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COMPARATIVE STUDY ON BEHAVIOUR FACTOR FOR COMPOSITE AND BARE STEEL FRAMES

One of the basic parameters for dynamic analysis of structures according to modern design regulations is the behavior factor, i.e. the q –factor. For different types of structural system configurations and ductility classes, in Eurocode 8, upper limits for this parameter are given. When it comes to Composite Steel-Concrete Frames, results in this area are relatively limited.

In this paper, using a static non-linear analysis, an evaluation of the q –factor has been performed for different types of configurations of Composite Steel-Concrete Moment Frames (Composite Frames). The individual components that make up the q-factor are determined and their variations are interpreted depending on the frame configuration. The influence of specific parameters, such as the span and number of stories of the frame, and member local characteristics are studied. The geometrical and material nonlinearity of composite cross sections are taken into account with the concept of distributed plasticity. The obtained results are compared with the proposed values for Bare Steel Moment Frames.

Keywords: behavior factor, steel-concrete moment frames, steel frames, nonlinear static analysis.

1. INTRODUCTION AND MAIN CONCEPTS

While analyzing structures under the influence of seismic dynamic loads, one of the main difficulties is to describe the behavior of the system outside the elastic region. Seismic codes, such as Eurocode 8 [7], ASCE [11], etc., recommend the use of simplified methods based on elastic linear analysis. The philosophy of treating structures affected by seismic inertial forces is based on the following formulation: Seismic forces obtained from the elastic response spectrum are reduced at the cost of the dissipative capacity of the structure [12]. In other words, part of the seismic forces is received by the construction with elastic behavior, while with the remaining part of the seismic force, the structure behaves non-linearly. To what extent the construction can behave linearly (and non-linearly, respectively), in principle depends on many parameters. Namely, all parameters are accumulated and (to a certain degree of precision) described by the q-factor. The recommended values of this factor are used, which are attached in Eurocode 8[14], Table 6.2. The tabular values for the q-factor are given for steel frames, and there are no additional provisions when instead of bare steel beam, a composite one is chosen. In this paper, during the analysis and quantification of the q-factor, the following parameters are considered: the number of levels and the span of the frame; the "column/beam" capacity ratio (the "strong columns/weak beams" criterion); the plastic rotation capacity of the columns; the ratio of permanent to variable loads (N/Np). This principle is used in the analysis of both the composite and bare steel moment frames. The obtained values are compared with those recommended by Eurocode 8, so the possible consequences of this choice of the q-factor are subject to discussion.

2. METHODOLOGY OF CALCULATING THE BEHAVIOUR FACTOR

The determination of the q-factor - for both steel and the composite frames - a Pushover analysis was carried out, with a monotonically incremental increase in the equivalent seismic horizontal force, with the gravity loads (in the seismic combination) being constantly present during the entire procedure. In this paper, the horizontal load is applied with a triangular shape, while the analysis is carried out with the SeismoStruct 2018 software package.

The segment bounded by points O-B describes the linear-elastic behavior, up to the limit of occurrence of the first plastic hinge (point B). The corresponding displacement at the occurrence of the first plastic hinge is denoted with Δ_y, it marks the beginning of the second phase of the monotonically increasing F-Δ curve. This part of the curve is generated as a result of the plastic capacity for the redistribution of internal forces, until reaching the ultimate state (point C), at a corresponding displacement Δ_{F_u}.The part bounded by the C-E curve is called the softening branch [9] and it dependent on the type of fracture mechanism and the intensity of the vertical loads. To determine the q-factor, the formulation according to [1] is used in this paper. The q-factor is determined as a product of the three parameters that are responsible for the dynamic behavior of a certain structural system, that is: R_Ω –design reserve strength; R_μ –ductility factor and R_ρ –redundancy factor.

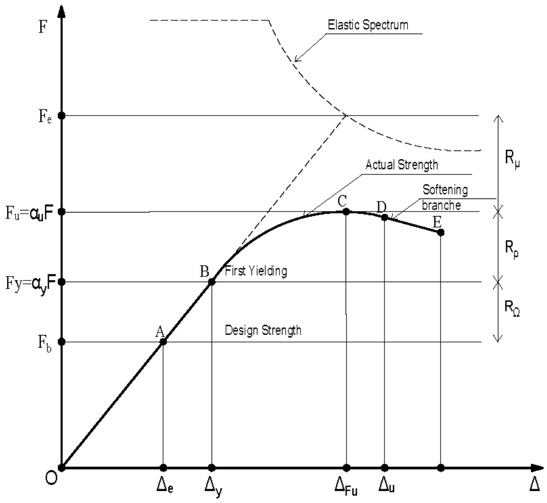

Figure 1. Typical pushover curve [1].

2.1 PERFORMANCE LIMITS ACCORDING TO FEMA 356

The nonlinear behavior of plastic hinges is described according to FEMA 356 criteria [3]. For this purpose, the force-deformation dependence is used, as in Figure 2.

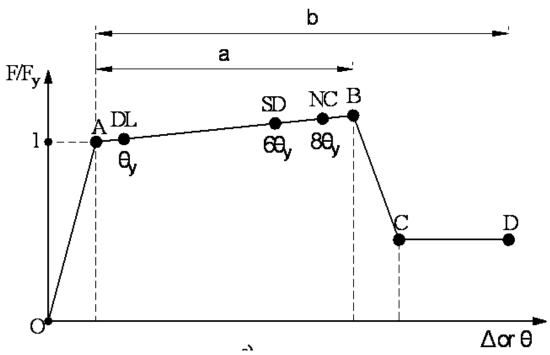

Figure 2. Performance curve as per FEMA356 [3].

Right next to point A, the limit DL - damage limitation is given, which corresponds to the occurrence of yielding in the beam element. The line AB depends on the material characteristics of the element and usually represents 10% of the slope of the line OA [3]. The point SD - corresponds to Significant Damage to the elements and is quantified by $6\theta_y$. The point NC-Near Collapse, represents a state close to failure of the element and it, according to Table 6.25, FEMA 356 [3], is quantified as: $8\theta_y$. It is evident that the value of the q-factor also depends on the plastic capacity of rotation of the elements. According to Eurocode 8, the criteria for the plastic rotation

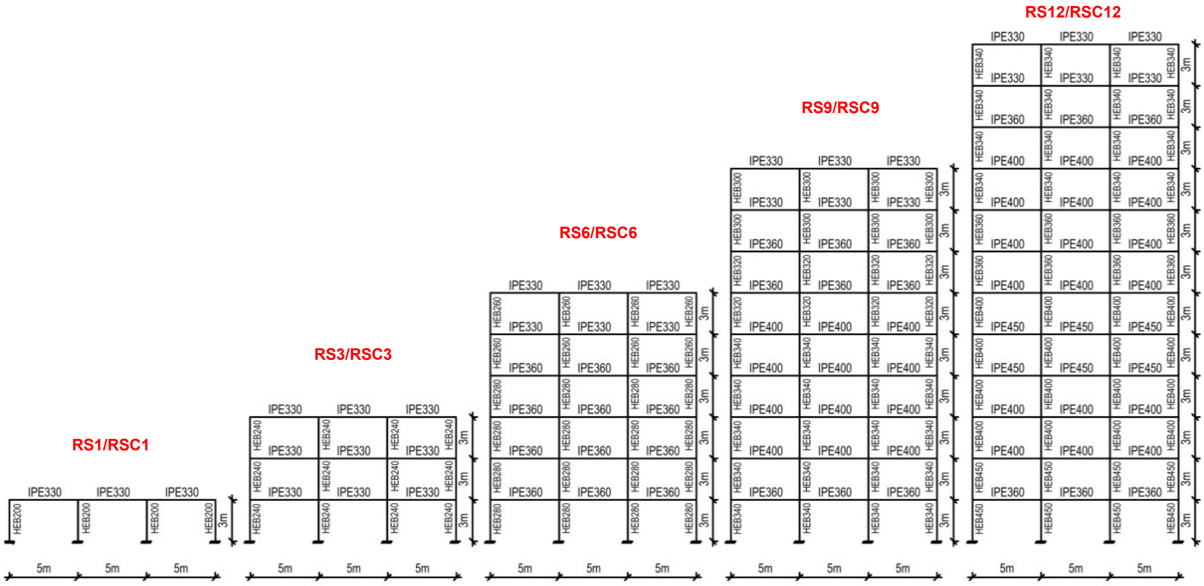

Figure 3. The studied frames.

capacity of the elements, which can define the ultimate capacity of the system, are not explicitly mentioned.

The ultimate capacity of rotation of the columns of the ground floor corresponding to the NC state, i.e. the maximum plastic rotation with a value of $8\theta_y$, has been chosen as the limit performance for the frames of this paper. In the following, the procedure according to the provisions of FEMA356 is given to determine the rotation capacity for beams and columns, respectively. M_p−the plastic moment of the element, L_b− the length of the beam, L_c −the length of the column, I_b −the moment of inertia of the beam, I_c−the moment of inertia of the column, N−the axial force in the columns (from the seismic combination) and N_p - the plastic axial bearing capacity of the column, the corresponding parameters from relations (5) and (6) are indicated. The rotation during plasticization of the beam and column, is given by the following expressions [3]:

$$\theta_{b,y} = \frac{M_p L_p}{6EI_b};\ \theta_{c,y} = \frac{M_p L_c}{6EI_c}\left(1 - \frac{N}{N_p}\right) \qquad (1)$$

In the second group, the same frame configurations are analyzed as composite frames and they are labeled as RSC1, RSC3, RSC6, RSC9 and RSC12. A composite slab d_c = 120 mm and an effective width of 1 m was chosen. In Figure 3, the analyzed frames are shown. The constant load varies from 27-45kN/m', while the variable load has a constant value of 11 kN/m'.

2.2 MODELING OF ELEMENTS

The nonlinear characteristics of the sections are modeled with the Distributed Plasticity approach. Distributed plasticity (Distributed Plasticity), compared to the Concentrated Plasticity (Lumped Plasticity), allows the distribution of plasticity along the entire length of the element and is not limited by the calibration of the parameters depending on the examined element, [6]. It is assumed that full interaction is provided between the section elements in the concrete and steel section.

3. STUDIED FRAMES

The subject of this paper's analysis are two groups of moment frames. In the first group, a series of steel moment frames with 1, 3, 6, 9, and 12 levels are analyzed and labeled as RS1, RS3, RS6, RS9, and RS12, respectively. The analysis was carried out with an assumed PGA=0.35g, damping coefficient ξ=5%, soil category B, Type 1 of the response and assumed value of the behavior factor q=4.

The advantage of this formulation comes into consideration especially when it is necessary to model the elements with variable stiffness along the length, such as the composite beams [5], [12]. Namely, with the distributed plasticity, the integration of the non-linear geometric and material behavior is carried out at the cross-sectional level. The cross-section is discretized into a series of infinitesimal cross-sections, called fiber-sections. At the same time, an axial dependence between stresses and dilations is established on each of these fiber-sections. That is, for each of the fiber sections, the normal stress is determined individually.

The longitudinal element is discretized into a finite number of Gaussian points. To obtain the influences along the length of the element, the Gaussian points are integrated. In this way, along the length of the section, the possible positions of the formation of the plastic hinges are controlled. The number of fiber - elements for the composite frame is 200. It is assumed S235, Strain - hardening 0.005 and E_s = 210 000 N/mm^2. While for concrete: f_{ck}= 30 N/mm^2, f_{ct} = 0.001 N/mm^2, crushing strain $\epsilon_c = 0.0022$ and E_c = 32836 N/mm^2. For this method to give solid results, the element should be divided along its length. Within this paper, a division of each of the elements into 6 equal parts has been made. Each of those 6 elements, is divided into 10 points for Gaussian integration

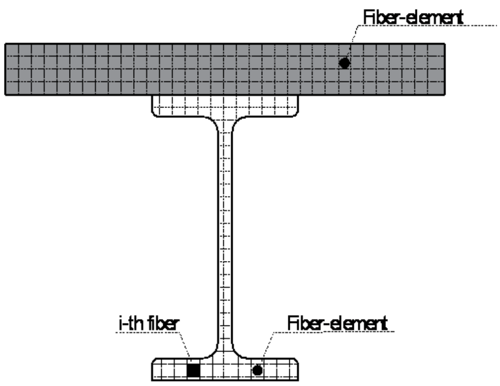

Figure 4. Discretization of the composite cross-section with "fiber" elements

4. ANALYSIS OF OBTAINED RESULTS

In this paragraph follows the presentation and discussion of the obtained results. In each of the graphs shown in Figure 5 the Pushover curves obtained from the nonlinear static analysis for determining the q-factor are shown.

In Figure 5a) it is observed that ultimate displacement corresponding to the performance limit PL (Performance Limit) ($8\theta_y=\theta_p$), is almost identical for RS1 and RSC1, respectively. The differences are evident in the displacements at the formation of the first plastic hinge, that is, the ductility factor for RS1 is 56% higher than RSC1(Figure 5a and 6c).

By increasing the number of levels, an increase of the overstrength factor ($\alpha = \mathrm{R}_\Omega \cdot \mathrm{R}_\rho$) is observed for both frames (RS3 and RSC3, 1.355 and 1.302, respectively (Figure 5b and 6c). Also, the ultimate horizontal force and the occurrence of the first plastic hinge, for RSC3 are 21.3% and 26% higher compared to the values obtained for RS3 (Figure 6c). On the other hand, the ductility factor of RS3 is 53% higher in comparison with RSC3 (Figure 6d). And in this case, the q-factor for the pure steel frame is 23% higher compared to the composite frame (Figure 6a). From Figure 6c) there is a drop again in the factor for the of RS6, with a total value of 1.17. The large value of the N/Np ratio can be considered as the reason for this decline. On the other hand, for RSC6, at the same value of N/Np, this factor is α_{sc}=1.24 (Figure 6c). There is also a decrease in ductility factor for the RS6 frame, which is 3.33 (Figure 6d). In the case of the composite frame (RSC6), there are no visible decreases in μ_{cs}. Namely, in this case, the q-factor for the steel frame is higher compared to the composite frame (Figure 6a).

From Figure 6d), a drop in the α_{sc} –factor for the of the steel frame is observed, which is α_s=1.109. This is due to the large value of the N/N$_p$ ratio, while in the composite frame, we have a visible increase in α_{sc} compared to RSC6, and a 25% increase compared to RS9. A decrease is also observed in the ductility factor of RS9, while in the case of RSC9, we have an unchanged value (Figure 6d). In this case, the q-factor of the steel frame is lower compared to the composite frame (Figure 6a). In the last group frames (RS12&RSC12), again the α –factor of the steel frame is lower in comparison to the composite frame, by 13.5% (Figure 6c). It is observed that the ductility factor for the steel frame has a value (μ_s=2.62) similar to that of the composite frame (μ_{cs}=2.74). As an implication of this, for RS12, the q-factor is 3.9 and it is also lower compared to RSC12. In the following, the course of changing several of the dynamic characteristics of the considered frames is given, by increasing the number of levels and, accordingly, by varying their stiffness characteristics.

From Figure 6a), the reduction of the q-factor can be clearly seen for both the steel and the composite frames, with an increase in the number of levels. From level 6 to level 12, the q-factor for both frame categories has similar values. The differences are more evident for lower heights.

On the other hand, from Figure 6 b), it is clearly seen that for the first group of frames (RS1 and RSC1), the relative floor displacements are

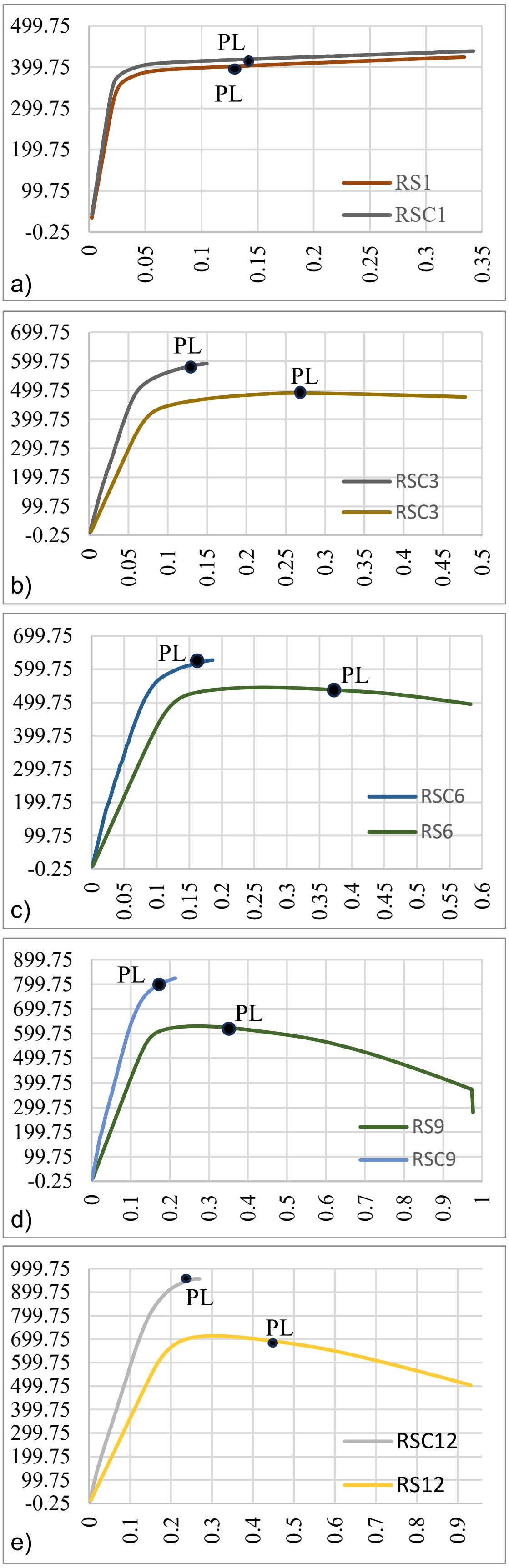

Figure 5. Comparison of Pushover curves for: a) RS1&RSC1; b) RS3&RSC3; c) RS6&RSC6; d) RS9&RSC9; e) RS12&RSC12.

almost identical. With the increase in the number of levels, in the composite frames, there is a gradual decrease in the relative storey displacements, by 18.1%, 32.2%, 43.7% and 31.2% for RSC3, RSC6, RSC9 and RSC12 (relative to RS3, RS6, RS9 and RS12), respectively.

Figure 6c), shows the overall global change in the overstrength factor for both group of frames. Except for RS3, all considered steel frames have a value of 1.17 for this parameter. Although these frames belong to the group of multi-storey and multi-bay frames, if the performance limit is defined in advance (as in this example with ultimate rotation capacity of the ground floor columns) obtained values for overstrength (α) factor would be less than 1.3.

On the other hand, for the composite frames, except for RSC1 (where α_{sc} =1.12), the values for α_{sc} gradually increase and they can be considered to be in the margins of the recommended value according to Eurocode 8, i.e. 1.3, although for them the ratio N/Np>0.25.

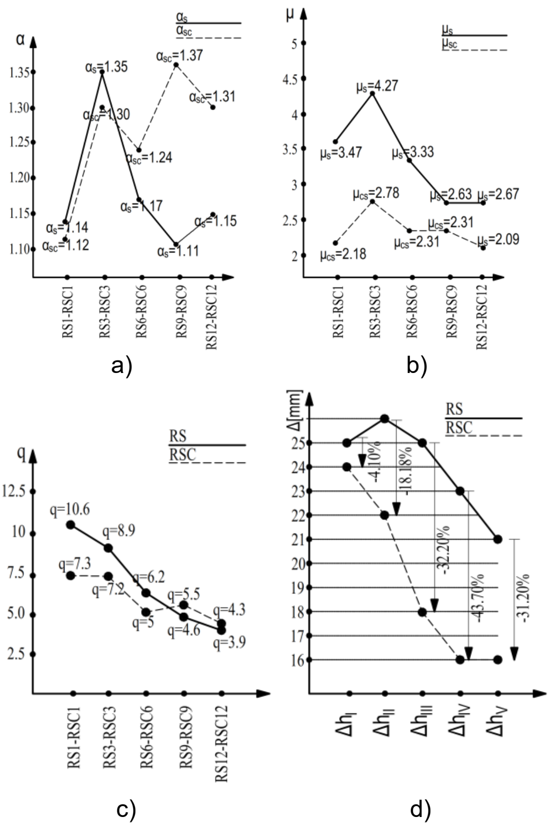

Figure 6. Graphical representation for the considered group of frames: a) behaviour factor, b) relative storey displacements, c) overstrength factor, d) ductility factor

The change in ductility factor for both frame categories is shown in Figure 6 d). In the case of steel frames, for storey 1 and storey 3, these values are relatively large (3.43 and 4.27, respectively). With increasing levels, a gradual decrease in this parameter is observed for steel frames. On the other hand, the situation with composite frames is different. Namely, there is a more constant distribution of the ductility factor and the same, regardless of the height change, ranges from 2 to 2.78.

4. CONCLUSION

In this paper, a sensitivity analysis is carried out for the two categories of moment frames, dimensioned according to the rules provided in EC3 and in EC8. The parameters that make up the q-factor are determined for both the composite and the steel frames. The influence of the effects of the composite beams, the number of frame levels, the stiffness characteristics ("Strong Columns/Weak Beams" criterion), the local behavior of the ground floor columns and the limit performance of the structures are taken into account, all in order to see the change of the behavior factor. In doing so, the following conclusions were drawn:

Composite beams have an influence on the increase of the design reserve capacity (R_Ω) and redundancy factor. (R_ρ) (up to 23%) in the composite frames compared to the pure steel frames. It is also observed that these parameters increase with an increase in the number of levels in composite frames (Figure 6, c));

Due to the fact that the frames were dimensioned by the dominant action of permanent loads, it is observed that although the ratio (N/Np)≤0.25 is not fulfilled for RSC6, RSC9 and for RSC12, still reserve capacity (R_Ω) and redundancy factor for the composite frames, is in the boundaries proposed by EC8. On the other hand, with steel frames, except for RS3 (where the criterion N/Np=0.232 is met), for all higher levels, lower value for $\alpha = R_\rho \cdot R_\Omega$ than 1.3 is obtained.

The ductility factor (R_μ) for steel frames is relatively high for RS1 and RS3, while with increasing levels, this parameter tends to decrease. In the case of composite frames, R_μ shows more even changes (Figure 6, c));

At composite frames, a reduction in horizontal relative storey displacements was also observed compared to the steel frames. For RSC6, RSC9 and RSC12, this reduction amounts to over 30%, compared to steel frames with the same number of levels.

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