Behaviour factors of MR steel frames with semi-rigid connections – a parametric study

C. Rebelo
Assistant Professor, Faculty of Sciences and Technology, University of Coimbra, Pinhal de Marrocos, Polo II, 3030 Coimbra, Portugal, email: crebelo@dec.uc.pt

L. Magalhães
Assistant, Polytechnic Institute of Castelo Branco, Av. Cidade de Zhuhai, n.º 6 - 5.º Dto, 6000 Castelo Branco, Portugal, e-mail: miguel.magalhaes@oninet.pt

ABSTRACT: In this paper same results are presented, related to the parametric study of the *q*-factor of steel moment resistant frames. The study considers semi-rigid beam-to-column joints with standard behaviour, characterized by their stiffness, resistance and deformation capacity. Three performance levels or limit states are considered, Serviceability Limit State (SLS), Damageability Limit State (DLS) and Ultimate Limit State (ULS).

1 INTRODUCTION

The structural safety check of building steel frames concerning seismic actions is usually carried out on an elastic-linear basis using design response spectra, which take into account the ductility (deformation capacity) and the energy dissipation capacity of the structure by means of the behaviour factor *q*. Adequate resistance of the structural elements (beams and columns) is required, including joints, whereas the necessary ductility to develop any post-elastic mechanism should be guaranteed.

Despite the inherent limitation of such a factor to take into account in a separate manner all the possible source and type of non-linearities that can arise from the extreme loading developed during an earthquake, it has been considered that it is very useful in design. In so far an effort has been made to clarify and identify every source of non-linearity which can have a decisive contribution to the behaviour factor, such as the type of structure, bracing, type of cross section, etc.

According to Eurocode 8 the characteristic semi-rigid behaviour of the joints in moment resisting (MR) steel frames is generally not included in their seismic behaviour. Connections in dissipative zones must have enough overstrength in order to allow for yielding of the beams, avoiding storey mechanisms and maximising the energy dissipation. However, connections exhibit, in general, semi-rigid behaviour and may contribute to the energy dissipation. Therefore they should be taken into account to the characterization of the *q*-factor.

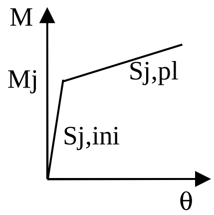
2 BEHAVIOUR FACTOR EVALUATION

2.1 *Characterization of semi-rigid connections*

The moment-rotation behaviour of the connections are modelled as a bilinear relation characterized by three parameters: initial stiffness ($S_{j,ini}$), post-elastic stiffness ($S_{j,pl}$) and maximum elastic bending moment (M_j). The parametric study used the characteristics given in table 1 for the connections, which were all combined to give 96 types of connections. Only those presented in bold were used in all the frames represented in table 3. The beams were in every case of the type IPE330 ($M_{pl,Rd}$=221kNm).

Table 1. Characteristics of the connections.

Elastic moment		Initial Stiffness	Post-elastic stiffness
Mj [KNm]		Sj,ini [KNm/rad]	Sj,pl [% of Sj,ini]
Pinned	50	$8EI_b/L = 39547$	0
	100	**$12{,}5EI_b/L = 61792{,}5$**	**0.1**
Partial resistant	**150**	**$25EI_b/L = 123585$**	**1**
	200	$37{,}5EI_b/L = 185377{,}5$	**10**
full resistant	250		
	300		



2.2 *Seismic action*

Since a non-linear dynamic time-history analysis had to be performed, accelerograms were simulated according to the specification included in Eurocode 8 (EC8) and to an adapted procedure described in Clough, 1975. Starting from a predefined time interval and a smooth power spectrum of the seismic acceleration, a time series is generated and modulated with a time function. Since the response spectrum of the resulting accelerogram will not have, in general, a form close to those given in EC8, the original power spectrum is repeatedly modified in order to generate a better approximation of the EC8 response spectrum considered.

Although an exact approximation is not possible, depending on the frequency interval chosen for the power spectrum, only a few iterations are usually needed to get a very close response spectrum, as the one presented in figure 1 for soil condition type A as defined in EC8, which is within ±5% of the EC8 response spectrum.

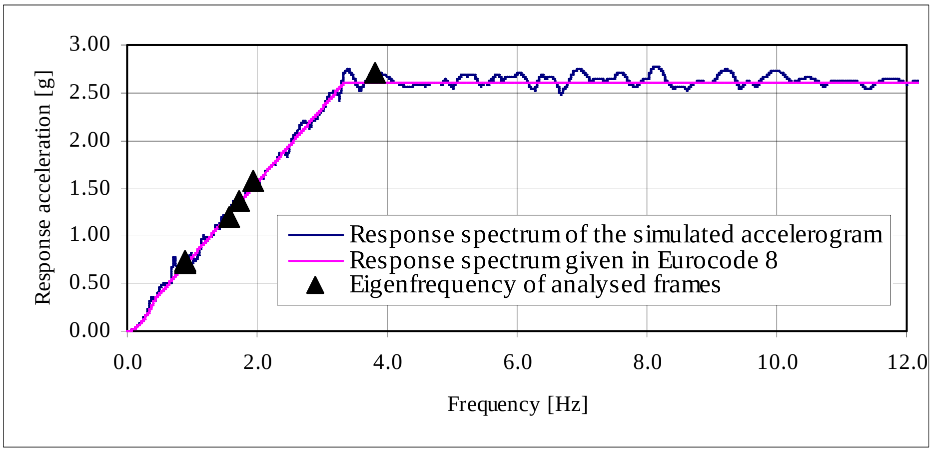


Figure 1. Comparison of response spectra for 5% damping, soil type A and 1g peak ground acceleration.

2.3 *Performance criteria*

According to Grecea, 2002,, structures designed against earthquakes have to comply several criteria related to strength, stiffness and ductility, which can be referred to conditions of interstorey drift, residual drift and rotation capacity of the connections.

Three limit states are considered in this study, in order to evaluate the corresponding behaviour factors. The Serviceability Limit State (SLS) concerns the case of low return period earthquakes (e.g. <20 years) and is quantified through a maximum allowed interstorey drift of 0.6% of the storey height. The Damageability Limit State (DLS) refers to rare earthquakes (475 years return period) and corresponds to serious structural and non structural damages, which can, however, be repaired without high costs or specially difficult repair techniques. Its quantification is based on an interstorey drift of 3% of the storey height. The Ultimate Limite State (ULS) is considered for very rare earthquakes (970 years return period). Although this limit state corresponds to very serious structural damages, it is expected that safety of people is guarantied avoiding collapse of the structure. Since large deformations are expected, the local ductility criteria are determinant for safety conditions to be verified. In this case maximum plastic rotations of 0.03rad in the connections are considered, according to AISC, 1997

The member ductility depends directly on the material ductility and on the cross section class, and it can be expressed in terms of rotational capacity. The rotation capacity can be evaluated as the ratio between the plastic rotation at the collapse state to the elastic one, according to the following formula:

$$R = \frac{\theta u}{\theta y} - 1$$

where θ_u is the ultimate plastic rotation and θ_y is the yielding rotation.

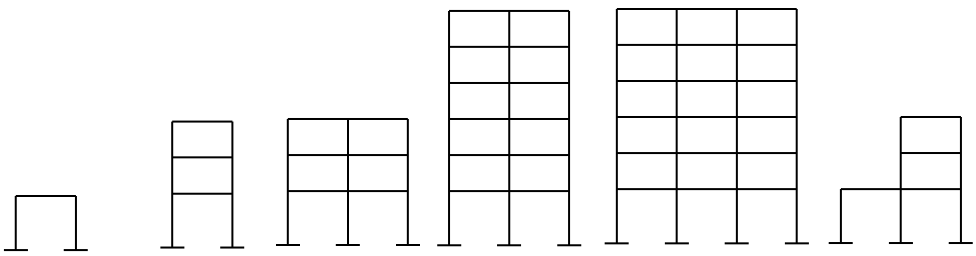
To calculate the ultimate and yielding rotation in order to compute the rotation capacity, different methods has been proposed. In this study we used the Mazzolani-Piluso semi-empirical method, recommended by ECCS, 1994. This method depends on the slenderness of the member cross-section, to take into account the lateral-torsional buckling, and on the axial force. The values obtained for the six different frame types are presented in table 3.

Table 2. Limit State Criteria for global behaviour factor evaluation.

Safety level	Plastic rotation in connections [rad]	Plastic rotation in members [rad]	Interstorey drift [% of storey height]
SLS	-	-	0.6
DLS	-	-	3.0
ULS	0.03	(*)	-

(*) Values depending on frame type and cross section geometry given in table 3

Table 3. Properties of the MR steel frames.

Type	P1	P3x1	P3x2	P6x2	P6x3	P3-2x2
Geometry						
Columns	HEB 260 $M_{pl,Rd}$=352.8 kNm					
Beams	IPE 330 $M_{pl,Rd}$=221.2 kNm					
Natural Frequency [Hz](*)	3.81	1.71	1.58	0.9	0.88	1.71
θ_u *[rad]*	0.101	0.075	0.075	0.063	0.063	0.075
θ_y *[rad]*	0.012	0.011	0.011	0.010	0.010	0.011

(*) obtained for rigid connections

2.4 *Methodology*

The evaluation of the behaviour factors is based on the following methodology [Dubina, 1994]:

1. The structure is submitted progressively to the accelerogram presented above multiplied by an amplification factor λ; the maximum amplification λ_y in elastic phase is registered;
2. The amplification is incremented to λ_u corresponding to reach the criterion established for the pertinent Limit State; λ_u is the value of the amplification corresponding to the interstorey limit drift for SLS an DLS and the amplification corresponding to the limit rotation in the connection or members for ULS.

The behaviour factor is defined by the following relation: $q = \dfrac{\lambda_u}{\lambda_y}$

2.5 *Geometric properties of the frames*

In this parametric research several types of moment resisting steel frames were considered. Their characteristics are presented in table 3. The natural frequency is plotted in figure 1 together with the corresponding spectral value.

2.6 *FE analysis*

The FE analysis took into account the geometric non-linearities and the non-linear behaviour of the beam-column connections and of the columns' bases. The analysis was performed by means of the computer program LUSAS.

3 PARAMETRIC STUDY AND RESULTS

3.1 *Influence of the connections using ULS criterion*

In respect to the variation of the plastic moment, lower levels of M_j allow greater plastic to elastic rates of rotation capacity of the connections. Therefore, increasing values of *q* are expected and verified when M_j decreases (table 4). The eventual exception is the frame P1, where the behaviour factor is controlled by the yielding of the columns' base and not by the non-linear behaviour of the connection as in all other cases.

When the initial stiffness ($S_{j,ini}$) is increased the *q*-factor increases, as expected, except for the 6-storey frames where the tendency is inverse of that one.

Increasing the post-elastic stiffness ($S_{j,pl}$) of the connections the *q*-factor increases in all situations.

3.2 *Influence of limit states and methodology in q-factor evaluation*

When SLS is the used criterion, the *q*-factor is in general *q*=1, except for the case of connections with high $S_{j,ini}$ and low M_j in which the plastification occurs for low acceleration multipliers.

As expected, the *q*-factors for DLS are, in general, lower then those for ULS. Exceptions are the cases of very low M_j used together with frame type P1, since in this case the limit state was attained through plastic rotation at the column basis.

The hardening stiffness of the connections in the post-elastic phase has different effect when DLS and ULS are considered (table 5), since an increase of $S_{j,pl}$ leads to an increase of the *q*-factor only when ULS is considered.

The methodology used here for the *q*-factor evaluation was compared with the Ballio-Setti method [ECCS, 1994] and some differences were detected. This method leads, in general, to lower *q*-factors when low values of the connections' resistance and stiffness are considered.

Table 4. Comparison of *q*-factors considering the variation of the three connections parameters for ULS.

Sj,pl = 0% Sj,pl = 1% Sj,pl = 10%

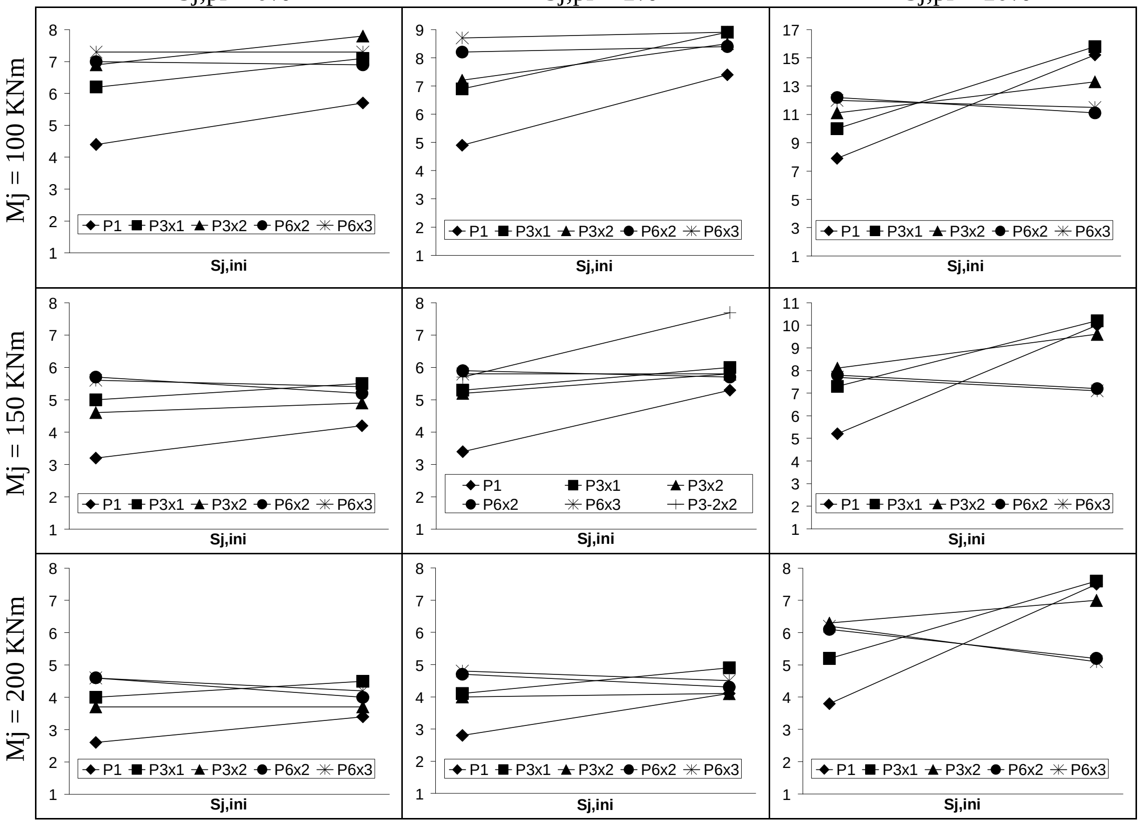
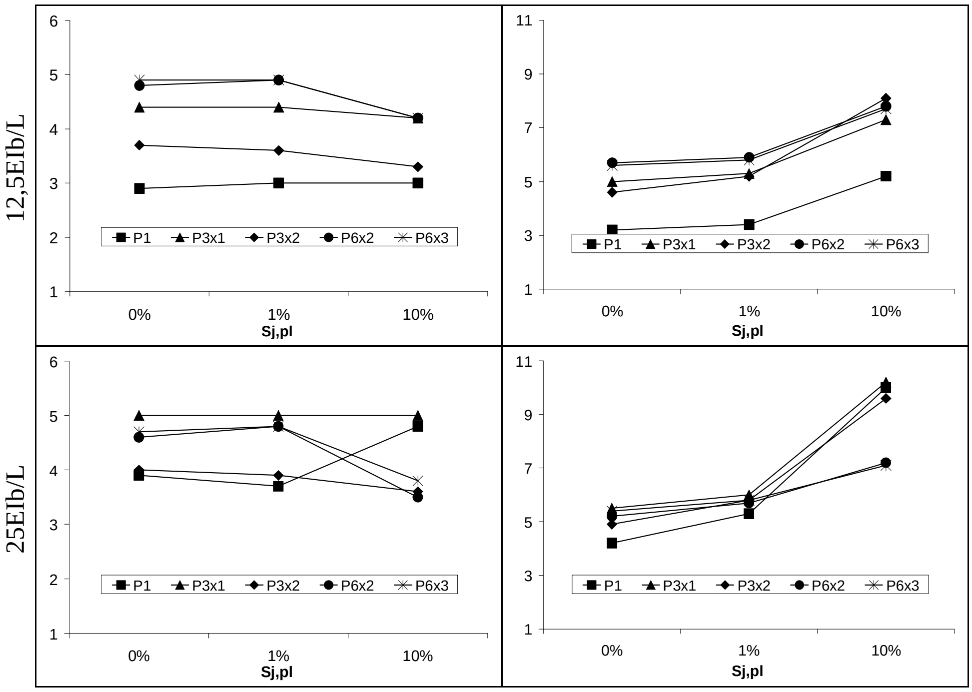


Table 5. Comparison of *q*-factors considering both limit states DLS and ULS and the connections stiffness

DLS ULS



4 CONCLUDING REMARKS

In this paper some preliminary results are presented concerning a parametric study of seismic behaviour factors for MR steel frames, obtained by non-linear dynamic time-history analysis and considering three limit states. Conclusions about the influence of the connections' stiffness and resistance are according to those expected when the criterion used is the ULS. For DLS q-factors are in general lower and the influence of the post-elastic connection behaviour does not followed the one found when ULS is the criterion adopted.

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