

EXPERIMENTAL BEHAVIOUR OF STANDARDISED EUROPEAN END-PLATE BEAM-TO-COLUMN STEEL JOINTS UNDER ARBITRARY CYCLIC LOADING

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Abstract. *This work presents the results of the study of the six experimental tests of standardised end-plate beam-to-column steel joints under arbitrary cyclic loading. The experimental programme is divided into two groups, varying the column section size. Each group has the first monotonic test and two more cyclic tests with arbitrary loading. The results are presented in terms of $M-\Phi$ curves, main mechanical joint properties and some relevant illustrations are presented. Finally, some conclusions can be drawn.*

1 INTRODUCTION

The recent publication of part 1-1 of Eurocode 8 [1] 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 (ULS);
- c) the effect of connection deformation on global drift is taken into account using nonlinear static (pushover) global analysis or non-linear 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 ductility class DCH and 25 mrad for structures of ductility class DCM with $q > 2$ [1]. 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 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 [FEMA, etc]. 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 [2] concerning pre-qualified moment resisting joints are not directly applicable.

It is the objective of this paper to report on an experimental test programme aimed at establishing pre-qualified beam-to-column moment resisting joints suitable for European practice. Double extended bolted end-plate beam-to-column joints were selected as an adequate joint typology, exhibiting partial strength. Conservatively, it was decided to adopt top and bottom transverse stiffeners at the column web panel.

It is noted that, in the literature, several researchers have carried out experimental work, including cyclic testing, on bolted extended end-plate joints with transverse stiffeners. In 1993, Plumier and Schleich [3] studied the contribution of the shear panel to the energy dissipation, strength and the rotation capacity of the joint. Pradhan and Bouwkamp [4] studied the interactive plastification of the beam-end and the column web panel zone in shear, the influence of the thickness of the shear panel and the contribution of the concrete. Yorgun and Bayramoglu [5] studied the effect of the gap between the end plate and the column flange on the performance of the joint under cyclic loading. Dubina *et al.* [6] studied the initial stiffness, moment capacity and plastic rotation capacity of end-plate joints. Similar studies were carried out by Summer and Murray [7]. Finally, Beg *et al.* [8] studied full-scale bolted end-plate connections, where the main goal was the study of the influence of loading speed in symmetric bolted connections. A thorough review of the behaviour of steel joints under cyclic loading is presented in Nogueiro *et al* [9].

2 DESCRIPTION OF THE TEST PROGRAMME

2.1 Test details and instrumentation

The experimental programme, here presented, corresponds to the first part of a wider study and comprises six external double end-plate joints. It is divided into two Groups, whereby the column section size is varied, as can be observed in table 1. End plates, 18 mm thick, were connected to the beam-ends by full strength 45° continuous fillet welds, shop welded in a down and up position. A manual metal arc welding procedure was used, with Autal Gold 70S electrodes. Hand tightened, full-threaded M24, 10.9 grade, in 26 mm diameter drilled holes were employed in all joints. All the material is steel grade S355. Figures 1 and 2 show the connection details.

Table 1: Bolted Beam-To-Column extended end-plate joints test programme.

Group 1 (J1)	Beam IPE	Column HEA	steel S355	type	Bending	Axial
Test J-1.1	360	320	“	Monotonic	M-	-
Test J-1.2	360	320	“	Cyclic	M-/M+	-
Test J-1.3	360	320	“	Cyclic	M-/M+	-
Group 3 (J3)	IPE	HEB	S355	type	Bending	Axial
Test J-3.1	360	320	“	Monotonic	M-	-
Test J-3.2	360	320	“	Cyclic	M-/M+	-
Test J-3.3	360	320	“	Cyclic	M-/M+	-

Each group has the first test with the loading applied monotonically, and two more tests with cyclic loading. The cyclic loading strategy consists of 2 distinct cyclic histories (i) increasing cyclic amplitude in the elastic range and constant amplitude loading at approximately $\phi_y \times 3$ and (ii) increasing cyclic amplitude in the elastic range and constant amplitude loading at approximately $\phi_y \times 6$. After twenty cycles, the amplitude loading was increased by 2.5 mrad. The tests were carried out with displacement control, with constant speed of 0.02mm/sec for the monotonic tests, 0.4 mm/sec for the first cyclic tests and 0.2 mm/sec for the second cyclic tests.

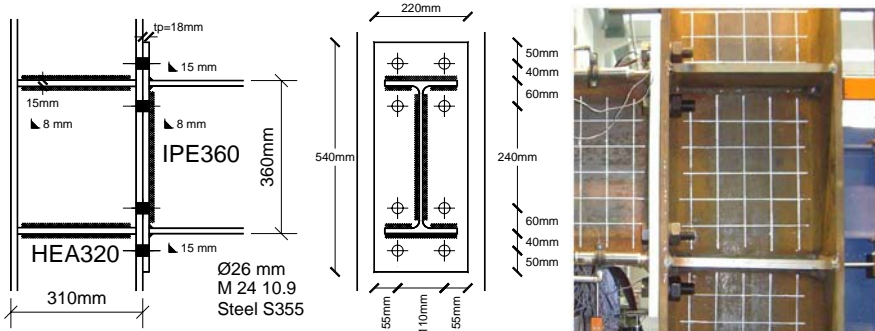


Figure 1: Detail of the joint for the Group 1; (a) Geometrical and (b) Illustration.

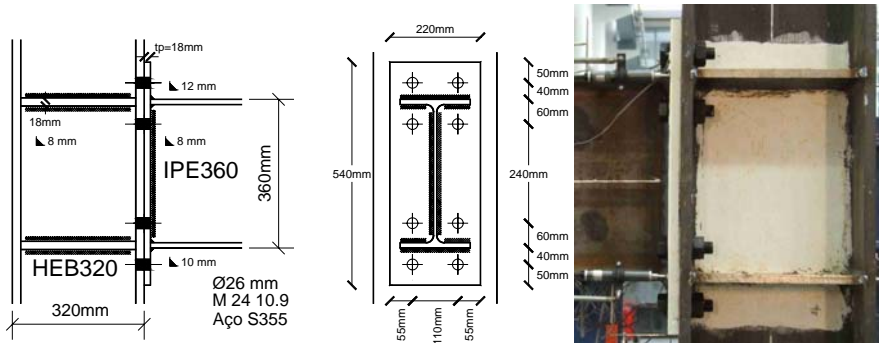


Figure 2: Detail of the joint for the Group 3; (a) Geometrical and (b) Illustration.

The choice of steel members and connection details result from the design of the Cardington building using EC0/EC1/EC3/EC4 [10] [11] [12] [13] and EC8 [1], with an alternative choice of columns (HEA and HEB) to match the seismic design criteria.

To get all the test information, the models are instrumented with displacement transducers, shown in figure 3a and electrical extensometers (figure 3b). All the instrumentation is connected to a data logger, as well as the load cells, which records all measurements every 10 seconds. The displacements were measured by means of TML displacements transducers. Strain gauges TML PFL-10-11-1L, FLK-1-11 and FRA-6-11 (general use, with 2%, 3% and 3% maximum strain, respectively) were used to measure the surface deformation in the points shown in the figure 3b. The bolt deformations were measured with special TML BTM 1C strain gauges embedded in the axis of the bolt.

The columns are 3.0 m high and have web transverse stiffeners in the direction of the beam flanges, and the beam is approximately 1.2 meters long. The loading is applied in the vertical direction, at the end of the cantilever beam by means of a 100 Ton hydraulic actuator, as can be seen in the figure 4a.

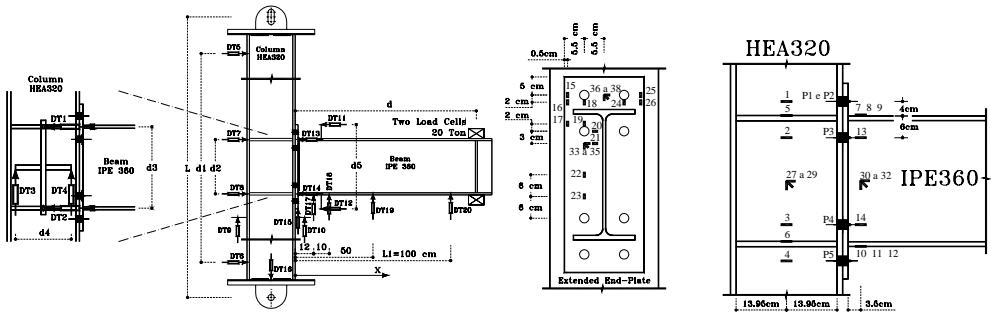


Figure 3: Plan of the models instrumentation; (a) displacements transducers and (b) electrical extensometers.

Figure 4b depicts the experimental test setup for joint J-3.3 which is comprised by a reaction wall in the left side, a loading steel beam at the top and a steel frame in the right side anchored to the floor. The models are supported through a concrete block with a steel pinned joint in the bottom. In the top of the model the connection to the steel beam has the vertical displacement free.

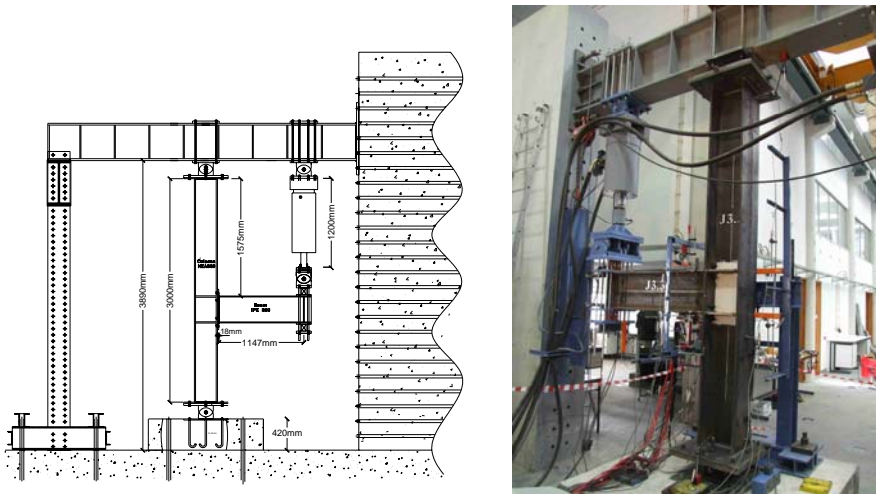


Figure 4: (a) Laboratorial test setup (back view) and (b) Illustration (front view).

2.2 Evaluation of the results

One of the main goals of this work it is to know the Bending-Moment versus Rotation joint curve ($M-\theta$), as well as knowing all their behaviour. The hysteretic curves can be obtained from different ways, this to say, from different points in the beam.

The applied bending moment is given by the applied force, F , at the end of the cantilever beam multiplied by the distance, d , between the load application point and the column flange, as can be observed in the figure 3a. The rotation of the joint is given by equation (2) and can be obtained from several redundant measurements along the beam, corrected by deducting the elastic rotation of the column and the beam and the rigid body rotations (equations (3) to (5)). The rigid body rotation is given by equation (6). The elastic rotation can be obtained by means of the equation (7), both for the elastic

rotation of the column and elastic beam rotation. The elastic rotation of the column is only valid for this test model, with the dimension shown in the figure 4a.

$$M=F \times d \quad (1)$$

$$\varnothing_{\text{joint}} = \varnothing_{\text{total}} - \varnothing_{\text{model}} - \varnothing_{\text{elastic}} \quad (2)$$

$$\varnothing_{\text{total}} = \text{artan} \frac{\delta_{DT20} - \delta_{DT15} - \delta_{DT16}}{L_1} = \text{artan} \frac{\delta_{DT19} - \delta_{DT15} - \delta_{DT16}}{0.50} \quad (3)$$

$$\varnothing_{\text{total}} = \text{artan} \frac{\delta_{DT11} - \delta_{DT12}}{d_5} \quad (4)$$

$$\varnothing_{\text{total}} = \text{artan} \frac{\delta_{DT1} - \delta_{DT2}}{d_3} + \text{artan} \frac{\delta_{DT3} - \delta_{DT4}}{d_4} \quad (5)$$

$$\varnothing_{\text{model}} = \text{artan} \frac{\delta_{DT5} - \delta_{DT6}}{d1} \quad (6)$$

$$\varnothing_{\text{elastic}} = (Mx - \frac{Fx^2}{2})/EI_b + 0.08898 \frac{ML}{EI_c} \quad (7)$$

3 RESULTS

3.1 Monotonic tests

Figures 5a and 5b below present the results of the monotonic tests. Besides establishing the failure modes of the joints, these tests help to establish the basic mechanical joint properties, like the initial stiffness, the strength, the yield rotation and the ultimate rotation. In addition, this information is very important to establish the cyclic loading strategy.

Theoretically, according to EC3 [12], joint J-1.1 presents an initial stiffness equal to 61694 KNm/rad, a moment resistance of 272.71 KNm and it is expected to fail from the column web panel in shear. As can be observed in figure 5a, an initial stiffness equal to approximately 69500 KNm/rad was obtained and a resistance approximately of 288 KNm, and a ultimate moment resistance of 419 KNm. The observed yield rotation was 4.14 mrad and the ultimate rotation approximately 70 mrad. The observed experimental failure mode was the column web panel in shear, although the model was further loaded up to end-plate failure, as can be seen in detail in figures 6a and 6b. At failure the model exhibits large deformation, in excess of 100 mrad. The column web panel and the extended end-plate were responsible for the majority of the energy dissipation.

Similarly, according to EC3 [12], joint J-3.1 presents an initial stiffness of 75400 KNm/rad, a moment resistance of 287.83 KNm and it is expected to fail from the column web panel in shear. As can be observed in figure 5b, an initial stiffness equal to approximately 100000 KNm/rad was obtained and a resistance approximately of 336 KNm, and a ultimate moment resistance of 477 KNm. The observed yield rotation was 3.37 mrad and the ultimate rotation approximately 47 mrad. The observed experimental failure mode was the column web panel in shear. No bolt failure was observed for both tests.

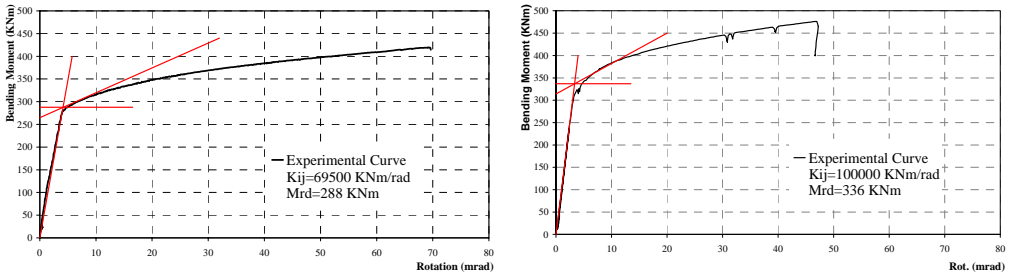


Figure 5: Experimental monotonic curve; (a) J-1.1 test and (b) J-3.1 test.

Both joints exhibit high strength and stiffness. It is important to note that, for test J-1.1, the nominal plastic strength of the connected beam (IPE360) is 361.75 KNm, giving a degree of partial strength of 0.80. It is noted that the real degree of partial strength is lower, given that the actual yield stress of the steel beam, is significantly higher (tensile coupon tests are being carried out at this stage). This behaviour was justified, rather than by the steel grade, by the transversal web column stiffeners and by the extended end-plate geometry, thickness and distance of the bolts.

To understand the failure initiation mechanism for test J-1.1, a metallographic study was carried out on a small specimen of the extended end-plate (figure 7). It can be concluded that failure occurred in the heat-affected zone (HAZ).

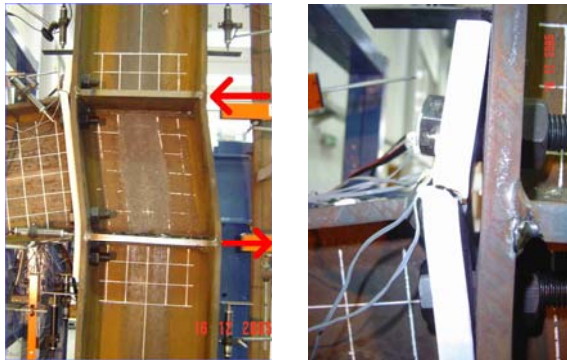


Figure 6: Illustration of the final deformation of the test J-1.1; (a) web column shear panel and (b) extended end-plate.



Figure 7: metallographic study: (a) J-1.1 test beam after loading; (b) specimen and (c) Microstructure of the material increased 50 times.

3.2 Cyclic tests

Figures 8a and 8b present the hysteretic $M-\theta$ experimental curves, respectively for J-1.2 and J-1.3 tests. The chosen load strategy, had the main propose to study the oligocyclic fatigue that is typical of steel behaviour usually called. Stable hysteretic curves for both tests with a great energy dissipation can be observed. The first cyclic test (J-1.2), which had lower amplitudes, reached 82 cycles against 22 reached for the J-1.3 test, as can be observed in figures 9a and 9b, respectively. The mode of failure observed for these two tests was the same, cracking in the extended end plate at the interface between the weld and the base material, as can be observed in figure 10a. A great part of the energy was dissipated by the column web panel (80%) and by the extended end-plate, as can be seen in figures 10b and 10c, respectively.

From these hysteretic curves, can be concluded that these joints do not exhibit any slippage, do not have strength deterioration and the deterioration of stiffness is low.

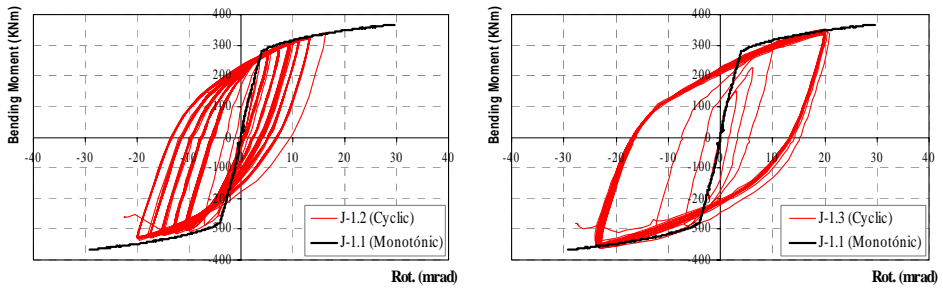


Figure 8: Hysteretic $M-\theta$ experimental curves; (a) J-1.2 test and (b) J-1.3 test.

Figures 11a and 11b present the hysteretic $M-\theta$ experimental curves, respectively for J-3.2 and J-3.3 tests. The cyclic J-3.2 test, reached failure after 26 cycles on the extended end-plate as can be seen in figure 14a. This point is highlighted with an arrow on the hysteretic curve. Beyond that, the loading was maintained until the extended end plate suffered complete rupture, along the extended part, as can be observed in figure 14b. From this point onwards, a hysteretic curve characteristic of flush end-plate joints was observed, with significant slippage. Despite this, the joints maintained significant resistance, at lower levels, and energy dissipation, as can be observed in the figure 13a. Finally, joint collapse occurred by bolt failure as can be seen in figure 14c.

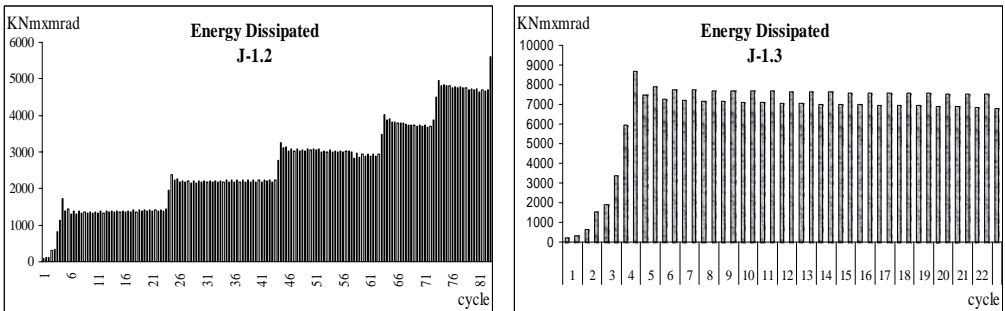


Figure 9: Energy dissipated; (a) J-1.2 test and (b) J-1.3 test.



Figure 10: Illustration of the test J-1.3; (a) end-plate failure; (b) column web panel and (c) end-plate deformation.

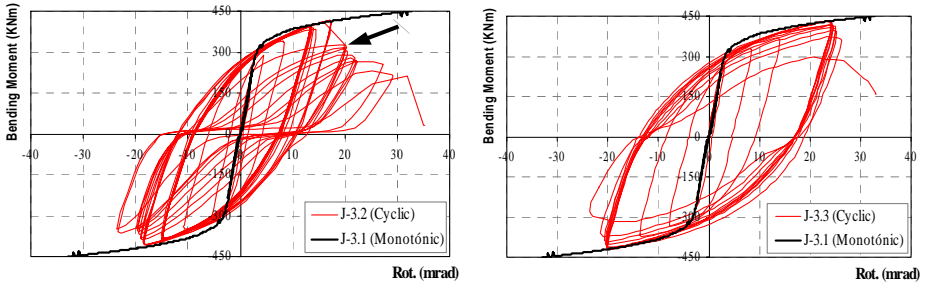


Figure 11: Hysteretic $M-\theta$ experimental curves; (a) J-3.2 test and (b) J-3.3 test.

The J-3.3 model endured 13 cycles only, as can be observed in figure 13b, less than for the J-1.3 test. The same conclusion is valid for tests J-1.2 and J-3.2. This is justified by the column section size, the great part of the joint deformation occurring in the column web panel between the transversal stiffeners, as can be observed in figure 12. For a stronger column, the extended end-plate is relatively weaker, and failure will occur at an earlier stage.

Final failure for the J-3.3 model occurred at the HAZ on the beam side, as can be seen in figure 15b and figure 15c, for a value of the maximum moment of 429.7 KNm.

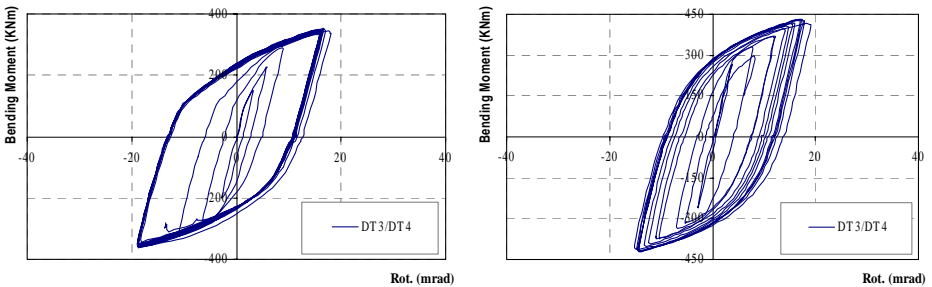


Figure 12: Hysteretic $M-\theta$ column web shear panel; (a) J-1.3 test and (b) J-3.3 test.

The models for Group 1 have less resistance than the models from Group 3, but present a better performance with respect to the energy dissipated, number of cycles to failure, and rotation capacity.

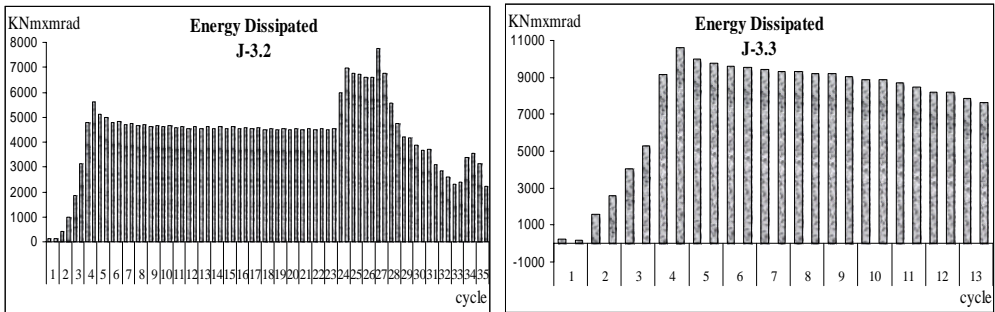


Figure 13: Energy dissipated; (a) J-3.2 test and (b) J-3.3 test.



Figure 14: Illustration of the test J-3.2; (a) Column web panel; (b) end-plate failure and (c) bolts failure.

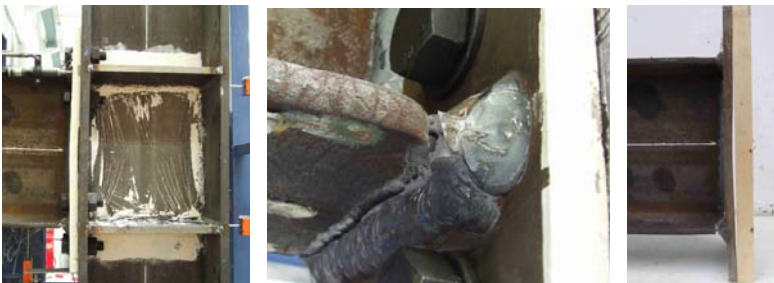


Figure 15: Illustration of the test J-3.3; (a) Column web panel; (b) beam flange failure and (c) end-plate deformation.

4 CONCLUSIONS

The results of six experimental tests on external steel joints are divided in two groups. Each group includes one monotonic and two cyclic tests. From these laboratorial tests some conclusions can be drawn: (i) the monotonic tests give information related to the main mechanical properties like rotation capacity, moment resistance and stiffness of the joint and modes of failure; (ii) a metallographic study for one of these tests showed that failure began in the heat affected zone; (iii) from the cyclic tests, various important aspects can be obtained as the number of cycles for each model before failure, the respectively energy dissipated, the maximum joint strength, the initial and post-limit stiffness; (iv) the stable trajectory for the hysteretic curves can be observed, due essentially to the transversal column web

stiffeners and to the extended end-plate geometry; (v) the joint behaviour is mainly conditioned by the column web shear panel, but also from the deformation of the extended end-plate.

These tests provide a wealth of information that should allow the reanalysis of the current prescription of Eurocode 8, an issue that is currently being actively pursued by the authors.

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