PERFORMANCE OF JOINTS IN STEEL STORAGE PALLET RACKS

Nadia Baldassino[(1)], Claudio Bernuzzi[(2)], Riccardo Zandonini[(1)]

[(1)] Department of Mechanical and Structural Engineering, University of Trento (I)
[(2)] Department of Structural Engineering, Politecnico di Milano (I)

ABSTRACT

Performance of pallet racking systems depends upon the efficiency of beam-end-connectors, which provide, together with column bases, sources of stiffness for down-aisle stability. Knowledge of the actual joint behaviour under static and seismic loading is of fundamental importance for a suitable definition of simplified moment-rotation joint relationships to use into design of semi-continuous frames.
This paper presents the preliminary results of research activities currently in progress in Italy focused on static and seismic behaviour of pallet racking beam-to-column joints.

1. INTRODUCTION

Steel storage pallet racks, which are usually manufactured from cold formed steel members, can be considered typical three-dimensional framed systems (Figure 1). Despite this fact, the design of pallet racks is quite complex, due to the particular geometry of their components. With reference to the European practice, generally beams are realised by means of boxed cross-section members and columns usually contain holes and/or perforations at regular intervals to allow beams and bracings to be attached without bolts or welds (Figure 2).

The behaviour of the perforated columns, which are in many cases thin-walled members, is affected by different buckling modes (local, distortional and global) as well as by their mutual interactions (Hancock, (1); Davies and Jiang, (2)). Furthermore, as shown in figure 1, bracing systems are generally placed only in the cross-aisle direction. The need to organise racking systems in such a way that the product is efficiently stored and sufficiently accessible, hampers in fact the presence of bracing systems in the down-aisle direction.

The model of semi-continuous sway frames (i.e., frames with semi-rigid joints (ECCS (3)), should hence be adopted for structural analysis of pallets racks, taking into account that the response of both beam-to-column and base-plate joints is typically non linear and, in addition, the performance of base-plate connections depends significantly on the level of the axial load (Godley (4); Markazi et al. (5)).

The performance of pallet racking systems significantly depends upon the efficiency of the beam-end-connectors, which provide support to the beams and are, together with column bases, the sole sources of stiffness for the down-aisle stability.

Knowledge of the actual joint behaviour is hence of fundamental importance for a suitable definition of simplified moment-rotation (M-Φ) joint relationships to use in the design analysis of pallet racks systems.

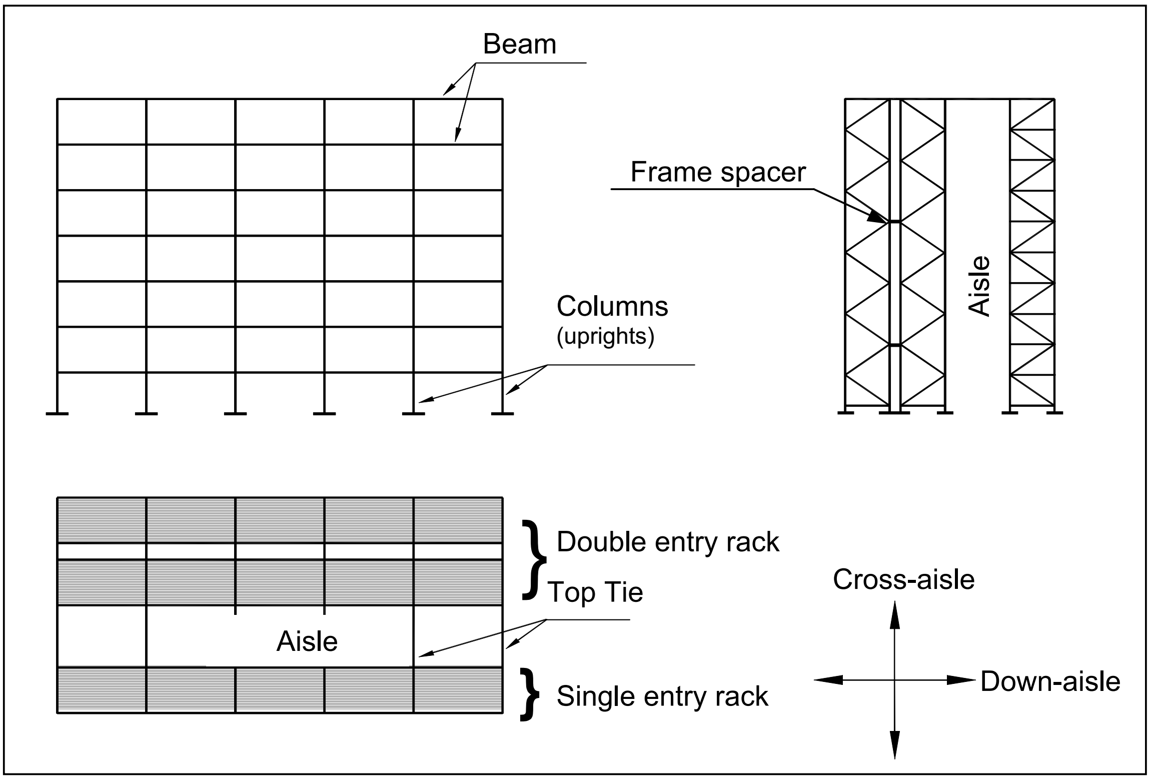


Figure1: Elevation and plan view of steel storage pallet racks.

Due to the great number of types and different geometry of the key rack components, pure theoretical approaches for rack design are not currently available.

Recent design standards for steel storage racks (RAL (6), AS (7), RMI (8), FEM (9)) require, specific tests to evaluate the performance of members as well as of joints in order to understand and to quantify main factors affecting the behaviour of the considered elements and, as a consequence, the response of the whole frame.

The experimental procedures proposed by these design standards, are mainly focused on the knowledge of the static behaviour of pallet racks.

As to racks in seismic zone, only the RMI specification (8) provides practical indications about the seismic design, while the standards for the earthquake resistance of structures don’t refer to rack systems.

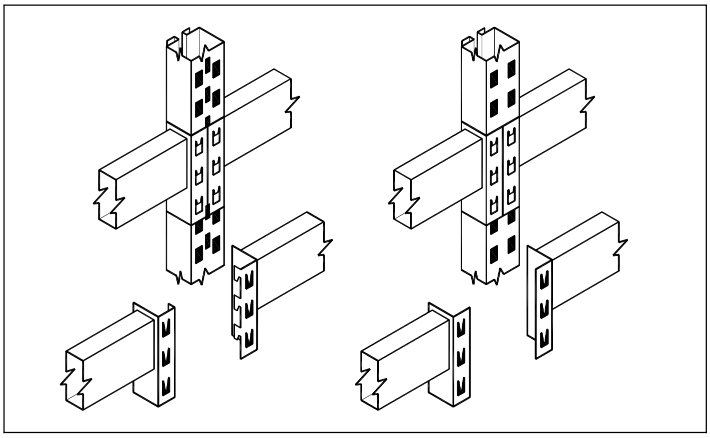


Figure 2: Typical beam-end-connectors of pallet racks.

It should be noted that a suitable design of pallet racks under seismic loading requires the knowledge of the actual cyclic behaviour of the key components, in order to define the performance of possible “dissipative” zones (i.e., the zones in which the energy associated with severe earthquakes could be dissipated). On the authors knowledge, only one research project has been carried out in the past with the aim of investigating the response of racks to dynamic loads (Chen (10)). Nowdays, a direct use of the so-called capacity design approach (Mazzolani and Piluso (11)) is actually prevented, which is based on the concept that the structure possesses sufficient strength, stiffness and absorption capabilities to dissipate the energy associated with severe earthquakes, developing "plastic" mechanisms in dissipative zones. However, the results of a numerical analysis carried out on several planar rack frame configurations in presence of monotonic loading (Baldassino and Bernuzzi (12)) can be considered, in order to have a general indication about the dissipative zones of pallet racks. In particular, it has been shown that frame collapse is generally due to the interaction between instability and plasticity in beam-to-column joints. Columns never achieved their ultimate strength, while, in a very limited number of cases, a plastic hinge occurred approximately at the beam midspan. It seems hence reasonable to assume dissipative zones located at the nodes between beam(s) and column, and the capability to dissipate energy of the racks systems can be considered strictly depending on their hysteretic behaviour (i.e., by their response to cyclic reversal loading). As a consequence, despite the lack of experimental data, a significant influence of beam-to-column joints is expected also on the response of the rack frames in presence of seismic loading.

Research activities on the static and seismic behaviour of pallets racks are in progress in Italy at the University of Trento (Baldassino et al. (13)) and at the Politecnico di Milano (Ballio et al. (14)). One of the main objectives of these studies is to develop simplified design procedures for pallet rack design.

This paper deals with the experimental phase of the researches. In particular, it is focused on the beam-to-column joint behaviour under monotonic and cyclic reversal loading.

The results of 238 monotonic tests on 61 different types of beam-to-column joints performed at the University of Trento are presented and discussed. Moreover, the cyclic behaviour of two different types of beam-to-column joints is presented and discussed on the basis of cyclic tests performed at the Politecnico di Milano.

2. JOINTS IN RACK SYSTEMS

As previously mentioned, the knowledge of the beam-to-column joint behaviour is of fundamental importance for the static and seismic design analysis of pallet racks, owing to the influence of joints on the overall frame performance.

The partial continuity of the rack frame in down-aisle direction is provided by beam-to column and base-plate connections. Moreover the nodal zone between beam(s) and column is expected to be a dissipative zone, influencing remarkably the capability to dissipate energy of the racks systems.

The behaviour of beam-to-column joints under static loading have been extensively investigated, while only few joint tests under cyclic reversal loading have been performed.

In the framework of this research project, the tested specimens consist of a short length column with the ends restrained to the employed counter frame. A cantilever beam is connected to the central zone of the column by the beam-end-connector to test. Specific testing and measuring systems have been designed to analyse the behaviour of the nodal zone (Baldassino et al. (13), Ballio et al. (14)).

Before describing the joint experimental programmes and the main results of the tests, it appears convenient to dwell on the definitions used in the following. In particular, a node is defined as the point at which the axes of two or more interconnected structural elements converge and a nodal zone can be identified where interaction between these members occurs. In this area (Figure 3), one or more joints and connections can be identified. The state of deformation produced by members and by their mutual interactions in the nodal zone is very complex and involves significant local distortions, in rack systems, as well as in multi-storey framed steel buildings (Bernuzzi et al. (15)). Generally, joint response can be described through the sole relationship between the moment in the plane of the down-aisle direction, M, and the associate rotation, Φ, at the beam end section.

In case of rack systems, joint response is mainly influenced by the deformation of beam-end-connectors and of the column nodal zone in shear and bending. As a consequence, these two contributions, indicated in figure 3, as Φ_{bec} e Φ_c, respectively, can be identified in the overall joint rotation Φ.

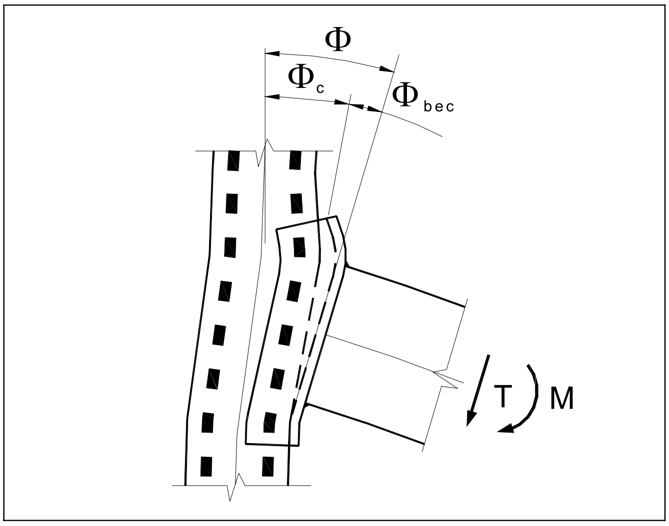


Figure 3: Definition of the main contributions to the overall joint rotation.

3. MONOTONIC TESTS

3.1 Experimental programme

The experimental analysis on beam-to-column connections under monotonic loading comprised of 238 tests on 61 different types of connections.

The tested specimens are characterised by different geometry of the connected members (i.e., beams and columns) as well as of beam-to-column connectors. In particular, it can be noted that:

- beams present close box section. Approximately 80% of the considered beams have regular rectangular sections. The remaining beams are characterised by shapes very similar to the rectangular one;
- columns have in general open perforated section (only 3.3% of the considered columns have close section without perforations). In some cases, columns are simple lipped channels (34.4%). In other cases, additional flanges (called rear flanges) are attached to the lips (37.7%). The remaining column sections have additional lips located at the ends of the rear flanges and normally point outwards (24.6%);
- beam-to-column connections are non symmetrical with reference to the cross- and down-aisle axes (Figure 1). The connection devices are welded to the beams and the connection is physically realised on one side of the column. The typologies of the considered beam-end-connectors are showed in figure 4.

For each type of specimen, four tests were generally executed: three under hogging moments, to appraise the connection behaviour in the usual service conditions, and one under actions generating sagging moments to evaluate the performance in presence of accidental upward action or of frame sway. Generally, tests were interrupted at a high level of connection rotation, out of the range of practical interest for the current usage of beam-end-connectors.

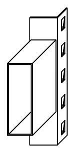
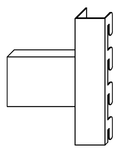
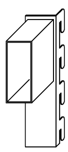
Type of connection		Percentage
C1		52.4
C2		6.5
C3		40.1

Figure 4: Typology of the tested beam-end-connectors.

3.2 Summary of the experimental results

Typical moment-rotation (M-Φ) joint curves obtained from the monotonic tests carried out on one type of joint are reported in figure 5.

The experimental curves are characterised by an initial slippage due to looseness of the beam-end-connector, and three branches can basically be identified under both hogging and sagging moments:

- elastic, characterised by significant value of the rotational stiffness;
- inelastic, with a progressive deterioration of stiffness;
- plastic, with a significant plateau and, in some cases, also a final softening branch.

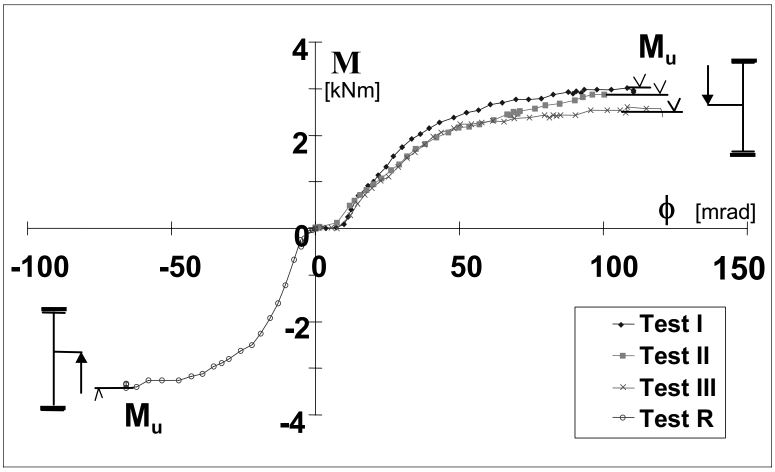


Figure 5. Typical moment-rotation joint curves.

Observed collapses are be due to tearing of the column material, yielding of the bracket material or fracture or yielding of the hook itself.

It shoul be noted that the initial slippage, which can be non negligible, is characterised by a great dispersion. The re-elaboration of the test data showed that under hogging moment, the mean value is 5.91 mrad with an associate standard deviation of 6.16 mrad.

For all the considered types of beam-end-connectors, response under sagging moment was generally characterised by values of rotational stiffness and bending capacity greater than those associated with hogging moment.

3.3 Joint classification

In order to select the frame model (simple, semi-continuous or rigid) to use for the design analysis, the same criteria proposed for joint classification in steel frameworks can be applied to beam-to-column joints for pallet rack systems (Eurocode 3 (16)). Furthermore, in order to have a general idea about the performances of the tested joints, the experimental M-Φ curves related to the response under hogging moments have been directly compared in non dimensional form, in accordance with the EC3 criteria of for classification of joint in unbraced frames. In particular, from the original M-Φ curve, a non dimensional $\overline{m}-\overline{\phi}$ relationship has been obtained and considered. Terms $\overline{m}$ and $\overline{\phi}$ are defined, as:

$$\overline{m} = \frac{M}{M_{p,b}} \tag{1a}$$

$$\overline{\phi} = \Phi \frac{EI_b}{L_b M_{p,b}} \tag{1b}$$

where E is the Young modulus, I_b and L_b are the second moment of area and the length of the beam, respectively, and $M_{p,b}$ represents the beam plastic moment.
With reference to all the $\overline{m} - \overline{\phi}$ joint curves, it should be remarked (13) that:

- for a great number of tests (approximately 31% of the tested specimens) joint response falls in the domain of flexible connection (as curve a in Fig. 6);
- in some cases (in total 14% of the examined joint curves) joints can be considered semi rigid, owing to the value of the rotational stiffness (as curve b in Fig. 6);
- in other cases (in total 9%) joints can be considered semi-rigid, on the basis of the value of the bending strength (as curve c in Fig. 6);
- approximately half (46%) of the tested joints can be considered semi-rigid with reference to both stiffness and strength (as curve d in Fig. 6).

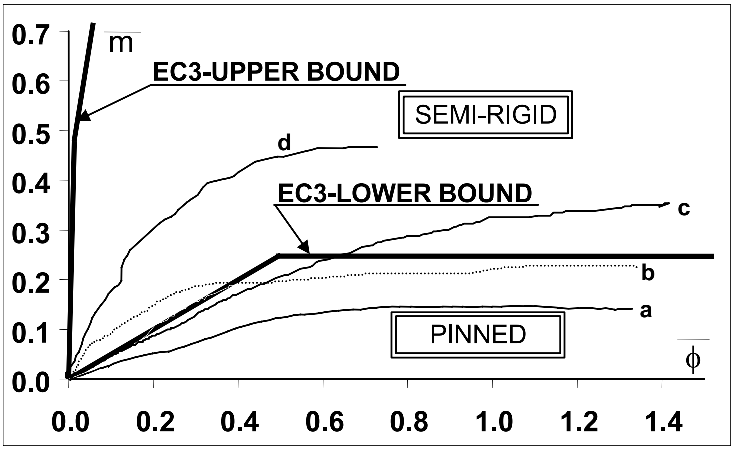


Figure 6: Typical non dimensional moment-rotation joint curves.

From these results related to EC3 joint classification, simple frame model should be used in many cases for the design analysis. However, as it appears from a numerical study on the analysis models for steel buildings (Bernuzzi and Zandonini (17)), joint influence on frame behaviour also in the case of flexible joints is non negligible. Semi-continuous frame design model should hence be always adopted to assess more accurately rack response.

4. CYCLIC TESTS

4.1 Experimental program

The cyclic tests have been performed on two types of beam-to-column connections, which are in the following, named conventionally, A and B and a total of 8 tests have been executed. With reference to the typologies of beam-end connectors showed in figure 4, specimens A are characterised by a connection type C1, while specimens B by a connection type C3.

The tests have been conducted by imposing a constant amplitude loading histories i.e., by performing cycles at the same level of the displacement of the beam end. Several tests have been executed with reference to both symmetrical and unsymmetrical loading histories.

All the tests were interrupted on the basis of the specimen response, directly appraised by the imposed load- beam end displacement relationship, being the scope of the research a general characterisation of the joint behaviour.

No brittle failures due to a sudden collapse of joint components were observed in all the specimens, despite the relevant deformations of the connection devices.

4.2 Summary of the experimental results

With reference to the cyclic tests as a general remark, common for both A and B specimens, it can be said that the form of the hysteresis loops is strictly influenced by the number of executed cycles. In particular, figures 7 and 8 can be considered, related to the tests executed on specimens A and B, respectively, with an imposed displacement of $\pm$75 mm (tests A150S and B150S). Joint response are here presented with reference to the relationship between the non-dimensional moment $\overline{m}$ (Eq. 1a) versus the joint rotation for some selected cycles. It can be noted that:

- the first cycle is very stable and similar to the ones associated with traditional steel components; reloading branches of the first cycle in plastic range are very close to the monotonic responses;
- after the first cycle, the form of the hysteresis loops changes significantly, owing to the influence of the residual deformations of the connection devices. In particular, increasing the number of the cycles, different forms of histeresis loops can be noted, depending on the connector types. In case of A joints, the moment-rotation curve is characterised by loops in which the stiffness of the reloading branches decreases progressively with the development of the test. Otherwise, in case of B joints, the effect of subsequent cycles is an initial branch with a very modest slope, the extension of which increases with the number of cycles. The stiffness of the reloading phases is practically constant and equal to the ones of the first cycle and of the monotonic tests. In case of unsymmetrical loading history, these remarks on the forms of the hysteresis loops are confirmed for both A and B joints;
- in correspondence of the zero load level, residual deformation in the tabs were observed, increasing with the number of the executed cycles. Furthermore, cracks appeared also in the tabs and in the column zones in the vicinity of the slots, the amplitude of which increased during the test.

Moreover, it should be noted that for both the types of tested joints, the cyclic response, except than for the first cycle, is significantly different from the ones associated with joints for traditional steel framed buildings, which are generally characterised by a satisfactory stable behaviour. As a consequence, other tests have been planned in order to analyse the relationship between the loading history and the joint response, with the aim of defining a simplified model capable of simulating the joint moment-rotation curve associated with the generic loading history.

5. CONCLUDING REMARKS

Research activities on the static and cyclic behaviour of beam-to-column joints in steel storage pallet racks has been presented, which are currently in progress in Italy. Monotonic tests have been performed on 61 different types of beam-to-column joints (238 tests), while two types of joints (8 tests) have been tested under cyclic loads.

The analysis of the experimental monotonic results shows that the joints are very flexible, if classified in accordance with Eurocode 3 criteria. However, the actual response of beam-end-connectors provides a non negligible degree of lateral stiffness of the frame and, as a consequence, semi-continuous frame model is always suggested for a more refined and “optimal” design analysis.

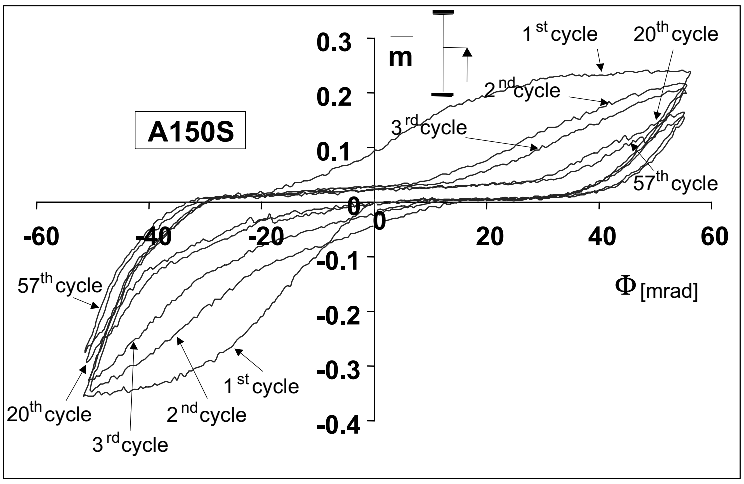


Figure 7: Selected cycles of the $\overline{m}$-Φ curves for A150S joint test.

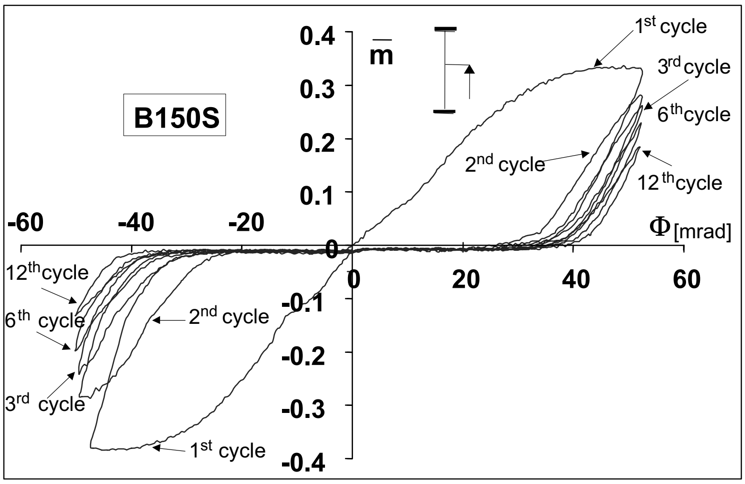


Figure 8: Selected cycles of the $\overline{m}$-Φ curves for B150S joint test.

As to the cyclic tests, it has been pointed out the relevant differences in the form of the hysteresis loops of rack joints in comparison with the ones associated with traditional steel components and the non-negligible influence of the connection systems on the joint behaviour.

This stresses the importance of the definition of an appropriate design philosophy for pallet racks in seismic zones.

Further tests are however required, which are planned for the next future. These tests will allow to investigate the behaviour of the second important source of stability to lateral load, i.e., base plate joints.

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REFERENCES

(1) Hancock, G.J. (1985). “Distortional Buckling of Steel Storage Rack Columns.” *Journal of Structural Engineering, ASCE* 111:12, 2770-2783.
(2) Davies, M. and Jiang, C. (1998). “Design for Distorsional Buckling.”, *Journal of Constructional Steel Research* 46:1-3, 174-175.
(3) ECCS-European Convention for Costructional Steelworks. (1992). *Analysis and Design of Steel Frames with Semi-Rigid Joints*, publication n. 67.
(4) Godley, M.H.R. (1991). Storage Racking, chapter 1*, Design of Cold Formed Steel Members*, Rhodes ed. Elsevier Applied Science, London, UK, 361-399.
(5) Markazi, F.D., Beale, R.G. and Godley, M.H.R. (1997). “Experimental Analysis of Semi-Rigid Boltless Connectors.”, *Thin-Walled Structures,* **28:1**, 57-87.
(6) RAL. (1990). *Storage and Associated Equipment*, RAL Deutsches Institut fur Gutersicherung und Kennzeichnung, (German Institute for Quality Assurance and Marketing).
(7) AS4084. (1993). *Steel Storage Racking*, Standards Australia.
(8) RMI. (1997). *Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks*, Rack Manufactures Institute.
(9) FEM. (1997). *Recommendation for the Design of Steel Pallet Racking and Shelving*, Section X of the Federation Europeenne de la Manutention.
(10) Chen, C.K. (1980). “Seismic study on industrial steel storage racks”, prepared for the Antional Science Foundation, URS/ John A. Blume & Associate Engineers, June.
(11) Mazzolani, F.M. and Piluso, V. (1996*). Theory and Design of Seismic Resistant Steel Frames*, Champmann and Hall, E&FN Spon.
(12) Baldassino, N. and Bernuzzi, C. (2000). “Analysis and behaviour of steel storage pallet racks.”, accepted for publication on *Thin-Walled Structures*.
(13) Baldassino, N., Bernuzzi, C., Zandonini, R. and Hancock, G. (1998). “Overall, local and distortional buckling in pallet racks.”, *Proc. of the Structural Stability Research Concil Conference (S.S.R.C.)*, September*,* Atlanta U.S.A. (to appear).
(14) Ballio, G., Bernuzzi, C. and Castiglioni, C.A. (1999). “An approach for the seismic design of steel storage pallet racks.”, spec. Ed. of *Stahlbau*, november.
(15) Bernuzzi, C., Zandonini, R. and Zanon, P. (1991). “Rotational behaviour of extended end plate.”, *Costruzioni Metalliche,* n°2, 74-103.
(16) CEN. (1994). *Eurocode 3: Design of Steel Structures – Part 1.1 General Rules and Rules for Buildings*, European Commitee for Standardization.

(17) Bernuzzi C. and Zandonini R. (1993). “Serviceability and Analysis Models of Steel Buildings.”, *International Colloquium on Structural Serviceability of Buildings*, Goteborg, Sweden, June.