# Behaviour Factor for seismic design of moment-resisting steel frames

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**SUMMARY**
Existing seismic codes are based on force-controlled design or capacity design, using the base shear concept. The most important parameter in this approach is the behaviour q-factor, which is based on the maximum capacity of structure to dissipate energy during the plastic deformations corresponding to ultimate limit state criterion. In this paper, the existing methods for determining the behaviour factor of moment-resisting steel frames are reviewed for both regular and irregular in elevation multi-storey moment-resisting steel frames. The effects of storeys, spans and regularity in elevation of frames on the behaviour factor were considered.

*Keywords: Moment resisting frames, behaviour factor, nonlinear analysis.*

## 1. INTRODUCTION

The main limit of traditional force-based design approach is that the performance cannot be predicted because the seismic behaviour of the structure is governed by phenomena which are not adequately captured in the simple design process. In fact, during the occurrence of an earthquake ground motion it is possible for steel moment-resisting frames to enter a region of non-linear behaviour. However, the nonlinear dynamic analysis requires step-by-step integration, which is very time consuming. As a consequence, the current design codes calculate the design force to be used for elastic analysis of structures from spectra based on linear behaviour together with the use of a behaviour factor that accounts approximately for the non-linear effects. In other words, an approximate inelastic spectrum is defined for specifying design actions of structures which are expected to respond inelastically to the design earthquake. Even if the distribution of forces resulting from such analyses may have little similarity to that expected during the actual earthquake, the concept of a factor used in design to reduce forces is adopted by most seismic codes in order to account for the nonlinear response of the structure associated with the material, the structural system and the design procedures. This factor is called behaviour factor ($q$-factor) in the European Code (2004) and response modification factor ($R$) in the American Codes (Uniform Building Code (1997), NEHRP Provisions (2003)). In SEOAC Guidelines (1999) $R$ is termed "the structural quality factor" or "the system performance factor". Some considerable differences in the numerical values of the behaviour factors specified in various codes for the same type of structure may be found. These discrepancies also derive from different partial safety factors used in each code for material resistances and applied loads.

## 2. FORCE REDUCTION FACTORS FOR SEISMIC DESIGN

Although the inelastic spectra are rigorously applicable to the inelastic behaviour of single-degree-of-freedom (SDOF) systems, these spectra are usually applied with satisfactory accuracy to multi-degree-of-freedom (MDOF) structures. It is also possible to use the design spectra - that represents a scaled down form of elastic pseudo-acceleration spectrum - to approximately estimate the response of inelastic MDOF systems. In other words, most seismic codes use the concept of design response spectrum defined by dividing the elastic response spectrum through a reduction factor. The ductility-dependent component of this behaviour factor is generally estimated on the basis of corresponding inelastic spectra. The overstrength-dependent component of $q$ is connected to the design procedure, and is generally estimated with static and dynamic inelastic procedures. ATC-63 (2008) introduces a separate factor relating to the structure's redundancy that is generally difficult to separate from the overstrength factor. The NEHRP Provisions define an empirical response modification ($R$ factor) to account for both damping and ductility inherent in a structural system at a displacement great enough to approach the maximum displacements of the system. The Eurocode 8 (2004) definition of the behaviour factor for steel structures explicitly accounts for the effects of ductility and redundancy, and for the effect of member overstrength. The reference behaviour factors assigned to steel moment resisting frames in EC8 are 4 and $5\alpha_u/\alpha_1$ for ductility classes medium (DCM) and high (DCH), respectively. The multiplier $\alpha_u/\alpha_1$ depends on the ultimate-to-first plasticity resistance ratio, which is related to the redundancy of the structure. A realistic estimate of this value may be determined from pushover analysis, but should not exceed 1.6. In the absence of a detailed evaluation, the approximate values recommended by EC8 are 1.1 for portal frames, 1.2 for single-bay multi-storey frames and 1.3 for multi-bay multi-storey frames. For regular structures in areas of low seismicity, a $q$=1.5÷2.0 may be adopted without applying dissipative design procedures, recognizing the presence of a minimal level of inherent overstrength and ductility. In this case, the structure would be classified as a low ductility class (DCL) for which global elastic analysis can be utilized, and the resistance of members and connections may be evaluated according to Eurocode 3 (2003) without any additional requirements. The values of the behaviour factors adopted in American codes (NEHRP and UBC) presuppose the existence of significant amounts of overstrength in the structures, which however can be relied upon without any check, as opposed to the Eurocode 8 procedure for steel structures. However, a direct code comparison between EC8 and US provisions is not consistent if only the level of force reduction is considered. For example, the suggested reduction factor $R$ in US provisions for regular structures with no specific ductility considerations is 3.0, which is again larger than the equivalent values in EC8. In the same way, the behaviour factor for high-ductility reinforced concrete frames is equal to 8.5 in UBC, but only 5 in EC8. A reliable comparison should involve not only the reduction factor but also the full design procedure. Since seismic design forces have a direct relation to the value adopted for the behaviour factor, a large number of studies have been performed over the years to assess this parameter. Maheri and Akbari (2003) investigated the behaviour factors of steel-braced reinforced concrete framed dual systems. Kappos (1999) focuses on the evaluation of behaviour factors for seismic design of structures, with due consideration to both their ductility and overstrength. Fathi et al. (2006) reviewed the existing methods for determining the behaviour factor of moment-resisting steel frames and their range of applicability. Costa et al. (2010) proposed a probabilistic methodology for the calibration of the $q$-factor. Lee et al.(1999) determined the ductility factor considering different hysteretic models.

## 3. BEHAVIOUR FACTOR METHODS

In the force based seismic design, the force is extracted from spectra based on linear behaviour together with the use of a reduction factor that modifies the linear system to an equivalent one to account approximately for the nonlinear effects. This force reduction factor or response modification factor (often called $q$-factor or $R$-factor) has an important role in the estimation of design force of a structure. Its value depends on the parameters that directly affect the energy dissipation capacity of the structure: ductility, added viscous damping and strength reserves coming from its redundancy and the overstrength of individual members. An appropriate definition of the $R$-factor is based on a ductility-dependent component, an overstrength-dependent component, and a damping dependent component:

$$R = R_S \cdot R_\mu \cdot R_\xi \tag{3.1}$$

In Eqn.3.1 $R_S$ is a strength reduction factor defined as the ratio between the real lateral strength of the structure and the design lateral strength. The ductility reduction factor $R_\mu$ is the ratio of the minimum lateral strength required to avoid yielding in the given inelastic SDOF system, to the minimum strength required to limit the inelastic deformations to the ductility demand ratio, $\mu$, when this system

is subjected to a given ground motion at its base. The damping reduction factor $R_\xi$ is typically set equal to 1.0, as in general is assumed the same damping ratio for the linear and nonlinear responses.
The various components of $R$ factor presented in Eqn. 3.1 have been extensively discussed in literature. In particular, the ductility dependent component $R_\mu$ has received considerable attention. This ductility factor is a measure of the global inelastic response of the structure and is expressed as a function of the displacement ductility. The relations proposed in literature are based on studies on single-degree-of-freedom (SDOF) systems subjected to different input ground motion. Newmark and Hall (1982) proposed a formulation based on statistical data analyses. Krawinkler and Nassar (1992) developed a relationship for SDOF systems on rock or stiff soil sites. Miranda and Bertero (1994) developed relationships for rock, alluvium, and soft soil sites using 124 recorded ground motions. Lam et al.(1998) studied the relationship between the ductility reduction factor and the ductility demand with a probability of exceedance approach. Ordaz et al. (1998) proposed a new rule to estimate the reduction factor based only on displacement elastic spectra.
The strength factor $R_S$ consider that the real lateral strength is greater than the design lateral strength because components are designed with capacities greater than the design actions, material strength exceed nominal strength, and because drift limitation and detailing requirements generally determinate an overstrength of structural members. This factor is generally expressed as follows:

$$R_S = R_\rho \cdot R_\Omega \tag{3.2}$$

where the redundancy factor $R_\rho$ is the ratio between the seismic action intensity at the development of a plastic collapse mechanism and that one corresponding to first yielding in the structure. The overstrength factor $R_\Omega$ is the ratio between the seismic action intensity corresponding to the formation of the first plastic hinge to that corresponding to the allowable stress state for the frame as defined by conventional design codes. The redundancy factor $R_\rho$ depends on the plastic redistribution capacity of the structure that is a function of structural type and redundancy. A redundant seismic framing system is composed of multiple vertical lines of framing, each designed and detailed to transfer earthquake-induced inertial forces to the foundation. The overstrength factor $R_\Omega$ is affected by the design method, the seismic zone and the local construction practices. In particular, the partial safety factors for materials and loadings and the verification method greatly influence the value of $R_\Omega$. Furthermore, this factor is very sensitive to ratio of gravity loads to seismic loads, resulting in greater reserve of strength in lower seismic zones. In the case of assessment using pushover analysis the relation expressing the global behaviour of the structure is the capacity curve, that is a base shear versus roof displacement relation obtained under monotonic increasing lateral loads. The lateral force distribution during pushover analysis should be defined to reproduce the inertia forces deriving from the earthquake ground motion. Since such forces depend on the response history of the building, the lateral load pattern should be modified during the analysis as an effect of structural yielding. In fact, as the damage progresses, the inertia forces are redistributed and the vibration properties of the structure change. This approaches can give better estimations of the inelastic response, but is conceptually complicated and computationally demanding for application in structural engineering practice. As an alternative, an invariant load pattern is generally allowed in code provisions for seismic assessment. In other words, the capacity of the structure is calculated in the hypothesis that the vibration properties remain unchanged in spite of structural yielding.
The force-displacement response curve obtained from pushover analysis is generally idealized by a bilinear elastic–perfectly plastic response curve, and Eqn. 3.1 may be expressed as follows:

$$R = \frac{V_e}{V_d} = \frac{V_e}{V_y} \cdot \frac{V_y}{V_1} \cdot \frac{V_1}{V_d} = R_\mu \cdot R_\rho \cdot R_\Omega \tag{3.3}$$

where $V_e$, $V_y$, $V_1$ and $V_d$ correspond to the structure’s elastic response strength, the idealized yield strength, the first significant yield strength and the allowable stress design strength, respectively.
The European definition of the behaviour factor for steel structures is

$$q = q_0 \cdot \alpha_u / \alpha_1 \tag{3.4}$$

where $q_0$ is the basic value of the behaviour factor, $\alpha_u$ represents the strength at the development of a plastic collapse mechanism, and $\alpha_1$ the force corresponding to first yielding in the structure. The comparison of Eqns. 3.3-3.4 reveal that $\alpha_u/\alpha_1$ represents the redundancy factor $R_\rho$, and $q_0=R_f=R_\mu R_\Omega$ is the structural response modification factor.
The evaluation of behaviour factor may be carried out with different methods:

1) Methods based on ductility factor theory (Newmark & Hall, 1973);
2) Methods based on dynamic inelastic response of single-degree-of-freedom systems (Ballio, 1985; Giuffrè et al.1983; Krawinkler et al.1992);
3) Methods based on an energy approach (Como & Lanni, 1979; Kato & Akiyama, 1980);
4) Methods based on the accumulation of damage (Ballio et al.1994; Castiglioni et al.2010).

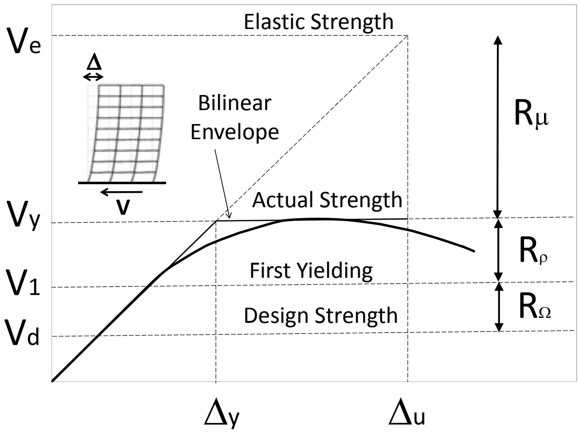


**Figure 3.1.** Base shear versus roof displacement relationship

## 4. ESTIMATION OF BEHAVIOUR FACTORS FOR MOMENT RESISTING FRAMES

In this paper the estimation of behaviour factors is carried out with respect to realistic code-designed moment resisting frames. Three different methods are used: 1) Static Approach; 2) Dynamic approach; 3) Mixed Approach. In the Static Approach the estimation is carried out from the force-displacement response curve obtained from pushover analysis with the formulation of Eqn. 3.3. In particular, two invariant load patterns are considered: 1) First Mode Distribution (FMD); 2) Uniform Distribution (UD). The bilinear representation of the capacity spectrum (BCS) is developed such that the elastic stiffness intersects the capacity curve at 60% of the yield base shear and the area under the capacity spectrum and the bilinear representation is the same. In the Dynamic Approach the behaviour factor is estimated from the Incremental Nonlinear Dynamic Analysis (IDA) of the moment resisting frame. The IDA consists in performing a series of nonlinear time-history analysis, using an acceleration input ground motion scaled to increasing amplitudes. The $q$-factor is defined as follows:

$$q = \frac{PGA_u}{PGA_d} \tag{4.1}$$

where $PGA_u$ is the peak ground acceleration at collapse and $PGA_d$ is the peak ground acceleration corresponding to first design yielding. In the Mixed Approach the behaviour factor is based on two components, the first one estimated with nonlinear dynamic analysis, the second one estimated with nonlinear pushover analysis, as follows:

$$q = \frac{PGA_u}{PGA_1} \cdot \frac{V_1}{V_d} \tag{4.2}$$

where $PGA_1$ and $V_1$ are the peak ground acceleration and the base shear corresponding to first yielding in the structure obtained, respectively, with nonlinear dynamic and nonlinear static analyses. The case studies are selected to be representative of regular and irregular in elevation steel moment resisting frames (Figs. 4.1-4.2). The frames are designed according the Italian Code Provisions (NTC08). The design seismic action is defined by soil class A, damping ratio $\xi$=5%, peak ground acceleration $PGA$=0.25$g$, behaviour factor $q$=6.5 for regular structures and $q$=6.5×0.80=5.2 for irregular structures. Steel members are made from Italian S275 ($f_y$=275 MPa). The interstorey height is 3.5m for the first floor and 3.0m for the other floors. The bay length is 5.00 m. A plastic hinge model implemented in SAP2000 nonlinear computer program is considered in the analyses. Sources of geometrical nonlinearity considered are both local and global. A bilinear model with kinematic strain-hardening of 0.5% is used for steel. According to FEMA 356 the modelling of nodal panel is avoided in the hypothesis that: 1) the expected shear strength of panel zones exceeds the flexural strength of the beams at a beam-column connection; 2) the stiffness of the panel zone is at least 10 times larger than the flexural stiffness of the beam. The ultimate plastic rotation $\theta_u$ is defined with two different formulations: a) $\theta_u$=0.03 rad; b) Ultimate plastic rotations defined according to FEMA 356 as a function of geometric and mechanical characteristics of steel members. The incremental dynamic analysis technique is used to evaluate the $q$-factor according to Dynamic Method and Mixed Method. At this aim, a set of 12 input ground motions are selected to be consistent to 5%-damped EC8 elastic spectrum for soil class A. In Fig.4.3 the envelope values of the spectral acceleration are compared to EC8 elastic response spectra. In Fig.4.4 the comparison between pushover analysis and Incremental Dynamic Analysis (IDA) is shown. Each IDA point is defined from the maximum total drift (maximum displacement/height) during dynamic analysis and the corresponding base shear. A good agreement between static and dynamic analysis is found. In Fig.4.5 the overstrength reduction factor $R_\Omega$ obtained with the Static Approach is plotted. Except in the case of irregular 5-storey building under a uniform lateral force distribution, the $R_\Omega$ factor is little sensitive to the lateral load pattern used during pushover analysis. On the contrary, $R_\Omega$ is greatly influenced by the number of stories of structure. In particular, the higher values are obtained for low-rise buildings that have greater reserve of strength because the ratio of gravity loads to seismic loads is very high.


**Figure 4.1.** Study cases: regular in elevation moment resisting frame structures.

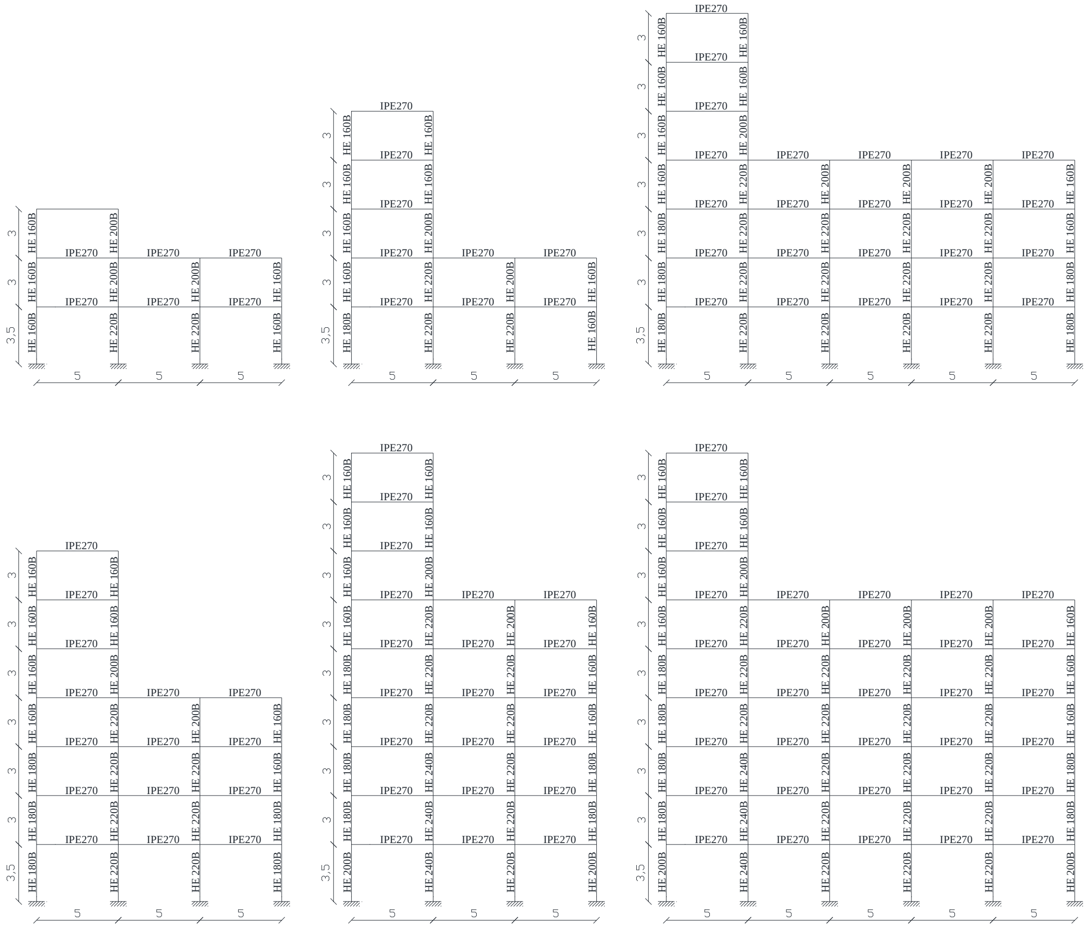

**Figure 4.2.** Study cases: irregular in elevation moment resisting frame structures.

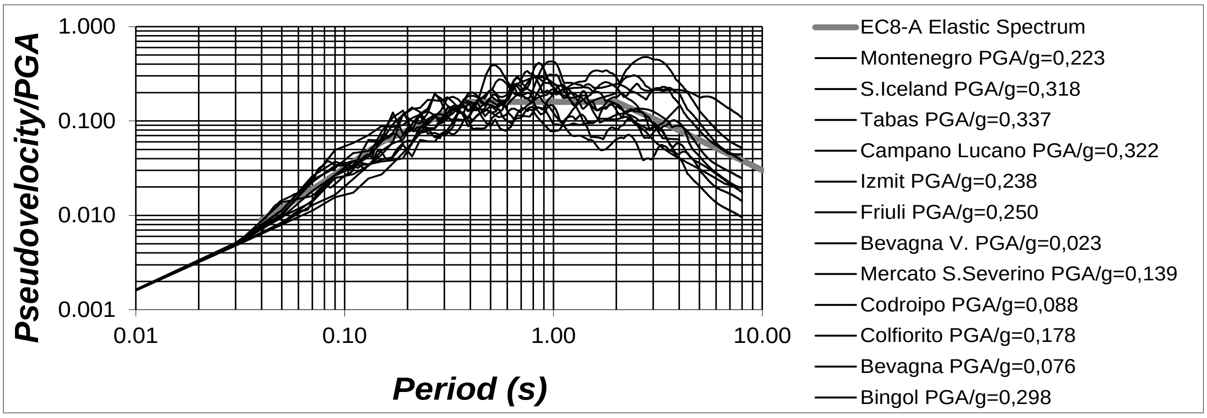


**Figure 4.3.** Ground motion response spectra and Eurocode 8 elastic response spectrum

In Fig.4.6 the redundancy factor $R_\rho$ is plotted. The value of $R_\rho$ is almost constant for the study cases considered that are all multi-bay multi-story frames with very similar redundancy and plastic redistribution capacity. The mean value of the redundancy factor is $R_\rho$=1.64 for both regular and irregular moment resisting frames. This value is greater than the value $R_\rho$=1.3 recommended by EC8 for multi-bay multi-story frames. In Fig.4.7 the ductility reduction factor $R_\mu$ is plotted. In case of irregular moment resisting frames pushover analysis reveals less ductile plastic hinge mechanisms deriving from concentration of plastic deformations. The higher values are obtained for low-rise buildings that generally develop global plastic failure mechanisms. In Figs.4.8 and 4.9 the behavior factor obtained, respectively, with ultimate plastic rotation $\theta_u$=0.03 rad and with plastic rotation defined according to FEMA 356 are reported.

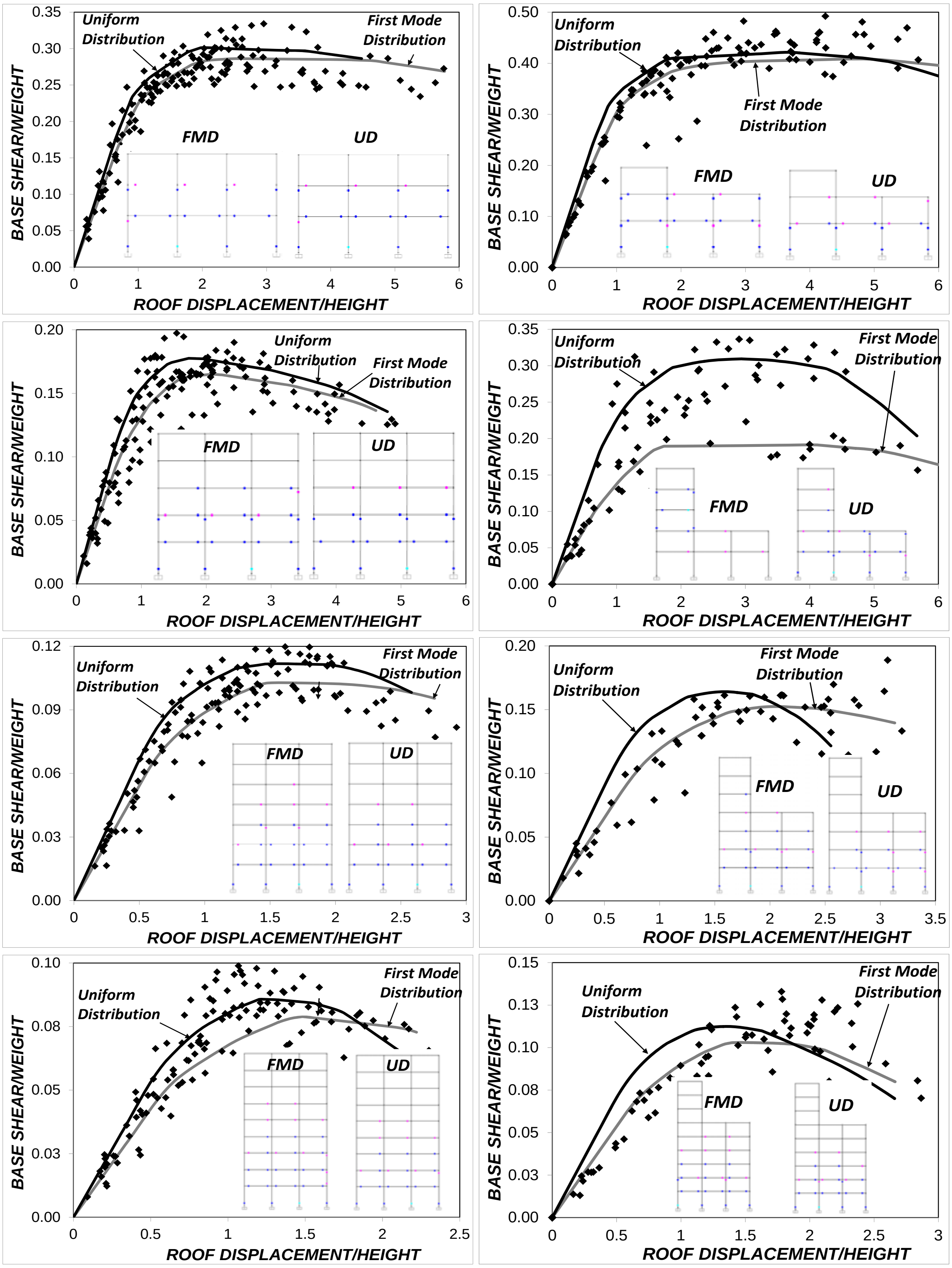


**Figure 4.4.** Base shear versus roof displacement. Pushover Analysis versus Incremental Dynamic Analysis

In particular, the results obtained with Static Approach, Dynamic Approach and Mixed Approach are compared. In each figure also the behavior factor of Italian Code Provisions ($q$=6.5 for regular structures and $q$=5.2 for irregular structures) is reported. In general, the Static Approach tends to understimate the behaviour factor if compared with the other approaches. In some cases the calculated behaviour factor is less than the code-specified value. In other words, the behaviour factor suggested by the Italian Code to accounts approximately for the non-linear effects may be not conservative with respect to the real inelastic behaviour of the structure. This result occurs for the more high-rise frames (regular 7-storey and 9-storey frames; irregular 9-storey frame) especially when static analysis is used. In these cases the compression failure of a first-story column limits the ultimate displacement capacity of the structure. This failure derives from the effect of axial force that reduces the plastic moment capacity of the first-story columns. In order to evaluate this effect in Fig.4.10 the FEMA plastic hinge rotation capacity of the external first-story columns is plotted as a function of the ratio of the axial force to the plastic axial force. For the 3-storey and the 5-storey moment resisting frames the value $\theta_u$=0.03 rad is lower than the plastic rotation estimated accounting for the axial force-bending moment interaction plastic behavior. For the 7-storey and 9-storey frames the increase of axial force in the external first-story column gives a dimensionless axial force greater than 0.30. As a consequence, the ultimate plastic rotation is significantly lower than 0.03 rad and the ductility of the column decreases accordingly. On the basis of these results, some modifications seem to be required to improve the inelastic behaviour of the structure and increase the coherence between the estimated behaviour factor and the value suggested in Italian Code. In this paper, a local ductility criterion is proposed to avoid the poor ductility properties exhibited by the columns with high axial loading. In particular, the design of columns is carried out with the following limitation on the dimensionless axial force: $N/N_{PL}$<0.3. In Fig.4.11 the behaviour factors for the frames designed with and without the aforementioned local ductility criterion are compared. The results show that the limitation of axial force improve the inelastic behaviour of the structure and the coherence between estimated and initially adopted $q$-factor.

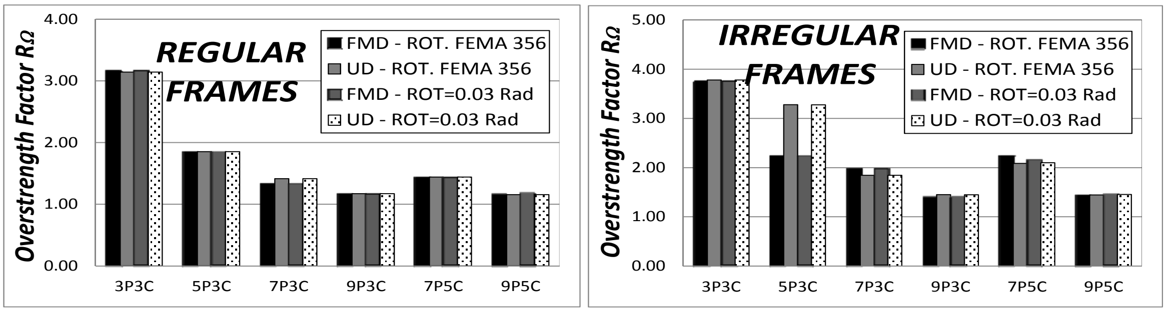


**Figure 4.5.** Overstrength Reduction factor $R_{\Omega}$

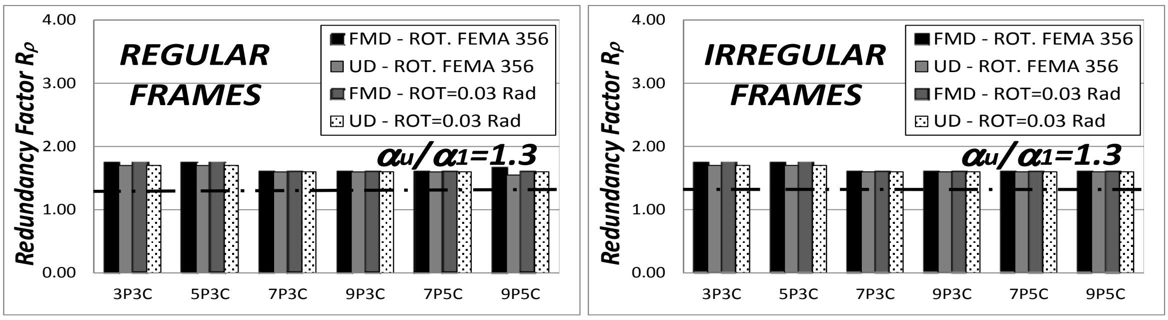


**Figure 4.6.** Redundancy Reduction factor $R_{\rho}$

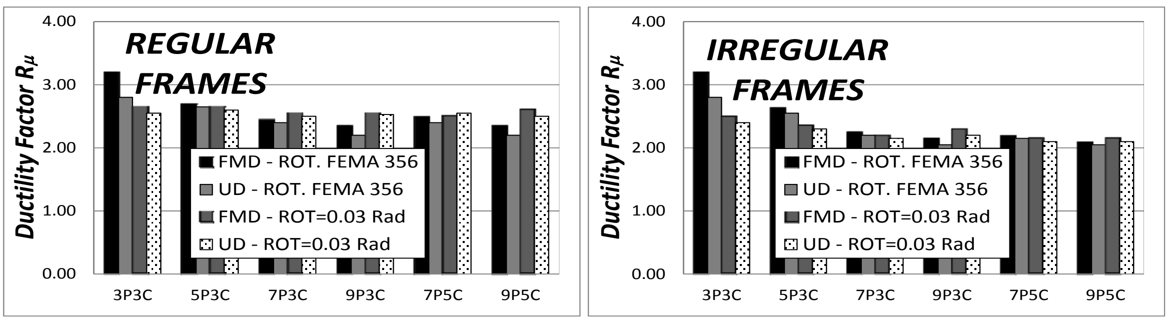


**Figure 4.7.** Ductility Reduction factor $R_{\mu}$

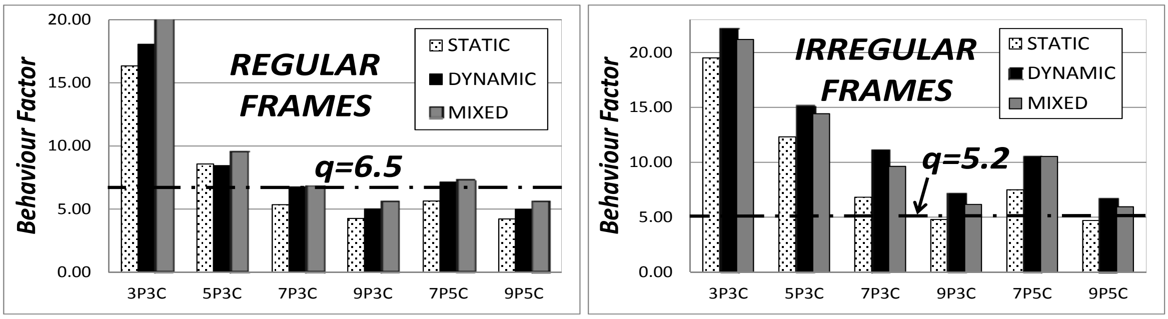


**Figure 4.8.** Behaviour Factor. Plastic rotation defined according to FEMA

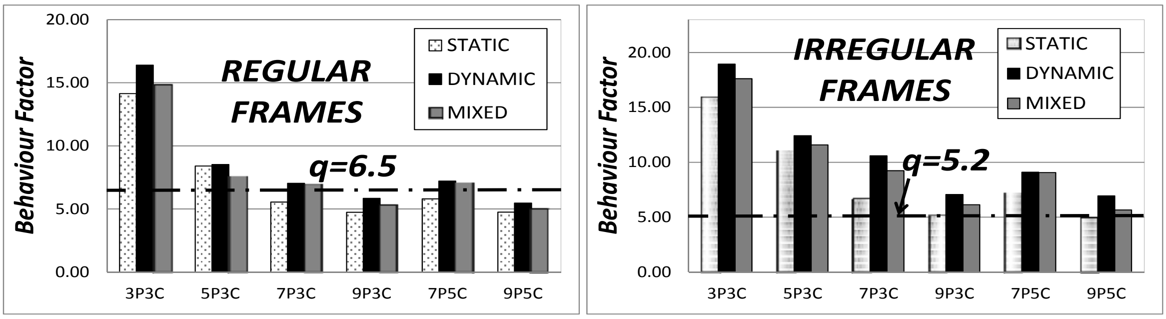


**Figure 4.9.** Behaviour Factor. Ultimate plastic rotation $\theta_u$=0.03 rad

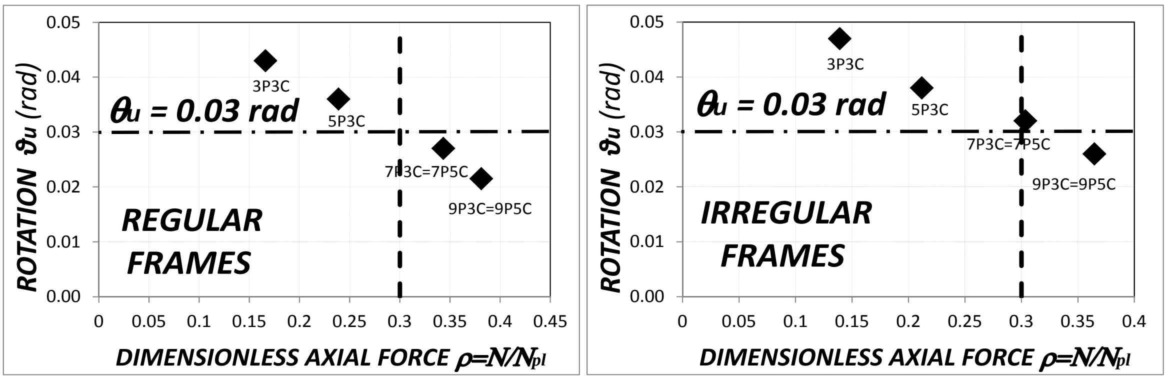


**Figure 4.10.** Ultimate plastic rotation in base columns

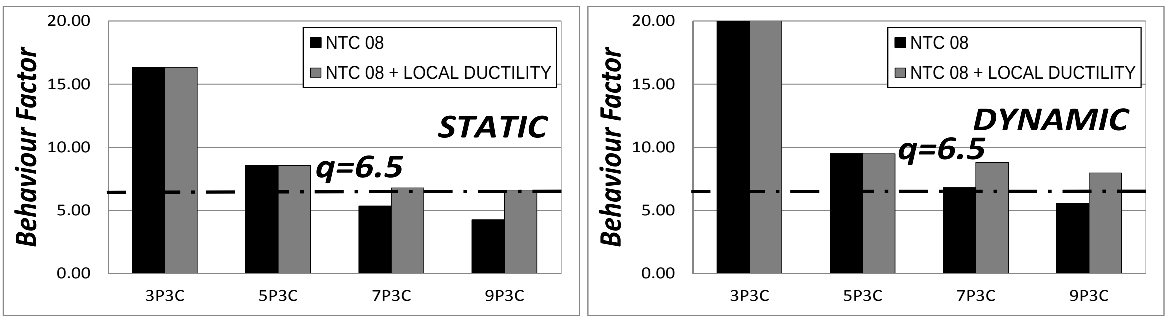


**Figure 4.11.** Behaviour Factor for frames designed with and without the local ductility criterion ($N/N_{PL}$<0.3)

## 5. CONCLUSIONS

In this paper, the behaviour factor that relates the real nonlinear dynamic response to simplified linear design response of moment-resisting steel frames is investigated. Results obtained show that the overstrength reduction factor recommended by EC8 and Italian Code for multi-bay multi-story frames is conservative. On the contrary, the structural response modification factor and, consequently, the behaviour factor proposed by these codes may be not conservative. This result derives from the effect of axial force that reduces the plastic moment capacity of the first-story columns in more high-rise steel frames. In these cases, the compression failure of a first-story column limits the ultimate displacement capacity of the structure. On the basis of these results, a local ductility criterion based on a limit of the axial force ratio is proposed to control the ductility of columns and so ensure that the recommended behaviour factor is conservative.

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