
To have an overview of the mechanisms formed from non-linear analysis, Table 4 gives for each of the coefficients β the number of plastic hinges formed at each performance level for the different models studied.

As shown in Table 4, the structures (01) and (02) present some differences in the number of plastic hinges and where a considerable number is obtained for the ratio β = 2.8 (of type (LS-CP); (C-D). As for the structure (03), we observe that a large number of plastic hinges are formed for the ratio β = 2.6 (of type D-E). We can still note a slight tendency to increase the number of hinges that form as the coefficient β is increased, indicating a progression of the mechanism on the upper stories.

6.2 Comparison of capacity and ductility curves on different structures

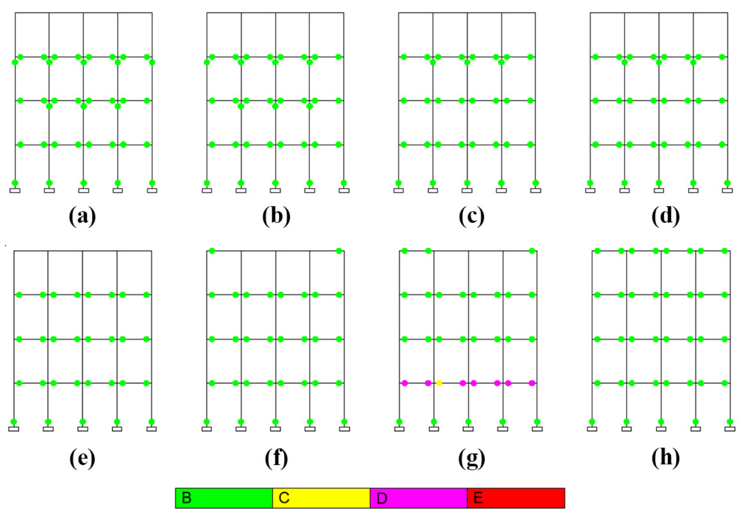
It is a question of making a comparison of all the capacity curves, and of the overall ductility of the three reinforced concrete frames studied with the different coefficients β assigned. Figures 10–12 show a comparison of the different capacity curves of the non-linear static analyses (push-over) carried out on the structures with the different coefficients β.

Figure 7 Mechanisms of structural failure (Model 01)



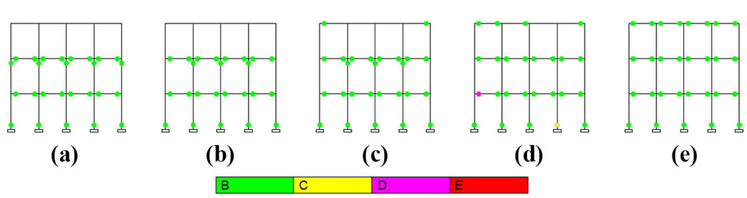
Notes: (a) $\beta = 1.2$; (b) $\beta = 1.4$; (c) $\beta = 1.6$; (d) $\beta = 1.8$; (e) $\beta = 2.0$; (f) $\beta = 2.4$; (g) $\beta = 2.6$; (h) $\beta = 2.8$

Figure 8 Mechanisms of structural failure (Model 02)



Notes: (a) $\beta = 1.2$; (b) $\beta = 1.4$; (c) $\beta = 1.6$; (d) $\beta = 1.8$; (e) $\beta = 2.0$; (f) $\beta = 2.4$; (g) $\beta = 2.6$; (h) $\beta = 2.8$

Figure 9 Mechanisms of structural failure (Model 03)



Notes: (a) $\beta = 1.2$; (b) $\beta = 1.6$; (c) $\beta = 2.0$; (d) $\beta = 2.4$; (e) $\beta = 2.6$

Table 4 Number of plastic hinges forming at the end of the analysis

		A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	Total
Structure (01)	$\beta = 1.2$	469	186	5	0	0	530	96	25	1,311
	$\beta = 1.4$	454	199	7	0	0	541	87	8	1,296
	$\beta = 1.6$	430	227	1	2	0	550	80	12	1,302
	$\beta = 1.8$	430	225	1	4	0	549	80	18	1,307
	$\beta = 2.0$	438	219	1	2	0	510	120	11	1,301
	$\beta = 2.4$	466	193	1	0	0	510	120	29	1,319
	$\beta = 2.8$	430	225	1	4	0	510	116	32	1,318
Structure (02)	$\beta = 1.2$	336	188	3	0	3	411	80	25	1,046
	$\beta = 1.4$	345	180	2	0	3	411	80	25	1,046
	$\beta = 1.6$	365	158	2	0	5	420	80	16	1,046
	$\beta = 1.8$	365	164	1	0	0	420	80	14	1,044
	$\beta = 2.0$	380	132	18	0	0	420	80	28	1,058
	$\beta = 2.4$	370	152	7	0	1	420	80	14	1,044
	$\beta = 2.6$	365	157	1	7	0	415	77	38	1,060
	$\beta = 2.8$	340	183	1	6	0	380	133	6	1,049
Structure (03)	$\beta = 1.2$	265	129	3	3	0	313	55	23	791
	$\beta = 1.4$	265	129	6	0	0	330	40	23	793
	$\beta = 1.6$	274	122	1	0	3	328	40	9	777
	$\beta = 2.0$	265	129	3	0	3	290	80	17	787
	$\beta = 2.4$	270	124	5	1	0	290	79	21	790
	$\beta = 2.6$	250	138	1	11	0	290	98	12	800

Figure 10 Capacity curves of the structure (01) with the different coefficients β

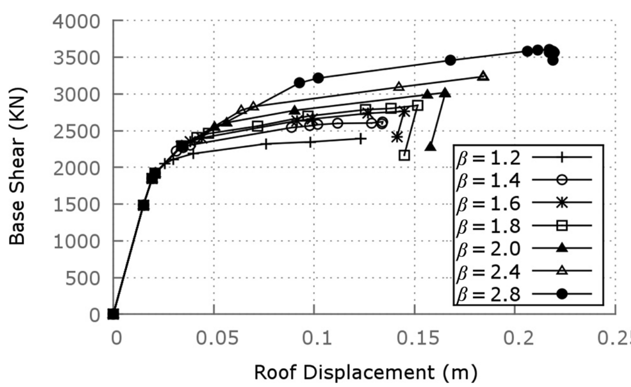


Figure 11 Capacity curves of the structure (02) with the different coefficients β

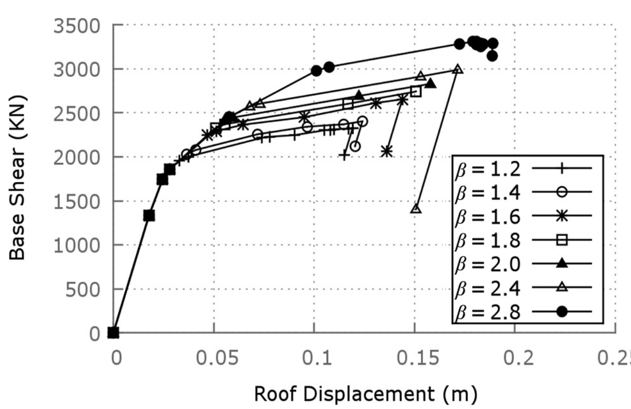
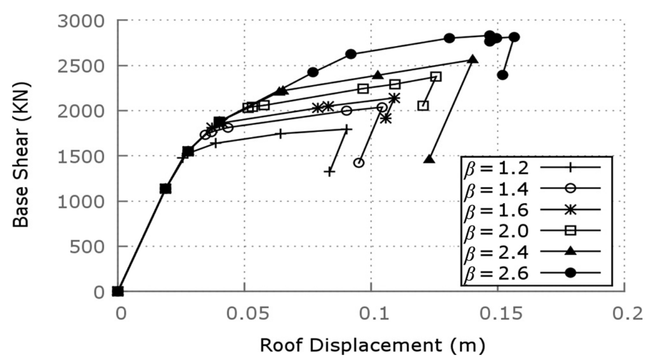


Figure 12 Capacity curves of the structure (03) with the different coefficients β

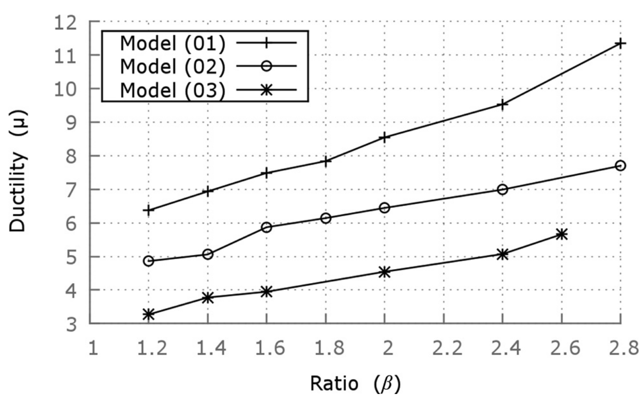


There is an increase in the bearing capacity of the structure as the ratio of the resistant moments of the columns on the beams is increased, an improvement in the ductility is also observed. All the curves display a similar elastic behavior, which shows that the variation in the β ratio has no significant impact on the elastic behavior of the structure. At the elastic limit, the value of the shear force is 1,848.474 KN, and the value of the displacement is 19.335 mm for the structure (01) on all the coefficients. While for structures (02) and (03), they indicate in particular a constant elastic stiffness with a value of 66,113.06 KN.m and 75,104.50 KN.m, respectively. Beyond the elastic limit, the structures exhibit different behavior; there is notably an improvement in the bearing capacity of the structures as the coefficient β increases, as well as an increasing ductility. To have a better appreciation of the results of the non-linear analysis carried out, Table 5 gives for each of the coefficients β the displacements and maximum shear forces obtained.

Table 5 Maximum values of the different capacity curves obtained

Structure (01)							
Values of β	$\beta = 1.2$	$\beta = 1.4$	$\beta = 1.6$	$\beta = 1.8$	$\beta = 2.0$	$\beta = 2.4$	$\beta = 2.8$
Shear (KN)	2,391.96	2,622.36	2,763.6	2,844.4	3,018.1	3,237.7	3,563.2
Displacement (m)	0.123224	0.1341	0.14477	0.15154	0.16525	0.1843	0.21945
Structure (02)							
Values of β	$\beta = 1.2$	$\beta = 1.4$	$\beta = 1.6$	$\beta = 1.8$	$\beta = 2.0$	$\beta = 2.4$	$\beta = 2.8$
Shear (KN)	2,327.44	2,403.96	2,654.4	2,741.86	2,831.12	2,989.9	3,288.3
Displacement (m)	0.119137	0.12404	0.1439	0.15056	0.1580	0.17141	0.1888
Structure (03)							
Values of β	$\beta = 1.2$	$\beta = 1.4$	$\beta = 1.6$	$\beta = 2.0$	$\beta = 2.4$	$\beta = 2.6$	
Shear (KN)	1,795.96	2,038.7	2,141.37	2,375.17	2,562.5	2,830.29	
Displacement (m)	0.090424	0.1044	0.109183	0.12566	0.14021	0.156421	

Figure 13 Ductility curves of the three structures with variation of the coefficient



We note from Table 5 that for the structure (01), the maximum shear force increases by 9.63, 5.38, 2.92, 6.1, 7.27, 10.05%, going gradually from β = 1.2 to β = 2.8. And, the maximum displacement also increases by 8.82%; 7.96, 4.67, 9.05, 19.07%. As for the structure (02), the results show an increase of 3.29, 10.41, 3.29, 3.25, 5.6, 9.98% of the maximum shear force at the base with the increase in the coefficient β, this translates into an increase in the bearing capacity. And, an increase in maximum displacement of 4.11, 16.01, 4.63, 4.75, 8.49, 10.14%, this indicates an increase in ductility. Finally, for structure (03), we note that the increase in maximum shear force increases by 13.5, 5.03, 10.91, 7.88, 10.45%, respectively, passing from β = 1.2 to 1.4, 1.6, 2.0, 2.4, 2.6. The increase in maximum displacement is around 15.45, 4.5, 15.09, 11.58, 11.56%. It can be seen that the increase in the coefficient β from 1.2 to 1.4 constitutes the most significant increase in terms of maximum shear force and maximum displacement.

Figure 13 shows a comparison of the different overall ductility curves of the three reinforced concrete frames studied, by the different non-linear static analyses (push-over) carried out on the structures with the different coefficients β.

The dissipation of energy increases as the coefficient β of the frame increases. The different structures studied, having a capacity ratio of 1.25 according to RPA (2003), achieve the lowest energy dissipation as shown in Figure 12, although plastic hinges developed in the frame but remains an intermediate and not a structural mechanism. We can see that the increase in the coefficient β makes very large plastic rotations, so more energy is dissipated by the beams by undergoing a large inelastic rotation.

The authors (Haselton *et al.*, 2011; Kuntz and Browning, 2003; Cagurangan, 2015) who studied the influence of the β ratio agree that the β ratio, necessary to achieve a global mechanism, increases as the number of floors in the structure increases, this study agrees with the aforementioned authors; in our study also we find that the structure with five and four levels required a coefficient β higher than the structure with three levels.

The results of this study are close to those found by Murty *et al.* (2012); however, a divergence exists regarding the capacitance coefficient required to achieve the global mechanism. Indeed, in this study the ratio is 2.8 compared to 3.6 found by Murty. The two structures are similar (five-level structures), the difference can be due to the different dimensions of the beams and columns, a study concerning the parameters influencing the capacity ratio to reach a global mechanism can be considered.

6.3 Influence of the capacity ratio on the probability of ruin of these structures

For the three studied structures, one supposes initially all the loadings, the deterministic geometric and mechanical properties, while the resistances (output variables) being normal random variables. The problem of reliability of structures will be treated using the method adapted by the principle of Hasofer–Lind (Benyahi *et al.*, 2018) for the determination of the probability of failure, and which is based on a quadratic method of approximation of the ruin surface of the structural system. We know a priori all the possible failure mechanisms of these structures for the different values of the ratio β.

The failure equations obtained from a state of damage by plasticization of the extreme sections of the elements of the structure can be used in this reliability calculation, and the function of ultimate limit state associated in the form of a request of maximum ductility in rotation G (V, U) is an implicit non-linear function (known numerically from the push-over analysis), whose system failure is observed when G (V, U) $\geq$ 0. Given the complexity of the finite element model, it is difficult to carry out the study by a direct coupling between the non-linear analysis program and the reliability program, so it becomes necessary to build a response surface.

In the design of these structures, the beam elements may be susceptible to damage; however, the failure of any section of load-bearing elements is assumed to be unacceptable. To assess the probability of a global structural system failing, criteria should be established to define the desired levels of safety. A criterion should be adopted for classifying the consequences of failure according to the size of the structures, thus making it possible to compare their reliability levels. Using the concept of reliability index, reliability classes (RC) have been defined according to EC90 – Eurocode (2002) to ensure minimum reliability.

For our study, we consider a reliability class (RC2), having an average consequence in terms of loss of human life, considerable economic, social or environmental consequences. The minimum value recommended by EC90 – Eurocode (2002) for the reliability index associated with the reliability class (RC2) is generally considered to lead to a structure with a value greater than 3.8; 3.72 for a reference period of 50; 100 years respectively. The probability of reaching a limit state of failure during a given period depends on the reliability index. For this, we choose a classification criterion whose value corresponds to safety levels for structural elements of reliability class RC2 (EC90. Eurocode, 2002), and so that the probability of failure is very low, i.e. a value conventional: ($Pf \leq 10^{-4}$). The values of the reliability and probability of failure indices, with the values of the ratio β, found by the reliability analysis of the three structures, are summarized in Table 6.

From Table 6, an estimate of the probability of ruin over the interval of the ratio β [1.2; 2.8], and where we can notice that model (03) gives greater reliability indices than the other models. This is due to better behavior of the less slender structures. We pass from a reliability index of structural systems (01); (02) and (03), from 1.36 to 3.68, from 1.42 to 3.78 and from 1.56 to 3.86, respectively. In other words, an increase in security of about more than twice. In Figure 14, the evolution of the ratio β as a function of the probability of failure of the structures is given.

The failure probabilities of the three structures are correctly approximated over a relatively wide range (from 10^{-4} to 10^{-1}). There is a relation between the probability of ruin with the variation of the coefficient β, as well as the height of the structure. It is noted that the increase in the coefficient β, induces a reduction in the probability of ruin of the structure. A decrease in the probability of ruin can be seen with the decrease in the height of the structure.

The probability of failure for the ratio $\beta = 2.8$ is very much lower than the required criterion ($P_f \leq 10^{-4}$), which shows that the design is safe. On the other hand, the probability of ruin for the ratio $\beta = 1.2$, close to what is recommended by regulation [Règlement Parasismique Algérien (RPA), 2003], is very far from the requested criterion, which is not safe. To do this, increasing the β ratio improves the probability of failure in a more rational way.

Based on the results obtained from the reliability study conducted, this study recommends a coefficient $\beta \geq 2.0$, from which the probability of failure of the three structures is less than 10^{-3} (EC90. Eurocode, 2002). The use of a coefficient allowing the structure to reach a global plastic mechanism can be quite expensive for construction companies, which is why resorting to a reliability study can be a good alternative.

Through the reliability study carried out, a capacity ratio greater than or equal to 2.0 is recommended; these results are in agreement with the recommendations made by Dooley and Bracci (2001) and close to those made by Kuntz and Browning (2003); the latter recommend a coefficient slightly less than 2.

It was shown that the increase in the capacity ratio decreased the probability of ruin of structures; these results are in agreement with those found by the authors (Dooley and Bracci, 2001; Haselton *et al.*, 2011; Cagurangan, 2015).

7. Conclusion

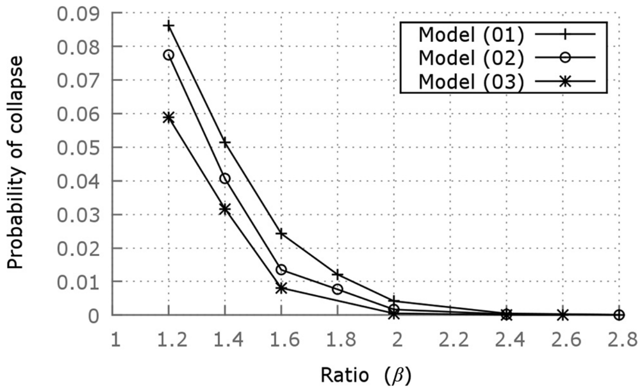
The results of this study are close to the results found by the studies of the authors cited above, in particular Murty *et al.* (2012) who recommend a coefficient β greater than 3 to avoid the formation of plastic hinges in the columns and the development of a favorable failure mechanism. This study was able to demonstrate that by applying the prescriptions of RPA (2003), the structures required a ratio of resistant moments of the columns to those of the beams, close to three.

The coefficient of the resistant moments of the columns on those of the beams has no influence on the elastic behavior of the structure; it only influences the post-elastic behavior of the structure. The coefficient recommended by RPA (2003) is not

Table 6 Values of the various reliability and probability of failure indices obtained

Structure (01)							
Values of β	$\beta = 1.2$	$\beta = 1.4$	$\beta = 1.6$	$\beta = 1.8$	$\beta = 2.0$	$\beta = 2.4$	$\beta = 2.8$
Reliability index	1.3649	1.6319	1.9728	2.2543	2.6387	3.2971	3.6866
Probability of failure	0.08615	0.05135	0.02426	0.01208	0.004161	0.000488	0.000113
Structure (02)							
Values of β	$\beta = 1.2$	$\beta = 1.4$	$\beta = 1.6$	$\beta = 1.8$	$\beta = 2.0$	$\beta = 2.4$	$\beta = 2.8$
Reliability index	1.4226	1.7436	2.2122	2.4247	2.9350	3.4620	3.7849
Probability of failure	0.0774	0.04061	0.01347	0.00766	0.0016	0.000268	0.00007684
Structure (03)							
Values of β	$\beta = 1.2$	$\beta = 1.4$	$\beta = 1.6$		$\beta = 2.0$	$\beta = 2.4$	$\beta = 2.6$
Reliability index	1.5646	1.8571	2.4049		3.2841	3.7112	3.8629
Probability of failure	0.05884	0.03164	0.008089		0.0005115	0.0001031	0.00005598

Figure 14 Probability curves of failure of the three structures with variation of the β ratio



sufficient to allow the structure to develop a structural mechanism; to allow optimal energy dissipation capacity for the structure. The increase in the coefficient β allows the increase in the number of stories in the failure mechanism, which is induced by the reduction in the formation of plastic hinges in the columns.

Among the perspectives that can be drawn from this study: study of the various parameters modifying the coefficient β necessary to achieve a global mechanism such as the example given by Murty *et al.* (2012) and this study.

In this study, we settled on an analysis in which the structure is deterministic. It is, however, entirely possible to take into account the uncertainty that may relate to the mechanical characteristics of the materials. The reliability model also allowed us to establish the criteria to define the desired levels of security.

References

ACI 318-02. American Concrete Institute (2002), “Building code requirements for structural concrete and commentary”, Farmington Hills, MI.

ACI 318-11. ACI Committee, American Concrete Institute, and International Organization for Standardization (2011), “Building code requirements for structural concrete and commentary”, American Concrete Institute.

ACI 318-71. American Concrete Institute Committee 318 (1971), “Building code requirements for reinforced concrete”, Detroit.

Altarejos-García, L., Escuder-Bueno, I., Serrano-Lombillo, A. and de Membrillera-Ortuño, M.G. (2012), “Methodology for estimating the probability of failure by sliding in concrete gravity dams in the context of risk analysis”, *Structural Safety*, Vols 36/37, pp. 1-13.

ASCE7-02. American Society of Civil Engineers (2002), “Minimum design loads for buildings and other structures”, Reston, VA.

ASCE7-05. American Society of Civil Engineers (2005), “Minimum design loads for buildings and other structures”, Reston, VA.

ATC-40. Applied Technology Council (1996), “Applied technology council, seismic evaluation end retrofit of concrete building”, Report No. SSC 96-01, Redwood City, CA, November 1996.

BAEL 91. Béton Armé aux Etats Limites (1992), “Règles techniques de conception et de calcul des ouvrages et constructions en béton armé suivant la méthode des Etats limites”, DTUP 18-702, Fascicule 62.

Balomenos, G.P., Genikomsou, A.S. and Polak, M.A. (2018), “Investigation of the effect of openings of interior reinforced concrete flat slabs”, *Structural Concrete*, Vol. 19 No. 6, pp. 1672-1681.

Benyahi, K., Bouafia, Y., Barboura, S. and Kachi, M.S. (2018), “Nonlinear analysis and reliability of metallic truss structures”, *Frontiers of Structural and Civil Engineering*, Vol. 12 No. 4, pp. 577-593.

Bouzid, L., Hamizi, M., Hannachi, N.E., Nekmouche, A. and Akkouche, K. (2020), “Plastic hinges mechano-reliability analysis in the beams of RC frames structures”, *World Journal of Engineering*, Vol. 17 No. 5, pp. 719-732.

Brunesi, E. and Parisi, F. (2017), “Progressive collapse fragility models of European reinforced concrete framed buildings based on pushdown analysis”, *Engineering Structures*, Vol. 152, pp. 579-596.

Cagurangan, C.K. (2015), *Effects of Strong-Column Weak-Beam Ratios on Collapse Capacities of Tall Reinforced Concrete Moment Frame Structures*, University of CA, Berkeley.

Cai, G.Q. and Elishakoff, I. (1994), “Refined second-order reliability analysis”, *Structural Safety*, Vol. 14 No. 4, pp. 267-276.

Calgaro, J.A. (1991), “Introduction à la réglementation technique”, *In Annales Des Ponts Et Chaussées (No. 60)*, University Of Illinois, Illinois.

Carvajal, C., Peyras, L., Felix, H., Royet, P., Bécue, J.P. and Philip, G. (2011), “Apport des méthodes de la sûreté de fonctionnement et de la fiabilité dans le cadre des études de dangers: exemple d’application sur le barrage du xoldocogaina”, *Colloque du Comité Français des Barrages et Réservoirs*, p. 11.

Cornell, C.A. (1968), “Engineering seismic risk analysis”, *Bulletin of the Seismological Society of America*, Vol. 58 No. 5, pp. 1583-1606.

CSI. Computers and Structures (2017), “CSI analysis reference manual for SAP2000, ETABS, and SAFE”, Computers and Structures (CSi): California.

D.T.R.-B.C. 2.2. Document technique réglementaire (1988), “Charges permanentes et charges d’exploitation”, Centre national de recherche appliquée en génie parasismique C.G. S, Alger.

Der Kiureghian, A., Lin, H.Z. and Hwang, S.J. (1987), “Second-order reliability approximations”, *Journal of Engineering Mechanics*, Vol. 113 No. 8, pp. 1208-1225.

Dooley, K.L. and Bracci, J.M. (2001), “Seismic evaluation of column-to-beam strength ratios in reinforced concrete frames”, *Structural Journal*, Vol. 98 No. 6, pp. 843-851.

Du, X. (2008), “Unified uncertainty analysis by the first order reliability method”, *Journal of Mechanical Design*, Vol. 130 No. 9.

EC90. Eurocode (2002), “Basis of structural design”, European standard, The European union Per Regulation.

EC98. Eurocode (2004), “Design for earthquake resistance, part 1: general rules, seismic actions and rules for buildings”, European standard EN 1998-1, European Committee for Standardization(CEN), Brussels.

FEMA-356. Federal Emergency Management Agency (2000), "Prestandard and commentary for seismic rehabilitation of building", Federal Emergency Management Agency, Washington, DC.

Fiessler, B., Rackwitz, R. and Neumann, H.J. (1979), "Quadratic limit states in structural reliability", *Journal of the Engineering Mechanics Division*, Vol. 105 No. 4, pp. 661-676.

Gökdemir, H. and Günaydin, A. (2018), "Investigation of strong column–weak beam ratio in multi-storey structures", *Anadolu Üniversitesi Bilim Ve Teknoloji Dergisi A-Uygulamalı Bilimler ve Mühendislik*, Vol. 19 No. 2, pp. 242-252.

Hammoudi, A., Moussaceb, K., Belebchouche, C. and Dahmoune, F. (2019), "Comparison of artificial neural network (ANN) and response surface methodology (RSM) prediction in compressive strength of recycled concrete aggregates", *Construction and Building Materials*, Vol. 209, pp. 425-436.

Haselton, C.B., Liel, A.B., Deierlein, G.G., Dean, B.S. and Chou, J.H. (2011), "Seismic collapse safety of reinforced concrete buildings, I: assessment of ductile moment frames", *Journal of Structural Engineering*, Vol. 137 No. 4, pp. 481-491.

Hohenbichler, M., Gollwitzer, S., Kruse, W. and Rackwitz, R. (1987), "New light on first-and second-order reliability methods", *Structural Safety*, Vol. 4 No. 4, pp. 267-284.

Huang, X., Li, Y., Zhang, Y. and Zhang, X. (2018), "A new direct second-order reliability analysis method", *Applied Mathematical Modelling*, Vol. 55, pp. 68-80.

IS 13920 (1993), "Ductile detailing of reinforced concrete structures subjected to seismic forces-code of practice", Bureau of Indian Standards, New Delhi.

IS 13920 (2016), "Ductile design and detailing of reinforced concrete structures subjected to seismic forces", First revision, Bureau of Indian Standards, New Delhi.

IS 1893 (2002), "Indian standard criteria for earthquake resistant design of structures", *Part 1: General Provisions and Buildings (Fifth Revision)*, Bureau of Indian Standards, New Delhi.

IS 1893. (Part 1) (2007), "Indian standard criteria for earthquake resistant design of structures", Bureau of Indian Standards, New Delhi.

IS 456 (2000), "Indian standard-plain and reinforced concrete, code of practice", Bureau of Indian Standards, New Delhi.

Kadukar, M.V., Kulkarni, R.B. and Deshpande, R.D. (2018), "Study on literature review of strong column weak beam behavior of frames", *International Journal for Research in Applied Science and Engineering Technology*, Vol. 6 No. 5, pp. 2915-22.

Karanjit, S. (2017), "Performance of RC frame structure with various beam to column flexural capacity ratio–a case study", *Journal of Science and Engineering*, Vol. 4, pp. 17-23.

Kuntz, G.L. and Browning, J. (2003), "Reduction of column yielding during earthquakes for reinforced concrete frames", American Concrete Institute.

Kurama, Y.C., Sritharan, S., Fleischman, R.B., Restrepo, J.I., Henry, R.S., Cleland, N.M., Ghosh, S.K. and Bonelli, P. (2018), "Seismic-resistant precast concrete structures: state of the art", *Journal of Structural Engineering*, Vol. 144 No. 4, p. 3118001.

Maier, H.R., Lence, B.J., Tolson, B.A. and Foschi, R.O. (2001), "First-order reliability method for estimating reliability, vulnerability, and resilience", *Water Resources Research*, Vol. 37 No. 3, pp. 779-790.

Mayer, M. (1926), "Die Sicherheit Der Bauwerke", Springer-Verlag.

Metropolis, N. and Ulam, S. (1949), "The Monte Carlo method", *Journal of the American Statistical Association*, Vol. 44 No. 247, pp. 335-341.

Murty, C.V.R., Goswami, R., Vijayanarayanan, A.R. and Mehta, V. (2012), *Earthquake Behavior of Buildings*, Gujarat State Disaster Management Authority, Gandhinagar, pp. 53-79.

Ni, P., Li, J., Hao, H. et al. (2020), "Reliability analysis and design optimization of nonlinear structures", *Reliability Engineering and System Safety*, Vol. 198, pp. 106-860.

Ning, N., Qu, W. and Ma, Z.J. (2016), "Design recommendations for achieving 'strong column-weak beam' in RC frames", *Engineering Structures*, Vol. 126, pp. 343-352.

Rackwitz, R. (1976), *Practical Probabilistic Approach to Design. First-Order Reliability Concepts for Design Codes*, Bulletin d'information du CEB, 112.

Ravindra, M.K., Lind, N.C. and Siu, W. (1974), "Illustrations of reliability-based design", *Journal of the Structural Division*, Vol. 100 No. 9, pp. 1789-1811.

Règlement Parasismique Algérien (RPA) (2003), "Algerian seismic code", National Center for Applied Research in Earthquake Engineering, RPA99, Algiers, Algeria.

Règlement Parasismique Marocain (RPS) (2002), *Le Règlement De Construction Parasismique*, Ministère de l'ATUHE, Secrétariat d'État à l'Habitat, Rabat.

Rzhanitzyn, R.A. (1949), *Design of Structures with Considerations of Plastic Properties of Materials*, Stroivoenmorizdat, Moscow.

Santana, J.A.D., Orozco, J.L., Furka, D., Furka, S., Matos, Y. C.B., Lantigua, D.F., Miranda, A.G. and González, M.C. (2021), "A new fuzzy-bayesian approach for the determination of failure probability due to thermal radiation in domino effect accidents", *Engineering Failure Analysis*, Vol. 120, p. 105106.

Saravanan, M., Goswami, R. and Palani, G.S. (2018), "Replaceable fuses in earthquake resistant steel structures: a review", *International Journal of Steel Structures*, Vol. 18 No. 3, pp. 868-879.

SNI 2847 (2013), *Persyaratan Beton Struktural Untuk Bangunan Gedung*, Badan Standardisasi Nasional, Jakarta.

Soares, R.C., Mohamed, A., Venturini, W.S. and Lemaire, M. (2002), "Reliability analysis of non-linear reinforced concrete frames using the response surface method", *Reliability Engineering & System Safety*, Vol. 75 No. 1, pp. 1-16.

Sofi, A., Muscolino, G. and Giunta, F. (2020), "Propagation of uncertain structural properties described by imprecise probability density functions via response surface method", *Probabilistic Engineering Mechanics*, Vol. 60, p. 103020.

Streletskii, N. S. (1947), "Statistical basis of the safety factor of structures", In Russian.

Sudarsana, I.K., Budiwati, I.A.M. and Aditya, P.W. (2014), "Effect of column to beam strength ratio on performance of reinforced concrete frames", *Proceedings of the International Conference on Engineering Technology and Industrial Application, The 1st ICETIA 2014.*

Surana, M., Singh, Y. and Lang, D.H. (2018), "Effect of strong-column weak-beam design provision on the seismic fragility of RC frame buildings", *International Journal of Advanced Structural Engineering*, Vol. 10 No. 2, pp. 131-141.
Tarawneh, A. and Majdalaweyh, S. (2020), "Design and reliability analysis of FRP-reinforced concrete columns", *Structures*, Vol. 28, pp. 1580-1588.
TEC 2007. Turkish Earthquake Design Code (2009), "Specifications for design of buildings in seismic regions", Ministry of Public Works.
Vijayanarayanan, A.R., Goswami, R. and Murty, C.V.R. (2017), "Simple linear elastic static analysis procedure to attain desired collapse mechanism for moment resisting frames", *In 16th World Conference on Earthquake, Santiago Chile*.
Zhang, Q. and Alam, M.S. (2020), "State-of-the-art review of seismic-resistant precast bridge columns", *Journal of Bridge Engineering*, Vol. 25 No. 10, p. 3120001.
Zhang, D., Han, X., Jiang, C., Liu, J. and Li, Q. (2017), "Time-dependent reliability analysis through response surface method", *Journal of Mechanical Design*, Vol. 139 No. 4.
Zhao, Y., Ono, T. and Et Kato, M. (2002), "Second-order third-moment reliability method", *Journal of Structural Engineering*, Vol. 128 No. 8, pp. 1087-1090.
Zhao, W., Fan, F. and Wang, W. (2017), "Non-linear partial least squares response surface method for structural reliability analysis", *Reliability Engineering & System Safety*, Vol. 161, pp. 69-77.

Further reading

Lemaire, M., Chateauneuf, A. and Mitteau, J.C. (2009), "Structural reliability", Editorial, ISTE Ltd and John Wiley and Sons.
Lu, Z.H., Hu, D.Z. and Zhao, Y.G. (2017), "Second-order fourth-moment method for structural reliability", *Journal of Engineering Mechanics*, Vol. 143 No. 4, p. 6016010.
NZS 3101. New Zealand Standard (2006), "Concrete structures standard", issued by standards council, New Zealand.

Corresponding author

Karim Benyahi can be contacted at: karim.benyahi@ummto.dz

For instructions on how to order reprints of this article, please visit our website:
www.emeraldgrouppublishing.com/licensing/reprints.htm
Or contact us for further details: **permissions@emeraldinsight.com**