Behaviour Factor of Ductile Code-Designed Reinforced Concrete Frames

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Received 30 October 2020; Revised 2 February 2021; Accepted 10 February 2021; Published 28 February 2021

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The current generation of seismic design codes is based on a linear elastic force-based approach that includes the nonlinear response of the structure implicitly through a response modification factor (named reduction factor \( R \) in American codes or behaviour factor \( q \) in European codes). However, the use of a prescribed behaviour factor that is constant for a given structural system may fail in providing structures with the same risk level. In this paper, the behaviour factor of reinforced concrete frame structures is estimated by means of nonlinear static (pushover) and nonlinear incremental dynamic analyses. For this purpose, regular reinforced concrete frames of three, five, seven, and nine storeys designed for high ductility class according to the European and Italian seismic codes are investigated, and realistic input ground motions are selected based on the design spectra. Verified analysis tools and refined structural models are used for nonlinear analysis. Overstrength, redundancy, and ductility response modification factors are estimated, and the effects of some parameters influencing the behaviour factor, including the number of bays and the number of storeys, are evaluated. The results are finally compared with those obtained from a previous paper for steel moment-resisting frames with the same geometry. According to the analysis results, the behaviour factors in the case of pushover analysis are significantly higher than those obtained in the case of nonlinear response history analysis. Thus, according to the pushover analysis, the behaviour factor provided by European and Italian standards seems highly conservative. On the contrary, the more refined nonlinear dynamic analysis shows that the code-prescribed value may be slightly nonconservative for middle-high-rise frame structures due to unfavourable premature collapse mechanisms based on column plastic hinging at the first storey. Thus, some modifications are desirable in local ductility criteria and/or structural detailing of high ductility columns to implicitly ensure that the recommended value of the behaviour factor is conservative.

1. Introduction

The structures may behave inelastically under earthquake strong ground motions. Thus, the nonlinear dynamic analysis (NDA) would be essential in predicting the seismic response, since it accounts for the redistribution of forces in the inelastic range. However, the nonlinear dynamic analysis implies more effort than linear static (LSA) or linear dynamic (LDA) analysis, in both performing the analysis since it requires additional data (i.e., hysteretic behaviour of materials and spectrum-compatible input records) and interpreting the results (i.e., failure mechanisms and acceptance criteria). Therefore, the NDA is perceived as too complicated and time-consuming in the structural design process. On the other side, the nonlinear static analysis (NSA) is generally less accurate than NDA, and the results should be validated especially for structures with important higher-mode and torsional effects [1–6]. Consequently, the current practice is generally founded on more simple and easy methods based on the linearly elastic analysis. Despite the recent development of deformation-based design (DBD) methods, the Force-Based Design (FBD) approach remains the standard method to design structures for seismic loads. According to the design linear elastic method, the design force level is defined by scaling the seismic forces with a single reduction factor (named response modification factor \( R \) in UBC97 [7], IBC [8], and NEHRP [9] or behaviour factor \( q \) in Eurocode 8 [10]) that aims to account for overstrength, energy dissipation, and plastic redistribution capacity. This factor that implicitly considers the inelastic behaviour is
defined as the ratio \( q = F_e/F_d \) of the maximum seismic force \( F_e \) on the structure that responds elastically to the design seismic force \( F_d \). Many studies in the literature were focused on formulas to evaluate the response modification factor and its components [11–14] that are primarily related to the period of the structure, ductility, damping, hysteretic behaviour, soil conditions, and distance to the epicentre of the earthquake. The first studies in the literature investigated the influence of the most important parameters (i.e., period, site effects, and ductility) on the response of structures using single-degree-of-freedom (SDOF) models [15, 16]. Some of them were implemented in design standards. For example, both UBC94 [7] and NEHRP 94 [17] were based on the studies of Wu et al. [12]. The outcomes of Newmark and Hall [11] were used in various US seismic codes [7–9, 18]. Neo et al. [12] studies of Wuet al. [12] demonstrated that the outcomes of Newmark and Hall were implemented in design standards. For example, both UBC94 [7] and NEHRP 94 [17] were based on the studies of Wu et al. [12]. The outcomes of Newmark and Hall [11] were used in various US seismic codes [7–9, 18]. The \( R-\mu-T \) relations proposed by Vidic et al. [19] inspired both Eurocode 8 [10] and Italian seismic code [20, 21]. More recent studies were dedicated to real complex buildings. The estimation of the components of the behaviour factor requires analysing realistic code-designed buildings, since material safety factors, design criteria, detailing provisions, and practical construction aspects may significantly affect the response reduction factors. Mwafy and Elnashi [22] presented a comprehensive study to calibrate the \( R \)-factors using 12 RC buildings designed with Eurocode 8 [10]. Zafar [23] conducted a parametric study involving RC framed buildings to evaluate the effect of dimensional and material properties on \( R \)-factor. AlHamaydeh et al. [24] studied the \( R \)-factors of three RC framed buildings of 4, 16, and 32 stories for two levels of seismicity in Dubai. Massumi et al. [25] examined the structural overstrength in 25 reinforced concrete framed structures and showed that the overstrength factor remains approximately constant (with average values of 2.5 and 1.7 for buildings designed using, respectively, the Iranian Code and North American Codes) for buildings with a number of floors between 4 and 10. Al-Ahmar and Al-Samara [26] investigated the effects of the number of stories and bays and the bay span on the seismic response modification factors of 25 special moment-resisting frames (SMRFs) buildings. The main weakness of the force-based design approach is that a predefined behaviour factor is used, which is constant regardless of the structure’s geometric configuration. The value proposed could be unrealistic, excessively conservative, or not representative of the actual nonlinear behaviour of the building. Most of the past studies in the literature have focused on evaluating the ductility component of the response reduction factor for single-degree-of-freedom (SDOF) systems. However, the overstrength is also very important in calibrating the force reduction factor and may vary widely depending on many factors such as the structural system, the ductility class, and the period of the structure. This paper aims to evaluate and clarify the above. For this purpose, the behaviour factors of moment-resisting RC-frame structures of high ductility class were evaluated performing inelastic pushover and dynamic analyses of RC frames of different heights and configurations. Three-, five-, seven-, and nine-storey reinforced concrete frame structures were designed according to the Italian Code [20]. Nonlinear static pushover analyses for uniform and modal load patterns were carried out to identify the load-displacement relationship and estimate the ductility, overstrength, and redundancy reduction factors. The incremental dynamic analysis using a set of time-history earthquake records was carried out to estimate the behaviour factors, and the results were compared with the values obtained by the pushover analysis. The effects of some parameters influencing the reduction factor, including the number of spans and the number of storeys, were investigated. The values calculated were related to the value of \( q \)-factor provided by the Italian Code [20]. The results were finally compared with those obtained from a previous study on steel moment-resisting frames having the same geometry.

2. Evolution of Seismic Codes and Behaviour Factor

First introduced in the ATC-3-06 report [27] in the late 1970s, the \( R \)-factor was then used in seismic codes to obtain economical designs based on simple elastic analysis but allowing plastic deformation under earthquake ground motion. Since the 1980s, in many studies, seismic codes, and documents, the \( R \)-factor has been decomposed into different components. For example, in ATC-19 [28] and ATC-34 [29], the \( R \)-factor was calculated as the product of three factors accounting for overstrength, redundancy, and ductility. Nowadays, most of the current seismic design codes (ASCE SEI/ASCE 7-05 [30], Eurocode 8 [10], BIS IS 1893 [31], NZS1170 [32], MDRA [33], NTC-2018 [26], and IBC [8]) are based on a reduction factor that implicitly accounts for the nonlinear response of the structure under the design earthquake ground motion. On the other side, many procedures for performance-based seismic assessment are included in codes and guidelines (ATC 19 [28], ATC 34, [29], ATC 40 [18], FEMA 440 [34], Annex B EC8 [10], FEMA 356 [35], FEMA 450 [36], FEMA 451 [37], and FEMA P695 [38]) and incorporate response reduction factors, since they are based on the capacity spectrum method [34], the displacement coefficient method [34], or the N2 method [39]. In general, the reduction factor depends on the parameters that may affect the nonlinear response of the building: mechanical behaviour of materials, design procedures, typology of the structure, construction details, structural regularity, and so on. However, the response modification factors provided by the different seismic codes may differ in both meaning and magnitude. The behaviour factor of Eurocode 8 [10] is based on the typology of structure, structural regularity, and ductility class but does not explicitly account for the overstrength. On the contrary, US [7, 35], Canadian [40], New Zealand [41], and Japanese [42] codes include this parameter in the force reduction factor definition. In Eurocode 8 [10], for a highly ductile frame structure, \( q = 4.5 \alpha_d/\alpha_1 \), where \( \alpha_d/\alpha_1 \) is the ratio of the seismic action that causes the development of a full plastic mechanism to the seismic action at the formation of the first plastic hinge. Since the ratio \( \alpha_d/\alpha_1 \) is 1.2 for single-bay multistorey frames and 1.3 for multibay and multistorey frames, the behaviour factor will range between 5.4 and 5.85. On the other side, the American seismic codes (UBC97 [7], IBC [8], and ASCE-7 [30]) provide a fixed value of the
reduction factor depending only on the structural resisting system. For example, a reduction factor \( R = 8 \) is assumed in ASCE-7 [30] for special (i.e., highly ductile) RC-frame structures. The National Building Code of Canada (NBCC) [40] for ductile moment-resisting RC-frame buildings prescribes a response modification factor that is the product of two factors, a ductility-related factor \( R_d = 4 \) and an overstrength-related factor \( R_o = 1.7 \). However, it should be underlined that a direct code comparison between European and American seismic code provisions is not consistent if only the level of force reduction is considered. A reliable comparison should involve not only the reduction factor but also the full design process.

3. Methods for Evaluating Behaviour Factor

The response modification factor is generally expressed as the products of different factors [27–29]:

\[
R = R_\mu \cdot R_\rho \cdot R_\Omega \cdot R_\xi,
\]

where \( R_\mu, R_\rho, R_\Omega, \) and \( R_\xi \) are ductility, redundancy, overstrength, and damping reduction factors. The damping reduction factor \( R_\xi \) is used to characterize energy dissipation given by viscous damping or hysteretic behaviour. It is included only if supplemental viscous damping devices are used [28]; otherwise \( R_\xi = 1 \). The meaning of the other parameters may be highlighted based on the pushover curve obtained by nonlinear static (pushover) analysis. Figure 1 shows the base shear force \( (F) \) versus roof displacement \( (\delta) \) relationship and its elastoplastic idealization according to EC8 [10] and NTC-2018 [21]. The parameters in this figure are defined as follows: design base shear force \( (F_d) \), first significant yield strength \( (F_\xi) \), idealized yield strength \( (F_y) \), elastic response strength \( (F_e) \), ultimate displacement \( (\delta_u) \), and yield displacement \( (\delta_y) \). Concerning Figure 1, the behaviour factor may be expressed as follows:

\[
R = \frac{F}{F_d} = \frac{F_x}{F_y} \cdot \frac{F_y}{F_\xi} \cdot \frac{F_\xi}{F_d} = R_\mu \cdot R_\rho \cdot R_\Omega \cdot R_\xi,
\]

where

\[
R_\mu = \frac{F_x}{F_y},
\]

\[
R_\rho = \frac{F_y}{F_\xi},
\]

\[
R_\Omega = \frac{F_\xi}{F_d},
\]

Figure 1: Base shear versus roof displacement relationship. Overstrength \((R_o)\), redundancy \((R_\rho)\), and ductility \((R_\mu)\) reduction factors.

The global ductility reduction factor \( R_\mu ; R_\Omega = q_0 \) is defined as the ratio between the minimum lateral strengths required to the elastic single-degree-of-freedom system (SDOF system) and the minimum strength required to limit the inelastic deformations to the ductility demand ratio \( \mu = \delta_u / \delta_y \). Many studies have been proposed in the literature developing relationships that are generally based on SDOF systems [11, 13, 14, 16, 43, 44]. In general, the ductility reduction factor \( R_\mu \) depends on both structural properties such as ductility, damping, and fundamental period and the characteristics of seismic ground motion. The redundancy factor \( R_\rho \) is defined as the ratio between the idealized yield strength (corresponding to the formation of the plastic collapse mechanism) and the first significant yield strength (corresponding to the formation of the first plastic hinge). The seismic framed building structures should be designed and detailed to transfer the seismic forces to the foundation through a redundant framing system. The overstrength factor \( R_\Omega \) is defined as the ratio between the first significant yield strength and the design base shear force. This parameter accounts for the overall structural overstrength, which is produced by the design assumptions that typically produce a conservative design: (1) design strength of materials with partial safety factors, (2) code requirements for reinforcement details, (3) drift limitations, (4) design engineering practice of rounding the steel rebars’ diameter, and (5) design simplifications concerning the methods of analysis (i.e., linear and static analysis) and verification (i.e., 30% rule for bidirectional design). Its value may be very sensitive to parameters such as the level of seismic intensity, the design method, and the ratio of gravity load to seismic load. In this paper, the behaviour factor was calculated using both nonlinear static (pushover) and nonlinear incremental dynamic analysis (IDA). In the nonlinear static analysis, the behaviour factor was calculated from equation (2) using the pushover curve to estimate the reduction factors. It should be highlighted that Eurocode 8 [10] provides the following expression of the behaviour factor:

\[
q = q_0 \cdot \frac{\alpha_u}{\alpha_1},
\]

where \( \alpha_u \) is the horizontal force multiplier to activate the mechanism and \( \alpha_1 \) is the horizontal force multiplier to first reach the flexural resistance in any member in the structure. Comparing equations (2)–(4) gives \( R_\mu, R_\Omega = q_0 \), and \( R_\rho = \alpha_u / \alpha_1 \). In the nonlinear incremental dynamic analysis (IDA), the behaviour factor was calculated using the formulation proposed by Mwafy and ElNash [22] as follows:

\[
q = \frac{a_{g,x}}{a_{g,d}}.
\]
The ultimate chord rotation capacity ($\theta_u$) and the plastic hinge length ($L_{pl}$) were estimated using the following expressions given in Italian Code [21] (NTC-Instructions, where $a_{g,u}$ and $a_{g,d}$ are the peak ground accelerations of the collapse and design earthquake, respectively. By supposing that the response acceleration spectra have the same dynamic amplification for both collapse and yielding limit states, $a_{g,u}$ and $a_{g,d}$ were calculated from IDA using spectrum-compatible earthquake ground motions of increasing intensity.

4. Design, Modelling, and Analysis of RC-Reinforced Concrete Frames

To investigate the relationship between the code-prescribed value and the calculated value of the behavior factor, symmetric-in-plan RC frames with different configurations were considered in the analysis. These frames were assumed to be the central one of a series of frames equally spaced at a distance of 5 m in typical residential buildings (Figure 2). The floor-to-floor height is 3.0 m for all storeys except the ground storey, which is kept as 3.5 m. The analyses were performed by varying the number of floors of the building, thus considering the following case studies (Figure 3): (1) 3-storey, 3-bay (3S3B); (2) 5-storey, 3-bay (5S3B); (3) 7-storey, 3-bay (7S3B); (4) 9-storey, 3-bay (9S3B); (5) 7-storey, 5-bay (7S5B); and (6) 9-storey, 5-bay (9S5B). The RC buildings were designed according to the Italian Code provisions [20] for high ductility class using the strong-column weak-beam design criterion. The design acceleration spectrum for soil class A, damping ratio of 5%, peak ground acceleration equal to 0.25 g, and behavior factor $q = 4.5 \times 1.3 = 5.85$ were considered to evaluate the seismic loads. The total dead load and live loads on floor slabs were assumed to be 4.89 and 2.0 kN/m$^2$, respectively. Concrete of class C25/30 and steel reinforcement of grade B450 were considered in the analysis. The building structures were analysed using the response spectrum analysis and considering the combination of actions for the seismic design situation. The action effects for the structural members were derived on the basis of capacity design considerations. Structural verifications were carried out to meet the strength and drift criteria specified by the Italian Building Code for new constructions. A damage limitation drift of 0.5% was considered for the serviceability limit state under the hypothesis that nonstructural elements of brittle materials are attached to the structure. A second drift criterion was given by the stability load combinations ty coefficient ($\theta$) for P-Delta effects that should not exceed 0.30 (Eq. 4.4.2 EN 1998-1 [10]; Eq. 7.3.3 NTC 2018 [20]).

Owing to their large lateral deformability, the steel moment-resisting frames (MRFs) compliant with these severe requirements may be largely overdesigned [45]. On the contrary, this is not the case of concrete moment-resisting frames where the requirements for lateral deformability and P-Delta effects are seldom critical. Tables 1 and 2 show the cross-sectional dimensions and the steel reinforcement, respectively, for columns and beams. It should be highlighted that, as is the case in current practice, the cross-sectional dimensions were first assumed before the design process (also considering architectural and construction needs) and then checked according to the Italian Code provisions [20]. This is because the column cross sections for the 3 and 5 storied buildings, as well as for the 7 and 9 buildings, are the same. In the same way, a little variation of reinforcement demands is observed when comparing 3S3B with 5S3B frame and 7S3B with 9S3B frame. The reason for this is that a minimum of three steel bars was placed on the short side of the column to limit the distance between consecutive bars. As is the case in current practice, this gives an overstrength of the columns of the lower frames (i.e., 3S3B compared to 5S3B frame and 7S3B compared to 9S3B frame). Longitudinal reinforcement was uniformly distributed around the perimeter to enhance the concrete confinement.

Once the buildings have been designed using a tridimensional model, the capacity curves were calculated based on a 2D pushover analysis on the plan frames. For this purpose, the nonlinear model of the framed structure was implemented in SAP2000 computer program [46]. It should be observed that, in general, the three-dimensional effects are expected to influence the structural response. Vertical and horizontal irregularities can certainly affect the collapse mechanism and increase the nonlinear torsional and higher-mode effects [47]. Moreover, there are cases where the three-dimensional geometry or the three-dimensional behavior is important to simulate. However, this paper aims to evaluate the general design provisions concerning the basic value of the behaviour factor for regular in plan and elevation framed buildings. Moreover, for most structural framing types, two-dimensional models are likely to be sufficient. Finally, the building code provisions regarding plan configuration and three-dimensional effects (e.g., redundancy and accidental torsion) are usually not structural system specific, so a two-dimensional model should be adequate to evaluate the design recommendations for the behaviour factor of RC buildings. The nonlinear behaviour was represented using a lumped plasticity (LP) hinge model based on fiber plastic hinges at both ends of the structural members. The element cross sections were divided into a number of fibres, which enables the geometric definition of the steel and confined and unconfined concrete regions within the section. User-defined hinge properties were used for the beams to simulate the variation of the steel reinforcements between the two ends of the beam. On the contrary, automatic hinge properties were used for the column, since the longitudinal steel reinforcement is constant in each column. Rigid-end offsets for beams and columns were considered in the analysis. The moment-curvature relationship was obtained through the integration of the nonlinear uniaxial stress-strain relation of the individual fibres forming the cross section. Three different constitutive models were used for the different parts of the section: unconfined concrete, confined concrete, and steel reinforcement. The concrete was modelled with the stress-strain relationship originally proposed by Mander et al. [48]. The steel was modelled with an elastic-plastic-hardening relationship. The moment-rotation relationship was defined based on the moment-curvature law of the plastic hinge section and the length of the plastic hinge. The ultimate chord rotation capacity ($\theta_u$) and the plastic hinge length of the plastic hinge ($L_{pl}$) were estimated using the following expressions given in Italian Code [21] (NTC-Instructions, 

Advances in Civil Engineering
Equations C8.7.2.5 and C8.7.2.6) and Eurocode 8 [49] (Annex A, Equations A.4 and A.5):

\[
\theta_u = \frac{1}{Y_{ef}} \left( \theta_y + (\phi_u - \phi_y) L_{pl} \left( 1 - \frac{0.5 L_{pl}}{L_v} \right) \right),
\]

\[
L_{pl} = 0.1 L_v + 0.17 h + 0.24 \frac{d_{bl} f_y}{\sqrt{f_c}}.
\]

where \( \phi_u \) is the ultimate curvature, \( \phi_y \) is the yield curvature, \( L_v \) is the shear length, \( h \) is the total section height, \( d_{bl} \) is the mean diameter of the longitudinal bars, \( f_y \) is the yield strength of the longitudinal reinforcement steel [MPa], and \( f_c \) is concrete compression strength [MPa]. Strength and stiffness degradation was considered using Takeda hysteresis model [50]. For calculating the reduction factor, the performance limit state of the structure should be defined. In general, both global limits (i.e., vertical and lateral load resistance, lateral drift, etc.) and local limits (i.e., displacement, acceleration, or rotation responses of single members) should be considered in the analysis. Currently, there are no drift limit requirements in both Italian Code [20] and Eurocode 8 [10]. Thus, only critical failure mechanisms are checked as the global limit. Moreover, the joint panel zones may be assumed to be strong enough to avoid any premature failure, since the capacity design approach was used.

**Figure 2:** Plan view of the buildings containing the studied frames.
Figure 3: Continued.
Table 1: Column dimensions and reinforcement details.

| Frame | Storey | Exterior columns | 1st interior columns | 2nd interior columns |
|-------|--------|------------------|----------------------|----------------------|
|       |        | Dimensions       | Reinforcement        | Dimensions           | Reinforcement        |
|       |        | 30 × 35          | 3 + 3ϕ18             | 30 × 40              | 4 + 4ϕ18             |
| 3S3B  | 1-2    |                  |                      |                      |                      |
|       | 3      | 30 × 35          | 3 + 3ϕ18             | 30 × 40              | 3 + 3ϕ18             |
|       |        |                  |                      |                      |                      |
| 5S3B  | 1      | 30 × 35          | 3 + 3ϕ18             | 30 × 40              | 5 + 5ϕ18             |
|       | 2      | 30 × 35          | 3 + 3ϕ18             | 30 × 40              | 4 + 4ϕ18             |
|       | 3      | 30 × 35          | 3 + 3ϕ18             | 30 × 40              | 3 + 3ϕ18             |
|       | 4-5    | 30 × 35          | 3 + 3ϕ18             | 30 × 35              | 3 + 3ϕ18             |
| 7S3B  | 1      | 30 × 40          | 3 + 3ϕ18             | 30 × 50              | 5 + 5ϕ18             |
|       | 2      | 30 × 35          | 3 + 3ϕ18             | 30 × 45              | 5 + 5ϕ18             |
|       | 3      | 30 × 35          | 3 + 3ϕ18             | 30 × 45              | 4 + 4ϕ18             |
|       | 4      | 30 × 35          | 3 + 3ϕ18             | 30 × 40              | 4 + 4ϕ18             |
|       | 5-7    | 30 × 35          | 3 + 3ϕ18             | 30 × 35              | 3 + 3ϕ18             |
| 9S3B  | 1      | 30 × 40          | 4 + 4ϕ18             | 30 × 55              | 6 + 6ϕ18             |
|       | 2      | 30 × 40          | 4 + 4ϕ18             | 30 × 55              | 5 + 5ϕ18             |
|       | 3      | 30 × 40          | 4 + 4ϕ18             | 30 × 50              | 5 + 5ϕ18             |
|       | 4      | 30 × 35          | 3 + 3ϕ18             | 30 × 45              | 5 + 5ϕ18             |
|       | 5      | 30 × 35          | 3 + 3ϕ18             | 30 × 40              | 4 + 4ϕ18             |
|       | 6-7    | 30 × 35          | 3 + 3ϕ18             | 30 × 40              | 3 + 3ϕ18             |
|       | 8-9    | 30 × 35          | 3 + 3ϕ18             | 30 × 35              | 3 + 3ϕ18             |
| 7S5B  | 1      | 30 × 35          | 4 + 4ϕ18             | 30 × 55              | 5 + 5ϕ18             |
|       | 2      | 30 × 35          | 3 + 3ϕ18             | 30 × 50              | 5 + 5ϕ18             |
|       | 3      | 30 × 35          | 3 + 3ϕ18             | 30 × 45              | 5 + 5ϕ18             |
|       | 4      | 30 × 35          | 3 + 3ϕ18             | 30 × 40              | 4 + 4ϕ18             |
|       | 5      | 30 × 35          | 3 + 3ϕ18             | 30 × 40              | 4 + 4ϕ18             |
|       | 6-7    | 30 × 35          | 3 + 3ϕ18             | 30 × 40              | 3 + 3ϕ18             |
|       | 8-9    | 30 × 35          | 3 + 3ϕ18             | 30 × 35              | 3 + 3ϕ18             |
| 9S5B  | 1      | 30 × 35          | 4 + 4ϕ18             | 30 × 55              | 6 + 6ϕ18             |
|       | 2      | 30 × 35          | 3 + 3ϕ18             | 30 × 50              | 5 + 5ϕ18             |
|       | 3      | 30 × 35          | 3 + 3ϕ18             | 30 × 45              | 5 + 5ϕ18             |
|       | 4      | 30 × 35          | 3 + 3ϕ18             | 30 × 40              | 4 + 4ϕ18             |
|       | 5      | 30 × 35          | 3 + 3ϕ18             | 30 × 40              | 4 + 4ϕ18             |
|       | 6-7    | 30 × 35          | 3 + 3ϕ18             | 30 × 40              | 3 + 3ϕ18             |
|       | 8-9    | 30 × 35          | 3 + 3ϕ18             | 30 × 35              | 3 + 3ϕ18             |
the chord rotation of RC members was used as the local limit. To this aim, the chord rotation capacity for the Limit State of Life Safety was assumed as 3/4 of the ultimate chord rotation $\theta_u$, assessed by equation (6). Both nonlinear static (pushover) analysis (NSA) and incremental dynamic analysis (IDA) were carried out. The structure was subjected to constant gravity loads and increasing lateral loads having different distribution patterns. Many procedures have been proposed in the literature based on adaptive and/or multimodal load patterns [1, 3, 5, 6, 51–53]. However, the case studies are regular in plan and elevation. Moreover, the number of floors is limited. Finally, the capacity design approach should avoid local or partial failure mechanisms. Thus, both higher mode contribution and variation of load pattern for yielding may be neglected, and the nonlinear static (pushover) analysis may apply two simple lateral load patterns: (1) a first mode load pattern related to the storey mass and the first mode shape displacement and (2) a uniform load pattern proportional to the storey mass.

A bilinear elastic-plastic or elastic-hardening idealization of the pushover curve is employed in all codes and guidelines. Eurocode 8 [10] follows an approach based on the elastic-perfectly-plastic idealization. The yield force is equal to the base shear force at the formation of the plastic mechanism. The initial stiffness of the idealized system is determined in such a way that the areas under the actual and the idealized pushover curves are equal (equivalence of energy). FEMA 273 [9], FEMA 356 [35], and FEMA 440 [34] gradually upgraded the bilinear fit by integrating rules to account for softening behavior. According to FEMA 356 [35], the initial stiffness is calculated at a base shear force equal to 60% of the nominal yield strength. The postyield slope is evaluated by balancing the area above and below the capacity curve up to the target displacement. This paper applies the elastic-perfectly-plastic idealization based on the Italian Building Code [20]. Thus, the initial stiffness was calculated using the 60% rule, and the yield strength was calculated with the equal area criterion that was extended up to the point of a 15% degradation of the maximum base shear to account for a limited softening. The bilinear idealizations employed by FEMA, Eurocode 8, and Italian Code were found to be conservative, since they provide higher equivalent periods [54]. In particular, the elastic-perfectly-plastic approximation of the Italian Code was found to give acceptable errors when matching the maximum strength and not exceeding the displacement that corresponds to 85% of the maximum shear. The incremental dynamic analysis (IDA) was performed according to the Italian Code [20] with a set of seven earthquakes scaled to increasing levels of intensity. The expected seismic effect was calculated as the average of the response quantities from all analyses. The input ground motions were selected from the European Strong-Motion Database (ESD), the SIMBAD database, and the Italian Accelerometric Archive (ITACA) [55, 56]. The spectrum compatibility to the target response spectrum was checked observing the rules of the Italian Code [20]: (1) the mean of the zero period spectral response acceleration values is not smaller than $a_{gs}$, where $s$ is the soil factor and $a_{gs}$ is the design ground acceleration on type A ground; (2) in the range of periods of interest no value of the mean elastic spectrum is less than 90% of the corresponding value of the target elastic spectrum. The earthquake data are plotted in Table 3. In Figure 4, the elastic acceleration spectrum of the selected records and their mean value is

| Frame | Storey | Dimensions | Section A | Section B | Section C |
|-------|-------|------------|-----------|-----------|-----------|
|       |       | Depth (mm) | Width (mm) | Top Bottom | Top Bottom | Top Bottom |
| 3S3B  | 1-2   | 500        | 300       | 4ϕ14      | 2ϕ14      | 2ϕ14 + 2ϕ18 | 1ϕ14 + 1ϕ18 |
|       | 3     | 500        | 300       | 4ϕ14      | 2ϕ14      | 2ϕ14 + 2ϕ18 | 1ϕ14 + 1ϕ18 |
| 5S3B  | 1-2   | 500        | 300       | 3ϕ18      | 2ϕ18      | 2ϕ14 + 2ϕ18 | 1ϕ14 + 1ϕ18 |
|       | 3-4   | 500        | 300       | 1ϕ14 + 2ϕ18 | 2ϕ18 | 1ϕ14 + 2ϕ18 | 1ϕ14 + 1ϕ18 |
|       | 5     | 500        | 300       | 2ϕ18      | 2ϕ18      | 2ϕ14 + 2ϕ18 | 1ϕ14 + 1ϕ18 |
| 7S3B  | 1-2   | 500        | 300       | 1ϕ14 + 3ϕ18 | 2ϕ18 | 1ϕ14 + 3ϕ18 | 2ϕ18 |
|       | 3-4   | 500        | 300       | 1ϕ14 + 3ϕ18 | 2ϕ18 | 3ϕ18      | 1ϕ14 + 1ϕ18 |
|       | 5-6   | 500        | 300       | 3ϕ18      | 2ϕ18      | 1ϕ14 + 2ϕ18 | 1ϕ14 + 1ϕ18 |
|       | 7     | 500        | 300       | 2ϕ18      | 2ϕ18      | 1ϕ14 + 2ϕ18 | 1ϕ14 + 1ϕ18 |
| 9S3B  | 1-3   | 500        | 300       | 1ϕ14 + 3ϕ18 | 2ϕ18 | 1ϕ14 + 3ϕ18 | 2ϕ18 |
|       | 4-5   | 500        | 300       | 1ϕ14 + 3ϕ18 | 2ϕ18 | 3ϕ18      | 1ϕ14 + 1ϕ18 |
|       | 6-8   | 500        | 300       | 2ϕ14 + 2ϕ18 | 2ϕ18 | 1ϕ14 + 2ϕ18 | 1ϕ14 + 1ϕ18 |
|       | 9     | 500        | 300       | 3ϕ18      | 2ϕ18      | 1ϕ14 + 2ϕ18 | 1ϕ14 + 1ϕ18 |
| 7S5B  | 1-4   | 500        | 300       | 1ϕ14 + 2ϕ18 | 2ϕ18 | 1ϕ14 + 2ϕ18 | 1ϕ14 + 1ϕ18 |
|       | 5     | 500        | 300       | 3ϕ18      | 2ϕ18      | 1ϕ14 + 2ϕ18 | 1ϕ14 + 1ϕ18 |
| 9S5B  | 1-3   | 500        | 300       | 1ϕ14 + 2ϕ18 | 2ϕ18 | 1ϕ14 + 2ϕ18 | 1ϕ14 + 1ϕ18 |
|       | 4-6   | 500        | 300       | 3ϕ18      | 2ϕ18      | 1ϕ14 + 2ϕ18 | 1ϕ14 + 1ϕ18 |
|       | 7-8   | 500        | 300       | 1ϕ14 + 3ϕ18 | 2ϕ18 | 1ϕ14 + 2ϕ18 | 1ϕ14 + 1ϕ18 |
|       | 9     | 500        | 300       | 1ϕ14 + 2ϕ18 | 2ϕ18 | 1ϕ14 + 2ϕ18 | 1ϕ14 + 1ϕ18 |
compared to the target spectrum. The nonlinear response history analysis was carried out using the Rayleigh damping model with tangent stiffness and constant proportionality coefficients based on 5.0% viscous damping in the first two natural modes.

5. Results and Discussion

Herein are presented and discussed the results obtained from nonlinear pushover analysis and incremental dynamic analyses for the different case studies. Figures 5 and 6 show the pushover curves (i.e., base shear versus roof displacement), their bilinear idealization, and all parameters necessary to calculate the behaviour factor according to equation (2): design base shear force \( F_d \), first significant yield strength \( F_1 \), idealized yield strength \( F_y \), and elastic response strength \( F_e \). It can be observed that, for a given number of storeys, the shear resistance of the structure significantly increases as the number of bays increases. In all cases examined, the displacement ductility assumes values ranging between 5.0 for the 75SB frame and 6.42 for the 95SB frame under the first mode load pattern. Figures 7 and 8 show the location of plastic hinges formed in RC members when the Life Safety Limit State is reached. No plastic hinges form in the beams of the highest stories for both uniform and first mode pushover analysis. According to the capacity design criterion, plastic hinges are permitted in beams but not in columns to ensure strong-column weak-beam design philosophy. However, the results highlight that column hinging mechanisms occur in many cases, even though often at high values of interstorey drift. As evidenced by other studies in the past [57], the column yielding could not be avoided, even in buildings designed for very high (i.e., values up to 1.4) strong-column weak-beam ratios (i.e., the ratio of the sum of moment capacities of columns to the sum of flexural strengths of beams). The designed “strong-column weak-beam” is not able to ensure a global failure mechanism and prevent the occurrence of plastic hinges in the columns under severe ground motions. Figures 9 and 10 show the distribution of the plastic hinges obtained from the nonlinear dynamic analysis under the Bevagna earthquake ground motion (Table 3). The envelope is plotted to view the maximum and/or minimum results over all the steps of the time-history analysis. As a consequence, the deformed shape

Table 3: Parameters of earthquake ground motions.

| Num. | Input        | Date          | PGA (m/s²) | Body-wave magnitude |
|------|--------------|---------------|------------|---------------------|
| 1    | Friuli       | 15.09.1976   | 2.45       | 5.09                |
| 2    | Tabas        | 16.09.1978   | 3.31       | 7.03                |
| 3    | Montenegro   | 15.04.1979   | 2.19       | 6.09                |
| 4    | South Iceland| 17.06.2000   | 3.12       | 6.05                |
| 5    | Campano Lucano | 23.11.1980  | 3.16       | 6.09                |
| 6    | Bingol       | 01.03.2003   | 2.92       | 6.03                |
| 7    | Izmit        | 17.08.1999   | 2.33       | 7.06                |
| 8    | Bevagna      | 26.09.1997   | 0.74       | 5.90                |
| 9    | Bevagna Valnerina | 19.09.1979 | 0.23       | 5.84                |
| 10   | Codroipo     | 06.05.1976   | 0.86       | 6.50                |
| 11   | Colfiorito   | 26.09.1997   | 1.75       | 5.90                |
| 12   | Mercato S. Severino | 23.11.1980 | 1.36       | 6.87                |

Figure 4: Spectrum compatibility for the selected records.
is based on the maximum or minimum displacement at each degree of freedom and thus it may not correspond to any particular state. The results show the column plastic hinging at large displacements, not only at the base but also in other locations even though the strong column concept was followed.
The parameters extracted from the pushover curves (i.e., $F_d$, $F_y$, $F_u$, and $F_p$) allow evaluating the seismic force reduction factors (i.e., overstrength $R_\text{q}$, redundancy $R_\rho$, and ductility factor $R_\mu$). The values of the force reduction factors are presented in Figure 11. The overstrength ratio ranges between 1.10 and 1.51 with higher values for uniform pushover analysis. It should be observed that the code-prescribed values are significantly higher. For example, ASCE 7-10 [58] prescribes an overstrength ratio of 3 for RC-frame structures. However, the values obtained are consistent with the results obtained by other studies. Zhu et al. [59] found that the overstrength ratio of RC-frame structures is between 1.2 and 1.7. Sharifi et al. [60] obtained values within the range of 1.2–2.2 for ordinary and special RC frames. In Figure 11, the redundancy reduction factor assumes values in the range 1.58–2.24, while the value recommended in Eurocode 8 [10] is 1.30. Thus, it can be concluded that Eurocode 8 [10] gives conservative estimates of the redundancy reduction factor. The number of stories seems to have the biggest influence on the redundancy response modification factor since it decreases with increasing the number of stories. The ductility reduction factor $R_\mu$ increases with both the number of stories and the number of bays. The number of bays also has shown some effects on all response modification factors (i.e., overstrength, redundancy, and ductility), since they increase with the number of bays. Finally, Figure 11 also shows the seismic behaviour factor calculated according to equation (2). In all cases examined here, the value of $q$-factor determined from the nonlinear static analysis was found sensibly higher than the code-prescribed value of 5.85. Moreover, the $q$-factor sensibly increases with increasing the number of bays, while it slightly decreases with increasing the number of stories, with the lowest value equal to 10.32. A similar trend was observed also in other studies in the literature [61, 62]. This result shows that the value of the seismic behavior factor depends, among others, on the height of a structure, the parameter of which is generally not taken into account by the seismic design codes. Figure 12 compares the $q$-factors determined based on four different methods. The first method (named “pushover analysis” in Figure 12) estimates the reduction factors from the bilinear idealization of the pushover curve (Figure 1) and then evaluates the $q$-factor using equation (2). The second method (named “incremental dynamic analysis” in Figure 12) calculates the $q$-factor from equation (5), where $a_{\text{g},u}$ and $a_{\text{g},d}$ are evaluated from the incremental dynamic analysis (IDA) as described in Section 3. The third method (named “capacity spectrum method” in Figure 12) uses the procedure of the current Italian Code [21] and Annex B of EN 1998-3 [10] to calculate the peak ground acceleration at collapse ($a_{\text{g},\text{max}}$) and then evaluates the $q$-factor according to equation (5). As is well known, this procedure for the determination of the target displacement ($\delta_t$) implements the capacity spectrum method [63] using the strength reduction rule proposed by Vidiani et al. [19]. The procedure was used as a simple tool to estimate the peak ground acceleration at collapse ($a_{\text{g},\text{max}}$). The displacement ($\delta_t$) corresponding to the ultimate limit state is known. The elastoplastic idealization of
the capacity curve gives the effective period \((T)\) and the ductility \((\mu)\), which are used to define the reduction rule \(R_\mu(T,\mu)\) according to Vidic et al. [19]. This allows obtaining the Inelastic Demand Response Spectrum (IDRS) from the Elastic Demand Response Spectrum (EDRS). Thus, the above-mentioned procedure was used to increase the peak ground acceleration until the target displacement equals the ultimate displacement (i.e., \(\delta_t = \delta_u\)). The corresponding peak ground acceleration gives the value of \(a_{g,u}\) to be used to calculate the \(q\)-factor using equation (5). The fourth method (named “Italian Code” in Figure 12) applies the code-prescribed formula, \(q = 4.5\alpha_u/\alpha_1\), for a highly ductile frame structure, where the ratio \(\alpha_u/\alpha_1\) is calculated as \(F_u/F_1\) from the pushover curve. Figure 12 shows that the \(q\)-factors calculated from IDA are generally lower than the corresponding values calculated by both the pushover analysis and the capacity spectrum method. The differences between static and dynamic approaches are essentially due to two contrasting effects: (1) large interstorey drift concentrated on the weakest storey and (2) higher mode contribution in the dynamic response. The concentration of storey drift in some stories under the earthquake ground motion has the effect of reducing the roof displacement corresponding to the interstorey drift capacity at Life Safety Limit State. On the contrary, the higher mode effects in the dynamic response may significantly increase the base shear and, thus, the overstrength reduction factor, especially in the case of high-rise frames. The formula \(q = 4.5\alpha_u/\alpha_1\) proposed by the Italian Code [20] for highly ductile frames gives values that are more consistent with IDA. Finally, it should be underlined that the code-prescribed value of 5.85 is slightly not conservative for the 7S3B frame structure. This results from an unfavourable premature collapse mechanism based on plastic column hinging at

Figure 7: Collapse mechanism and distribution of plastic hinges at Life Safety Limit State. Pushover analysis of 3-bay RC frames.
**Figure 8:** Collapse mechanism and distribution of plastic hinges at Life Safety Limit State. Pushover analysis of 5-bay RC frames.

**Figure 9:** Distribution of plastic hinges. Nonlinear dynamic analysis of 3-bay RC frames under the Bevagna earthquake.
the first storey during the dynamic analysis. The chord rotation demand reaches the ultimate capacity in one of these columns without involving large plastic deformation of beams. The collapse occurs at a small value of inter-storey drift, thus affecting the value of $q$-factor obtained from the dynamic analysis which is lower than 5.85. In this case, the design criteria and detailing rules of both Eurocode 8 [10] and Italian Code [20] for highly ductile frames were not completely effective in assuring that the calculated behavior factor is higher than the code prefixed value. The consistency between the calculated and prefixed values of $q$-factor might be improved by introducing more rigorous rules for the detailing of primary seismic columns for local ductility.
Finally, a comparison between RC frames and the corresponding steel moment-resisting frames was carried out. To this aim, the results obtained were compared with a previous research paper [64] that focused on the effects of the parameters influencing the response reduction factor of steel moment-resisting frames (MRFs). Based on the results of this study, a local ductility criterion was proposed to improve the provisions of the 2008 version of the Italian Code [65]. In particular, the following limitation on the dimensionless axial force was proposed to counteract the effect of the poor ductility properties exhibited by columns of MRFs under high axial loading:

\[
\frac{N_{Ed}}{N_{pl,Ed}} \leq 0.3,
\]  

(8)

where \(N_{Ed}\) is the axial force demand of the column and \(N_{pl,Ed}\) is the lower-bound axial compressive strength of the column. This local ductility criterion has then been introduced in the current version of the Italian Code (§4.2.4.1.2.) [20]. The comparison between RC frames and steel moment-resisting frames was carried out considering frames with the same geometry (3-, 5-, 7-, and 9- storey frames). In Figure 13, the behaviour factors calculated using pushover analysis and incremental dynamic analysis are compared. Results for steel MRFs refer to both the 2008 version [65] and the 2018 version [20] of the Italian Code. In the same figures, the code-prescribed values of the behaviour factor (i.e., \(q_d = 5.85\) for RC frames and \(q_d = 6.50\) for steel MRFs) are plotted. The results show that the pushover analysis provides much higher values of the behaviour factor for RC frames compared to steel MRFs. On the contrary, the incremental dynamic analysis shows slightly lower values for RC frames than for steel MRFs as happens, moreover, for the code-prescribed values. The 9S3B steel MRFs designed with NTC-2008 [65] have a calculated behaviour factor of 5.54 which is lower than the prefixed value of 6.50 for MRFs. On the contrary, the steel MRFs designed with NTC-2018 [20] show
estimated behaviour factors that are always higher than the code-prescribed value. This results from the local ductility criterion of equation (8) introduced in the 2018 version of the Italian Code [20]. On the other side, the calculated behaviour factor of the 7S3B RC frame is 5.16 which is lower than the code-prescribed value of 5.85. Thus, in this case, the design procedure is nonconservative. To overcome this problem, additional local ductility criteria and/or structural detailing should be provided to implicitly ensure that the columns exhibit a more ductile behaviour and the recommended value of the behaviour factor is conservative.

6. Conclusions

Based on the analytical results, the following conclusions can be drawn:

(i) The results from pushover analysis seem to highlight that both European and Italian standards are too conservative. The overstrength and redundancy factors are generally higher than the code-prescribed values. The overstrength factors are found to be between 1.10 and 1.51, while the redundancy reduction factors range between 1.58 and 2.24. The numbers of stories and bays seem to influence the response reduction factors. The redundancy response modification factor increases with the number of bays, while it decreases with the number of stories. The ductility reduction factor $R_q$ increases with both the number of stories and the number of bays. The $q$-factor decreases with increasing the number of stories, whereas the height of the structure is not taken into account by the seismic design codes.

(ii) Compared to more refined nonlinear incremental dynamic analyses, both pushover analysis and capacity spectrum method are shown to greatly overestimate the calculated behavior factor. On the contrary, the formula proposed by both European and Italian standards seems to provide more accurate values if the redundancy factor $a_d/a_1$ is calculated using pushover analysis.

(iii) The value of the behaviour factor prescribed by the European and Italian seismic codes may be overestimated, especially for middle-high-rise frame structures. In the case studies examined, this happens for the 7S3B frame structure (where the prescribed value of 5.85 is higher than the calculated value of 5.16) due to an unfavourable premature collapse mechanism based on column plastic hinging at the first storey.

(iv) The behaviour factor calculated from IDA for steel MRFs is slightly higher than that for RC frames, as is the case for the code-prescribed values. The code-prescribed value of the behaviour factor for steel MRFs is always conservative, since the limitation on the dimensionless axial force proposed by Ferraioli et al. [64] has been introduced in the current version of the Italian Code [20]. On the contrary, the code-prescribed value may be nonconservative for middle-high-rise RC frames due to plastic hinging of the first storey columns at the base and lack of sufficient local ductility for the increased deformation demand associated with the soft storey. In this case, some changes to the standards are desirable in local ductility criteria and/or structural detailing of high ductility frame structures to implicitly ensure that the inelastic response of the structure is consistent with the prefixed behaviour factor.

Finally, it should be highlighted that this study is limited to regular in plan and elevation low/middle-rise buildings where both torsional and higher mode effects are negligible. More research is needed for high rise and irregular buildings. Moreover, a wider set of geometrical parameters such as unequal span lengths and interstorey heights should be considered.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The author declares that there are no conflicts of interest.

Acknowledgments

The research described in this paper was supported by the DPC/RELUIS Project 2019-2021 – WP12: “Contribution to standards for steel and composite steel-concrete structures for civil and industrial buildings,” funded by the Italian Department of Civil Protection (DPC).

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Advances in Civil Engineering 17

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