INFLUENCE OF JOINT SLIPPAGE ON THE SEISMIC RESPONSE OF STEEL FRAMES

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ABSTRACT

The present paper presents an evaluation of the influence of joint slippage on the seismic response of steel frames. Using three alternative hysteretic joint models to simulate the connections of a representative low-rise steel frame, and a series of dynamic time-history analyses for a typical seismic record, it is found that slippage affects the response of the structure and should be allowed for in design.

1 INTRODUCTION

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]. Steel and composite joints subjected to cyclic or dynamic excitation are characterised by a hysteretic response whereby the joint exhibits progressive degradation of its moment-rotation response that eventually leads to failure. Bolted end-plate beam-to-column joints, in particular, may present a hysteretic response with slippage [2], thus potentially resulting in higher horizontal drift and a redistribution of internal forces. This issue has been addressed by Della Corte *et al* [3], who carried out a parametric study based on a regular six-storey by two spans plane frame and the Kobe accelerogram, and concluded that the shape of the hysteretic models clearly influenced the ductility demand on the various joints.

Previous work by the authors [4] investigated the effect of slippage on the cyclic response of steel frames. In that study, a 20% variation in bending moments and significant redistribution of internal forces was noticed. However, those results were only approximate, because of the unavailability of a joint element with proper degradation and slippage in the computer code. It is the objective of this paper to assess the influence of slippage on the seismic response of steel frames. To this purpose, the following methodology was implemented: (i) development of a spring element able to reproduce a hysteretic behaviour with and without slippage, (ii) implementation of the spring element within the specialised nonlinear code Seismosoft [5], (iii) selection of two typical composite joints, illustrated in Figure 1 together with their characteristic cyclic behaviour, (iv) choice of a representative low-rise frame [4] and seismic event (Kobe), and (v) consideration of a parametric study, combining 3 different hysteretic models (frames PA1, PA2 and PA3), two PGA (peak ground acceleration) for the Kobe seismic event (0.3 g and 0.6g) and a single unique connection (E9) for all beam-to-column joints, whereby dynamic time histories are evaluated using the software Seismosoft.

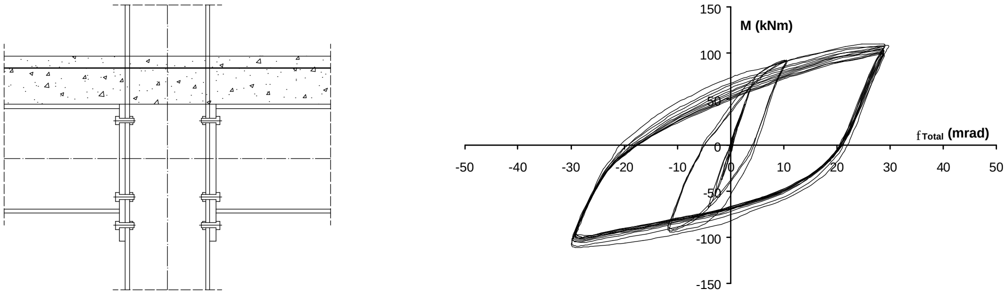


Fig. 1a - Typical cyclic moment-rotation curves in internal composite joints; joint E11.

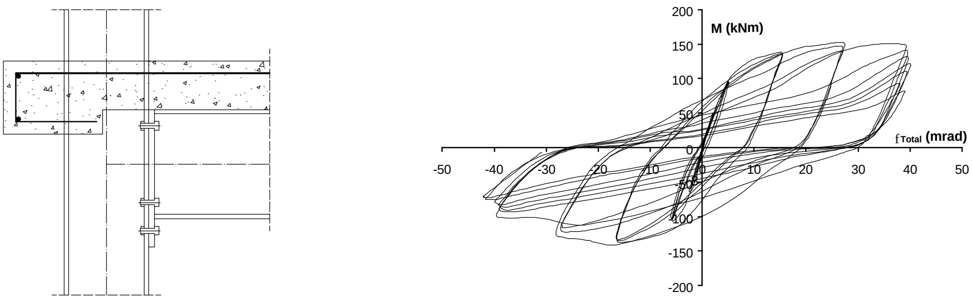


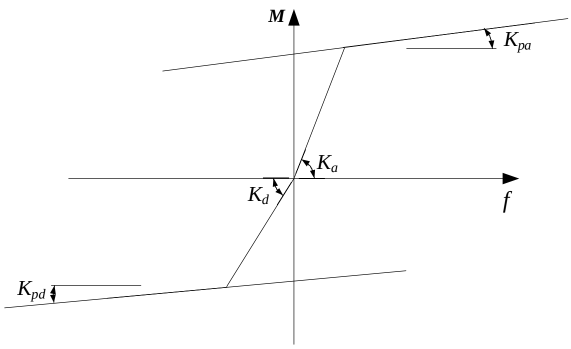
Fig. 1b - Typical cyclic moment-rotation curves in external composite joints; joint E9.

2 HYSTERETIC MODELS

In order to assess the influence of the hysteretic models, three alternative formulations were used to reproduce the behaviour of the joints: (i) an elastic-plastic bi-linear law, (ii) a hysteretic model without slippage based on the Richard-Abbott model [6], and (iii) a hysteretic model with slippage based on the Richard-Abbott modified model [3]. All three models are briefly described in the following.

2.1 Elastic-plastic bi-linear law

The elastic-plastic bi-linear law, taken as the basic model for further comparison, is illustrated in Figure 2 for the two selected joints.



Joint E9		Joint E11	
Ka	25810 KNm/rad	Ka	16830 KNm/rad
Kpa	1200 KNm/rad	Kpa	941.5 KNm/rad
Kd	24570 KNm/rad	Kd	16500 KNm/rad
Kpd	1200 KNm/rad	Kpd	825.0 KNm/rad
M^{+}_{rd}	130 KNm	M^{+}_{rd}	90 KNm
M^{-}_{rd}	110 KNm	M^{-}_{rd}	80 KNm

Fig. 2 - Elastic-plastic bi-linear law.

2.2 Richard-Abbott model

The Richard-Abbott model is based on a formula developed in 1975 [6] to reproduce the elastic-plastic behaviour of several materials and was initially used to simulate the static monotonic response of joints, later applied to cyclic situations [7] and further modified to deal with asymmetrical joints with respect to the centroidal axis, as is the case of composite joints [2]. Figure 3 shows the application of this model to the selected joints. It is clear that this model cannot simulate very well the effect of slippage.

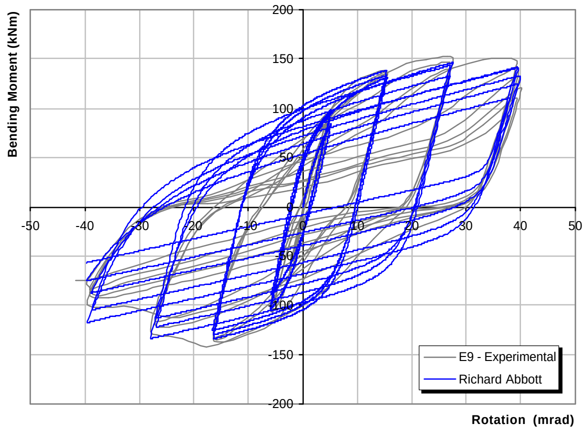


Fig. 3a – Hysteretic curve for joint E9.

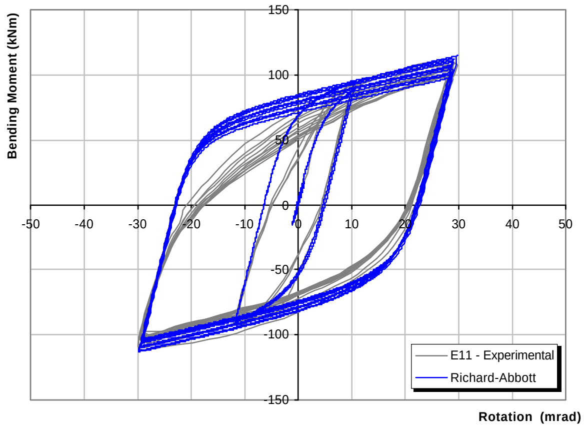


Fig. 3b – Hysteretic curve for joint E11.

2.3 Modified Richard-Abbott model

This model was modified by Della Corte et al. [3] to include pinching. Figure 4 illustrates its application to the chosen joint typologies, depicting good agreement with the experimental results.

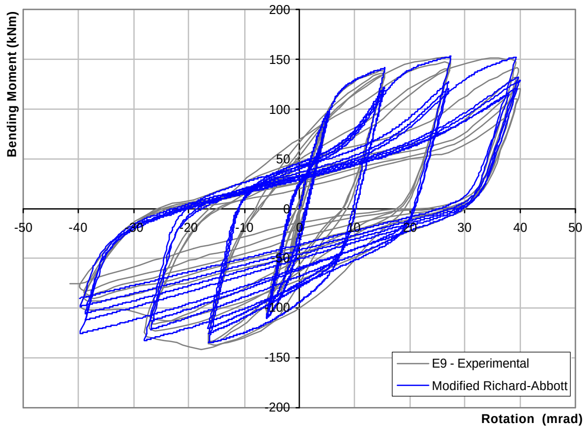


Fig. 4a – Hysteretic curve for joint E9.

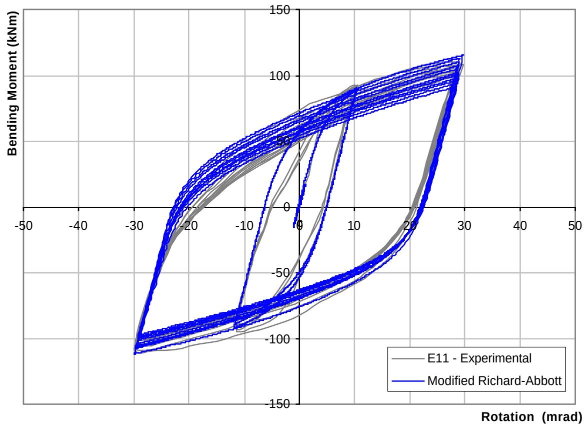


Fig. 4b – Hysteretic curve for joint E11.

3. ANALYSES OF MOMENT RESISTING FRAME

3.1. Description of the structure

As briefly referred in the introduction, a typical low-rise office building was selected - Figure 5a. The structure consists of a two-storey building, with a inner service area (lifts, staircases, WC and storage and ducts), surrounded by a flexible office area, without structural members.

The structural layout consists of an orthogonal grid with five alignments with 3 spans of 7.5-5-7.5m in the transverse direction and 4 alignments with 4 equal spans of 7.5 m in the longitudinal direction. The total height of the steel frames is 7 m (3.5 m in each floor). The structure has HEA 220 columns and composite beams supporting a concrete slab - Figure 5b. Table 1 describes the dead and live loads considered in the study.

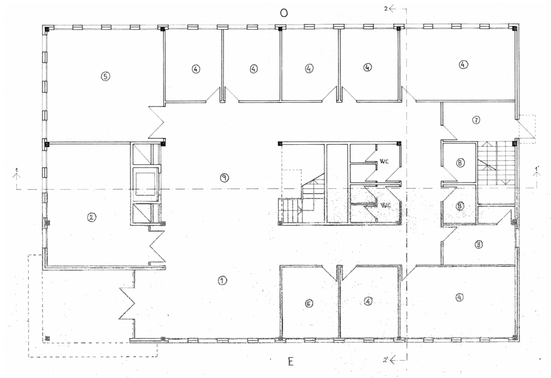


Fig. 5a – Architectural layout of the case study building

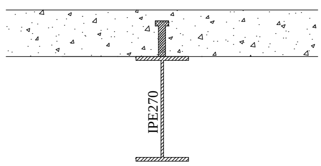


Fig. 5b – Composite cross section.

Table 1: Dead and Live loads

Loads	
Dead loads (self-weight, finishes, etc.)	3.0 kN/m^2
Live load	2.0 kN/m^2

Given the symmetry of the structure, a major axis internal frame was selected to represent the structural response of the building -Figure 6. This figure presents the frame geometry and identifies the structural joints E9 and E11.

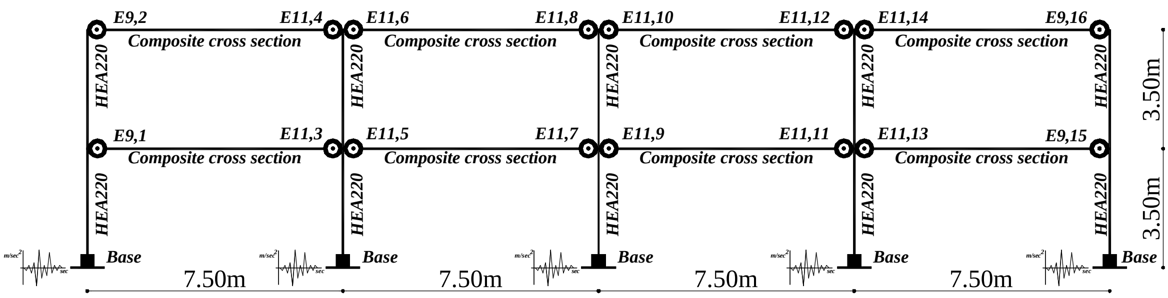


Fig. 6 - Reference frame.

As already mentioned, for the same frame, three different joint models were used: (i) the elastic-plastic bi-linear law - frame PA1, (ii) the hysteretic model without slippage - frame PA2 and (iii) the modified Richard Abbott model, accounting for slippage - frame PA3. The initial stiffness for all three joint models is the same, and defined according the results of the experimental tests [2]. Thus, for all cases, the fundamental period of the frame is T=1.075 sec.

4.2. Description of the analyses

The program SeismoStruct [5] is used for all non-linear dynamic analyses performed and a damping of 2% was considered. For these analyses the Kobe accelerogram, with a peak ground acceleration of 0.6g (PGA=0.6g), was chosen.

Different analyses were carried out in this work to study the influence of joint slippage on the seismic behaviour of steel frames as well as the seismic intensity and the type of the joint models adopted. Firstly, and modelling differently the non-linear behaviour of joints E9 and E11, according to some test results (see Figure 1), the three models PA1, PA2 and PA3 were studied and the results for a PGA equal of 0.3g (half of the Kobe PGA) analysed. In the second analysis, the same models were adopted for the PGA=0.6g. In the third analysis all the frame joints were modelled using joint E9 and the accelerogram used with a PGA of 0.3g. In this paper, mainly the results for the first type of analysis are presented but some comments and discussion are made regarding the outcomes obtained for the different parametric studies.

4.3. Results and discussion

In the following, the results obtained for the first type of analysis are presented in terms of: rotation *versus* time (Figures 7 and 9), moment *versus* time (Figures 8 and 10), story drift *versus* time for the two storeys (Figures 11 and 12) and moment-rotation for joints E9,1 and E11,13 (see Figure 6), those subjected to higher internal forces and displacements.

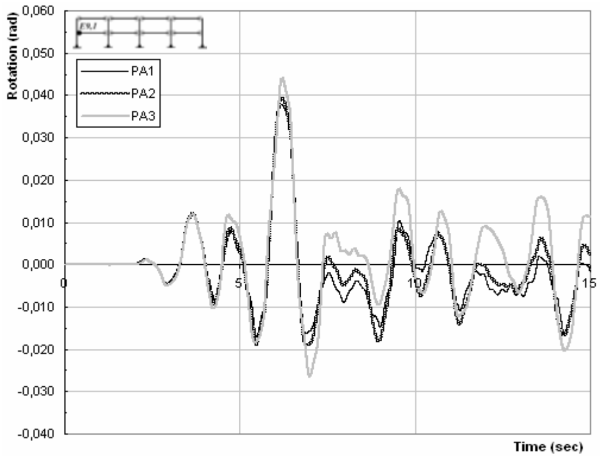


Fig. 7 –Curve rotation/time for joint E9,1, PGA=0.3g.

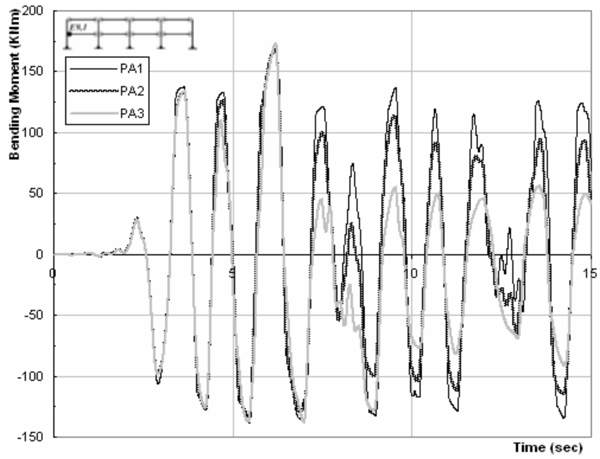


Fig. 8 – Curve B. Mom./time for joint E9,1, PGA=0.3g.

Knowing that behaviour of the joint E9 it is more influenced by the slippage phenomenon than joint E11, as shown in Figures 1a and 1b, it is expected that the results obtained with the three models (PA1, PA2 and PA3) present more differences in terms of deformations and internal forces for the joints E9 than for joints E11.

To ease the comparison of the different results, the individual area of each curve was evaluated, called total area in Table 2, and named intensity. It can be observed that the intensity of deformation of joint E9,1 for the frames PA1 and PA2 increases 3.6 %, but the differences between PA2 and PA3 are more pronounced (18.4%). This increase is mainly due to the effect of the joint slippage.

As expected, the strength intensity decreases from the PA1 model to PA3: 10.5% from PA1 to PA2 and 13.8% from PA2 to PA3. When the seismic intensity considered was higher (PGA=0.6g) this decrease of strength became more considerable, (23.9% from the frame PA1 to PA2 and 21.1 % from PA2 to PA3) mainly due to the increase of strength degradation and the plastic deformations, as well as, for PA3, the increase of the slippage phenomenon.

Table 2 also presents the values for the maximum rotations and maximum bending moments. As was expectable, the deformation increases from PA1 to PA3 and the maximum moment, due to strength degradation, decreases from PA1 to PA3.

Table 2: Rotation and Bending Moment values for the joint E9,1.

	Frame	Rot.$_{max}$ (mrad)	total area (rad x sec)	Δ total area %	B.Mom.$_{max}$ (KNm)	total area (KNm x sec)	Δ total area %
PGA=0.3g	PA1	37.84	0.0928	-	169.36	1010	-
	PA2	37.84	0.0963	+3.6	167.45	904	-10.5
	PA3	44.12	0.118	+18.4	173.63	779	-13.8

Figures 9 and 10 present, respectively, the rotation and moments *versus* time for joint E11,13. It is easily observed that the effect of the slippage in this joint is less obvious than in the previous case. This was expected according to the test results presented in Figure 1.

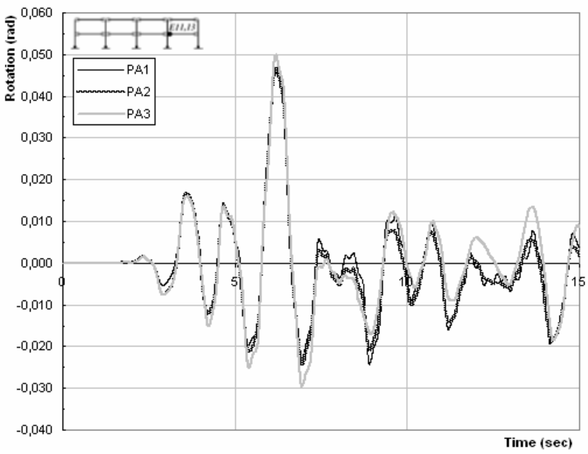


Fig. 9 – Curve rotation/time for joint E11,13, PGA=0.3g.

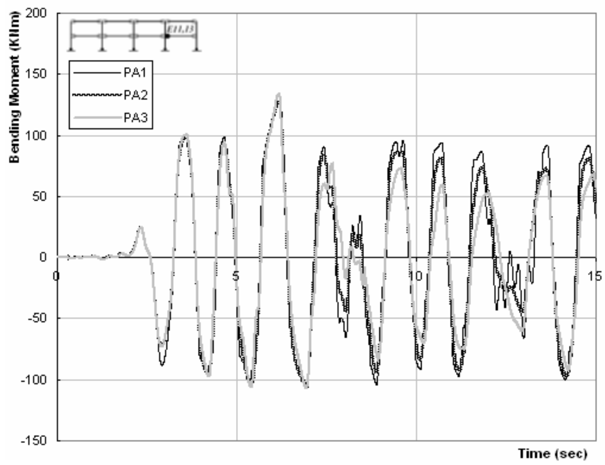


Fig. 10 – Curve B.Mom./time for joint E11,13, PGA=0.3g.

According to the results presented in Table 3, a strength intensity damage of 5.6% is found from PA1 to PA2 and 10.1% from PA2 to PA3. Even for this joint, the difference of intensity of deformation for frames PA1 and PA2 is small, and increases 5% for frame PA3 for this analysis and 11.8 % when a PGA equal to 0.6g is considered, due especially to the effect of slippage.

From all the results obtained, one can conclude that the effect of the joint slippage in the global frame seismic behaviour is very important, and should be considered in the analyses, mainly if the seismic intensity is high. The PA3 joint model can contribute more adequately to the dissipation of the seismic energy as it accounts for the slippage phenomenon.

Table 3: Rotation and Bending Moment values for the joint E11,13.

	Frame	Rot.$_{max}$ (mrad)	total area (rad x sec)	Δ total area %	B.Mom.$_{max}$ (KNm)	total area (KNm x sec)	Δ total area %
PGA=0.3g	PA1	45.96	0.114	-	128.23	769	-
	PA2	46.58	0.114	0	127.00	726	-5.6
	PA3	49.86	0.120	+5.0	134.39	653	-10.1

Figures 11 and 12 represent the evolution of the inter-story drift with time, for the first and the second floor, respectively. From the values obtained with this analysis (for a PGA= 0.3g), storey drifts and joint deformations in particular, compared with some capacity values proposed in some codes [8,9], one can say that the seismic action considered in this analysis corresponds to a ultimate limit state. In fact, for a PGA of 0.6g, the frame structure reaches collapse.

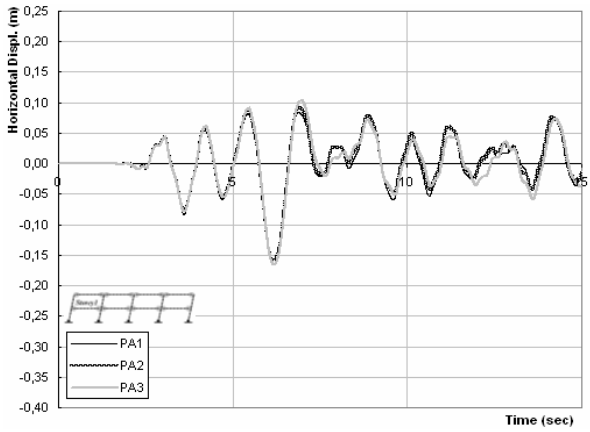


Fig.11 –Storey drift for the floor 1, PGA=0.3g.

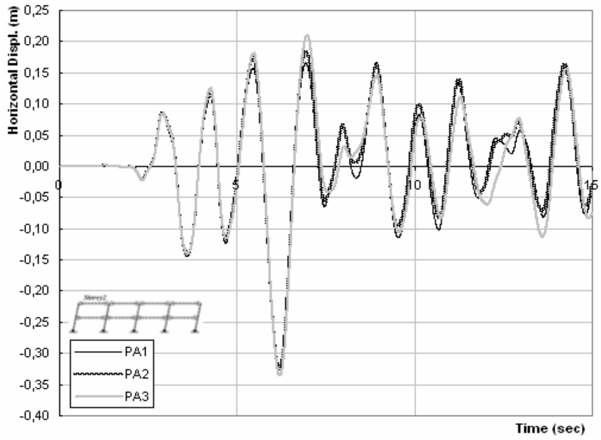


Fig.12 –Storey drift for the floor 2, PGA=0.3g.

Table 4 presents the maximum values of inter-storey drift for the two stories, as well as the individual area defined for each curve. The values obtained for the different models are very similar, thus the inter-storey drift values are not very sensitive to the slippage phenomenon.

Table 4: Storey Drifts values.

Frame		$St1drift._{max}$ (m)	total area (m x sec)	Δ total area %	$St2drift._{max}$ (m)	total area (m x sec)	Δ total area %
PGA=0.3g	PA1	-0.158	0.948	-	-0.322	0.472	-
	PA2	-0.159	0.965	+1.8	-0.326	0.474	+0.4
	PA3	-0.165	0.990	+2.5	-0.336	0.482	+1.7

The following figures show the hysteretic bending moment – rotation curves, for the joints E9,1 and E11,13. It is clear that for this seismic intensity (PGA= 0.3g) the frame behaviour is non-linear.

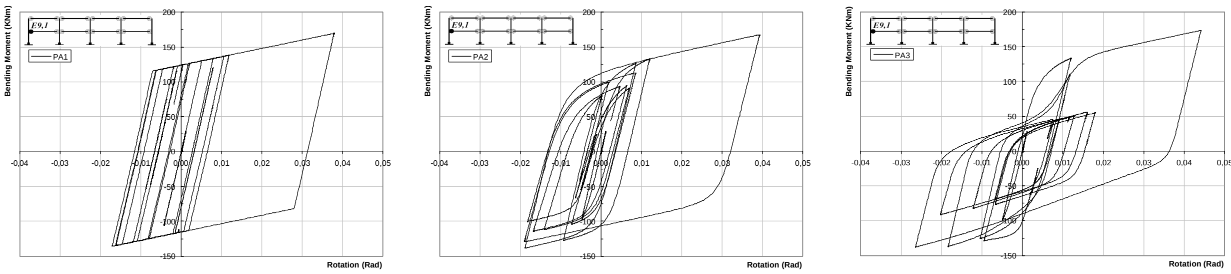


Fig. 13 – Hysteretic curves for the joint E9,1 for PA1, PA2 and PA3 frames, PGA=0.3g.

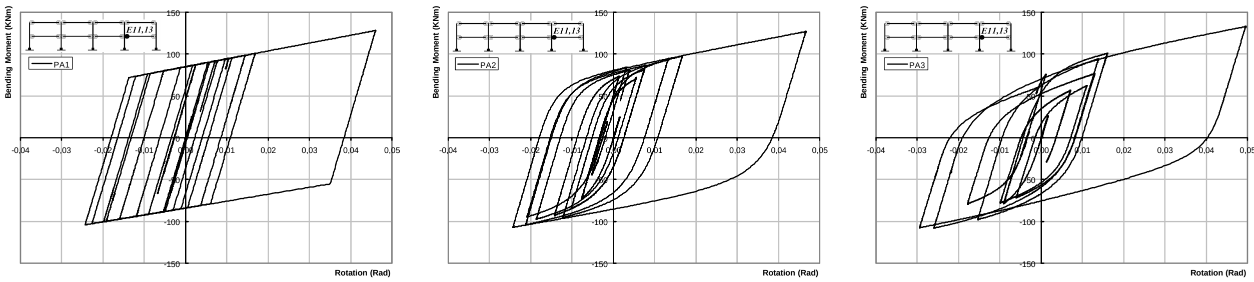


Fig. 14 – Hysteretic curves for the joint E11,13 for PA1, PA2 and PA3 frames, PGA=0.3g.

4. CONCLUSIONS

The objective of this paper was to evaluate the influence of the joint slippage on the seismic response of a two storey four bays moment-resisting (MR) steel frame. Using a two storey frame designed to represent a typical office building and real joint details, the following conclusions could be made:

- the joint deformation increases with the degree of joint slippage by about 20%, thus increasing the ductility demand;
- a moderate seismic event (50% of the PGA of the Kobe earthquake) results in circa 40 mrad of joint rotation, and corresponds to a ultimate load condition;
- the inter storey drift values are not very sensitive to joint slippage.

Given that only a seismic record was used on a single frame configuration, the conclusions must be further verified with an enlarged parametric study.

REFERENCES

[1] Mazzolani, F.M. and Piluso, V., "*Theory and design of seismic resistant steel frames*", E & FN Spon, London (1996).

[2] Simões, R., Simões da Silva, L. and Cruz, P., "Behaviour of end-plate beam-to-column composite joints under cyclic loading", *International Journal of Steel and Composite Structures*, 1(3), 355-376 (2001).

[3] Della Corte, G., De Matteis, G. and Landolfo, R., Influence of Connection Modelling on Seismic Response of Moment Resisting Steel Frames. In: Mazzolani FM, editor. Moment resistant connections of steel buildings frames in seismic areas. London: E. & F.N. Spon; 2000.

[4] Nogueiro, P., Simões da Silva, L. and Bento, R., "Influence of joint slippage on the cyclic response of steel frames", in Topping, B.H.V. (ed.), *Proceedings of 9th International Conference on Civil and Structural Engineering Computing*, Civil-Comp Press, Stirling, United Kingdom, paper 66 (2003).

[5] SeismoStruct, "Computer program for static and dynamic nonlinear analysis of framed structures" [online]. Available from URL: http://www.seismosoft.com (2004).

[6] Richard, R. M. and Abbott, B. J. (1975) Versatile Elasto-Plastic Stress-Strain Formula. *Journal of the Engineering Mechanics Division*, ASCE, 101, EM4, 511-515.

[7] De Martino, A., Faella, C. and Mazzolani, F. M. (1984) Simulation of Beam-to-Column Joint Behaviour under Cyclic Loads. *Construzioni Metalliche*, 6, 346-356.

[8] FEMA, Seismic Design criteria for new moment-resisting steel frames construction. Federal Emergency Management Agency Report no. 350, 2001.

[9] Structural Engineers Association of California (SEAOC). Vision 2000 a framework for performance-based engineering. Sacramento (CA): Structural Engineers Association of California; 1995.

KEYWORDS

Structural Engineering, Steel Structures, Buildings, Component Method, Beam-to-column Joints, Dynamic Behaviour, Seismic Behaviour.