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SEISMIC BEHAVIOUR OF STEEL BRACED FRAMES WITH DUCTILE INERD™ CONNECTIONS

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Abstract. *The paper presents some experimental results of the European research program INERD, on the seismic behaviour of steel braced frames with dissipative connections, involving among the others Instituto Superior Tecnico (P), Politecnico di Milano (I) and National Technique University of Athens (GR). Four different types of new dissipative connections were studied, involving the analysis of constructional parameters such as type of dissipative connection, slenderness of the bracing element, stiffness of the connection or thickness of the plates. The study of these connections was developed in three stages: experimental study of the connections themselves; experimental study of frames with such connections; numerical simulation of the cyclic behaviour of connections and of connected frames. Based on experimental studies and numerical simulation models, practical design rules were derived to allow for the proper dimensioning of structures with the developed dissipative connections.*

1 INTRODUCTION

In modern codes, the seismic design of steel structures is achieved by considering that, at the ultimate limit state (corresponding to the maximum expected earthquake in the geographic area where a building is located), the structure should still stand up, allowing for permanent deformations. To achieve such behaviour, a high number of reliable dissipative zones should develop, in which the earthquake energy is dissipated by local plastic cyclic mechanisms. Actually, dissipative zones only in the main structural profiles can be considered, because reliable data of energy dissipation exist only for that type of elements. On the contrary, designing structures in which seismic energy dissipation takes place in the connections of the structural elements is not possible without experimental tests that demonstrate their actual dissipative capacity.

The paper presents the experimental activity performed on concentric bracing with dissipative connections on steel frames, subjected to cyclic loads. The capacity to dissipate energy and to sustain large plastic deformations without collapse, due to steel ductility and connection geometry, is the fundamental feature of the tested connections. Moreover after their plastic deformation, it is possible to replace them quite easily, so the number of structural elements that need to be restored in the post-seismic process is small.

2 DESCRIPTION OF THE INERD™ DEVICES

Two main types of dissipative connections between bracing and columns were developed: Pins (two typologies) and U devices, both dissipating energy by flexural deformation.

The pin devices consist of two external eye-bars bolted to the adjacent column, two internal eye-bars welded to the diagonal and a pin running through the eye-bars. The pins behave mainly as a four-points bent short beam due to the axial load of the diagonal. The load acts through the two internal eye-bars. The two external eye-bars, which are bolted to the column, provide support to the pin.

The U device consists of a U-shaped thick plate that is bolted to the brace and to the column. The U-device can work in two different ways, depending on the geometry of the connection (Figure 1). In U1 typology, the bolts on the diagonal are mainly in shear, in U2 they are mainly in tension.

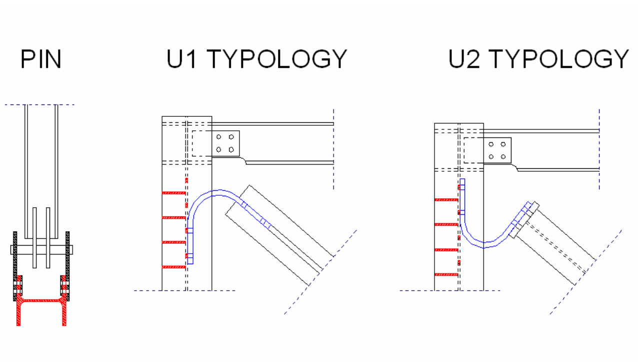

Figure 1: Pin and U-spring typologies connections.

By an appropriate sizing of the devices, the main structural members remain elastic and don't buckle, while damage is restricted to the devices and eventually also to the connection plates (eye-bars) which can be easily replaced once they are largely deformed after an earthquake.

Braced frames with INERD™ connections exhibit the following benefits compared to conventional steel frames:

- Better compliance with seismic design criteria.
- Protection of compression braces against buckling.
- Activation of all braces, either in compression or in tension, even at large storey drifts.
- Inelastic deformation limited to small parts that can be easily replaced.
- Easy inexpensive repair after strong earthquakes, if required.
- Reduction of overall structural costs for the same performance.

3 EXPERIMENTAL PHASE

The experimental phase of the research was carried out in Lisbon for the characterization of the behaviour of the dissipative devices and in Milan, where the large scale frame behavior was tested. In order to compare the experimental results, the same dissipative devices were used.

The experiments performed in Lisbon, applying a quasi-static loading, were twenty on PIN devices and seventy-eight on U-devices. Two pin typologies were adopted, rounded section and rectangular section bars, both types tested with two different distances, 50 and 70 mm, between the inner eye bars. The rounded pins (Figure 2) avoid torsion stresses in the pin as well as stress concentrations in the sharp edges of the eye-bars, but are more expensive.

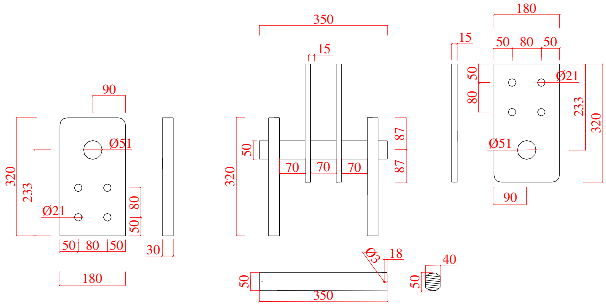

Figure 2: pin device with rounded section and distance 70 mm.

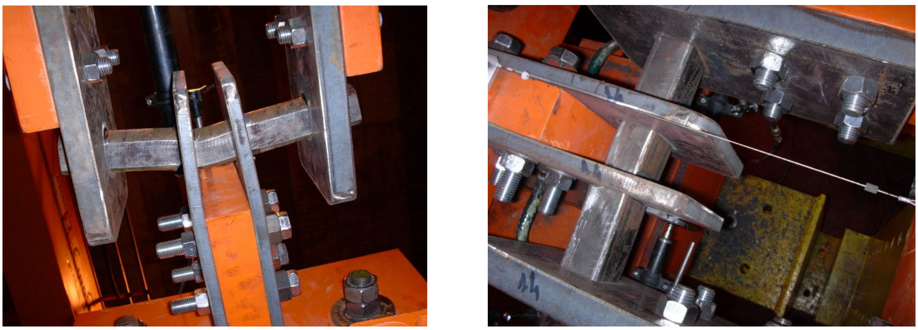

Figure 3: Test on a rectangular pin; distance 50 mm, in Lisbon.

The U devices were tested with two different thickness, 25 and 30 mm, two different radius, 100 and 125 mm, four different opening angles: 30°, 39°, 45° and 50°. Both types of geometry U1 and U2 were tested. During the monotonic tests the maximum displacement reached was about 20 cm. Figures 3 and 4 represent respectively the tests performed in Lisbon on rectangular pins and on U spring devices.

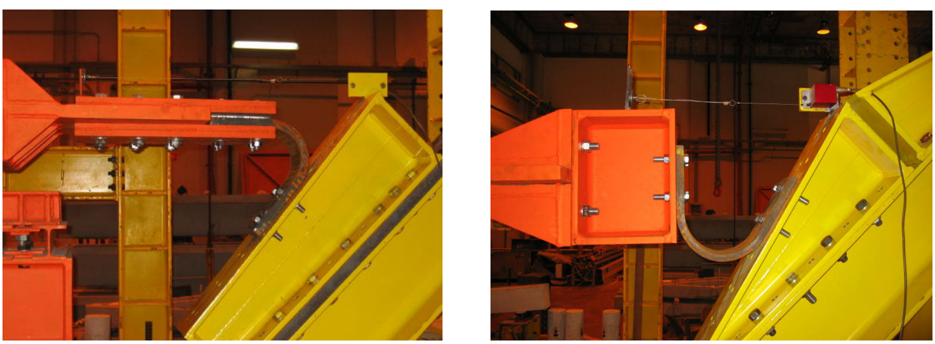

Figure 4: Arrangement of the tests on springs in Lisbon.

In Milan Sixteen experimental quasi-static cyclic, displacement controlled tests were performed on full scale frames, eight with pin devices, and eight with U devices.

The specimens were designed so that the main structural members remained elastic, while the plastic deformation was concentrated in the “dissipative devices” connecting the diagonals to the columns. The columns are HE240 B, the top beam is a HE200 B and the diagonals are HE160 B. Out plane displacement of the specimens was prevented. For some tests were used strain gauges in the diagonals, which give a better evaluation of the force in each diagonal with respect to the teorical values from the equilibrium of the frame. Figure 5 shows an overall view of the specimens tested in Milan.

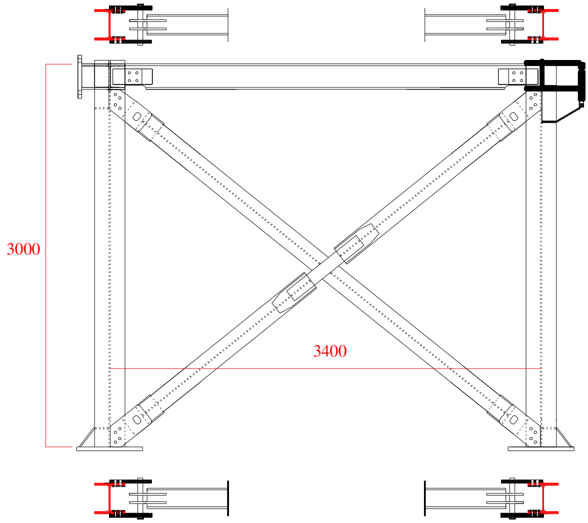

Figure 5: Geometry of the frame tested in Milan, with pin devices.

4 LOADING HISTORIES

The tests were performed and analysed according to the ECCS “Recommended Testing Procedure for Assessing the Behaviour of Structural Steel Elements under Cyclic Loads”.

The testing procedure may or may not include preliminary monotonic displacement tests. The first case, called complete testing procedure, was adopted in Lisbon, while the second, short testing procedure, in Milan. The conventional limit of the elastic range F_y was defined according the ECCS recommendations.

The test can be stopped at any level of displacement with regard to specific code or research requirements. In many cases it was stopped only at the breaking of the spring or of the pin.

4.1 Characterization tests

4.1.1 Pin devices

In addition to monotonic displacement tests, constant cyclic displacement tests were performed, in order to study their fatigue behaviour. For each pin type four different displacement values were imposed: ±15 - 20 – 25 - 30 mm. Totally 20 experiments were done. The monotonic test was stopped usually when it reached a great deformation. Rarely the devices were broken.

4.1.2 U devices

Differently from the pins tests, both compression and tension tests were performed, due to the different behaviour. The ECCS tests were stopped at the failure of the device (Figures 6, 7). Moreover four constant displacements (± 20 – 40 – 60 – 80 mm) fatigue tests were carried out. Totally, 78 tests were done.

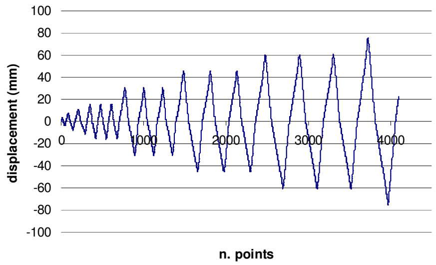

Figure 6: ECCS loading history U devices.

Figure 7: U device deformation with cracks.

4.2 Full scale tests

4.2.1 Pin devices

In Milan were applied to the frames two different loading histories: ECCS and a seismic time history. The ECCS test was stopped at an interstorey drift of ±70 mm, corresponding to a local deformation of the pin of about 8 times the yield displacement.

The seismic displacement time history applied was obtained from the Greek Thessaloniki earthquake (M=6.5, PGA=0.15g) of June 20, 1978, whose epicentre was close to the city. The seismic effect on buildings was classified VIII degree MCS.

4.2.2 U devices

Only the ECCS test was performed. Tests were stopped when at least one spring was broken. The loading history was the same adopted for the pins. The maximum frame displacement reached was 120 mm.

5 REPRESENTATIVE EXPERIMENTAL DATA

In the following results of two representative experiments performed on frame structures are presented, focusing the attention on pin devices. Let us consider tests n. 2 and n. 4, characterized by rounded pins, with a 70 mm distance between bearing points, respectively with the ECCS and the seismic loading history (Figure 8). Test n. 2 was continued up to the complete failure of one pin (top right position - PIN A) in correspondence of the eye-bar of the diagonal. Almost all the energy was absorbed by the pins (figure10).

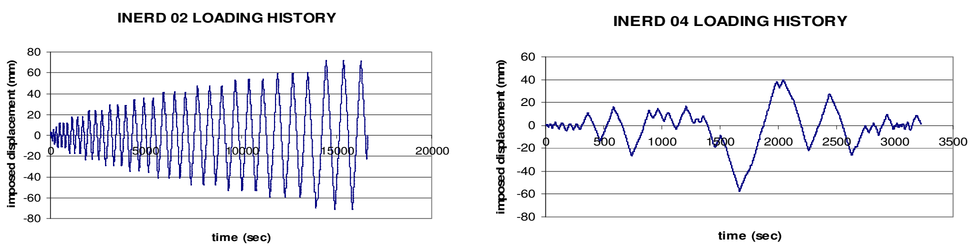

Figure 8: ECCS loading history test 2 – Seismic loading History test 4.

In seismic loading, only a few cycles imposed are comparable in size to the large ones of the ECCS test. The maximum displacement is from –57 mm to +39 mm. The energy absorbed from the specimen is significant only for the large cycles, and negligible for the others (Figure 10). The four pins after the test are only slightly deformed. Tables 1, 2 and the following figures 9 -11 report some main results.

Table 1: Results of test 2 and test 4.

Test	n.° Cycles	Envelope Energy (kNmm)	F_y (kN)	K_y ($kNmm^{-1}$)	e_y (mm)
2	34	83830,65	340,00	36,96	9,20
4	21	45051,97	360,00	37,11	9,70

Table 2: Maximum cycle of test 2 and test 4.

Test	Negative Cycle		Positive Cycle	
	ΔV_{max} (mm)	F_{max} (kN)	ΔV_{max} (mm)	F_{max} (kN)
2	-71,63	-876,84	71,89	944,89
4	-57,25	-667,70	39,86	629,25

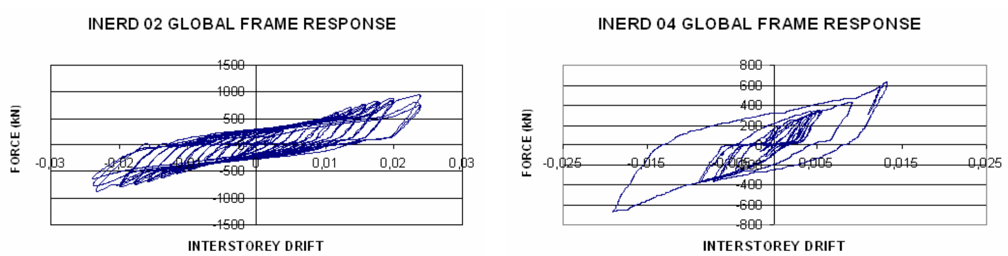

Figure 9: Global frame response of test 2 and test 4.

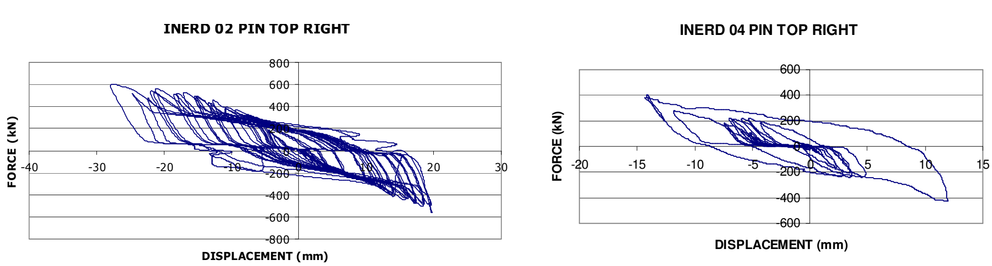

Figure 10: Top left pin of test 2 and test 4.

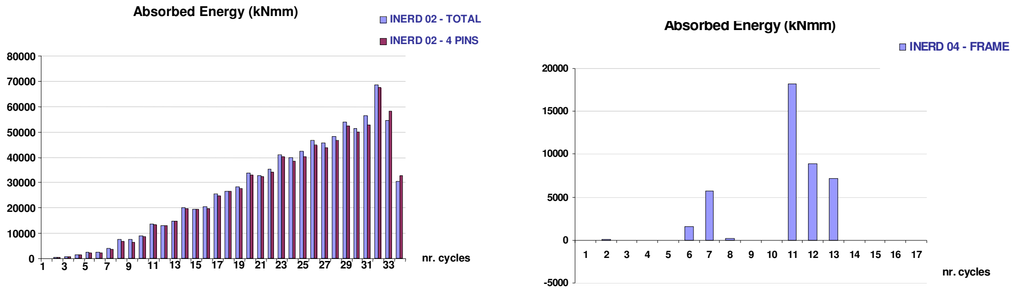

Figure 11: Absorbed energy in test 2 and in test 4.

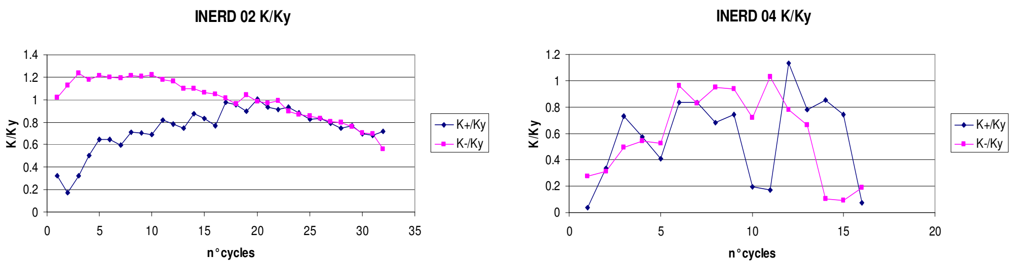

Figure 12: K/Ky for test 2 and test 4.

6 CONCLUSION

Modern seismic design codes impose limitations to the interstorey drifts. In order to comply with these limitations, MR steel frames cannot completely develop their ductility capacity, due to their large flexibility. On the contrary, braced frames, which exhibit high stiffness, while complying with the deformability limitations, exhibit a brittle behaviour. From the design point of view, this means that braced frames should be designed with a q-factor equal to 2.0. The advantage of braced frames with dissipative connections, a structural typology encompassed by EC-8, is that they exhibit a large stiffness at SLS, complying with the allowable interstorey drift, while at ULS they can develop a large ductility, hence allowing for quite larger design q-factor values (of the order of 3.5 to 4.0).

The tests performed show the effectiveness of the INERD™ dissipative connection devices to absorb energy and absorb large inter-storey displacements with the frame structure in the elastic field, maintaining stiffness and bearing capacity.

The INERD™ dissipative connection devices described in this paper are covered by an international patent n. EP05101722.6 (2005).

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