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MODELLING CONNECTIONS OF MOMENT RESISTING STEEL FRAMES FOR SEISMIC ANALYSIS

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Abstract. *The seismic response of moment resisting steel frames depends on the behaviour of the beam-to-column connections, in particular when these are considered to be dissipative zones. This paper aims to contribute to a better understanding of modelling joints of steel moment resisting frames concerning their seismic behaviour. The modified Richard-Abbott constitutes a sophisticated model and is used here to reproduce the cyclic behaviour of the steel joints. A parametric study is carried out in order to evaluate the influence of stiffness degradation, strength degradation and hardening effect of the connections in global response of the structure. For the study case the maximum global displacements increase for higher hardening effect. This conclusion can be relevant because for moment resisting frames the horizontal displacement is usually the controlling design criterion in seismic design.*

1 INTRODUCTION

The competitiveness of steel and composite construction, especially in seismic areas, requires the presentation and consideration of solutions that clearly demonstrate added value concerning structural performance. The influence of the real behaviour of steel and composite joints on the seismic response of steel frames has long been recognized as a crucial aspect to ensure safe structural response [1]. The following questions are thus pertinent:

- How relevant is it to model the hysteretic behaviour of dissipative joints in the structural analysis?
- How sophisticated should the hysteretic model be?
- How relevant are the hysteretic parameters in the structural analysis?

This paper aims to contribute to a better understanding of modelling joints of steel moment resisting frames concerning their seismic behaviour. It is the main objective of this study to assess the influence of the hysteretic parameters of joint model in the global seismic response.

The recent publication of part 1-1 of Eurocode 8 [2] provides some rules for the design and detailing of joints subjected to seismic loading. In particular, for moment resisting frames, it is specifically allowed to use dissipative semi-rigid and/or partial strength connections, provided that all of the following requirements are verified:

a) the connections have a rotation capacity consistent with the global deformations;

b) members framing into the connections are demonstrated to be stable at the ultimate limit state;

c) the effect of connection deformation on global drift is taken into account using nonlinear static (pushover) global analysis or non-linear dynamic time history analysis.

Additionally, the connection design should be such that the rotation capacity of the plastic hinge region is not less than 35 mrad for structures of high ductility class DCH and 25 mrad for structures of medium ductility class DCM with the behaviour coefficient q greater than 2 ($q > 2$). The rotation capacity of the plastic hinge region should be ensured under cyclic loading without degradation of strength and stiffness greater than 20%. This requirement is valid independently of the intended location of the dissipative zones. The column web panel shear deformation should not contribute for more than 30% of the plastic rotation capability. Finally, the adequacy of design should be supported by experimental evidence whereby strength and ductility of members and their connections under cyclic loading should be supported by experimental evidence, in order to conform to the specific requirements defined above. This applies to partial and full strength connections in or adjacent to dissipative zones.

It is clear that Eurocode 8 [2] opens the way for the application of analytical procedures to justify connection design options, while still requiring experimental evidence to support the various options. In contrast, North American practice, following the Kobe and Northridge earthquakes, was directed in a pragmatic way towards establishing standard joints that would be pre-qualified for seismic resistance [3]. This approach, although less versatile, would certainly be of interest for the European industry, especially if it could overcome uncertainties that would require experimental validation. Unfortunately, North American design practice and usual ranges of steel sections are clearly different from European design practice. Thus, the benefits of the SAC research programme [4] concerning pre-qualified moment resisting joints are not directly applicable.

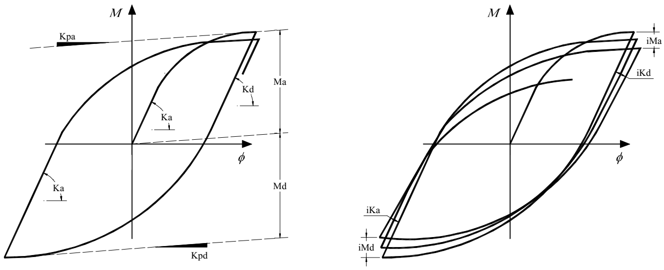
2 STEEL JOINTS UNDER CYCLIC LOADING

Predicting the behaviour of steel and composite joints is quite complex, because it combines several phenomena such as: material non-linearity (plasticity, strain-hardening), non-linear contact and slip, geometrical non-linearity (local instability), residual stress conditions and complicated geometrical configurations [5]. Although numerical approaches using non-linear finite elements could deal with all the complexities of joint behaviour, they require lengthy procedures and are very sensitive to the modeling and analysis options [6].

For static monotonic situations it is nowadays possible to accurately predict the moment-rotation response of a fairly wide range of joint configurations by applying the principles of the component method [7][8]. Under cyclic loading, the behaviour of steel joints is further complicated by successive static loading and unloading. The joint moment-rotation curve is characterized by hysteretic loops with progressive degradation of strength and stiffness that eventually lead to failure of the joint. This typical behavior is usually called oligocyclic fatigue, in close analogy with the behavior of steel under repeated cyclic loading stressed into the plastic range. A typical natural event that, for simplicity, is usually approximated by cyclic loading is an earthquake. Usually, seismic events provoke relatively high amplitudes of rotation in the joint area, so that steel repeatedly reaches the plastic range and the joint fails after a relatively small number of cycles.

2.1 Modified Richard-Abbott Model

For cyclic loading the usual approach is to develop multi-parameter mathematical expressions that are able to reproduce the range of hysteretic behaviors for a given group of steel joint typologies. The modified Richard-Abbott model, illustrated in Figure 1, constitutes such a sophisticated model that can adequately reproduce the cyclic behaviour of steel joints. Subsequently, the values of the parameters are calibrated to satisfactorily correlate to a range of section sizes for a given group of joint typologies [9].



a) Degradation of stiffness b) Degradation of strength

Figure 1: Modified Richard-Abbott model

The structure system under cyclic actions may have its failure characterized by deterioration of mechanical properties. Cyclic action in inelastic range produces accumulation of plastic deformation, until ductility of system is locally exhausted and failure occurs due to fracture [9]. This can be taken into consideration for strength ($M_{o,red}$), stiffness ($K_{o,red}$) and hardening effect using the following expressions:

$$M_{o,red} = M_o \left(1 - i_M \times \frac{E_h}{M_y \times \phi_{u,0}}\right) \tag{1}$$

$$K_{o,red} = K_o \left(1 - i_k \times \frac{E_h}{K_0 \times \phi_{u,0}}\right) \tag{2}$$

$$\begin{aligned} M_{0,ini} &= M_0 && \text{if } \phi_{\max} \leq \phi_y \\ M_{0,ini} &= M_0 \left(1 + H \times \frac{\phi_{\max} - \phi_y}{\phi_y}\right) && \text{if } \phi_{\max} \geq \phi_y \end{aligned} \tag{3}$$

where $\phi_{u,0}$ is the corresponding ultimate value in the case of one single excursion from the origin (monotonic loading), ϕ_{max} is the maximum value of deformation reached in the loading history (in either positive or negative direction), ϕ_y is the conventional yielding value of deformation, E_h is the hysteretic energy accumulated in all previous experienced excursions, M_y represents the conventional yield resistance of the joint, M_0 and $M_{0,ini}$ are the initial and increased value of strength, respectively, K_0 the initial stiffness, i is an empirical parameter related to damage rate and H is an empirical coefficient defining the level of the isotropic hardening.

3 STUDY CASE

3.1 Methodology

The adopted methodology was the study of a predesigned plane frame, modeling the structural elements, beams and columns, in the linear elastic range connected by springs that simulate the structural links. The numerical calculation uses the Software *Seismostruct* [10], which includes the numerical implementation of the modified Richard-Abbott model, capable of simulating the generic cyclic behaviour of steel and composite connections [9].

Thus, the structural seismic performance is evaluated through non-linear dynamic analyses, verifying the variation of maximum horizontal displacements with height, the corresponding inter-storey drifts, maximum total base shear, maximum total support moment and maximum rotation measured in links.

3.2 Structure description

The studied structural system is the steel frame with two bays and three storeys represented in Figure 2.

The choice of steel members, connection details and the geometry of the study frame is based on design of the Cardington Building [11,12], with an alternative choice of columns (HEA) and beams (IPE) to match the seismic design criteria and to correspond to southern European practice and simplifying the frame through reduction of the number of storeys and bays. Remark that all steel members are grade S355.

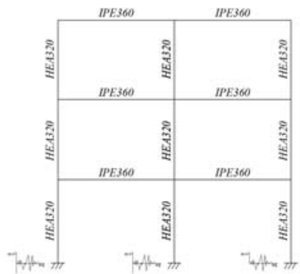


Figure 2: Structural system

3.3 Connection

The reference connection used in this structure, a double-extended beam-to-column steel connection with transverse stiffeners, was tested in laboratory by Nogueiro *et al* [13] and corresponds to the group J1 of the test program. Figure 3 illustrates it configuration as well as the evidenced experimental behaviour.

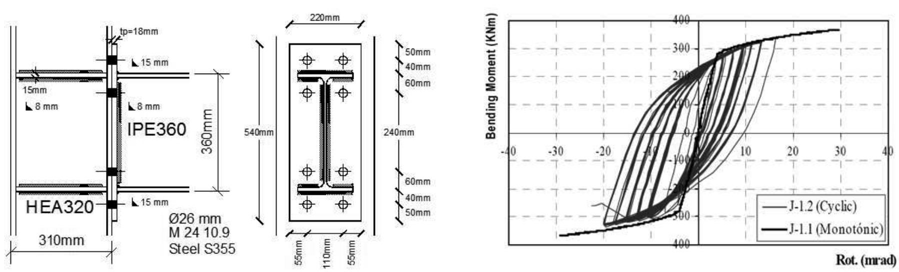


Figure 3: Joint J1; a) Geometry; b) Experimental hysteretic curve Bending moment-Rotation [13]

The connection presents semi-rigid behaviour with partial resistance. It is noted that the hysteretic response is very stable. No pinching or strength degradation is noted and only small stiffness degradation was observed. Table 1 reproduces the model parameters, obtained by numerical calibration with the experimental results. [9]

Table 1: Model parameters for joints J1

K_a KNm/rad	M_a KNm	K_{pa} KNm/rad	n_a	K_{ap} KNm/rad	M_{ap} KNm	K_{pap} KNm/rad	n_{ap}	t_{1a}	t_{2a}	C_a	i_{Ka}	i_{Ma}	H_a	E_{maxa} rad
69500	285	5500	1	0	0	0	0	0	0	0	2	0	0	0,1
K_d KNm/rad	M_d KNm	K_{pd} KNm/rad	nd	K_{dp} KNm/rad	M_{dp} KNm	K_{pdp} KNm/rad	n_{dp}	t_{1d}	t_{2d}	C_d	i_{Kd}	i_{Md}	H_d	E_{maxd} rad
69500	285	5500	1	0	0	0	0	0	0	0	2	0	0	0,1

3.4 Non-linear dynamic analysis

For the evaluation of the seismic loading according to Eurocode 8 [2], soil type B and critical damping of 2% were selected. The chosen peak ground acceleration was 0.45g for a near-field earthquake. For the non-linear dynamic analyses, artificial accelerograms compatible with the target response spectrum and the chosen peak ground acceleration were generated using the software *Gosca* [14]. Figure 4 shows the selected artificial accelerogram.

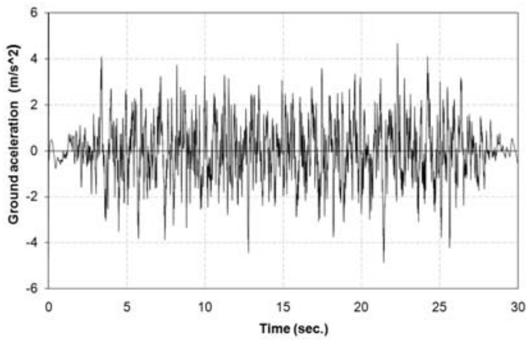


Figure 4: Artificial Accelerogram.

The time range for the system integration of non-linear motion equations considered in analysis was 7,32 x 10^{-3} seconds. Note that nonlinear dynamic analysis allows to incorporate the strength degradation and stiffness degradation of respective links in global behavior of the structures.

4 PARAMETRIC STUDY

A parametric study is carried out in order to evaluate the influence of stiffness degradation, strength degradation and hardening effect of the connection on the structural response of the moment resisting frame. Thus, the following empirical coefficients were varied for descending and ascending branch: i_K (calibration coefficient related to the stiffness damage rate), i_M (calibration coefficient related to the strength damage rate) and H_h (calibration coefficient that defines the level of isotropic hardening).

Table 2 lists the four frames that were used in analysis. Frame J1 is taken as reference and corresponds to the structure with reference joints J1. The other analyzed frames correspond to frames with joints similar to J1, but increasing each single empirical coefficient. The used value corresponds to the maximum value found in the parameters calibration performed by Nogueiro *et al* [9] for the Richard-Abbott mathematical model with a series of experimental tests results for double-extended beam-to-column steel joint subjected to cyclic loading.

Table 2: Analyzed parameters

Frames	Parameters		
	i_k	i_m	H
Frame_J1	2	0	0
Frame_i_K	60	0	0
Frame_i_M	2	0,05	0
Frame_H	2	0	0,02

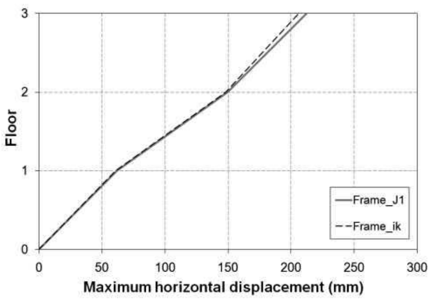
4.1 Results and discussion

Figure 5 illustrates the maximum horizontal displacement and the corresponding interstorey drift for Frame_J1 and Frame_i_K. It is observed that up to the second floor the curves are similar, however, in the last floor the Frame_J1 presented larger displacement (212 mm) and interstorey drift. Comparing to Frame_J1, Frame_i_K presents a 3,8% smaller displacement (204 mm) and a 7,3% smaller interstorey drift.

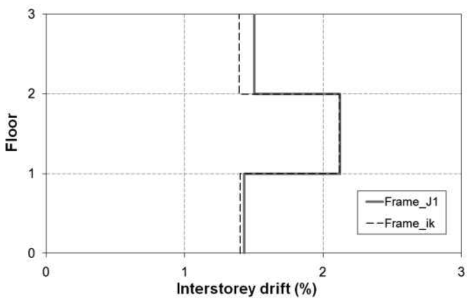
As referred above, the parameter i_M was also analyzed, therefore Figure 6a and Figure 6b show the comparative curves of displacement and interstorey drift relatively to the reference frame (Frame_J1) and Frame_i_M. Analyzing these figures, it can be observed that the displacement in last floor of Frame_J1 presents an increase of 3,9%, but no variation was observed up to second floor. Frame_i_M presents larger interstorey drift between the first and second floor, but it presents smaller interstorey drift between second and third floor being the major variation for this case.

Finally, the Figure 7a and Figure 7b illustrate the curves of maximum horizontal displacement and interstorey drift, respectively, to Frame_J1 and Frame_H. In this case, increasing the value of parameter H, the Frame_H presented greater displacements in all floors and higher interstorey drift up to the second floor.

It was verified that for every analyzed frames the maximum roof displacement did not reach the reference value for Ultimate Limit State (2,5% of height = 320mm) [15].

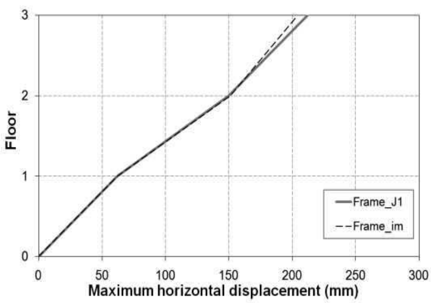


a) Maximum horizontal displacement

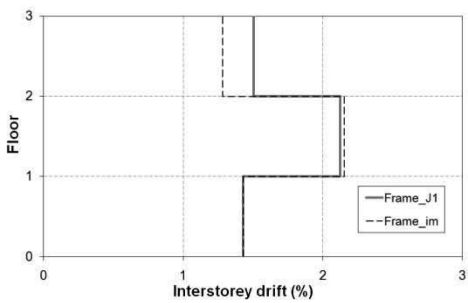


b) Interstorey drift

Figure 5: Analysis of the parameter i_K

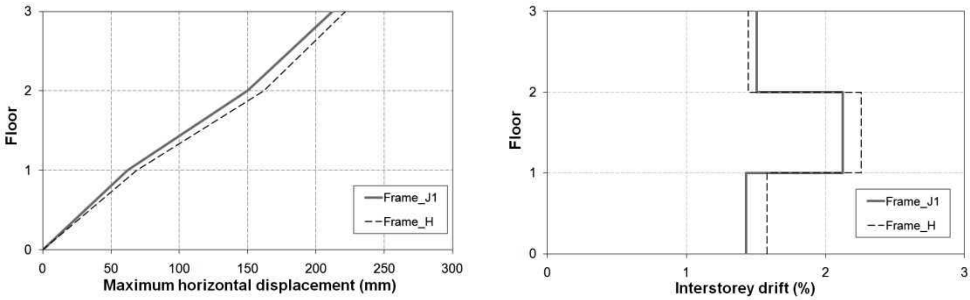


a) Maximum horizontal displacement



b) Interstorey drift

Figure 6: Analysis of the parameter i_M



a) Maximum horizontal displacement b) Interstorey drift

Figure 7: Analysis of the parameter H

The internal forces on the frame's base and maximum rotations observed in links were also analyzed. Table 3 lists the results obtained from the four analyses previously performed. It reveals that for increasing parameters i_K (Frame_i_K) and i_M (Frame_i_M) both moment and force decrease and for increasing *H* parameter total moment and shear force increase. The major difference presented (12,3%) was found in moment corresponding to Frame_H. It can also be observed that the total support moment is not sensitive to variations of the parameter i_M.

According to Table 3, the smaller maximum rotation is for reference frame (16,6 mrad) and the largest rotation (18,7 mrad) corresponds to Frame_H, as well as the maximum displacement. Remark that these rotations are not from the same link for each frame.

Table 3: Internal forces in base of frame and maximum rotations in links

Frames	Total support moment		Total support shear force		Maximum rotation in links	
	Maximum Value (kNm)	Δ (%)	Maximum Value (kN)	Δ (%)	Rot.(mrad)	Δ (%)
Frame_J1	1608,08		514,97		16,59	
Frame_i_K	1485,24	-7,64	457,24	-11,21	16,66	+0,38
Frame_i_M	1590,24	-1,11	505,80	-1,78	16,84	+1,49
Frame_H	1806,12	+12,32	564,88	+9,69	18,74	+12,94

5 CONCLUSION

The objective of this paper was to evaluate the influence of stiffness degradation, strength degradation and hardening effect of the connection on the seismic response of a three storey two bays moment resisting steel frame. It was evaluated the displacements, internal forces in base of frame and rotations in a selected link.

For the study case it can be concluded that the maximum global displacements increases for higher hardening effect and decreases for higher stiffness degradation and strength degradation. This conclusion can be relevant because for moment resisting frames the horizontal displacement is usually the controlling design criterion in seismic design. It can also be concluded that maximum rotation in links is not very sensitive to stiffness degradation and strength degradation.

Further studies will take into account variations of different seismic records and frame typologies.

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