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STEEL BEAM-TO-COLUMN CONNECTIONS UNDER LOW-CYCLE FATIGUE EXPERIMENTAL AND NUMERICAL RESEARCH

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ABSTRACT

An experimental and numerical research was carried out to investigate the cyclic behaviour of steel beam-to-column connections. Several full scale specimens were submitted, in a multi-specimen testing program, to constant amplitude displacement histories, in order to develop a cumulative damage model, and lead to the assessment of possible classes of fatigue resistance.

Variable amplitude tests were carried out in order to verify the validity of proposed model. In particular, it is shown that the S-N lines given by Codes for high cycle fatigue can be adopted for interpreting the low cycle fatigue behaviour of connections. Miner's rule (1945) can be adopted, together with the previously defined S-N curves and with a cycle counting method to define a unified cumulative damage model valid for both high and low cycle fatigue.

Based on the experimental results of this and previous research programs carried out by the authors, a general failure criterion is also proposed for steel connections and members under low-cycle fatigue, which considers the actual energy dissipation capacity of the component, through plastic deformations.

KEYWORDS

Connections; fatigue; damage; failure; cyclic behaviour; experimental tests.

INTRODUCTION

For the economic and reliable design of a structure in seismic areas it is fundamental to know the cyclic behaviour of all structural elements, as well as how to assess damage accumulation in order to predict structural failure.

In fact, the most recent recommendations and codes for the seismic design of structures consider the possibility of an inelastic design encompassing damage under strong ground motion. In order to apply this design methodology, which is the most appropriate in the case of ductile materials as steel, it is necessary to identify reliable damage accumulation models and failure criterion, as well as damage assessment procedures.

This paper presents the results of a research program carried out in cooperation between the Instituto Superior Técnico of Lisbon and the Politecnico of Milano. Four different typologies of beam-to-column connections were realized and tested in a multi-specimen program in order to verify the possibility of application of the cumulative damage model proposed by Ballio and Castiglioni (1995), and the identification of a unified failure criterion, as proposed by Calado and Azevedo (1989) and Calado and Castiglioni (1995).

Aim of the research was also the assessment of classes of low cycle fatigue resistance for connections, similar to those existing for structural details under high cycle fatigue (e.g. EC3, CEN 1992), because these are important parameters governing the engineering choice during the design process.

TESTING PROGRAM AND LOADING HISTORIES

Test Set-Up

The experimental set-up used in these tests (Calado and Castiglioni, 1995) 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, which has several pairs of holes allowing to set the jack at different heights. The specimens are connected to the supporting girder. The lateral frames were designed to prevent specimens lateral displacement. An automatic testing technique was developed to allow computerized control of the power jackscrew and of all the transducers used to monitor the specimen.

Types of Test Specimens

The specimens were as close as possible to full size in order to minimize size effects. They consisted of a beam attached to a column by means of four different details, which represent frequent solutions adopted in steel construction for beam-to-column connections: bolted web and flange cleats (BCC1), extended end plate (BCC2), flange plates with web cleats (BCC3), and welded flange with bolted web cleats (BCC4). For each typology several specimens were builted 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 flange cleats specimens 100x100x10 angles in Fe360 were adopted. The bolts used in this type of connection were M16 grade 8.8 without preloading. The bolts adopted in flange plates with web cleats specimens and in extended end plate connections were M16 grade 8.8 preloaded according to EC3 provisions, with 150 Nm and 224 Nm torque respectively, while all welds were full penetration butt welds.

Specimens were fabricated in “as field” conditions, with procedures of workmanship and quality control as required by applicable standards. This applies particularly to welding and bolting. Specimens were instrumented with electrical displacement transducers. They measure the displacement of the specimen supporting plates, the vertical displacement of the joint and the relative and absolute rotation of the cross-section and joint. The basic monotonic stress-strain properties of specimens material were determined before the test.

Loading Histories Adopted

The choice of a loading 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 (ATC, 1992), 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 Δv of the displacement cycle in the plastic range adopted in this research were comprised between 5.00 and 12.00 v_y where v_y is the yield displacement of the connection. All specimens were initially subjected to four cycles in the elastic range, respectively with amplitude of 0.50 v_y, 1.00 v_y, 1.50 v_y and 2.00 v_y.

EXPERIMENTAL RESULTS AND EVALUATION OF PERFORMANCE

The test results for each typology of beam-to-column connections are shown in Fig.s 1-4 as hysteresis loops in terms of moment-rotation diagram (M-ϕ) and correspondent failure mode. Some comments concerning the behaviour of each type of connection during the test, until failure are also presented.

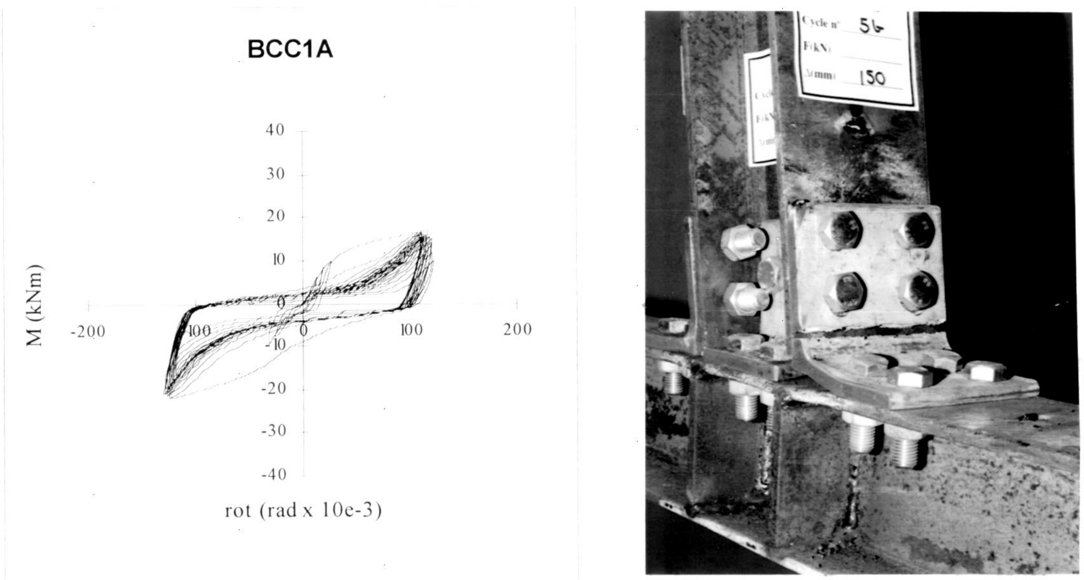

Fig. 1. Experimental moment-rotation diagram and failure mode of BCC1 connection type.

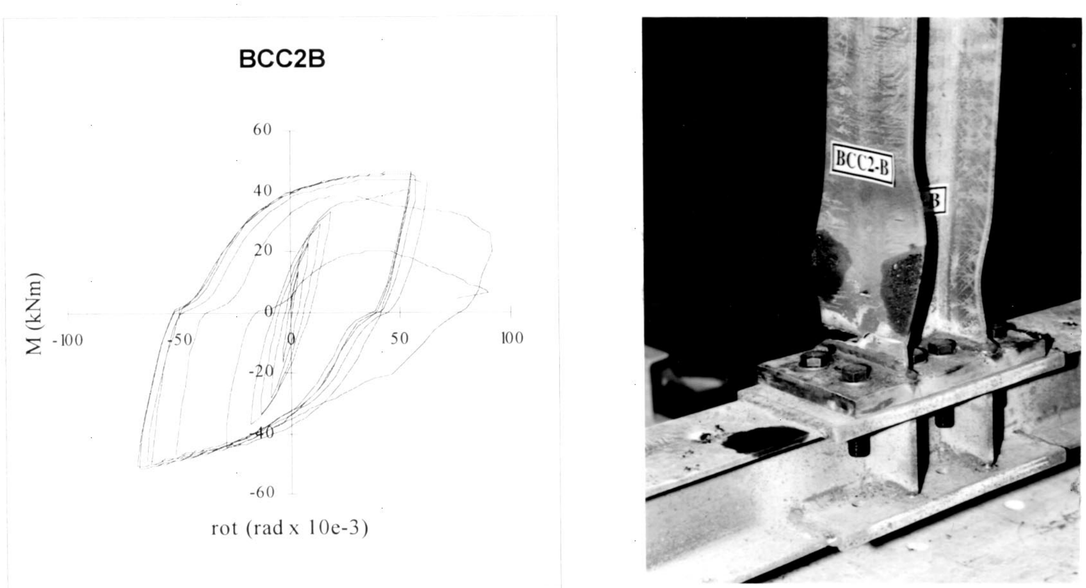

Fig. 2. Experimental moment-rotation diagram and failure mode of BCC2 connection type.

Figures 5 and 6 show the behaviour of each connection tested under constant amplitude displacement in terms of absorbed energy ratio in each cycle. The value of the absorbed energy ratio in the ith cycle is the ratio of the real absorbed energy to the absorbed energy if the connection had an elasto-perfectly plastic behaviour. The values shown in Fig.s 5 and 6 were obtained from the moment rotation diagrams shown in Fig.s 1 - 4.

<u>Bolted web and flanges cleats connections (BCC1)</u>. This type of connection is characterized by large slippage between bolts, that seems to increase, even though the displacement amplitude is constant, with the number of cycles. Increase of plastic deformations in the angles was observed during the test producing a progressive reduction of the absorbed energy (Fig. 5). Slip occurred mainly between the beam flange and the connected angle leg due to ovalization of the holes in the beam flange and in the angle leg. For all specimens, failure was due to a horizontal crack that started in the middle of each leg of the angles when they were in tension. These cracks grew with increasing the number of cycles until complete failure of the angle occured.

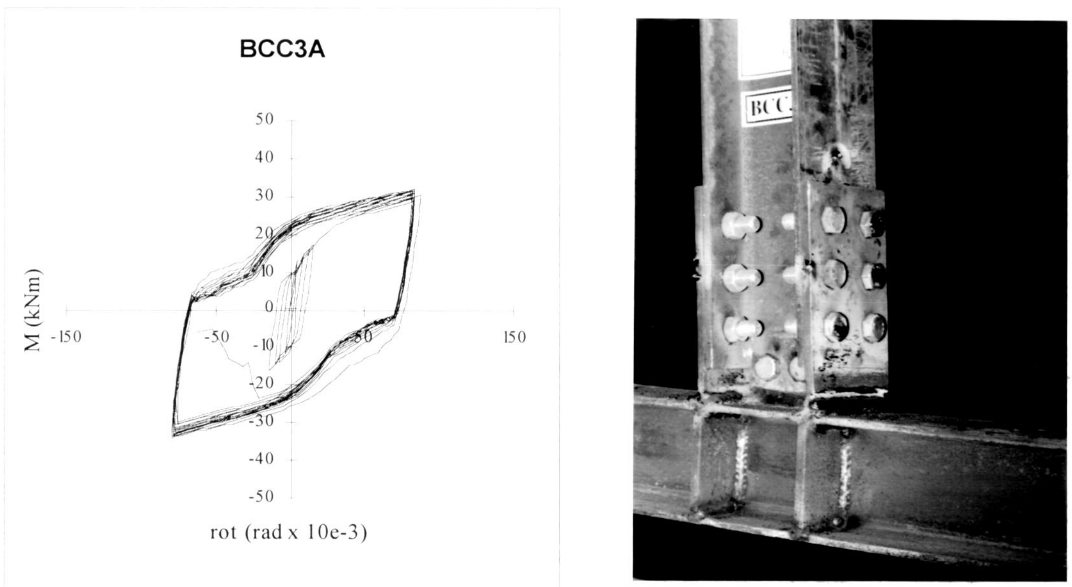

Fig. 3. Experimental moment-rotation diagram and failure mode of BCC3 connection type.

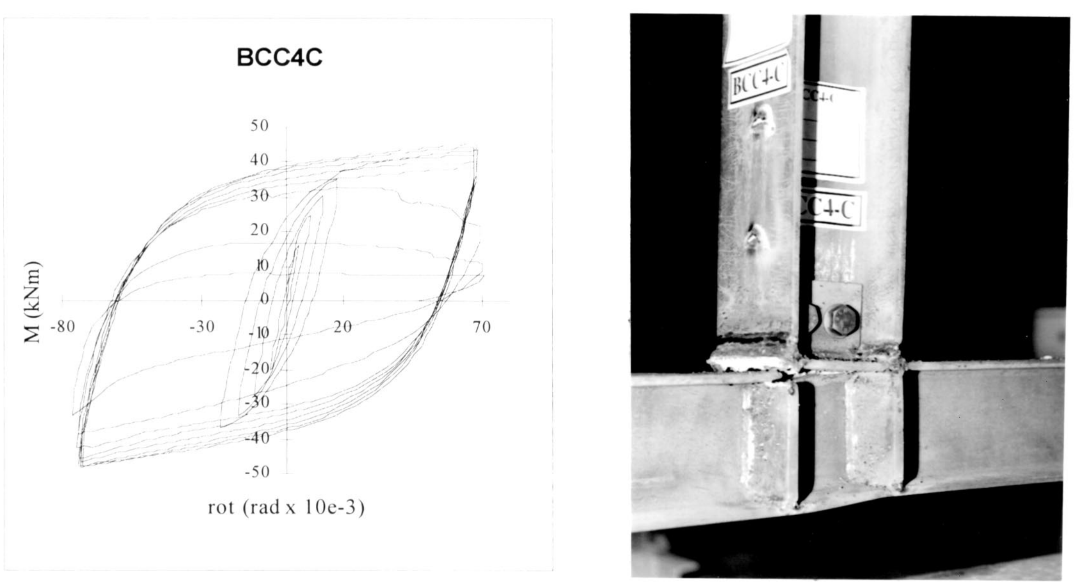

Fig. 4. Experimental moment-rotation diagram and failure mode of BCC4 connection type.

<u>Extended end plate connections (BCC2)</u>. Connections of this type are characterized by regular histeresis loops without any slippage and with a regular reduction of the absorbed energy (Fig. 5) and of the strength

at reversals. For all specimen a plastic hinge was formed in the beam, approximately at a distance from the connection equal to the height of the beam. The effect of bending and tension in these zone induced the development of cracks which started in the flange of the beam. The effect of local buckling of the beam produced a reduction of the absorbed energy and of the strength at reversals.

<u>Flange plates with web cleats connections (BCC3)</u>. This type of connection exhibits slippage between the flange plates and the beam flange due to the ovalization of the holes, but this phenomena has lower importance when compared with web and flanges cleats connections. During the cyclic test a progressive deterioration of the absorbed energy (Fig. 6) was also observed. The flange plates behaved as the angles in BCC1 type under bending deformation but without the separation between beam and column. Failure was due to cracking in the plates connecting the beam flange with the column ones. These cracks were growing with increasing of the number of cycle until complete failure of the angle's cross-section occured.

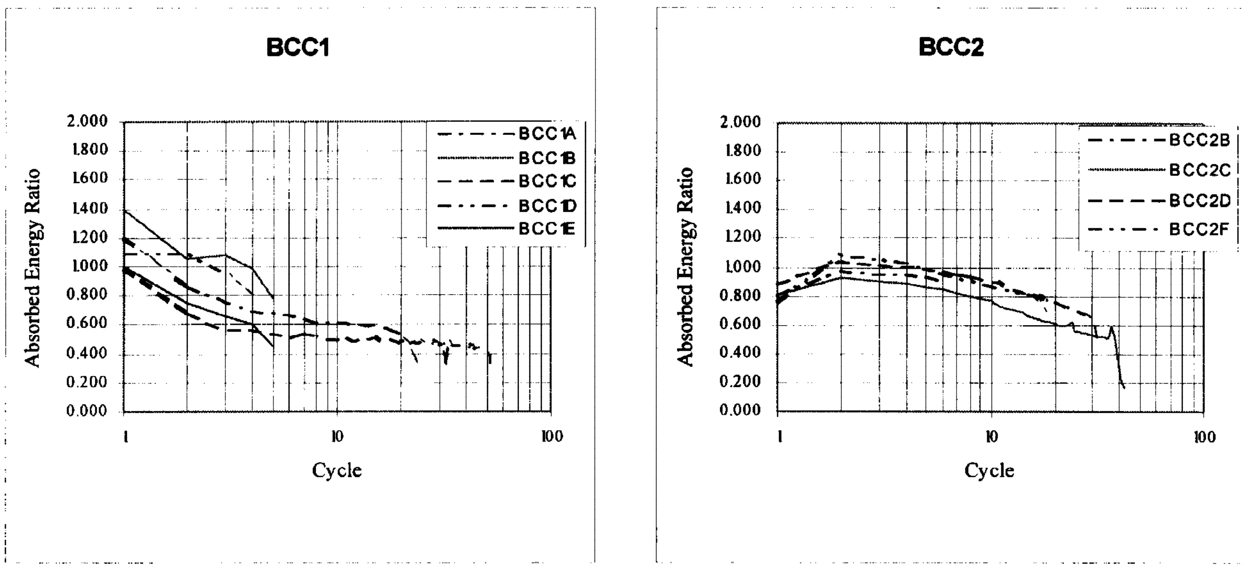

Fig. 5. Absorbed energy ratio versus number of the cycle for BCC1 and BCC2 connection types.

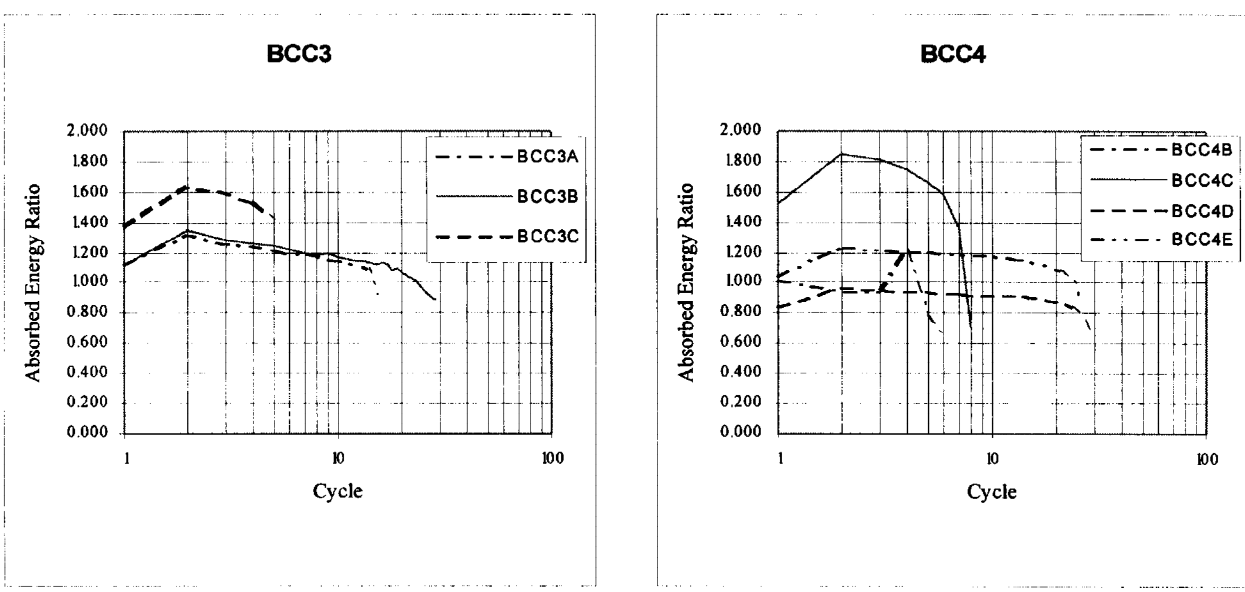

Fig. 6. Absorbed energy ratio versus number of the cycle for BCC3 and BCC4 connection types.

<u>Welded flange with bolted web cleats connections (BCC4)</u>. This type of connection is characterized by great regularity in the hysteresis loops and by gradual deterioration (Fig. 6) and no initial problems, due to the high elasticity of the connection. In all tests large plastic deformations were observed in the central

panel of the column resulting in some cases in large cracks near the stiffeners, as shown in Fig. 5. The effect of bending and tension in the beam flanges induced the formation of cracks which started, for all specimens, near the weld connecting of the beam flange with the column ones. At the end of the test the web of the beam exhibits large holes ovalization due to high stresses in the bolts of the web cleats.

CUMULATIVE DAMAGE MODEL

From tests results it was noticed that, in good agreement with previous studies (Coffin, 1954; Mason, 1954), for all structural components (beams, beam columns, welded joints, beam-to-column connections) (Calado and Castiglioni, 1995; Castiglioni, 1995; Ballio and Castiglioni, 1995) the relationships which best fitted the experimental results in terms of cycle amplitude Δv (normalized on the yield displacement v_y) and number of cycles to failure N_f, are exponential functions of the type $N_f = a\,(\Delta v/v_y)^b$, with a and b constant parameters to be defined and calibrated on the experimental test results. If the cycle amplitude Δv can be correlated to the stress range in the component $\Delta\sigma$, this kind of relationships become similar to the Wöhler S-N lines (Wöhler, 1860) usually adopted in high-cycle fatigue design. Starting from these considerations, Ballio and Castiglioni (1995) recently proposed an approach to unify the design and damage assessment procedures for steel structures under low and/or high cycle fatigue.

According to Ballio and Castiglioni (1995) the damage model for steel can be represented, in terms of generalized displacements s, by the following equation:

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

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

FAILURE CRITERION

Based on extensive experimental and numerical research on beams, beam-columns, welded connections and beam-to-column connections (Calado and Azevedo, 1989; Calado and Ferreira, 1994; Castiglioni, 1995; Ballio and Castiglioni, 1995) the following failure criterion can be formulated having a general validity for structural steel components under variable amplitude loading:

$$\frac{\eta_f}{\eta_0} \leq \alpha \qquad (2)$$

where η_f represents the ratio of the real absorbed energy at the last cycle before collapse (E_{cf}) to the energy that might be absorbed in the same cycle if the structural member had an elasto-plastic behaviour (E_{cppf}), while η_0 represents the ratio of the real absorbed energy in the first cycle in plastic range (E_{c0}) to the energy that might be absorbed in the same cycle if the structural member had an elasto-plastic behaviour (E_{cpp0}).

In equation 2, the value of α should be determined by fitting the experimental results. Based on the results obtained in a multi-specimen testing program for beam-to-column connections it was observed that the value of α seems to depend on the type of connection. The values obtained for α in all tests range between

0.44 and 0.75. The difference between these values is related with the type of failure observed. However, based on these experimental results and on the results obtained for the same type of connections but under random cyclic loading, it seems that a value between 0.50 and 0.70 should be considered. The proposed value of α can be adopted for a safe assessment of the damage cumulated in the beam-to-column connections. Hence, 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.

RE-ANALYSIS OF TESTS RESULTS

Figure 7 shows the test data for beam-to-column connections tested under constant amplitude loading (BCCi-Lisbon) together with those of tests performed with variable amplitude loading on double angle connections (U.C.-Berkley by Astaneh, Nader and Malik, 1989), top-and-seat angle connections (S.U.N.Y.-Buffalo by Mander, Pekcan and Chen, 1995), top-and-seat angle connections (VPH V.A.-Lisbon by Calado and Ferreira, 1994) and beam-to-column connections (BCCi-V.A.-Lisbon by Calado and Castiglioni, 1995).

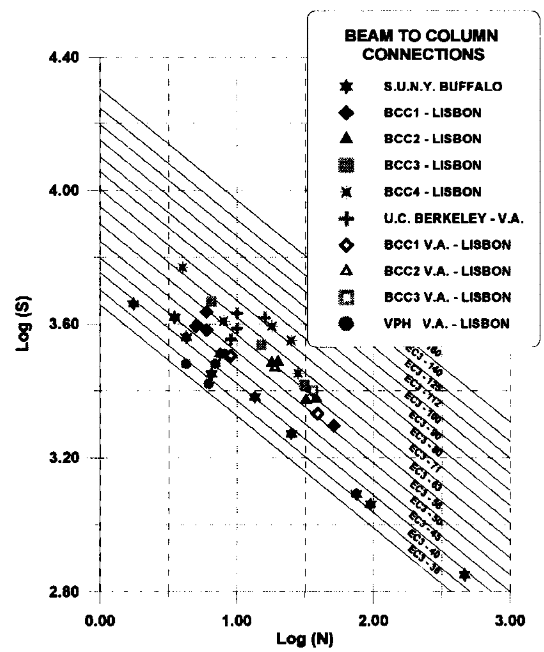

Fig. 7. Fatigue strength of beam-to-column connections.

It can be noticed from Fig. 7 that beam-to-column connections of the same typology, belong to the same fatigue strength category. In fact, top-and-seat angle connections tested in Lisbon (VPH V.A.-Lisbon) had HEA120 and 100x100x10 profiles while those tested in State University of New York at Buffalo (S.U.N.Y. - Buffalo) had W8x31, W8x21 and L6x4x3/8 profiles. Furthermore, specimens tested in Lisbon were subjected to variable amplitude while those tested in Buffalo were submited to constant amplitude loading histories showing that the parameter which seems to govern the fatigue behavior of connection is its typology.

These results are directly related with the strength of the connection. For instance, top-and-seat angle connections are less stiff when compared with flange plates with web cleats or welded flange with bolted web cleats, and for that reason they can be fitted by S-N category 40 while the other are fitted by category 63. However, 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.

CONCLUSIONS

The fatigue damage model proposed in this research was validated by constant and variable amplitude cyclic test results on different typologies of beam-to-column connections tested in different laboratories (Lisbon, Berkeley and Buffalo). This validation was carried out over the range of amplitudes expected in the response of steel structures under earthquakes of average intensity. The main issue becomes, in this case, the assessment of fatigue strength categories of various typologies of connections, i. e., of the appropriate S-N curve to be associated with each type of detail. This can be done either by means of 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 consistent results for a number of structural components has been proposed. The validity of such criterion must however be furtherly investigated and extended to other structural details.

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