Low-cycle fatigue behaviour and damage assessment of semi-rigid beam-to-column connections in steel

Autor(en): **Castiglioni, Carlo A. / Calado, Luis**

Objekttyp: **Article**

Zeitschrift: **IABSE reports = Rapports AIPC = IVBH Berichte**

Band (Jahr): **75 (1996)**

PDF erstellt am: **23.06.2024**

Persistenter Link: https://doi.org/10.5169/seals-56915

Low-Cycle Fatigue Behaviour and Damage Assessment of Semi-Rigid Beam-to-Column Connections in Steel

Carlo A. Castiglioni
Associate Professor
Structural Engineering Dept.
Politecnico di Milano
Italy

Luis Calado
Associate Professor
Instituto Superior Técnico
Civil Engineering Dept.
Portugal

Carlo A. Castiglioni, born 1956, received his laurea in Civil Engineering from Politecnico di Milano in 1980. He was part of the Visiting Faculty at Lehigh University, USA, in 1984. After being appointed Associate Professor at La Sapienza University in Rome in 1992, he is presently at Politecnico di Milano. His main research fields are high and low cycle fatigue of steel structures.

Luis Calado, born 1958, received his MSc in Structural Engineering in 1985 and his PhD in Civil Engineering in 1989 from Technical University of Lisbon. Presently he is Associate Professor at the Civil Engineering Department of the Technical University of Lisbon. His main research field is seismic behaviour and design of steel structures. He is also partner in a consulting company for structural design.

Summary

A research was carried out in co-operation between Politecnico di Milano and Instituto Superior Técnico to investigate the behaviour of semi-rigid beam-to-column connections in steel under low-cycle fatigue and to try to establish possible classes of low-cycle fatigue resistance similar to those existing for structural details under high cycle fatigue. Different typologies of beam-to-column connections were experimentally and numerically studied, and a general failure criterion was proposed for steel components under low-cycle fatigue.

1. Introduction

Following the Northridge Earthquake of January 17, 1994 several cases were reported of steel frame buildings which did not collapse, yet exhibited significant structural damage, with local failures of steel structural members or their connections. Such failures, however, did not result in severe overall deformations, thus remaining hidden behind undamaged architectural panels. A variety of local collapses was observed, among which the recurrent case was identified in the failure of welded beam-to-column joints in moment resisting space frames. Investigations [1] have been carried out about the nature and the causes of the observed failures and have shown that several topics pose unsolved problems, thus providing subjects of primary importance for research programs.

In design of steel structures it is in general assumed that the behaviour of connections is rigid or pinned, although their real behaviour is intermediate. Interest in this type of approach is recently decreased because was recognised that it does not allow a full comprehension of the real structural behaviour, and that leads to an economically non competitive design.

On the contrary, interest in semi-rigid connections is widely increased in the last 15 years, and several numerical and experimental research studies were performed all over the world. Most of these studies were carried out on the static behaviour of connections [2], and limited information are available on their cyclic response [3-7]. In addition to these and other experimental researches, a number of numerical models were also developed by various authors. Most of these models are empirical, and need experimental results for calibrating the various parameters assumed as governing the joint behaviour.

Since 1985 a co-operation between the Instituto Superior Técnico in Lisbon and the Politecnico of Milano was activated and several research programs in the field of the seismic behaviour of steel structures were performed.

This paper presents the results of a research on the low cycle fatigue behaviour of semi-rigid steel beam-to-column connections. Four different typologies of connections were realised and tested in a multi-specimen program in order to obtain information regarding the behaviour of different connections, and to verify the validity of the damage accumulation model and failure criterion presented in [8].

2. Experimental research

2.1 Test set-up

The experimental set-up was designed in order to simulate the conditions of different members or connections within the frame structure. It consists mainly in a foundation, a supporting girder, a reaction wall, a power jackscrew and a lateral frame. The power jackscrew, which displays a 1000 kN capacity and a 400 mm stroke, is attached by means of pretensioned bolts to the reaction element. Specimens are connected to the supporting girder. Lateral frames were designed to prevent specimens lateral displacement.

2.2 Test specimens

The specimens consisted of a beam attached to a column by means of four different details, which represent frequent solutions adopted in steel construction: bolted web and flanges cleats (BCC1), extended end plate (BCC2), flange plates (welded to the column and bolted to the beam) with web cleats (BCC3), and beam flanges welded to the column with bolted web cleats (BCC4). For each typology several specimens were realised and tested, according to a multi-specimen testing program.

The profile used for columns and beams in all specimens was a HEA120 in Fe360. For the web and flanges cleats specimens 100x100x10 angles in Fe360 were adopted. Bolts used in all specimens were M16, grade 8.8; in the case of flange plates with web cleats specimens and of extended end plate connections, bolts were preloaded according to EC3 provisions. All welds were full penetration butt welds.

Specimens were fabricated in order to simulate field conditions, following the procedures of workmanship and quality control as required by applicable standards. This applies particularly to welding and bolting. Specimens were instrumented with LVDTs, measuring namely, the vertical displacement of the joint and the relative and absolute rotation of the cross-section and joint.

2.3 Loading histories

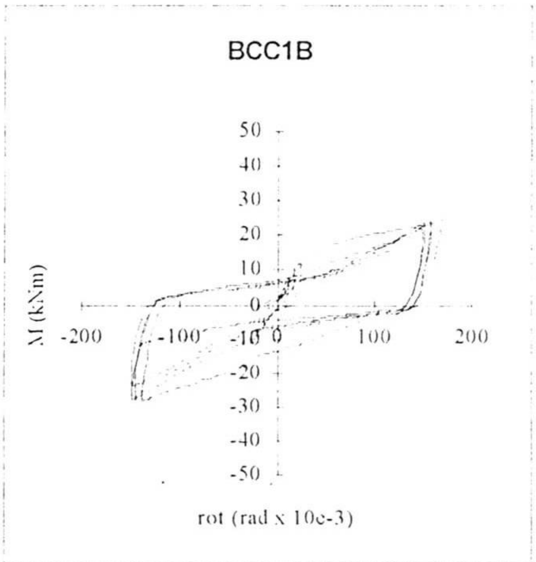
The choice of a testing history associated to a testing program depends on the purpose of the experiment, type of test specimen and type of anticipated failure mode. However, as it is also clearly stated in ATC Guidelines [9], a multi-specimen testing program is needed if a cumulative damage model is to be developed for the purpose of assessing the performance of a component under arbitrary loading histories. In particular a cumulative damage model may be adopted to evaluate the cumulative effect of inelastic cycles on a limit state of acceptable behaviour. The deformation amplitudes for the tests should be selected so that they cover the range of interest for performance assessment. The total amplitude of the cyclic rotation $\Delta\phi$ adopted in this research ranges between 5.00 and 12.00 ϕ_y where ϕ_y is the yield rotation of the connection. All specimens were initially subjected to four cycles in the elastic range with total amplitude of 0.50 ϕ_y, 1.00 ϕ_y, 1.50 ϕ_y and 2.00 ϕ_y.

3. Experimental results

Test results as hysteresis loops in a moment-rotation diagram (M-ϕ) and respective failure mode for each typology of beam-to-column connection are presented in Figures 1 - 4, together with some comments on the behaviour of each type of connections.

3.1 Bolted web and flanges cleats connections (BCC1)

These connections are characterised by large slippage, that seems to increase, with the number of cycles, although the displacement amplitude is constant.



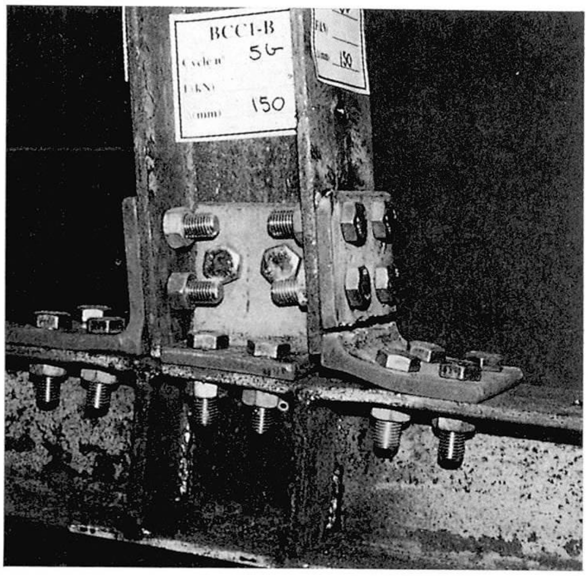
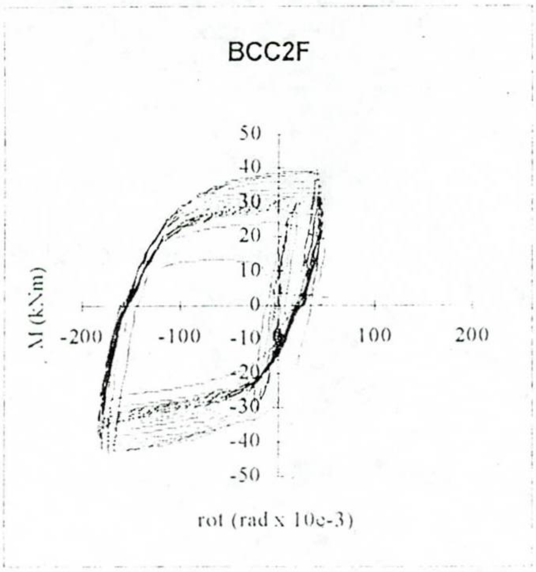


Fig. 1. Experimental moment-rotation diagram and failure mode of BCC1 connection type.

Increment of plastic deformations in the angles produced progressive deterioration of absorbed energy (Fig. 5). Slip occurred mainly between the beam flange and the angle leg due to ovalization of the holes. For all specimens collapse was caused by cracking in the angles under tensile loading which propagated, with increasing the number of cycles, until complete failure.

3.2 Extended end plate connections (BCC2)

These connections are characterised by regular histeresis loops without any slippage and with regular deterioration of the absorbed energy (Fig. 5) and the maximum moment at the end of each cycle (Fig. 2). For all specimens, a plastic hinge developed in the beam approximately at a distance equal to the height of the profile. Local buckling produces deterioration of absorbed energy and maximum moment capacity. Large deformations at plastic hinge induce cracking in the beam flange leading, in a few cycles, to complete failure of the cross-section.



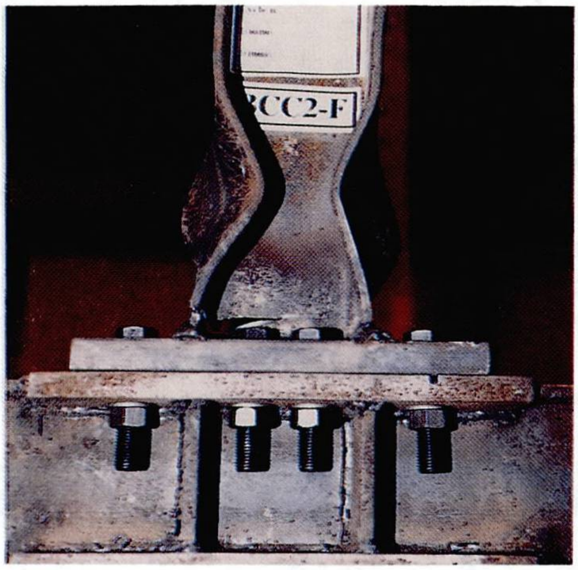


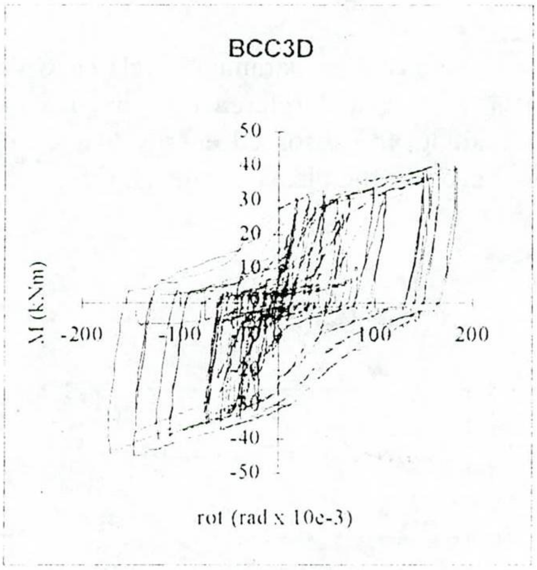
Fig. 2. Experimental moment-rotation diagram and failure mode of BCC2 connection type.

3.2 Bolted flange plates (welded to the column) with web cleats (BCC3)

This type of connection exhibited slippage between flange plates and beam flange due to ovalization of the holes, but this phenomenon has lower importance when compared with web and flanges cleats connections. During the test, a progressive deterioration of the absorbed energy (Fig. 6) took place. The flange plate had a similar behaviour as the angles in BCC1 type under bending deformation but no evident separation between beam and column was observed. Failure was due to cracking at welding connecting the flange plates to the column flange. These cracks propagated with increasing the number of cycles until complete failure.

3.4 Welded flanges with bolted web cleats connections (BCC4)

These connections are characterised by high elasticity and great regularity in the shape of the loops with gradual deterioration of the absorbed energy (Fig. 6). Large plastic deformations developed in the panel zone of the column. In most cases failure was due to cracking at toe of welding connecting the beam flanges to the column one. These cracks propagated with increasing the number of cycles until complete failure. Propagation was observed either in the beam flange or in the column one, where due to the presence of other weldings connecting transversal stiffeners to the column web and flanges, complete separation of the column flange in the panel zone was observed (Fig. 4). Large ovalization was always observed due to high bearing stresses in the bolts of the web cleats.



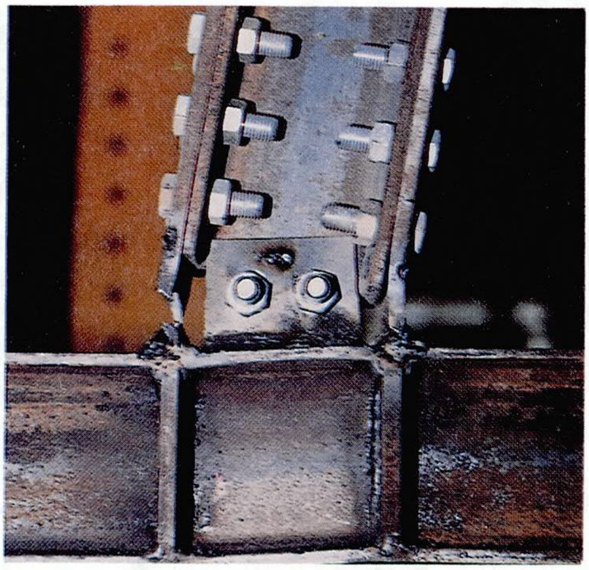


Fig. 3. Experimental moment-rotation diagram and failure mode of BCC3 connection type.

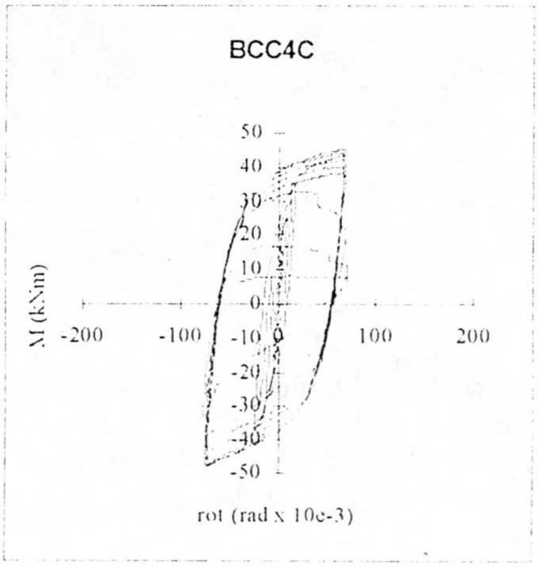


Fig. 4. Experimental moment-rotation diagram and failure mode of BCC4 connection type.

4. Failure criterion

As previously stated, aim of the research was also the development of a failure criterion, to be adopted together with a damage model for the assessment of low cycle fatigue resistance. It is particularly interesting to formulate some failure criterion based on the achievement of a given level of deterioration of the mechanical properties of the material. In fact, by means of such a collapse criterion, the limit state at which a structural component is considered out-of-service, can be a-priori defined. Such a situation, of course, may not coincide with actual collapse of the component. However, in order to be applied in standard design procedures, such a collapse criterion must allow an assessment of the failure conditions as close to reality as possible, and always on the safe side.

Previous experience indicated that such a criterion might be based on parameters related with the energy absorption capacity of the component. For this reason, with reference to the present research program, that considered constant amplitude loading, the absorbed energy E in each cycle has been related to the one E_0 absorbed in the first cycle in the plastic range, as shown in Figs. 5 and 6.

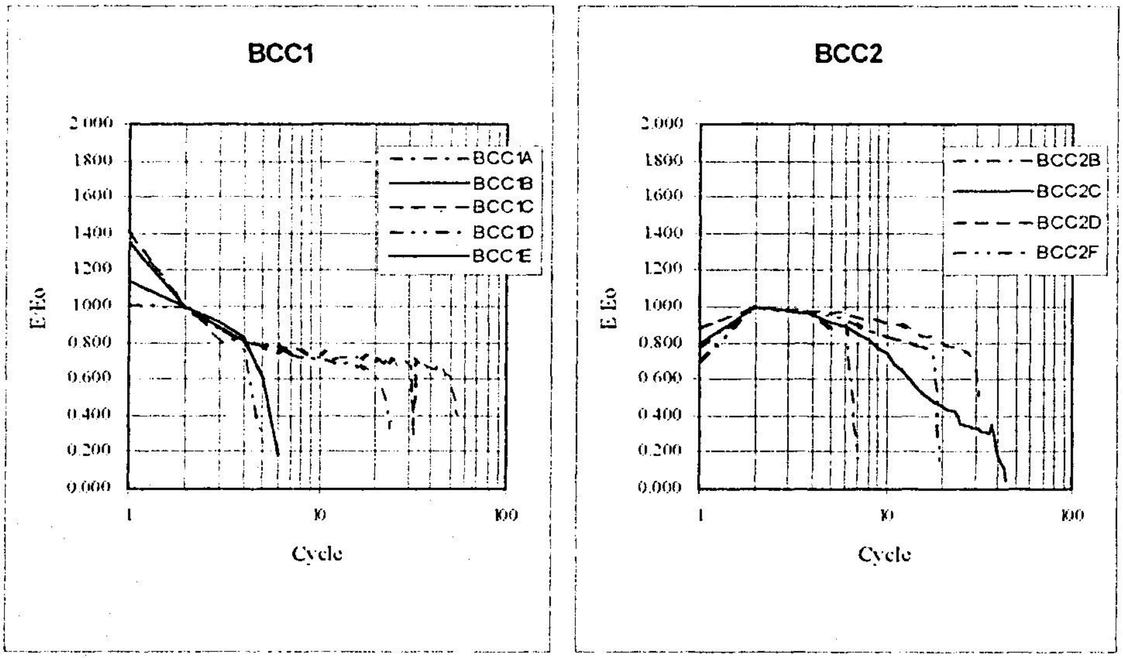


Fig. 5. E/E_0 versus number of the cycle for BCC1 and BCC2 connection types.

It can be noticed that after a stage of nearly constant reduction of the ratio E/E_0, collapse is reached in a few cycles (representing nearly 5% of the total number of cycles to failure), in which an abrupt reduction of the energy absorption capacity takes place. It can also be noticed that this abrupt reduction occurs when the ratio E/E_0 reaches a value which seems to be a constant, independently on the typology of the connection.

Based on this experimental evidence the following failure criterion can be formulated having a general validity for structural steel components:

$$\frac{E}{E_0} \leq \alpha \tag{1}$$

In this equation α is a parameter whose value represents a limit to the reduction of energy absorption capacity beyond which it might be assumed that failure occurs, and should be determined by fitting the experimental results. The current research program showed that values for α ranging between 0.50 and 0.70 can be adopted. It should however be noticed that, in general, the difference between the number of cycles corresponding to the extreme values of the suggested range is a very small percentage of the total number of cycles, and that a value of α equal to 0.50 might be assumed.

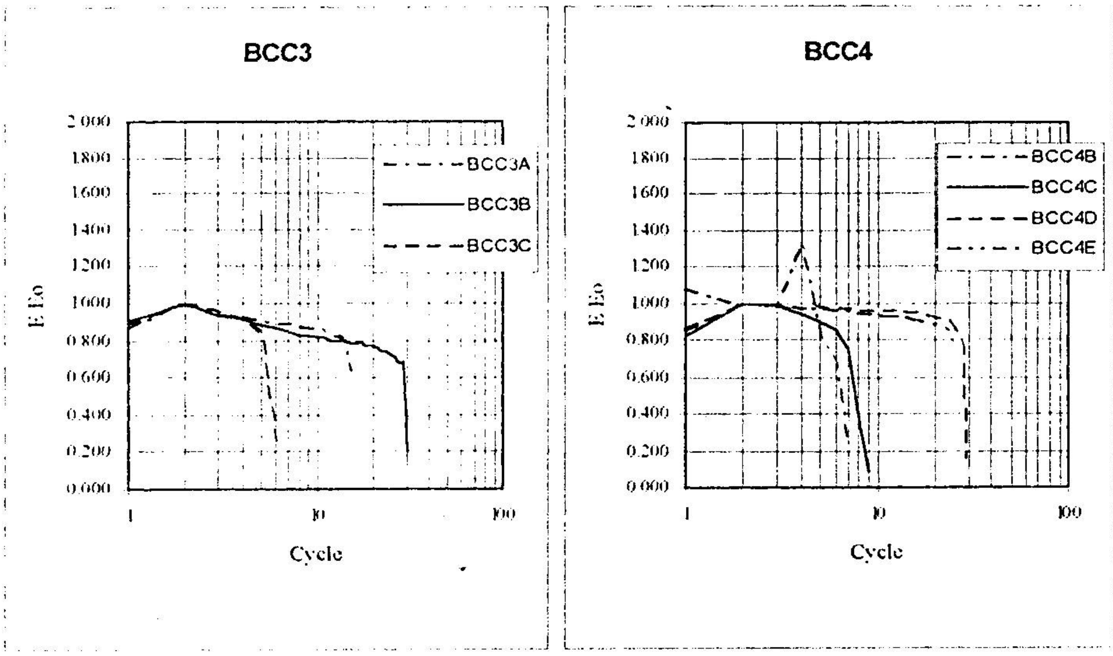


Fig. 6. E/E_0 versus number of the cycle for BCC3 and BCC4 connection types

This value is not to be considered as the best fit of experimental results, but can be regarded as possible reference value in damage assessment procedures. Of course, to be adopted for design purposes, an appropriate safety factor should be assumed, based on safety and reliability considerations.

5. Cumulative damage model

From these tests results as well as those carried out by other authors [10,11] it was noticed that, in good agreement with other previous studies [12,13], for all structural components (beams, beam columns, welded joints, beam-to-column connections), the relationships which best fitted the experimental results in terms of cycle amplitude and number of cycles to failure N_f, were exponential functions of the type:

$$N_f = a\left(\frac{\Delta s}{s_y}\right)^b \qquad (2)$$

where a and b are constant parameters to be calibrated on experimental results, Δs is the cycle amplitude in terms of generalised displacement component, and s_y is the yield value of s.

the design and damage assessment procedures for steel structures under low and/or high cycle fatigue.

In practice, by adopting the S-N curves of EC3 [16], and adopting Miner's rule [17], the proposed damage model becomes:

$$D = \frac{1}{K} \sum_{1}^{L} n_i \left(\frac{\Delta s_i}{s_y} \sigma(F_y) \right)^3 \tag{3}$$

where K is a constant value tabulated by EC3, depending on the fatigue strength category of the detail, n_i is the number of occurrences of cycles having an amplitude Δs_i, the summation is extended to the number L of different cycle amplitudes Δs_i to be considered, and $\sigma(F_y)$ is the stress corresponding to first yield [15].

An equivalent stress range of constant amplitude, that equals the variable amplitude damage for the total number of applied stress cycles, can also be derived:

$$\Delta\sigma^{*}_{eq} = \left[\sum_{1}^{L} \frac{n_i}{N_f} \left(\frac{\Delta s_i}{s_y} \sigma(F_y) \right)^3 \right]^{\frac{1}{3}} \tag{4}$$

This value should be used when plotting, on S-N diagrams, data from variable amplitude tests.

6. Re-elaboration of test results

Figure 7 shows the test data for beam-to-column connections tested under constant amplitude loading (BCCi-Lisbon) together with tests results performed with variable loading amplitude on double angle connections (U.C.-Berkley) [4], top-and-seat angle connections (S.U.N.Y.-Buffalo) [18], top-and-seat angle connections (VPH V.A.-Lisbon) [19] and beam-to-column connections (BCCi-V.A.-Lisbon) [20].

Beam-to-column connections of the same typology, belong to the same fatigue strength category, despite the adopted profiles are different. Furthermore, the fatigue strength of the connection can be directly related to its ductility. For example, bolted web and flange cleats are less ductile then the connection with welded flanges and bolted web cleats. This results, in lower fatigue strength for BCC1 connections and higher for BCC4. BCC2 and BCC3 connections show a similar behaviour, intermediate between the previous ones.

Independently on the category of fatigue resistance pertinent to each typology of the connection, it is important to notice that the slope of the line fitting (in a log-log plot) the low cycle fatigue test data, is nearly -3. This is in good agreement with the results of research on high cycle fatigue.

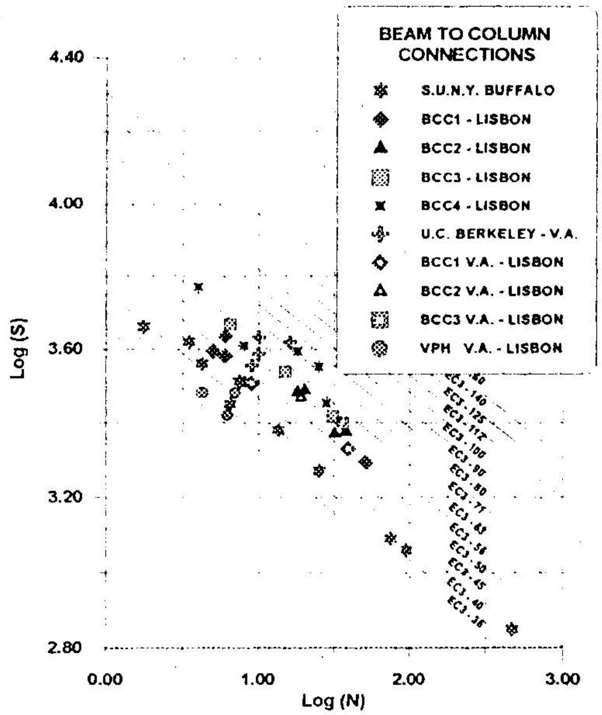


Fig. 7. Fatigue strength of beam-to-column connections.

7. Concluding remarks

This paper presents an approach to the study of the seismic behaviour of beam-to-column connections which is based on methodologies commonly adopted for fatigue damage assessment. In particular, it is tried to extrapolate the procedures usually adopted for high cycle fatigue to the low cycle fatigue range, which is the behaviour to be expect in structural steel members and connections under seismic actions.

In order to be adopted in current design practice, the proposed model requires the definition of the fatigue strength category of various typologies of the connections, i. e., of the appropriate S-N curve to be associated with each type of detail. This can be done either by means extensive experimental research or by numerical modeling. Such models should, however, be calibrated on tests results. In any case a reliable failure criterion must be defined, allowing conservative definition of the number of cycles to failure, i. e. of the conditions corresponding to specimen collapse. A possible failure criterion having a general validity and giving consisting results for a number of structural components has been proposed in this paper.

8. Acknowledgements

The financial support from JNICT, from M.U.R.S.T. 40% and from E.C. "Human Capital and Mobility" "Seismic Protection of the Built Heritage" is gratefully acknowledged.

9. References

[1] SAC Joint Venture, (1995), Interim Guidelines: Evaluation, Repair, Modification and Design of Steel Moment Frames, Report N° SAC-95-02, *Federal Emergency Menagement Agency, FEMA 267.*

[2] Nethercot, D. and Zandonini, R., (1989), Methods of prediction of joint behaviour, *Structural Connections: Stability and Strength. Ed. R. Narayanan, Elsevier Applied Publishers.*

[3] Ballio, G., Calado, L., De Martino, A., Faella, C. and Mazzolani, F., (1987), Indagine sperimentale sul comportamento ciclico di nodi trave-colonna in acciaio, *Costruzioni Metalliche*, N° 2.

[4] Astaneh, A. Nader, M. N. and Malik, L. (1989), Cyclic behaviour of double angle connections, *ASCE, Journal of Structural Engineering*, vol. 115, n.5.

[5] Bernuzzi, C., R. Zandonini and P. Zanon, (1992), Semi-Rigid Steel Connections Under Cyclic Loads, *First World Conference on Constructional Steel Design*, Acapulco.

[6] Popov, E. (1985), Flexibility of steel seismic moment connections. *Connection flexibility and steel frames*, Edited by Wai-Fah Chen.

[7] Korol, R., A. Ghobarah and A. Osman (1990). Extended end-plate connections under cyclic loading: behaviour and design. *Journal of Constructional Steel Research*, vol. 16.

[8] Ballio, G., Calado, L. and Castiglioni, C.A., (1996), Low Cycle Fatigue and Fracture of Structural Steel Members and Connections, *Fatigue & Fracture of Engineering Materials & Structures.* (To be published)

[9] ATC - Applied Technology Council (1992), Guidelines for cyclic seismic testing of components of steel structures, *ATC 24.*

[10] Ballio, G. and Castiglioni, C.A. (1994), Seismic behavior of steel sections, *Journal of Constructional Steel Research*, n.29, pp 21-54

[11] Ballio G. and Chen Y. (1993), An experimental research on beam to column joints: exterior connections, *Proc. C.T.A.*, Viareggio, pp. 110-120.

[12] Coffin, L. F. (1954), A study of the effects of cyclic thermal stresses on a ductile method, trans. *ASME*, 76.

[13] Mason, S. (1954), Behaviour of material under conditions of thermal stress, *Heat Transfer Symposium, NACA TN 2933.*

[14] Wöhler A. (1860), *Zeitschrift für Bauwesen*, vol.10.

[15] Ballio, G., Castiglioni, C.A. (1995), A unified approach for the design of steel structures under low and/or high cycle fatigue, *Journal of Constructional Steel Research*, n.34,pp75-101

[16] CEN (1992), Eurocode 3: Design of Steel Structures-Part 1-1: General rules and rules for buildings, *ENV1993-1-1*, February.

[17] Miner, M.A. (1945), Cumulative damage in fatigue, *Journal of Applied Mechanics*, Sept.

[18] Mander, J.B., Chen, S.S. and Pekcan, G. (1994), Low-cycle fatigue behavior of semi-rigid top-and-seat angle connections, *Engineering Journal, American Institute of Steel Construction*, 3rd quarter, pp 111-122

[19] Calado L. and Ferreira J. (1994), Cyclic behaviour of steel beam to column connections: an experimental research, *Proc. Int. Workshop on Behaviour of Steel Str. in Seismic Areas*, Timisoara, E and FN SPON, pp. 381-389.

[20] Calado, L and Castiglioni, C., (1995), Low-cycle fatigue testing of semi-rigid beam-to-column connections. *3rd International Workshop on Connections in Steel Structures*, Trento.