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# An alternative approach to improve the capacity design concept for moment resisting reinforced concrete (RC) frame systems

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## Abstract

Capacity design concept for reinforced concrete frame systems is based on a hierarchy of the strength and stiffness properties of the structural elements so that the seismic energy dissipation mechanism occurs in a certain way. This theoretical concept of seismic energy dissipation by allowing the occurrence of plastic hinges at the end of the beams and the columns located at the ground floor was not observed / identified in post-earthquake inspection of damaged RC frame structures. On the other hand, various failure mechanisms were observed that are not compliant with theoretical considerations specified in the seismic design codes. The damages produced during the latest seismic events, (2020 Zagreb earthquake, 2020 Aegean Sea earthquake, 2020 Caribbean earthquake, 2020 Puerto Rico earthquake, 2020 Mexico earthquake, etc.) raised some concerns related to the theoretical ductile failure mechanism (Strong Column – Weak Beams, SCWB) versus the practical approach. Consequently, a possible improvement of the capacity design concept through the consideration of different values for the behaviour factor "q" applied to structural elements (beams and columns) was investigated and presented in this paper. The goal was to reach the expected theoretical structural degradation mechanism. Thus, by considering different values for the behaviour factor "q", at the design stage, for beams and columns, it was possible to reach a favourable value for the ratio Kc/Kb between the bending stiffness of columns (Kc) and beams (Kb). Consequently, a good correlation between the real seismic response and the theoretical mechanism of structural deformation was obtained.

***Key words:*** Behaviour factor, soft storey, bending stiffness, over-strength, ductile failure mechanism

## 1 Introduction

The earthquakes that took place all over the world during the last decades resulted in many casualties, material and financial losses. The year 2020 went down in history with a series of earthquakes that caused the collapse of countless old buildings and the severe damage of various structural systems, including multi-storey reinforced concrete (RC) frame structures. In all these cases, the earthquakes action develops unfavourable seismic energy dissipation mechanisms in the structural (lateral) system of buildings. In these conditions, arise many uncertainties regarding the seismic design effectiveness of seismic-resistant structures. Thus, in some recent research papers [1, 2] a theoretical method to improve the design capacity concept (Fig.1) for the RC frame systems located in seismic areas is presented in terms of analytical results.

## 2 Bending stiffness ratio effects for Strong Columns – Weak Beams (SCWB)

The design capacity concept has suffered many improvements due to the observations of earthquake effects produced in the last 50 years. Currently, there are two approaches to the concept of **S**trong **C**olumns – **W**eak **B**eams (SCWB) seismic energy dissipation mechanism (Fig.1) for RC moment resisting frame systems that are used in practice [3, 4]: 1) the beams have a superior bending stiffness compared to that of the columns [5] and 2) it is proposed to use a higher base shear force than the value resulting from the calculation [6], in order to minimize the consequences of design deficiencies [7].
Haselton et al. [1] performed an extensive study on the seismic response of the RC frame structures with GF to GF+19F height regime using nonlinear dynamic analysis. The study revealed that the RC columns were the main source of seismic energy dissipation by large deformations in nonlinear inelastic domain for each of the considered RC frame structures. The same effects regarding inelastic RC columns strains were observed in more recent research works[5, 8, 9] for a series of GF+1F moment resisting RC frame models without considering the influence of the infill walls on the seismic response [10]. Moreover, the initial study illustrates the influence of the bending stiffness ratio between beams and columns on the seismic response of RC frame structures and the risk of collapse. Thus, for a set of RC moment resisting frame structures designed according to the ductile concept with GF+3F and GF+11F height regime, the bending stiffness ratio had a special influence for the assessment of the seismic energy dissipation mode, as seen in Fig.2. In this situation, for $K_c/K_b$ = 0.4 - 0.8 and the GF+3F set of RC moment resisting frame systems, the risk of collapse was major due to the formation of a weak ground floor mechanism.
As the ratio between $K_c/K_b$ increased to 1.0-1.2, the risk of collapse of the set of RC frame structures decreased. Thus, the seismic energy dissipation mechanism included the formation of plastic hinges not only on the ground-floor columns but also at the end

of beams (Fig.2). The most adequate seismic behaviour for this type of RC frame structures was recorded for $K_c/K_b$ = 2.0-3.0. The number of dissipative elements (the number of RC beams with nonlinear inelastic deformations) increased and the distribution of the plastic hinges matched the theoretical one generally assumed for the ductile design concept. The risk of collapse decreased [1] and the RC columns recorded some local deformations without significant incursions in the nonlinear inelastic domain.

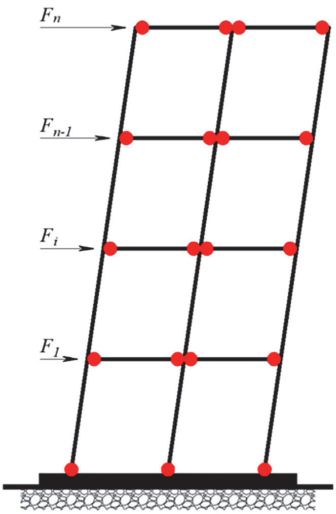


**Figure 1. The idealized global seismic response for pure RC moment resisting frame structure [11, 12]**

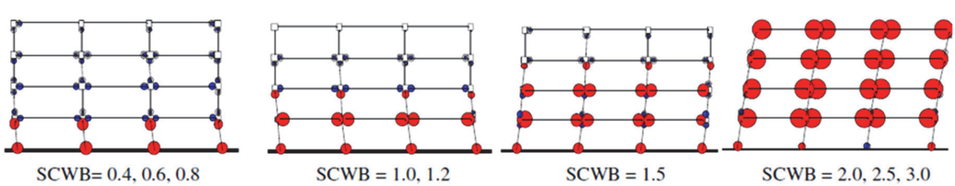


**Figure 2. Dominant collapse mechanisms for GF+3F RC frame structures set with different values of $K_c/K_b$ [1]**

The Romanian seismic design norm P100-1/2013 [3] uses a $K_c/K_b$ = 1 ratio (equal bending stiffness for both RC columns and RC beams) for a single value of the seismic load reduction factor "q". This design principle could present a real problem as it can lead to an increase in the risk of collapse risk of the RC moment resisting frame structures [1]. Moreover, the seismic energy dissipation mechanism does not correspond, conceptually, to the ductile concept [13]. Therefore, a different approach may be needed to improve the capacity design, using much higher $K_c/K_b$ values (2.0-3.0).

# 3 Capacity design concept of the RC moment resisting frame systems

## 3.1 Main aspects regarding the practical problems of the capacity design concept

As mentioned in previous research works [14, 15] it was deemed necessary to eliminate the P-$\delta$ effect by introducing linear elastic elements in the RC frame structure to avoid the global collapse. This RC moment resisting frame structure with mixed stiffness considered in the current seismic design norms [3, 16] can be represented as shown in Fig. 1. The SCWB seismic response mechanism [11, 12] theoretically represents the RC frame structure with mixed stiffness described in [14]. The RC columns form a common body with RC beam-column joints and become vertical linear elastic elements. Moreover, the RC beams become the primary role of the seismic energy dissipation through local deformations, e.g. plastic hinges. In real life practice, there is a large slab influence on the overall stiffness of the RC beams and the RC beam-column joints rigidity [5, 8, 9, 17-22]. These structural aspects can be observed by means of the nonlinear inelastic seismic response of the RC frame structures designed only for gravitational loads [23-25]. Similar observations could be made based on the analytical approach for assessing the seismic response of the RC frame systems designed according to current seismic norms, especially when applying the method according to the Japanese seismic code [26]. Finally, it can be concluded that the design of these RC structural systems requires special attention when designing the RC columns [27].

## 3.2 Theoretical and analytical aspects regarding the improvement of the ductile design concept

The theoretical aspect outlined in the study of Haselton et al. [1] for the implications of $K_c/K_b$ = 2.0-3.0 ratio could become the solution for a favourable seismic response without significant nonlinear post-elastic behaviour of the RC columns. Thus, the linear elastic structural elements (RC columns, RC beam-column joints) maintain their integrity through oversizing and the ductile structural elements dissipate the seismic energy by means of plastic hinges. This aspect was emphasized in a recent research work [2] on assessing the seismic behaviour of a GF+3F RC moment resisting frame. The 2D model considered a single value for the behaviour factor, in accordance with the actual seismic regulations [3, 26]. The investigated RC frame structure corresponds to the class of RC moment resisting frame systems studied by Haselton et al. [1]. In these conditions, the capacity curve presents the ductility of RC beams, the main dissipative elements, and the reaching of the entire overstrength capacity, as seen in Fig. 3. Consequently, the ductility source can be directly expressed through the reduction factor "q" of seismic forces (Eq. (1), Eq. (2)).

$$q = q_{\mu} \cdot q_s = q_{\mu} \cdot q_{Sd} q_{R} \tag{1}$$

$$q_s = q_{Sd} q_{R} \tag{2}$$

where [3]: "q" – total reduction factor for the seismic forces; "$q_{\mu}$" - reduction factor for the seismic forces depending on the ductility of the structure; "$q_S$" – overstrength factor for the structure, depending on: "$q_{Sd}$" – design overstrength and "$q_R$" – redundancy or plastic redistribution capacity of depending on the load carrying capacity.

According to the design concept and the "F-d" curve shown in Fig. 3, it is not possible to include the influence of the seismic energy dissipation capacity to the RC columns because these RC structural elements present linear elastic response. Under these circumstances, the RC columns can be designed for $F_y$ force corresponding to the global nonlinear inelastic response of the RC frame system. There is, however, a slight chance that the RC frame structure will not form the maximum number of plastic hinges at the end regions of the RC beams.

Consequently, the design of the superstructure depends on two calculation steps with two different "q" values: step 1) using the seismic load reduction value proposed by P100-1 [3] that is the $F_d$ value of the seismic force and step 2) using the seismic load reduction value corresponding to the yielding of the structural RC frame system. In the latter case, the RC columns should be designed for a minimum value of $F_y$ of the seismic force.

This idea is also found in the P100-1 norm [3] for unbraced steel frame structures yet in another form. The value of over-strength factor $\Omega_T$ initially assigned to design values of internal forces and moment in the seismic design stage of the steel columns (Eq. (3), Eq. (4) and Eq. (5)), is used as a value that affects the contribution of the seismic action from the very stage of generating the load combination that includes the seismic force. Under these conditions, a linear dependency was obtained between the equivalent lateral static loads and the design values of the internal forces and moments (Note 2, point 6.6.3. (2), P100-1 norm [3]). However, this linear relationship could not be validated either in the design practice or by means of analytical procedures. It still reflects the need to simplify the design process of structural systems by directly obtaining the design values for the internal forces and moments from a simple algebraic equation.

$$N_{Ed} = N_{Ed,G} + \Omega_T \cdot N_{Ed,E} \tag{3}$$

$$M_{Ed} = M_{Ed,G} + \Omega_T \cdot M_{Ed,E} \tag{4}$$

$$V_{Ed} = V_{Ed,G} + \Omega_T \cdot V_{Ed,E} \tag{5}$$

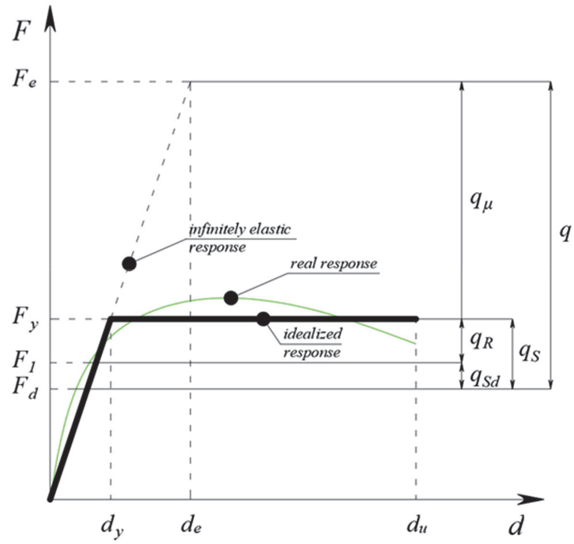


**Figure 3.** **"F-d" capacity curve and notation of seismic reduction factors (where: F – lateral force; $F_d$ - design base shear force; $F_1$ - – base shear force corresponding to the formation of the first plastic hinge in the structural system; $F_y$ - force corresponding to the global nonlinear inelastic response of the RC frame system; $F_e$ - force corresponding to infinitely linear elastic response of the RC frame system; d – lateral displacement [3]**

According to the Romanian seismic design norm P100-1:2013, point 6.6.3. (2), Note 2 [3]: the values of $N_{Ed}$, $M_{Ed}$ and $V_{Ed}$ are obtained from the seismic load combination, where the seismic action is multiplied by $\Omega_T$. It is, therefore, also possible to specify a different required bending stiffness constant (and not an equivalent stiffness) for the RC columns and the RC beams to ensure the SCWB seismic response mechanism by means of the "k" factor (Eq. (6)):

$$k = F_y/F_1 \tag{6}$$

For the case presented in a recent study [2] and considering the graph shown in Fig. 3, the bending constant stiffness difference between RC columns and RC beams is given in Eq. (7):

$$k = F_y/F_1 = 476.22 \text{ kN} / 299.15 \text{ kN} = 1.59 \text{ times} \tag{7}$$

Under these conditions the RC columns would be designed for $F_y$ horizontal force and the RC beams for $F_d$ lateral force. Consequently, the RC columns would have a bending stiffness 1.59 times higher than the RC beams. This aspect can be specified due to the fact that the pure RC moment resisting frame structure was designed for a single "q" value and a single value of the bending stiffness 0.5EI for both the beams and the columns.

A very important aspect that needs to be specified for this improved design method is the by default fulfilment of the majority of seismic conditions present in the current design norm [3] without the need for all verification steps (e.g., sum of the moments for RC columns around the beam-column joint is greater than the moments for the RC beams). Theoretically, this laborious method of nonlinear static design should be performed for every RC moment resisting frame structure because it is a fundamental step to assess $F_e$, $F_y$, $F_1$ and $F_d$ forces. The improved design concept is not an ordinary method for the structural engineer. There is also uncertainty about the formation of the maximum number of plastic hinges at the end regions of RC beams and their deformation capabilities due to seismic action. Therefore, the over-strength factor for the design of the RC columns needs to be considered, as given in Eq. (8):

$$\alpha = q_u = F_e/F_y \tag{8}$$

which for the study presented in [2] would mean:

$$\alpha = q_u = F_e/F_y = 788.88\ \text{kN} / 476.22\ \text{kN} = 1.65 \tag{9}$$

Consequently, the design base force for the RC columns becomes (Eq. (10)):

$$F_{columns} = \alpha \cdot F_y = F_e \text{ or } q = 1 \tag{10}$$

Under these conditions, the ratio between the bending stiffness of the columns and the beams for a GF+3F reinforced concrete frame structure [2] (Fig. 3 and Fig. 4) as considered in the current research is (Eq. (11)):

$$\text{k} = F_e/F_1 = 788.88\ \text{kN} / 299.15\ \text{kN} = 2.64 \text{ times} \tag{11}$$

Thus, $K_c/K_b$ ratio falls in the 2.0-3.0 range. This valid value range for a favourable seismic response of the GF+3F moment resisting RC frame structure was also obtained in the analytical study conducted by Haselton et al. [1]. So, the improved design capacity concept through lateral forces is validated from a theoretical and analytical perspective.

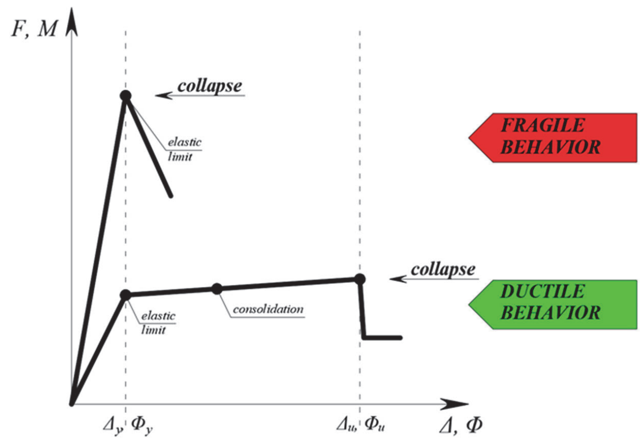


**Figure 4. "F-Δ" (F-d) capacity curve for the same $\Delta_y$ ($d_y$) lateral displacement of the structure in the case of different behaviour factors utilization for beams and columns during the seismic design stage**

From the practical perspective of implementing the improved design capacity concept in the future seismic design normative (with application of 3 different behaviour factors during the design for the superstructure and infrastructure, respectively), it should be mentioned that:

$$q_{inf} < q_c < q_b \tag{12}$$

where:
- the RC beams should be designed considering the "$q_b$" value proposed by P100-1 norm [3];
- the RC columns should be designed considering the "$q_c$" value (where: $q_c < q_b$);
- the infrastructure design with $q_{inf} < q_c$ condition.

## 4 Conclusions

The improvement of the ductile design concept facilitates the calculation methodology and leads to the verification of all the mathematical equations from the seismic design regulations. The approach facilitates the occurrence of the **S**trong **C**olumns-**W**eak **B**eams (SCWB) seismic energy dissipation mechanism without major changes of the ductile concept and seismic response of the "mixed" RC moment resisting frame structure with linear elastic RC elements and nonlinear inelastic RC structural components.
The research study conducted by the authors, based on previous data from the scientific literature, highlights the possibility of introducing an over-strength factor during the design step of the RC columns with two general consequences: a) for the RC structural elements designed to exhibit a linear elastic response, the over-strength factor equates to an increase in the horizontal lateral force (behaviour factor decrease); b) for the RC

structural elements designed to respond in the nonlinear inelastic domain, the over-strength factor leads to a decrease in the magnitude of the horizontal lateral force (behaviour factor increase). Under these circumstances, the design of the RC columns with low "q" values leads to a nonlinear inelastic response and a fragile behaviour.
The over-strength factor leads to a larger cross section for the RC columns. Therefore, this type of RC structural system tends to be similar to the RC coupled walls structures. Evidence of this structural orientation is provided in the retrofitting method according to the Japanese norm [26] of the RC frame structures with RC coupled wall systems by means of RC columns consolidation / strengthening.

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