16th World Conference on Earthquake, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 1559
Registration Code: S-N1461761407

EVALUATION OF LOADING PROTOCOLS FOR ASSESSING LOCAL SEISMIC DEMANDS IN STEEL BUILDINGS DESIGNED TO EC8

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Abstract

A study for the derivation of loading protocols for quasi-static cyclic testing of beam-to-column sub-assemblies of steel buildings designed to modern seismic guidelines is presented in this paper. The adopted methodology follows the general guidelines developed in the US in the early 2000's for experimental investigations into the cyclic behaviour of steel beam-column moment-resisting connections and later incorporated into national standards (e.g. AISC 341-10) and relevant guideline documents (e.g. FEMA 461).

In previously published research, loading protocols to be used in cyclic testing have been strongly dependent on the sub-structure or component to be tested as well as the seismic hazard scenario considered as a basis of experimental programmes. Accordingly, this study focuses on steel building typologies (namely, moment-resisting frames) designed to European codes for medium and high seismicity scenarios, considered representative of the greater European and Middle Eastern region.

A large set of non-linear response history analyses was performed on the numerical models of the designed buildings, using an ensemble of ground motions, scaled to match European design spectra. The numerical models used to investigate the seismic performance were developed taking into account the cyclic response of the selected joints, including the stiffness and strength characteristics of the column web panel zones and the connections. Cumulative and maximum rotation demands obtained from the analyses, in terms of inter-storey drift response histories, provided the basis for the derivation of the protocols in order to represent more accurately design earthquake demands and to compare with standardised loading procedures.

The results indicate a dependence of the derived load protocol characteristics on the seismic hazard level assumed for the analyses and the quantitative characteristics (e.g. number of significant cycles) of the ground motion records employed. The developed loading protocols are compared with existing and widely-used testing protocols, and recommendations for their use in the assessment of steel framed structures designed to European procedures are put forward.

Keywords: loading protocols; connection testing; steel moment frames; seismic design; Eurocodes

1. Introduction

A methodology for deriving loading histories for use in cyclic tests of steel beam-to-column subassemblages of steel moment resisting frames and its application to building typologies, designed to European seismic codes, is discussed in this paper.

The basis of the proposed methodology has been presented by Krawinkler et al [1] as part of the SAC project research after the 1994 Northridge earthquake. In the aforementioned research, the results of nonlinear time-history analyses on steel moment frames were used for developing cyclic loading protocols for components and assemblies of special moment resisting frames. Similarly, the CUREE protocols for wood-frame structures have been developed by Krawinkler et al. [2]. Protocols for short links in eccentrically braced frames were proposed by Richards and Uang [3], and were subsequently incorporated in the AISC 341 Seismic Provisions [4]. Recommended generic testing protocols for quasi-static cyclic testing of structural and non-structural components are presented in FEMA 461 [5]. A testing protocol for the seismic qualification of non-structural systems is presented by Retamales et al. [6], and a drift protocol for testing non-structural window systems has been developed by Hutchinson et al. [7]. Also, a methodology for the development of a cyclic load protocol for testing the cyclic response of anchored non-structural components and systems has been proposed by Hutchinson and Wood [8].

More recently, a set of cyclic loading protocols for European regions of low to moderate seismicity has been developed by Mergos and Beyer [9] on the basis of nonlinear response-history analyses of a set of single-degree-of-freedom idealisations, corresponding to various types of structural systems. Finally, a near-fault loading protocol has been proposed by Lanning et al. [10] for testing and prequalification of buckling-restrained bracing systems, intended to be used in long-span bridges. The methodology suggested by Mergos and Beyer [9], which is substantially based on the original work by Krawinkler et al. [2], serves, with appropriate adaptations, as a basis for the present study.

2. Methodology

The key steps of the methodology are presented hereafter, followed by an example of application of the loading protocol derivation procedure to a 6-storey frame.

1. Definition of seismic hazard. Two cases were considered, one corresponding to medium seismicity and a second corresponding to high seismicity. The medium seismicity case (MH) corresponds to ground motions with peak ground acceleration of 0.25g and compatible with EN 1998-1 [11], Soil Type B, 5% damping, and 475-year return period design spectrum (Type 1). The high seismic hazard case (HH) comprises ground motions with peak ground acceleration of 0.35g, compatible with the EN 1998-1 spectrum (Type 1).

2. Selection and initial scaling of ground motions. Two sets of 7 ground motions each were selected for the MH and HH seismicity levels by scaling original recorded ground acceleration records with “far-field” characteristics and performing spectral matching to the design spectra mentioned in Step (1). The two suites of ground motions were also used to evaluate the seismic performance of the various frame typologies considered. The frequency content of the selected acceleration records were not altered.

3. Definition of limit state. The proposed loading protocols for quasi-static cycling testing should impose (on the connection assemblies) deformations that are consistent with the “Near Collapse” (NC) limit state, as a minimum. According to EN 1998-3 [12], the NC limit state corresponds to a ground motion with return period of 2475 years (i.e. probability of exceedance of 2% in 50 years. Therefore the “design” ground motions have to be also scaled by a minimum importance factor $\gamma_I = 1.73$, in order to represent deformation demands consistent with the NC limit state [11].

4. Response-history analyses. The scaled ground motion sets were used as acceleration input for a series of non-linear response-history analyses of the frames. The numerical analyses were carried out by means of the open-source software OpenSEES [13]. The two-dimensional OpenSEES models of the frames were built following the distributed plasticity approach, by means of force-based elements with fibre sections, for the

idealisation of beams and columns. For this study, the beam-column connections were assumed to be full-strength and rigid. Also, the panel zones were considered to be strong, which is in tune with the design philosophy of the European code.

5. Selection of response quantities. The inter-storey drifts were selected as the response quantities that were used as input for the definition of the loading protocols, as they better represent the deformation demands on the connections during a seismic event [1]. For each frame typology and seismicity level, inter-storey drifts were recorded form the most severely affected storey, in terms of the absolute maximum drift.

6. Cycle counting. Each inter-storey drift response history (signal) was divided into a “pre-peak” and a “post-peak” part. The pre-peak part is defined as the portion of the signal before either the maximum positive peak or the minimum negative peak. Only the pre-peak signal was used in further analysis according to the recommendations in FEMA 461 [5], to prevent the derivation of an over-conservative loading protocol. The cycle counting algorithm ([14,15]) as implemented by Nieslony [16] was then applied to the pre-peak signal. For each signal, a sequence of normalised (with respect to the maximum) cycle amplitudes was produced. The cycle amplitudes were arranged with respect to zero, sorted in decreasing order and cycles with normalised amplitude less than 0.1 were omitted. The procedure was applied to each selected signal from the numerical analyses.

7. Statistical processing. A set of normalised amplitude sequences was processed as follows: the median normalised cycle amplitudes were calculated as the median of the 1st, 2nd, 3rd, etc. largest cycle for all ground motions in a specific set. It is evident that the median of the first cycle is equal to unity and the medians for the rest of the cycles are smaller than one. As with the case of a single ground motion in Step (6), cycles with median normalised amplitude less than 0.1 were omitted.

8. Calculation of the empirical CDF. The empirical cumulative density function (CDF) was constructed for each median normalised cycle amplitude sequence. The CDF describes the distribution of the amplitudes within the sequence and their rate of decrease (or increase, if sorting is in increasing order).

9. Construction of the loading protocol. The construction of the loading protocol followed the approach by Mergos and Beyer [9] and the general guidelines by Krawinkler [1]. The proposed loading protocols were characterised by 2 cycles of constant amplitude per loading step, and 7 loading steps as a minimum. An iterative process was followed, during which the amplitudes for each loading step were adjusted so that the empirical CDF of the proposed protocol matches the CDF obtained in Step (8) from the numerical analyses. The number of loading steps was gradually increased until the cumulative deformation (proportional to the sum of amplitudes) of the protocol was equal or larger than the cumulative deformation calculated for the median sequence of amplitudes from the numerical analyses.

10. Smoothing of the protocol. An exponential function was fitted by least-squares regression to the loading step amplitudes derived in Step (9), with the appropriate boundary conditions for the largest and the minimum cycle. The rate of increase of the amplitudes was determined by a single exponential parameter α. Thus, the derived protocol is fully described by the number of loading steps from Step (8) and the value of the amplitude increase rate parameter.

A schematic flow-chart of the aforementioned methodology is presented in Fig. 1.

Example of application (single signal):

An example of load protocol construction from a single signal is depicted in Fig. 2 to Fig. 7. In this case, the input signal is the inter-storey drift response history for the 4th storey of a 6-storey moment-resisting frame analysed for a high seismicity record at an intensity corresponding to two times the NC level (i.e. ~350%). The specific level of intensity has been chosen for the definition of a loading protocol, as it produces maximum drift demands of the order of 35 to 40 mrad, which corresponds to the generally accepted ultimate rotational capacity of the connections.

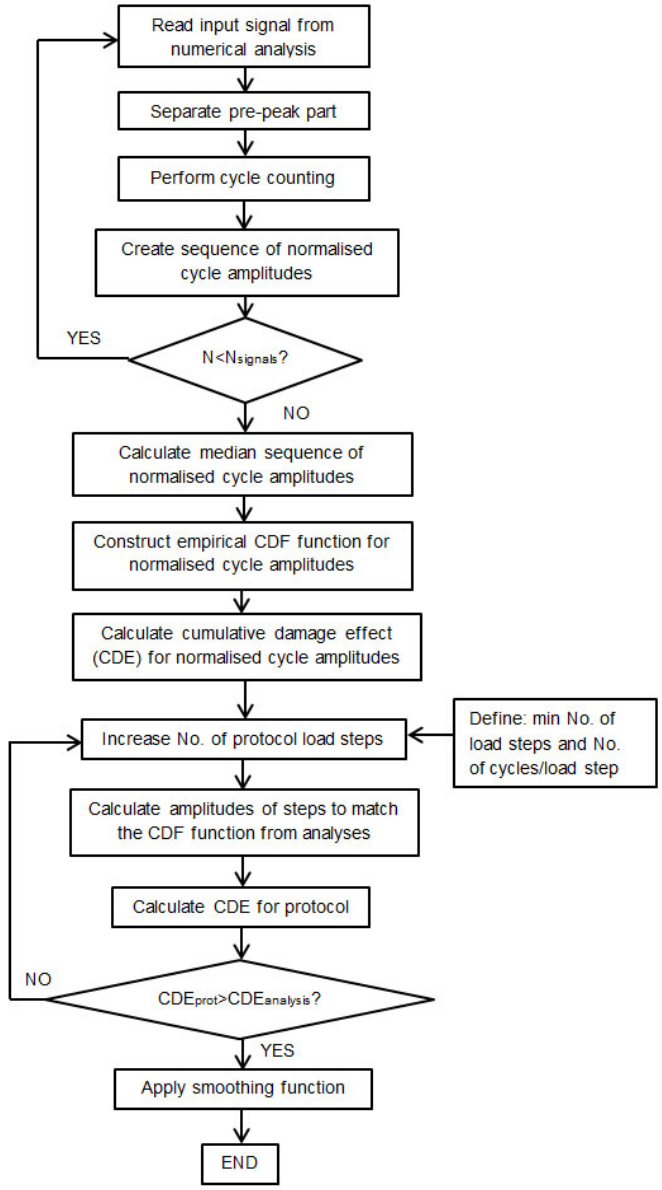

Fig. 1 - Flow chart of the methodology for the derivation of the loading protocols

The selected input signal is presented in Fig. 2. The pre-peak portion which is used for the construction of the protocol is highlighted. It is assumed that only the pre-peak cycles contribute to the cumulative damage to the connections. The rain-flow counting algorithm is applied to the signal and outputs a sequence of cycles with amplitudes and means that are plotted in Fig. 3. It is noteworthy that for the specific drift response history, the cycles with the maximum counted amplitudes have very small means, which is compatible with the arrangement of cycle amplitudes with respect to zero. The normalised sorted amplitude sequence is presented in Fig. 4; a total of 42 cycles are counted with amplitudes ranging from 0.1 to 1.0.

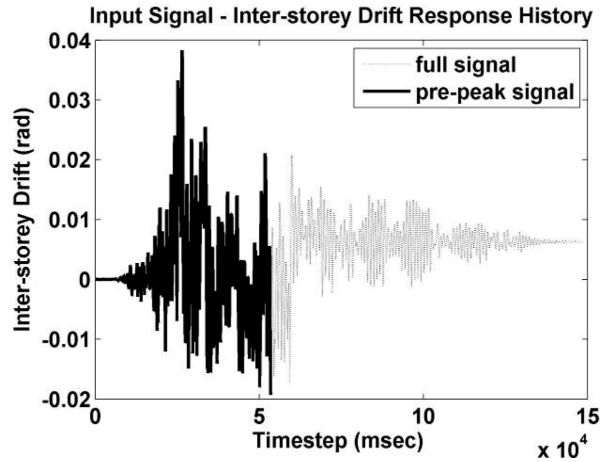

Fig. 2 - Inter-storey drift signal with the “pre-peak” part highlighted

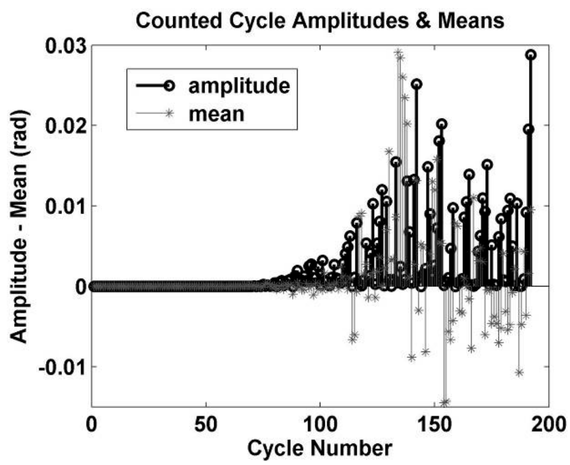

Fig. 3 - Cycle amplitudes and means from rain-flow counting

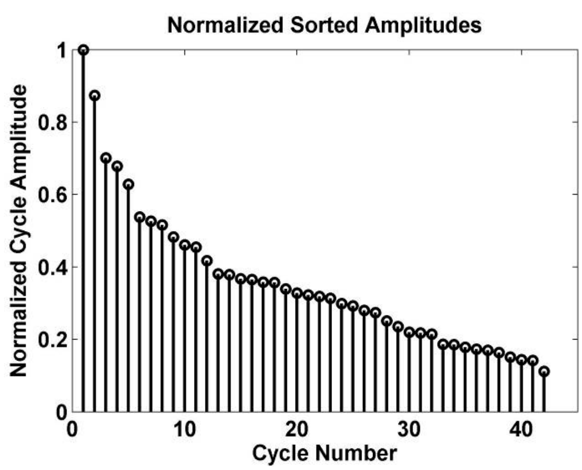

Fig. 4 - Normalised amplitudes sorted in decreasing order

The empirical cumulative distribution function constructed from the counted cycles of the drift response history is presented in Fig. 5. The iterative fitting process is then applied and a tentative loading protocol comprising 18 load steps (36 cycles) is constructed (dotted line in Fig. 5).

A smoothing function is then applied to the sequence of amplitudes according to the equation:

$$f(x) = -0.43 + 0.53\exp\left[\left(x/n\right)^{a}\right] \quad (1)$$

where n is the total number of load steps (equal to 18) and a is the rate parameter (in this case, equal to 2.2). The smoothed load step sequence is presented in Fig. 6, while the constructed protocol is presented in Fig. 7.

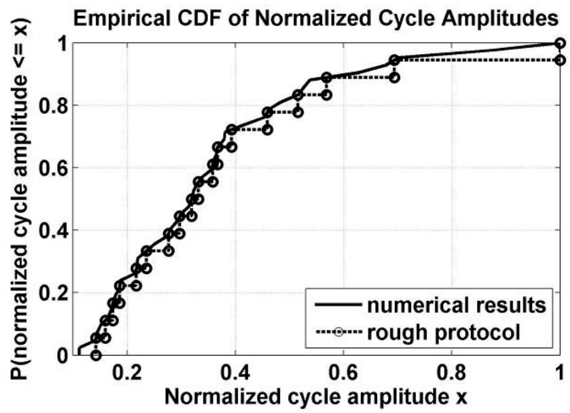

Fig. 5 - Matching of empirical CDF's (numerical analysis vs. protocol)

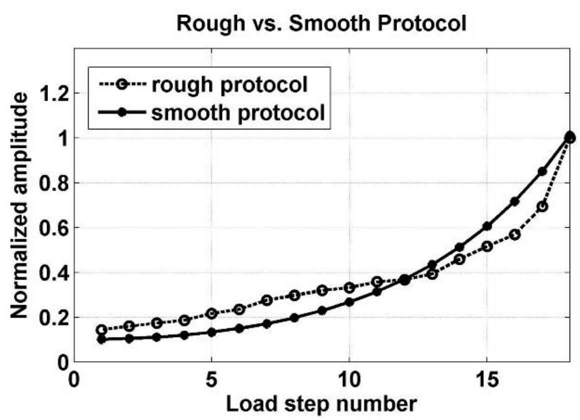

Fig. 6 - Smoothing of constructed protocol

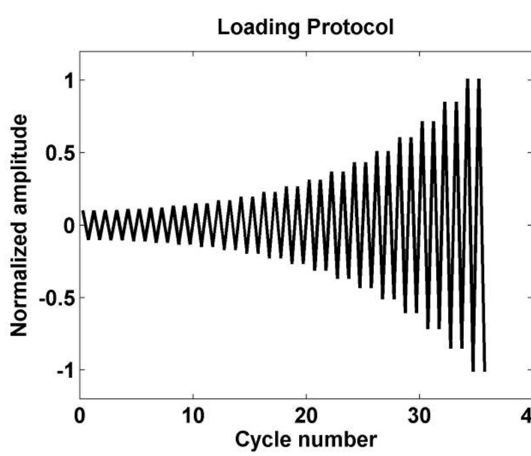

Fig. 7 - The derived protocol (from single signal)

As already mentioned, the methodology can be applied to a single input signal, or alternatively, to a set of signals. In the context of this study, the pool of signals are inter-storey drifts, calculated from response history analyses of various typologies of steel frames, using two suites of seven accelerograms each that correspond to two levels of seismic hazard. The inter-storey drift signals are selected and grouped per typology and per seismicity level, as discussed in Section 3 below. The ordered normalised sequences of counted cycle amplitudes are constructed and the median sequences are calculated as follows: the first element of the median sequence is 1 (as all sample values are normalised); the second element is the median value of all second largest cycle amplitudes of the sample; the third value is the median of all third largest values, and so on. According to FEMA 461 [5], the use of median values is appropriate, as the objective of testing is to obtain fragility data in which the effect of record-to-record variablility should be represented as an average.

3. Application to steel moment frames designed to EC8

The methodology described in Section 2 has been followed for the construction of tentative loading protocols, based on numerically computed drift demands from non-linear time-history analyses of moment-resisting frame typologies. The analyses have been grouped by typology and by seismicity level into the following two cases: (i) medium seismicity moment-resisting frames (MRF-MH), and (ii) high seismicity moment-resisting frames (MRF-HH). Each seismicity group comprises 3-storey (11.5 m high) and 6-storey (22 m high) frames, with 3 bays and a bay span ranging from 6 m to 8 m. The aforementioned dimensions are considered to be typical of European structures.

For the ground motion set corresponding to each seismicity case, median sequences of normalised amplitudes were derived and subsequently used for constructing a loading protocol representative of the drift demands at each seismic hazard level. It should be noted that the input signals were extracted from the storeys that sustained the maximum absolute inter-storey drifts, for each frame typology. In general, for the frame typologies of this study, the maximum inter-storey drifts were recorded at the 4th (mostly) and 3rd storey of the 6–storey frames and at the 1st (ground) storey of the 3-storey variations. It is worth noting that no soft-storey mechanisms formed, at least up to twice the ground motion intensity corresponding to the NC limit state.

The selected drift response histories correspond to a seismic intensity level consistent with the NC limit state, as a minimum. In cases where the recorded maximum drift demands were significantly lower than the standard 35 mrad ultimate rotation limit, the drift response histories were extracted from analyses at higher ground motion intensities, up to twice the NC level.

The derived median loading protocols for the moment-frame cases (i) and (ii) are presented in the plots in Fig. 8 to Fig. 11. The derived protocols (Fig. 8 and Fig. 10) for the medium and high seismicity cases are presented, along with comparative plots of the respective CDF functions (Fig. 9 and Fig. 11), where the empirical CDF obtained from the numerical results (purple dashed line) and the resulting rough (blue step plot) and smoothed protocol (red step plot) are counter-posed on the respective plots for the standard AISC protocol (black step plot) and the FEMA 461 recommended protocol (green step plot).

Notably, the required number of cycles for the medium seismicity case (16 cycles) is significantly lower than that for the high seismicity case (24 cycles) and both are lower than the 30 cycles imposed by the AISC protocol (Fig. 12). The rate of increase of amplitudes is more “linear” in the medium seismicity case (Fig. 9), as a result of the lower value of the exponential rate parameter ($a = 1.50$). In the high seismicity case, the amplitude increase rate is larger ($a = 2.67$) and a very good match is observed between the CDF of the proposed protocol and that of the AISC protocol (Fig. 11).

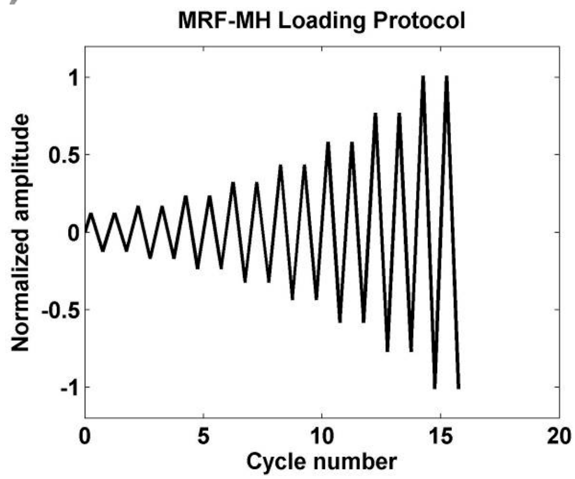

Fig. 8 - MRF-MH protocol (median)

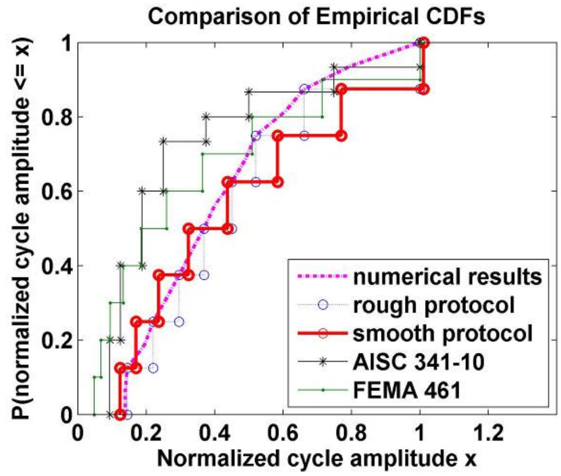

Fig. 9 - MRF-MH protocol – comparison of CDFs

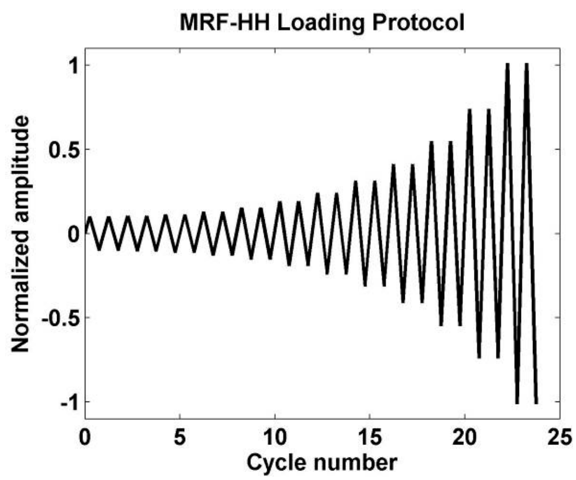

Fig. 10 - MRF-HH protocol (median)

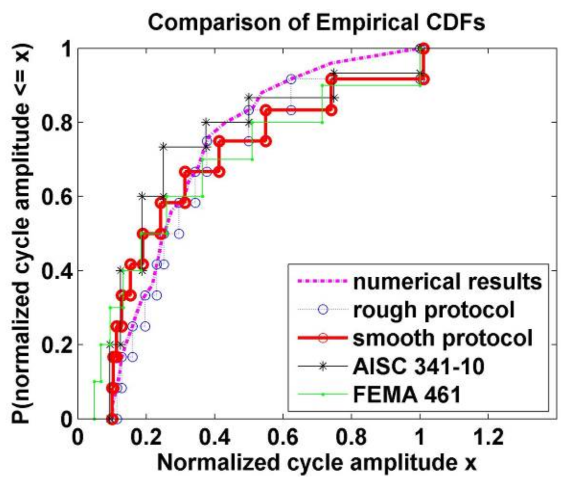

Fig. 11 - MRF-HH protocol – comparison of CDFs

An overview of the median demand parameters, in terms of number of “damaging” cycles and sum of normalised amplitudes (proportional to the cumulative damage effect), is presented in Table 1.

A comparison of the basic characteristics of the two cases of derived protocols is presented in Table 2 and the respective basic protocol characteristics are compared with the parameters of the AISC and FEMA-461 protocols. Except for the shape of the envelopes of the protocols, which is reflected in the corresponding empirical CDFs, the most important parameters are the total number of the imposed cycles as well as the total cumulative imposed deformation, which is expressed in this study as the sum of normalised amplitudes.

By comparison of Table 1 and Table 2, the numerically calculated median numbers of damaging cycles (as defined in the methodology) for the braced frame cases range from 16 (for the MH case) to 25 (for the HH case), which is larger than the assumed minimum number of 14 damaging cycles for the constructed loading protocols (7 load steps with 2 cycles per step).

The standard AISC protocol turns out to be only about 7% more demanding, in terms of cumulative deformation than the most severe case (ii) which corresponds to the high seismicity moment frames. On the other hand, the FEMA 461 protocol is less demanding, always in terms of cumulative imposed deformation, than both protocols derived from analyses of medium and high seismicity moment frames. From these results, it could be preliminarily concluded that the use of the AISC protocol for testing beam-column assemblies of European frames is a safe but rather conservative approach.

Table 1 - Median parameters calculated from numerical analyses

Analysis Group	No. of "Damaging" Cycles	Sum of Norm. Amplitudes
(i) MRF-MH	16	6.88
(ii) MRF-HH	25	8.17

Table 2 - Basic parameters of the loading protocols

Protocol Type	No. of Load Steps	No. of Cycles per Load Step	No. of Cycles	Sum of Norm. Amplitudes	Rate Parameter "α"
(i) MRF-MH	8	2	16	7.31	1.50
(ii) MRF-HH	12	2	24	8.13	2.67
AISC (2010)	8	variable	30	8.69	N/A
FEMA-461 (2007)	10	2	20	6.76	N/A

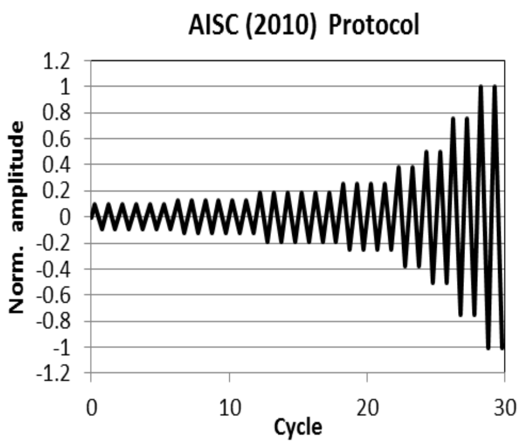

Fig. 12 – AISC 341 standard loading protocol

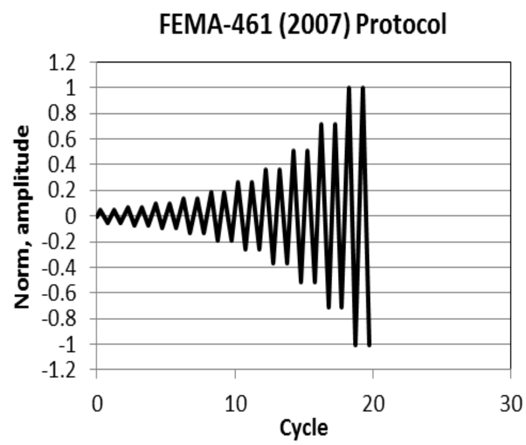

Fig. 13 - FEMA-461 loading protocol

4. Concluding remarks

A methodology was presented for the construction of loading protocols for the testing of beam-to-column connection assemblies for use in earthquake-resistant moment frames, designed to EC8. The current approach closely follows well-established and published methods for constructing loading protocols and is applied to the specific structural typologies for predetermined seismic hazard levels corresponding to medium and high seismicity scenarios in a European region.

The results show that the proposed derived protocols are characterised by lower cumulative deformation demands than those imposed by the widely used and standardised AISC protocol for quasi-static cyclic testing of components of moment-resisting frames. This is mainly due to the absence of the initial "non-damaging" cycles that are present in the standardised AISC loading history. On the other hand, the rate of increase of the amplitude of the loading steps is more severe in the case of the derived protocols (especially in the medium seismicity case), as evidenced by the shapes of the respective empirical cumulative density functions.

Further investigation aimed at deriving appropriate loading histories for moment connections of steel frames designed to the Eurocodes, using a wider selection of frame design cases and a larger pool of ground motion records, is needed in order to better evaluate the findings of the current study, especially concerning additional assessment of whether the demands imposed by standard protocols are overly conservative.

5. Acknowledgements

The authors acknowledge the financial support of the Research Fund for Coal and Steel of the European Community within the Project EQUALJOINTS: "European Prequalified Steel Joints", Grant No RFCS-CT-2013-00021, for the work described in this study. The related discussions with collaborators from the Universities of Naples, Liège, Timisoara and Coimbra, as well as ARCELOR and ECCS, are gratefully acknowledged.

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