

CYCLIC MODELLING OF BOLTED BEAM-TO-COLUMN CONNECTIONS: COMPONENT APPROACH

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1 INTRODUCTION

According to the traditional design philosophy of seismic resistant steel structures, the dissipation of earthquake input energy has to be provided by the yielding of some selected zones, namely dissipative zones, which have to be properly detailed to provide adequate ductility supply and energy dissipation capacity.

Concerning the location of dissipative zones in MR-frames, design criteria for beam-to-column joints play a role of paramount importance. Beam-to-column joints can be designed to develop either full strength or partial strength aiming to reach a given performance requirement. In the former case, the seismic input energy is dissipated by excursions in plastic range occurring at the beam ends, relying on the ability of the steel member to develop wide and stable hysteresis loops. In the latter case, beam-to-column joints are designed to have a flexural strength less than the one of the connected beam, so that the seismic input energy is dissipated by yielding of the joint components.

Under this point of view it is necessary to distinguish between dissipative and non-dissipative components, i.e. between dissipative and non-dissipative failure mechanisms (Plumier, 1994, Astaneh, 1995).

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4 It is well known that local buckling phenomena do not usually allow the development of
5 highly dissipative mechanisms. This is the case of the column web in compression and
6 the case of plates in compression. For this reason the use of stiffeners, such as
7 continuity plates, is commonly suggested. Regarding beam flange and web in
8 compression, local buckling phenomenon can result in a dissipative behaviour provided
9 that the width-to-thickness ratios of the plate elements constituting the beam section are
10 limited to assure a ductile behaviour, i.e. class 1 cross-sections are adopted.
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21 Regarding bolt behaviour, it is important to note that, both under normal and shear
22 stresses, their limited plastic deformation capacity and fatigue life can lead to the brittle
23 collapse of the joint, so that bolts have to be designed with sufficient overstrength to
24 prevent brittle failure modes. Moreover, the cyclic behaviour of bolted connections can
25 be strongly affected by bolt plastic deformation, because the occurrence of such plastic
26 deformations can lead to pinching phenomena of hysteresis loops of bolted components,
27 i.e. end-plate, column flange and flange cleat in bending.
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37 Similarly, aiming to assure a dissipative behaviour of joints, failure of welds has to be
38 absolutely avoided, because of their brittle collapse mechanism. For this reason, it is
39 suggested to provide sufficient over-strength of welds with respect to the design flexural
40 resistance of the joint. The yielding of the panel zone in shear can be admitted, provided
41 that excessive distortion does not occur, because excessive panel deformations can lead
42 to the premature fracture of welds. To this scope, Eurocode 8 (CEN 2005a), requires
43 with reference to MR-frames that column web panel in shear has not to contribute for
44 more than 30% to the plastic rotation capacity of the joint.
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In conclusion, an accurate design should aim to properly balance the plastic engagement of the joint components, paying attention to avoid brittle and non-dissipative mechanisms.

The economic advantages deriving by the adoption of partial strength beam-to-column joints within MR-frames have been already pointed out by different authors in past works. In both braced and unbraced frames, the use of partial strength connections can lead to important savings in the gravity load system (Bjorhovde and Colson, 1991). In unbraced frames subjected to seismic loads the use of partial strength connections can lead to an increased ductility as well as to lower design forces due to period shifts (Elnashai and Elghazouli, 1994).

Since Northridge and Kobe earthquakes, numerous research efforts have been devoted to enhance the performance of moment connections in steel structures under seismic loads. Such seismic events showed the vulnerability of some types of welded connections subjected to cyclic actions. Since then, different solutions have been proposed for the retrofit of existing structures and for the design of new structures, in particular four approaches have been followed: the improvement of unreinforced welded connections, the strengthening of moment connections by adding cover plates, ribs or haunches, the application of reduced beam section approach aiming to promote the yielding of the beam far away from welds and the study of bolted solutions developed in order to improve connections performance under cyclic loads (Mahin et al., 2002).

The inelastic behaviour of bolted connections is definitely more complex compared to welded joints, simply because more components can be involved in the dissipation mechanism, such as the end-plate, angles or tee stubs in bending (depending on the

connection typology), the column flange in bending, the panel zone in shear, the column web in tension/compression.

With reference to bolted end-plate connections many research programs have been developed in past years aiming to identify the monotonic and the cyclic behaviour of the joint components and of the beam-to-column joint as a whole. Such connections exhibit a distinctively non-linear behaviour resulting from a lot of phenomena like elasto-plastic deformations, contact or slip between the connecting elements. The analysis of these phenomena and their interpretation is very complex, so that test results provide a fundamental basis for the development and the calibration of mechanical models for predicting the joint behaviour. Under this point of view, experimental tests can be aimed to provide the behaviour of each joint component and/or the whole joint moment-rotation curve. According to the component approach codified in Eurocode 3 (CEN, 2005b) for beam-to-column joints subjected to monotonic loading conditions, the whole beam-to-column joint is modelled by means of a combination of both rigid and deformable elements that represent the joint components, leading to a mechanical model whose aim is the evaluation of the joint moment-rotation curve starting from the knowledge of the force versus displacement behaviour of the joint components. This approach can be extended also to beam-to-column joints under cyclic actions (Bernuzzi et al., 1996, Nemati et al., 1999, Rassati et al., 2004, Nogueiro et al., 2007). The goal is the prediction of the cyclic behaviour of the connection starting from the modelling of the cyclic response of each joint component. The distinction between dissipative and non dissipative components allows to focus the attention on the modelling of the cyclic behaviour of the dissipative components, assuming that a correct design of the joint is made by assuring an adequate overstrength of brittle components (Faella et al., 2000).

In particular, it is of fundamental importance to design welds with adequate overstrength preferring the adoption of full penetration welds.

Even though full strength connections are usually preferred and their use is suggested by seismic codes, Eurocode 8 has opened the door to the possibility of adopting partial strength joints for seismic purposes. In particular, with reference to MR-frames, dissipative semi-rigid and/or partial strength connections are allowed provided that connections have a rotation capacity consistent with global deformations (CEN, 2005a), members framing into the connections are demonstrated to be stable at the ultimate limit state (ULS) and the effect of connection deformation on global drifts is taken into account using non-linear static global analysis or non-linear time history analysis.

Most of all, Eurocode 8 requires a plastic rotation supply of connections depending on the ductility class of the structure. In particular, benchmark values equal to 35 mrad for structures designed for ductility class high (DCH) and 25 mrad for structures designed for ductility class medium (DCM) have to be assured.

In this work, the possibility to extend the component approach to the prediction of the cyclic behaviour of bolted beam-to-column connections is analyzed. To this scope, reference has been made to experimental tests carried out at the University of Salerno and described in a companion paper (Iannone et al., 2009). The setting up of experimental tests has been performed taking care of measuring the displacements of each joint component and the joint as a whole. In particular, the comparison between the sum of the energy dissipated by each joint component and that dissipated by the joint as a whole has underlined that the extension of the component approach to the prediction of the cyclic behaviour of beam-to-column joints is feasible, provided that the joint components are properly identified and modelled (Iannone et al., 2009).

Starting from such result, in this paper, the implementation of the component method for predicting the joint rotational response under cyclic loading conditions is carried out and the accuracy of the proposed model is verified in terms of energy dissipation by means of the comparison between the experimental moment-rotation curves and those provided by the proposed model.

Even though in the technical literature there are works dealing with the modelling of the cyclic response of beam-to-column joints since 1980s (Moncarz and Gerstle, 1981; Ballio et al., 1987), such works were based on mathematical models concerning the overall flexural response of the joints. Conversely, the prediction of the cyclic response of joints by means of a component approach is a relatively young research topic (Nemati et al., 1999; Rassati et al., 2004; Nogueiro et al., 2007) considering, above all, that only the problem of predicting the monotonic response is codified in Eurocode 3 (CEN 2005b). The novel contribution of the work herein presented consists in providing not only a cyclic mechanical model for the assembly of the single joint components, but also the analytical rules for predicting the behaviour of such components starting from their geometrical and mechanical properties to be combined with empirical rules to account for stiffness and strength degradation and pinching effects based, again, on past component modelling efforts, rather than on calibration procedures regarding the joint as a whole. In fact, the novelty of the contribution is to underline how degradation rules of stiffness and strength and pinching rules derived from single component testing can be properly applied to predict the whole joint rotational response. Any recalibration of parameters would lead to something more similar to a curve fitting rather than to a prediction model which, conversely, is the goal of the present work.

2 MECHANICAL MODEL FOR BOLTED BEAM-TO-COLUMN JOINTS

Methods available in technical literature regarding the modelling of the cyclic behaviour of beam-to-column joints can be divided into three groups: mathematical models, mechanical models and finite element models. Mathematical models are based on curve fitting of joint moment-rotation curves, so that their limits can be easily understood. In fact, mathematical models can be developed only if experimental tests are available, so that their application is limited to tested structural details.

Conversely, mechanical models are based on an appropriate combination of the cyclic response of the joint dissipative components. The potentialities of such a kind of approach with respect to mathematical modelling can be easily recognized in the variety of connections which can be modelled, provided that the modelling of the cyclic response of the joint components is available and the accuracy of the mechanical model adopted for combining the joint components is verified. The starting point of this approach is the check of the possibility to obtain the dissipation capacity of the whole joint as the sum of those of the single components, as testified in a companion paper (Iannone et al., 2010). In addition, because of the complexity of finite element models and their computational effort, mechanical models appear to be an effective and practical prediction of the beam-to-column behaviour under cyclic actions.

In this paper, the prediction of the cyclic rotational response of bolted connections is carried out by means of the mechanical model depicted in Figs.1-2, with reference to bolted end-plate connections (a) and to bolted tee-stub connections (b). In particular four sources of energy dissipation are considered: column flange in bending (cfb), column web in tension and compression (cwt-c), column web in shear (cws) and end-

plate in bending (epb)/ tee-stub in bending (tsb). The mechanical model is aimed at the prediction of the rotational cyclic response of bolted connections having two bolt rows in tension, because, as depicted in Figs.1-2, the behaviour of the two bolt rows in tension is modelled by means of only one spring element for each component (cfb and epb or cfb and tsb), represented by an equivalent T-stub.

The use of the mechanical model requires three steps to be performed:

- Modelling of the cyclic behaviour of each joint component;
- Assembling of the joint components;
- Evaluation of joint cyclic moment-rotation curve.

In the following sections, further details concerning modelling of the joint components and their assembling are provided. Successively, the proposed model is applied to simulate the joint rotational response of the specimens tested during an experimental program carried out at Salerno University whose results are presented in a companion paper. Finally, the accuracy of the proposed mechanical model is investigated by comparison with test results.

3 CYCLIC MODELLING OF DISSIPATIVE COMPONENTS

3.1 Equivalent “T-stub” under cyclic actions

Following the same approach of Eurocode 3 (CEN, 2005b), bolted joint components such as the end-plate in bending, the column flange in bending and the tee-stub in bending can be modelled by means of an “equivalent T-stub”. Aiming to develop a

model for the prediction of the cyclic behaviour of bolted steel connections, it is of paramount importance the use of a cyclic model sufficiently reliable for predicting T-stub response under cyclic actions. In this work, the model proposed by Piluso and Rizzano (2008) is applied.

According to such model the main parameter to define degradation laws of stiffness and strength is the energy cumulated at collapse under cyclic conditions. This is related to the energy absorbed under monotonic loads up to a displacement equal to the displacement amplitude of the i -th cycle, of the displacement history under investigation, by means of a nondimensional parameter. Such non-dimensional parameter is a function of the ratio between the ultimate displacement of the T-stub under monotonic loading conditions and the plastic part of the displacement occurring in the i -th cycle. The ultimate displacement under monotonic loading conditions can be evaluated depending on the material mechanical properties, by means of a coefficient C , and on the geometrical properties of the T-stub, namely m and n , that are respectively defined as the distance between the bolt axis and the plastic hinge corresponding to the flange-to-web connection and the distance between the prying force and the bolt axis, and t_f , i.e. the plate thickness (Piluso et al., 2001a,b).

By means of a regression analysis of experimental data dealing with isolated bolted T-stubs (i.e. the single joint component) under cyclic actions, the following relationship has been proposed derived (Piluso and Rizzano, 2008).:

$$\frac{E_{cc}}{E_0} = a_0 \left(\frac{t_f \delta_p}{2Cm^2} \right)^{-b_0} \quad (1)$$

where E_{cc} is the energy cumulated at collapse, assuming the conventional collapse as the attainment of 50% deterioration of energy dissipation capacity, E_0 is the area below the monotonic curve (i.e. the absorbed energy) up to a displacement equal to the one corresponding to the i -th cycle, C is a parameter depending on the material mechanical properties (Piluso et al., 2001), δ_p is the plastic part of the displacement corresponding to the i -th cycle and a_0 and b_0 are two regression parameters, given in Table 1.

The implementation of the model requires the preliminary evaluation of monotonic force-displacement curve by means of the theoretical approach proposed by Piluso et al. (2001a, b), starting from the knowledge of the geometrical and mechanical properties of the equivalent T-stub.

As soon as the T-stub monotonic behaviour has been theoretically predicted, for a given displacement amplitude the hysteresis loop is modelled by means of a multilinear approximation.

On the basis of experimental results on isolated T-stubs, the authors have observed that in constant amplitude cyclic tests, the points corresponding to the load inversion remain practically unchanged during loading and unloading process. These points, i.e. A and D in Fig.3, can be identified starting from the maximum displacement achieved at the i -th cycle and the corresponding load on the monotonic force-displacement curve, as depicted in the same figure.

As mentioned before, the characteristic points of the generic loading or unloading branch are defined provided that stiffness and strength degradation laws are known. Such degradation laws have been derived by means of a regression analysis of experimental data on isolated T-stubs, relating the degradation to the ratio between the maximum displacement of the i -th cycle (δ_{max}) and the threshold amplitude ($2\delta_y$) and to

the ratio between the energy dissipated in the previous loading history up to the i -th cycle and the energy cumulated at collapse, derived by means of Eq.(1):

$$\frac{F_i}{F_{max}} = 1 - a_1 \left(\frac{\delta_{max}}{2\delta_y} \right)^{a_2} \left(\frac{E_{ic}}{E_{cc}} \right)^{a_3} \quad (2)$$

$$\frac{K_i}{K_0} = 1 - b_1 \left(\frac{\delta_{max}}{2\delta_y} \right)^{b_2} \left(\frac{E_{ic}}{E_{cc}} \right)^{b_3} \quad (3)$$

where F_i and K_i are the force and stiffness at the i -th cycle, F_{max} is T-stub strength corresponding on the monotonic curve to the displacement δ_{max} of the i -th cycle, K_0 is the initial stiffness of the T-stub without bolt preloading (Faella et al., 1998), δ_y is the yield displacement, E_{ic} is the energy dissipated up to the i -th cycle and $a_1, a_2, a_3, b_1, b_2, b_3$ are regression parameters obtained by curve fitting of test data on isolated T-stubs. The values of the regression parameters required for the application of the model are reported in Table 1.

The main steps for the application of the model for predicting the cyclic response of the equivalent T-stub can be summarized as follows:

1. Prediction of monotonic force-displacement curve by means of Piluso et al. approach (2001a, b);
2. Computation of the energy E_0 absorbed under monotonic loads up to a displacement δ_{max} equal to the one of the i -th cycle:

3. Computation of the energy dissipation capacity E_{cc} corresponding to the displacement δ_{max} of the i -th cycle by means of Eq.(1);
4. Evaluation of the force F_{max} corresponding to δ_{max} on the monotonic force-displacement curve;
5. Definition of the strength degradation law by means of Eq.(2);
6. Definition of the stiffness degradation law by means of Eq.(3);
7. Evaluation of the degraded values of the force F_i and of the stiffness K_i for the current cycle;
8. Definition of the current bilinear branch of the cyclic response.

3.2 Modelling of the panel zone in shear

Since some seismic codes (CEN, 2005b) allow yielding of the panel zone prior to the full development of the moment carrying capacity of connected beams, many research efforts have been addressed to the study of the behaviour under load reversal of column web panels.

Many models have been proposed in last two decades to account for panel zone behaviour under both monotonic and cyclic loading conditions following two different approaches: the analytical modelling and the FEM modelling. In the former case, the panel zone is idealized as a dimensionless region, representing its behaviour by means of a rotational spring connecting two nodes at the same coordinates. Such a kind of approach allows to account for the additional P- Δ effects on the columns due to the shear deformation of the panel zone provided that rigid end offsets are adopted. On the other hand, FEM models are able to account for the actual dimension of the panel zone

by modelling the shear panel response by means of an assemblage of sub-elements representing the deformation modes of the panel zone.

In this work, Kim and Engelhardt model has been chosen due to the best agreement with the experimental results (Kim and Engelhardt, 1996, 2002). The authors, reviewing the existing literature since the 70s, have recently developed a monotonic and a cyclic model for the prediction of the panel zone under shear loads enhancing existing models (Kim and Engelhardt, 1996, 2002).

One of the main features of the model, originally developed by Cofie and Krawinkler to model cyclic stress-strain behaviour of steel, resides in the rules for the movement of the bound lines. In such a model the cyclic steady state curve, defined as the locus of peak moments obtained by cycling the shear panel at various rotation amplitudes, is used to describe the bound lines shift at the i -th cycle (Krawinkler et al., 1983). The following expression for the cyclic steady state curve has been proposed by the authors:

$$\frac{\gamma}{\gamma_n} = \frac{M_s}{M_n} + \left(\frac{M_s}{\xi M_n} \right)^c \quad (4)$$

Where M_s is the ordinate of the cyclic steady state curve M_n is the normalizing moment assumed equal to the panel zone yield moment plus two times the plastic moment of the column flange, γ_n is the rotation corresponding to the normalizing moment, given by the ratio between the normalizing moment and the panel elastic stiffness evaluated according to Kim and Engelhardt monotonic model for panel zone in shear and ξ and c are two parameters empirically determined, given in Table 2.

The first step to apply the model is the evaluation of the monotonic law for the prediction of the first loading branch. The monotonic moment-rotation relationship is idealized by means of a quadrilinear curve, defined by two post-elastic branches and a strain-hardening branch (Fig.4). After the first semi-cycle, the i -th loading or unloading branch is constituted by a linear elastic branch, whose elastic limit factor is equal to 1.4 times the monotonic value M_y (Fig.4) as determined on the basis of available experimental results, followed by a non linear curve derived according to the expression proposed by Dafalias (1975, 1976).

The non linear branch, according to Dafalias' bounding surface theory, is defined by means of the following parameters:

- the distance d_{in} between the bound line and the end point of the previous linear branch (Fig.4);
- shape factor ψ whose value depends on the accumulated plastic rotation θ_p during the previous loading history;
- the slope K_p^{bl} of the bound line (Fig.4).

In particular, the factor ψ governs the shape of the i -th non linear branch. Kim and Engelhardt have found that a shape factor equal to 20 for small rotation amplitudes and equal to 40 for large rotation amplitude provides a good agreement with available experimental tests (Kim and Engelhardt, 1996). Thus, a shape factor varying with a Boltzman function has been proposed:

$$\psi = 40 + \frac{(20 - 40)}{\left[1 + e^{\frac{(\theta_p - 0.213)}{0.074}} \right]} \quad (5)$$

where θ_p is the accumulated plastic rotation at the i-th cycle.

According to Dafalias' theory, the plastic part of the non linear curve is defined by means of its plastic stiffness K_p^A (Fig.5). This is related to the distance from the bound line of the generic point d_A , to d_{in} , to ψ and to the stiffness of the bound line by means of the following expression:

$$K_p^A = K_p^{bl} \left[1 + \psi \frac{d_A}{d_{in} - d_A} \right] \quad (6)$$

By means of comparison with experimental data a slope of the bound line $K_p^{bl} = 0.008K_e$ is assumed, where K_e is the initial stiffness of the shear panel defined by Kim and Engelhardt monotonic model (Kim and Engelhardt, 2002). Finally, the tangent stiffness at the point A of the inelastic curve is determined accounting for its elastic part by means of the following relationship:

$$K_t^A = \frac{K_e K_p^A}{K_e + K_p^A} \quad (7)$$

Finally, the procedure to shift the bound line at the i-th loading or unloading cycle can be summarized in the following steps:

- compute the mean value of the bending moment and the mean value of the rotation corresponding to the previous semi-cycle, given by:

$$M_m = \frac{(M_i + M_{i-1})}{2} \quad (8)$$

$$\gamma_m = \frac{(\gamma_i + \gamma_{i-1})}{2} \quad (9)$$

- compute the semi-amplitude of moment range and rotation range corresponding to the previous semi-cycle, given by:

$$M_a = \frac{|M_i - M_{i-1}|}{2} \quad (10)$$

$$\gamma_a = \frac{|\gamma_i - \gamma_{i-1}|}{2} \quad (11)$$

- calculate the difference ΔM between the moment amplitude M_a and M_s , which is defined as the moment on the cyclic steady state curve corresponding to the rotation amplitude γ_a ;
- if ΔM is positive cyclic hardening is predicted and bound lines are moved outward by an amount equal to $2F_H \Delta M$;
- if ΔM is negative cyclic softening is predicted and bound lines are moved inward by an amount equal to $2F_S \Delta M$;

- finally bound lines are moved to account for the mean moment relaxation by an amount equal to $F_R M_m$.

The hardening, softening and mean moment relaxation coefficients, namely F_H, F_S and F_R respectively, are given in Table 2 as experimentally evaluated by Cofie and Krawinkler (1983,1985).

Reference is made to the original work for further details regarding the monotonic behaviour and the steps needed to apply the model.

3.3 Modelling of column web under tension and compression

The model adopted in this paper to account for the panel zone behaviour under cyclic tension and compression is the one proposed by Cofie and Krawinkler (1983,1985) and, therefore, it is analogous to the one of Kim and Engelhardt for the shear panel. The main differences between the above models are constituted by the assumption of a constant shape factor ψ and of a non-linear monotonic law. Three curves are needed to apply the model: the monotonic curve, the cyclic steady state curve and the hysteretic curve consistent with Dafalias' bounding surface theory.

The monotonic curve is constituted by means of a linear branch followed by a yield plateau branch and a non linear hardening branch defined by a Ramberg-Osgood relationship. The branch are given by the following equations:

$$\sigma = E \varepsilon \qquad \text{for} \qquad 0 < \varepsilon < \varepsilon_y \qquad (12)$$

$$\sigma = f_y \qquad \text{for} \qquad \varepsilon_y < \varepsilon < 14 \varepsilon_y \qquad (13)$$

$$\frac{\varepsilon}{\varepsilon_y} = \frac{\sigma}{f_y} + \left(\frac{\sigma}{K f_y} \right)^{1/n} \quad \text{for} \quad \varepsilon > 14\varepsilon_y \quad (14)$$

where the coefficients K and n given in Table 3, as obtained from curve fitting of experimental data reported in Krawinkler et al. studies on A36 steel. As already seen for the modelling of the shear panel, the cyclic steady state curve is a stable reference curve during the whole cyclic loading history representing the saturation curve of the material (Fig.6). This has been determined by Cofie and Krawinkler by means of constant amplitude cycles and it is described by a Ramberg-Osgood relationship:

$$\frac{\varepsilon}{\varepsilon_y} = \frac{\sigma}{f_y} + \left(\frac{\sigma}{K' f_y} \right)^{1/n'} \quad (15)$$

the parameters K' and n' are given in Table 3. Further details on the application of the model can be found in the original work of the authors. The parameters needed for the application of the model are reported in Table.3.

4 ASSEMBLING OF COMPONENTS

In order to obtain the cyclic moment-rotation curve of bolted beam-to-column joints starting from the knowledge of the cyclic force-displacement behaviour of the joint components, a computer program has been developed.

The computer program is constituted by a series of subroutines, corresponding to the different joint components, providing the component displacement for a given force level. With reference to the loading phase of the cyclic response, the main program

working step-by-step for increasing values of the bending moment compute the force level in each joint component, because of the knowledge of the lever arm. Therefore, for each joint component the displacement corresponding to the given force level can be computed, accounting for the rules governing their cyclic behaviour. The knowledge of the displacements of each joint component allows the evaluation of the joint rotation according to the mechanical models presented in Section 2. This process is carried out step-by-step, increasing the bending moment, until the computed rotation assumes a value equal to the one corresponding to the end of the loading semi-cycle of the applied history.

Similarly, regarding the unloading phase, the analysis is carried out by progressively decreasing the bending moment and computing the force levels in the joint components. The computation of the component displacements, accounting for the previous loading history, allows the evaluation of the corresponding rotation values. This process continues up to the rotation value corresponding to the end of the unloading phase, as fixed by the applied rotation history.

The above procedure is repeated for the following loading-unloading cycles to compute the numerical prediction of the joint cyclic flexural response corresponding to a given rotation history.

5 ACCURACY OF THE PROPOSED MECHANICAL MODEL

In order to verify the accuracy of the proposed model, the numerical simulation of the cyclic rotational response of the beam-to-column connections tested at the University of Salerno, presented in a companion paper (Iannone et al. 2010), has been carried out. In

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addition, to enlarge the experimental sample for model validation, other three tests collected from the technical literature have been considered.

Regarding the authors' own tests, attention has been focused on three partial strength bolted beam-to-column connections. All the specimens are characterized by the same coupling of beam and column, but with different details of the connection elements. In particular, HEB200 and IPE270 profiles have been used for the column and the beam, respectively. The first joint, namely EEP-CYC01 (Fig.7), is characterized by a panel zone without continuity plates and by an end-plate whose resistance has been calibrated to significantly reduce its plastic engage so that, as confirmed by experimental evidence the energy dissipation mainly occurs in the shear panel. The second and the third joint, namely EEP-CYC02 (Fig.8) and TS-CYC04 (Fig.9) have been designed aiming to engage in plastic range the end-plate in bending, in case of EEP-CYC 02 specimen, and the tee-stub in bending, in case of TS-CYC04 specimen. To this scope, continuity plates and supplementary web plates have been added to the panel zone. The equivalent T-stubs modelling the end-plate and the tee-stub, respectively, of these two joints are characterized by flanges having different thickness and bolt location.

Regarding the experimental tests collected from technical literature, those performed by Bernuzzi et al. (1995), Nogueiro et al. (2006) and Yang & Kim (2006) have been considered. The joints under investigation are characterized by different details. In particular, Bernuzzi test, namely FPC/B (Fig.10), consists of a beam stub of an IPE 300 section connected by means of a flush end-plate connection to a rigid counterbeam. This testing condition approximately reproduces the case of a beam-to-column joint with negligible column deformability. Nogueiro et al. test, namely J-1.3 (Fig.11), is composed by the assemblage of an IPE 360 beam and an HEA 320 column coupled

with an extended end-plate connection whose end-plate thickness is equal to 18 mm fastened with eight M24 bolts (10.9 grade). The steel grades of plates and members is S355. The column panel zone is strengthened by means of 15 mm continuity plates. In this case, the experimental test has evidenced that the joint response of this specimen has been mainly governed by the shear panel plastic engagement with a minor contribution of the end-plate. Finally, Yang & Kim test, namely FW (Fig.12), is constituted by H-250x125x6x9 beam fully welded to a H-125x125x6.5x9 column without stiffeners on the column panel zone, i.e. with neither supplementary web plates nor continuity plates. All welds were checked by means of magnetic particle testing to assure the absence of defects. The base material of plates and steel members is SS400. The test is characterized by a dissipation mainly concentrated in the panels in tension and compression and in the shear panel.

The comparison between the experimental results and the numerical results deriving from application of the developed mechanical model shows a good agreement both in terms of cyclic moment-rotation curves and in terms of energy dissipation. Obviously, the accuracy of the developed mechanical model in predicting the cyclic rotational behaviour of beam-to-column joints is mainly related to the accuracy in the modelling of the cyclic force-displacement response of the weakest joint component. Therefore, in the case of specimen EEP-CYC 01 and J-1.3, where dissipation mainly occurs in the shear panel, the good accuracy (Fig.13-14) testifies the goodness of Kim and Engelhardt model adopted for the modelling of the panel zone in shear. In both cases the shape of the hysteresis loops is accurately predicted in terms of stiffness and peak moment. Notwithstanding, a slight overestimation of the energy dissipation capacity occurs (Fig.15-16), because of a slight overestimation of resistance in the monotonic envelope.

In addition, the accuracy of simulation of FW test is mainly governed by the accuracy of the modelling of shear panel and panels in tension/compression which are the two components engaged in plastic range (Fig.17). The comparison in terms of energy dissipation indicates that beyond Kim and Engelhardt model, even Cofie and Krawinkler model is sufficiently reliable, allowing a satisfactory prediction of the joint rotational response (Fig.18). Furthermore, regarding the accuracy in the modelling of the cyclic response of specimens EEP-CYC 02 (Fig.19), TS-CYC 04 (Fig.20) and FPC/B (Fig.21), it is mainly related to the accuracy in the modelling of the cyclic force-displacement response of the equivalent T-stub modelling the end-plate in bending and the tee-stub in bending, respectively. In fact, the two specimens are characterized by the plastic engage of the T-stub representing the weakest joint component. In particular, the experimental cyclic response is characterized by a more significant pinching of hysteresis loops compared to the numerical prediction. The pinching of the hysteresis loops is due to the whole reloading branch, not only to the initial reloading stiffness. Therefore, such pinching is affected by the degradation of stiffness and strength as the number of cycles increases. It is a highly complicated phenomenon, because it is affected not only by the geometrical properties, but also by the fabrication process which, in turn, affects stiffness and strength degradation rules. This is confirmed not only by the experimental results presented in this work, but also by experimental tests dealing with isolated T-stubs carried out both by the authors (Piluso and Rizzano, 2008) and by other researchers (Clemente et al., 2004, 2005). In particular, according to Clemente, Noè and Rassati (2004, 2005), experimental results on the cyclic response of isolated T-stubs can be classified under three different aspects: *a*) behavioral differences among same-sized T-stubs fabricated with different methods; *b*) behavioral differences

among different-sized T-stubs with the same fabrication method; *c*) differences between the cyclic and the monotonic behavior of the specimens. As far as issue *a*) is concerned (Clemente et al., 2004, 2005) have found results in agreement with those presented in this paper stating that force-displacement curves of same sized T-stubs obtained from hot rolled shapes and by welding of plates are characterized by different amount of pinching. This is caused by localized plastic deformations of the T-stub flanges which tend to pull away from the plate surface. In addition, welded T-stubs show early signs of stiffness and strength degradation, mostly due to the formation of cracks in the welds. Therefore, experimental evidence shows that stiffness and strength degradation rules are significantly affected by the fabrication process, because of the heat affected zone.

In this framework, it is useful to observe that the model for the cyclic response of the equivalent T-stub was mainly calibrated on the basis of the experimental tests regarding the cyclic response of isolated T-stubs made of rolled profiles. Therefore, as already underlined in (Piluso and Rizzano, 2008), an improvement of the model could be expected provided that additional experimental tests on isolated T-stubs composed by welding are carried out. In fact, the modelling of bolted connections by means of the component approach requires the definition of an equivalent T-stub to derive the force versus displacement behaviour of the column flange in bending and of the end-plate in bending/ tee-stub in bending (depending on the connection typology). It is evident that there is no any difference between rolled T-stubs and welded T-stubs when stiffness and strength are of concern, because the flange-to-web connection typology of the T-stub slightly affects only the *m* parameter providing the distance between the bolt axis and the plastic hinge arising at the flange-to-web connection according to Eurocode 3 (CEN, 2005b). Conversely, it is also to be recognized that, as soon as the cyclic behaviour is of

concern, stiffness and strength degradation rules and, as a consequence, pinching phenomena can be better modelled by using two equivalent T-stubs: one calibrated on the cyclic response of rolled T-stubs to be used for the column flange in bending and the second one calibrated on the cyclic response of welded T-stubs to be used for modelling either the end-plate in bending or the T-stub in bending, depending on the connection typology.

Therefore, it is expected that when the cyclic behaviour of the beam-to-column joint is mainly governed, like in the examined cases, by the end-plate in bending or T-stub in bending, an improved prediction of the whole cyclic behaviour and, in particular, of pinching and degradation phenomena can be obtained by means of degradation rules properly calibrated on a wider experimental sample dealing with isolated welded T-stubs under cyclic actions.

As a result of the more marked pinching occurring in the experimental hysteresis loops, the mechanical model provides also in these cases a slight overestimation of the energy dissipation, as depicted in Fig. 22 for EEP-CYC 02 specimen, in Fig.23 for specimen TS-CYC 04 and in Fig.24 for Bernuzzi et al. test FPC/B. Moreover, the maximum ratios between the dissipated energy predicted by the model and that provided by experimental evidence are summarized in Tab.4 for the six tests considered. Finally, in order to give a more complete picture of the joint model performance, the comparison has been extended to the peak moment response and to the stiffness evaluated at each semi-cycle on unloading branches (Figs. 25-28). In all cases the model appears to be sufficiently reliable providing a good correlation with the experimental results.

The results are really encouraging about the possibility of accurately predicting the cyclic response of bolted connections by means of the component approach. The

accuracy of the developed mechanical model can be improved provided that additional test results on the cyclic response of isolated joint components are available to improve the modelling of the cyclic force versus displacement behaviour of the joint components. Moreover, considering that the cyclic model relies on empirical parameters calibrated on the basis of cyclic experimental tests with only a symmetrical loading history, other tests with generic loading history are necessary to prove the sufficiency of the model for any loading history.

6 CONCLUSIONS

In this paper, a mechanical model for predicting the cyclic response of bolted joints has been developed starting from models available in the technical literature for the modelling of the cyclic behaviour of each joint component. The main feature of the model is that it relies on the component approach in the same fashion of the approach codified in Eurocode 3 for monotonic loading conditions. The accuracy of the model has been investigated by means of a comparison in terms of energy dissipation, peak moment response and stiffness of unloading branches, between the results of an experimental program performed at the University of Salerno and those provided by the proposed model. A further validation has been obtained by extending such comparison to some experimental tests taken from the technical literature. In particular, a good accuracy with experimental test results has been obtained in terms of strength, stiffness and energy dissipation. The models by Cofie and Krawinkler and by Kim and Engelhardt for the panel zone in tension and compression and for the panel zone in shear, respectively, assure a good agreement with the experimental results. Also the approach proposed by Piluso and Rizzano (2008) in previous works for modelling the

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4 cyclic behaviour of bolted T-stubs appears sufficiently accurate. Nevertheless, the
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6 setting up of more accurate stiffness and strength degradation laws for T-stubs
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8 composed by welding can lead to the model improvement for those joints where
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10 significant plastic engage occurs in the flanges of the equivalent T-stub modelling the
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12 bolt row behaviour.
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16 The obtained results are encouraging about the possibility of extending the component
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18 approach to cyclic loading conditions. To this scope, further research efforts are needed
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20 to improve the accuracy of the modelling of the cyclic force versus displacement
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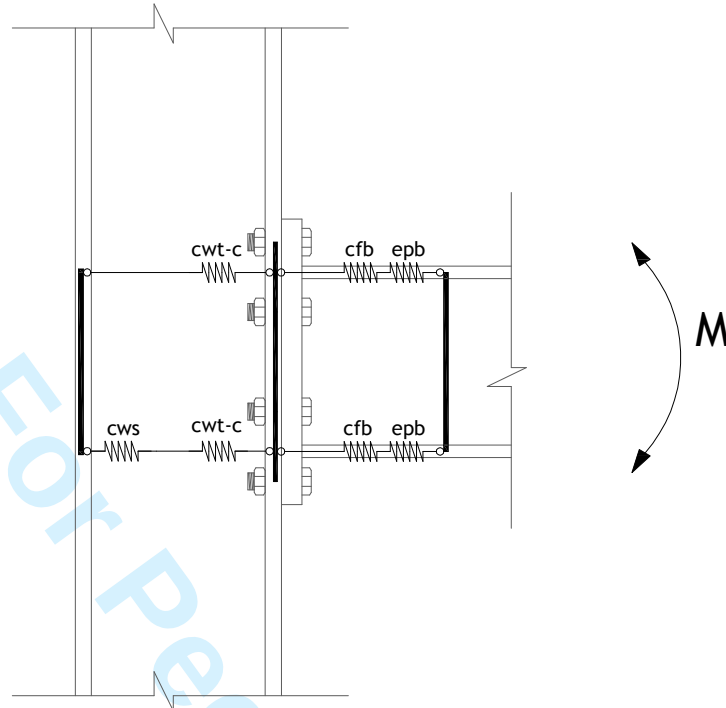


Fig. 1: Mechanical model for bolted extended end-plate connections

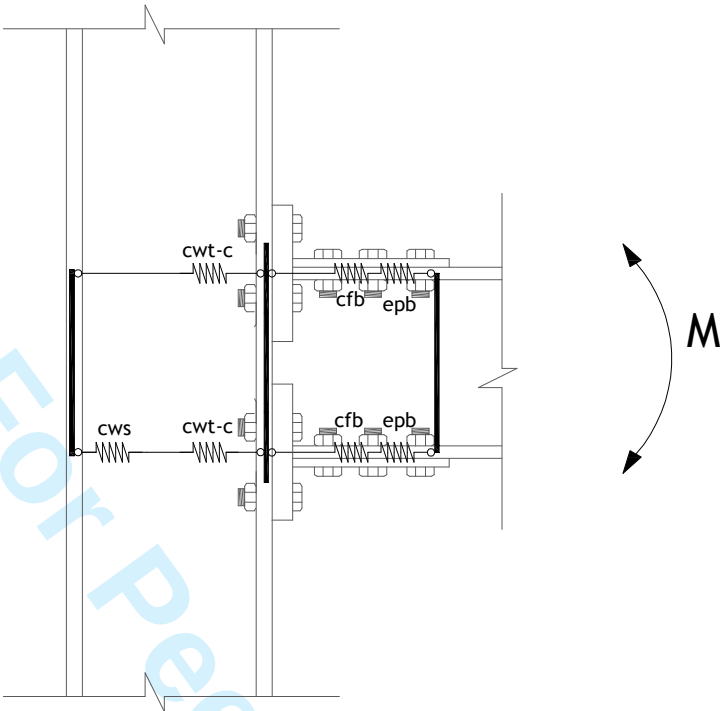


Fig. 2: Mechanical model for double tee connections

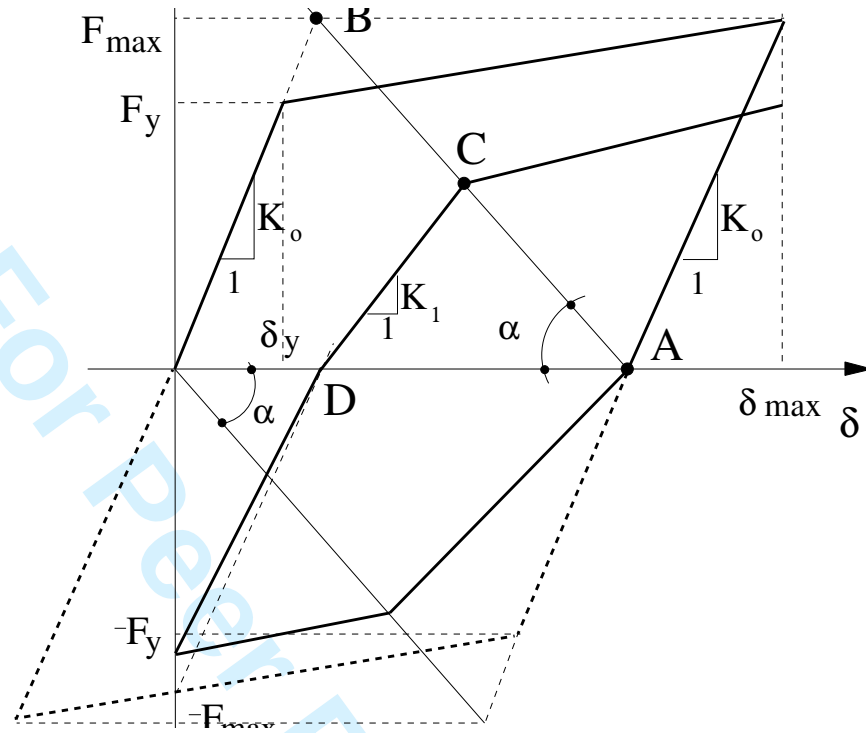


Fig. 3: Piluso et al. T-stub cyclic model

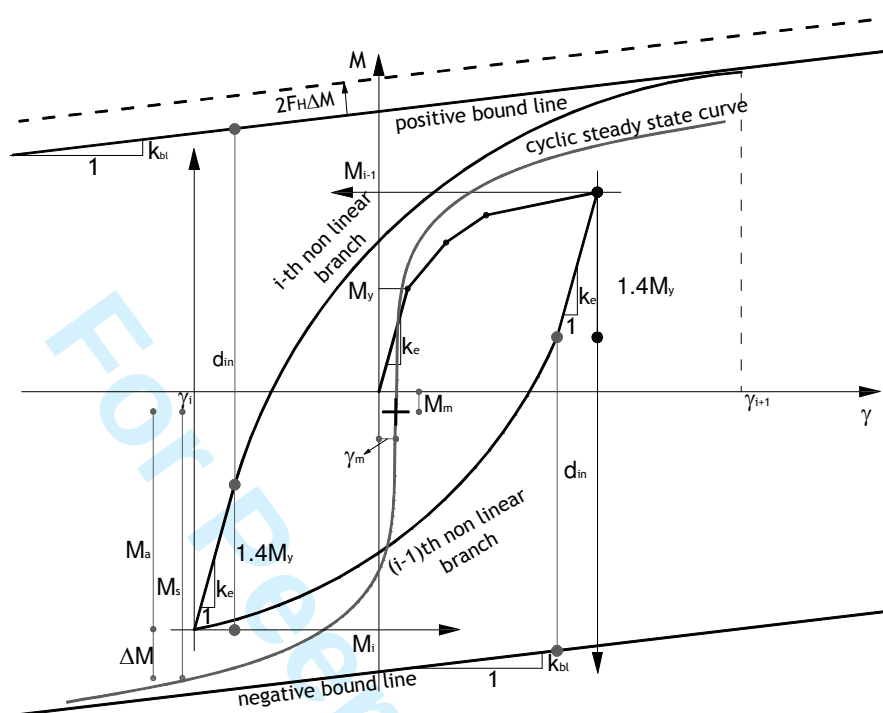


Fig. 4: Shear panel cyclic curve

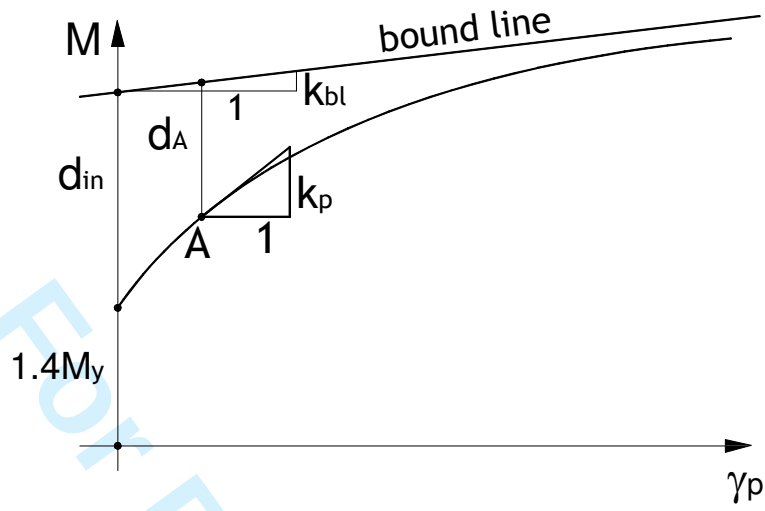


Fig. 5: Dafalias-Popov model for Hysteresis Curve

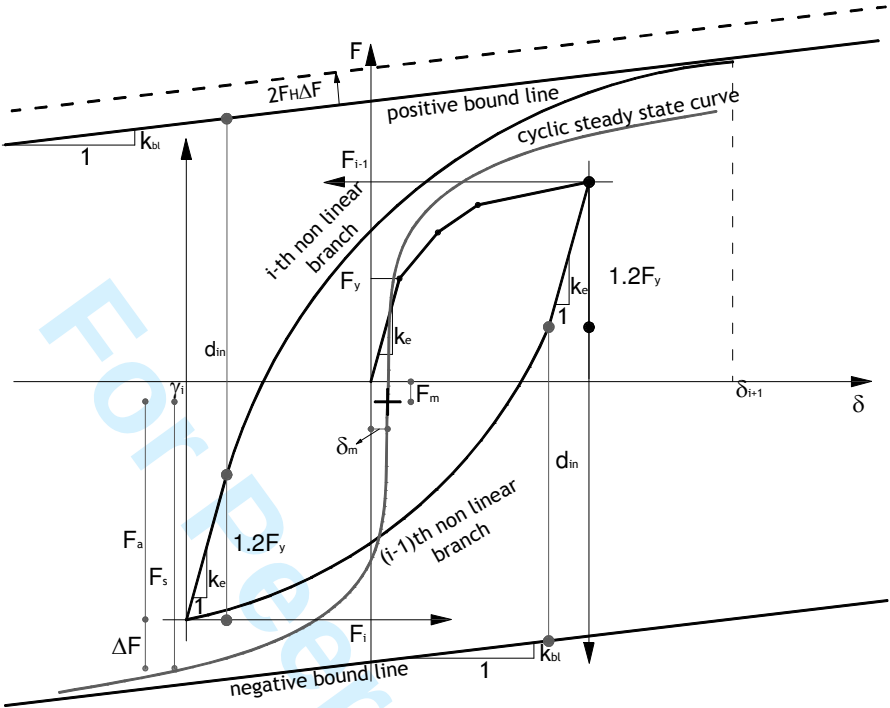


Fig. 6: Cyclic curve at the i -th cycle for panels in tension and compression

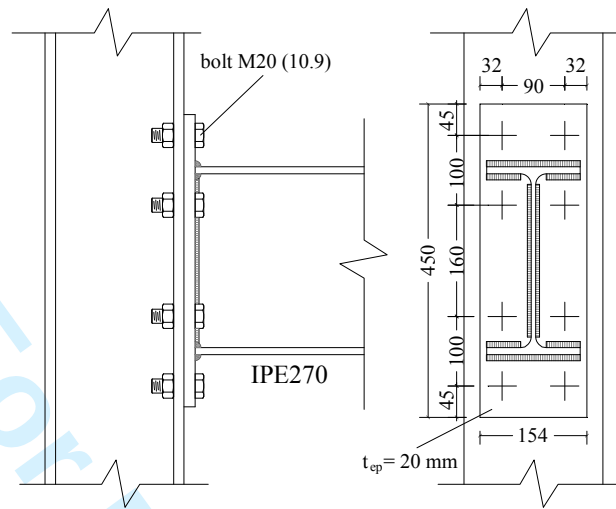


Fig.7 – Geometry of EEP-CYC01 specimen

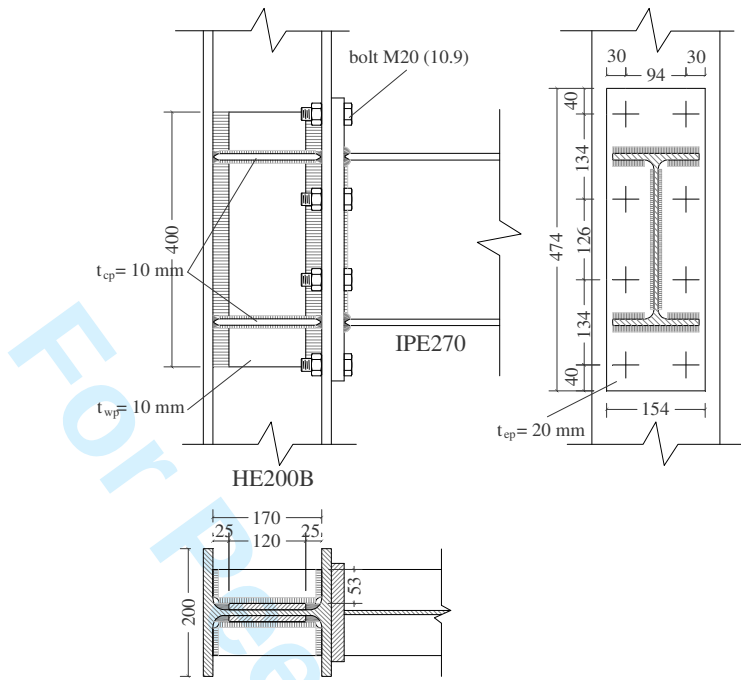
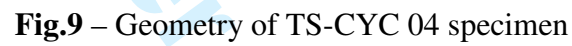


Fig.8 – Geometry of EEP-CYC02 specimen



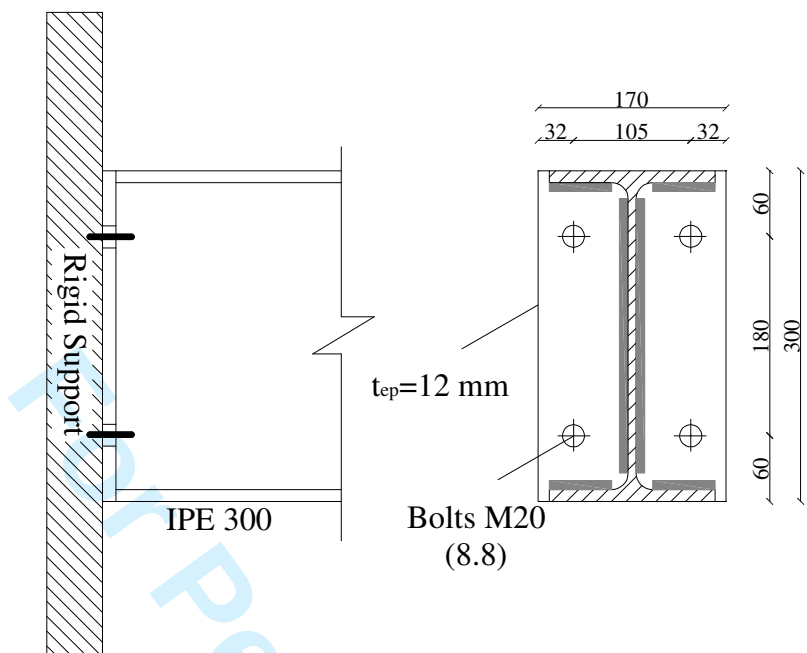


Fig.10 – Geometry of FPC/B specimen (redrawn from Bernuzzi et al., 1996)

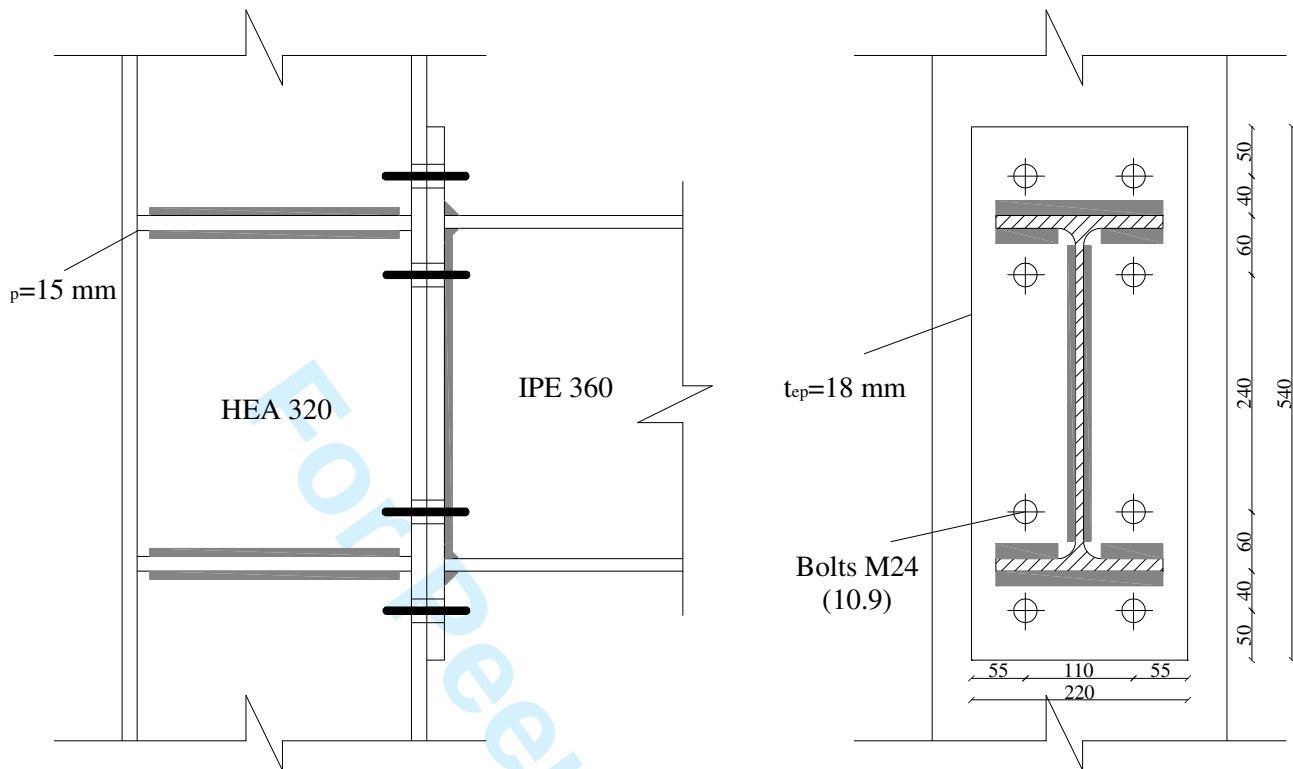


Fig.11 – Geometry of J-1.3 specimen (redrawn from Nogueiro et al., 2006)

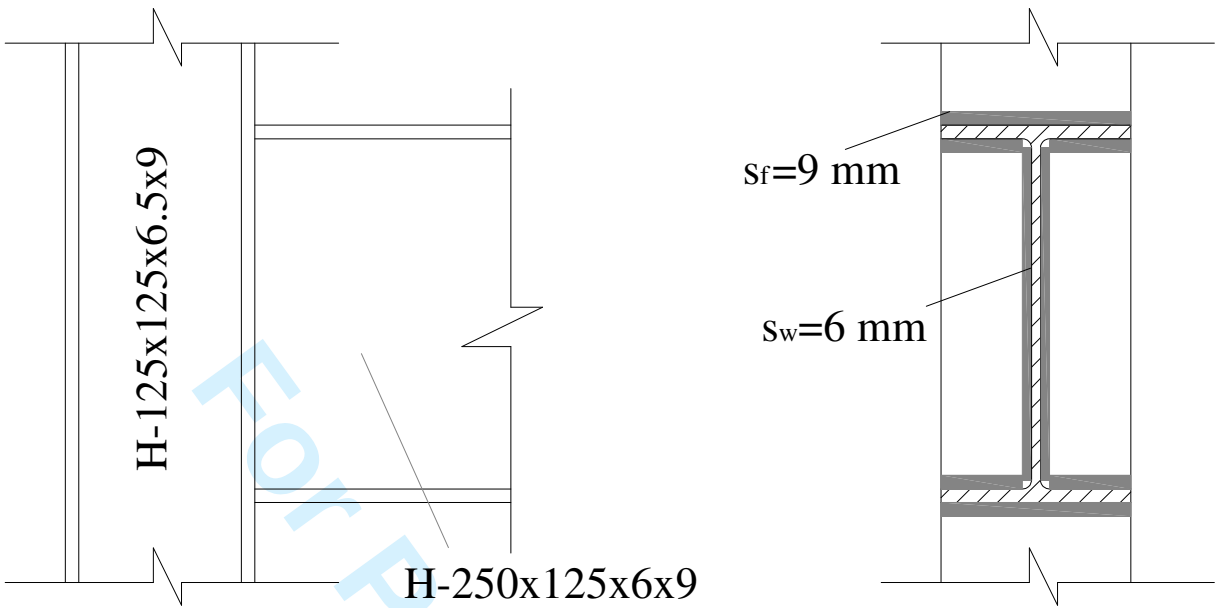


Fig.12 – Geometry of FW specimen (redrawn from Yang & Kim, 2006)

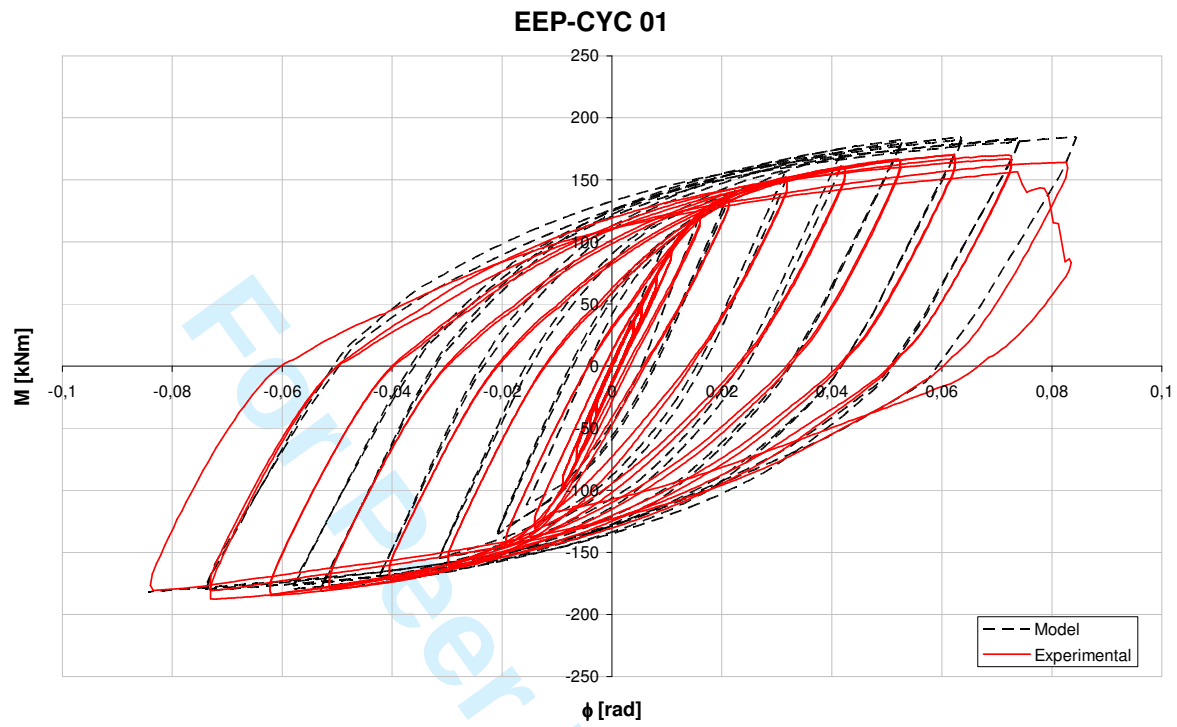


Fig. 13: Theoretical-Experimental comparison for specimen EEP-CYC 01

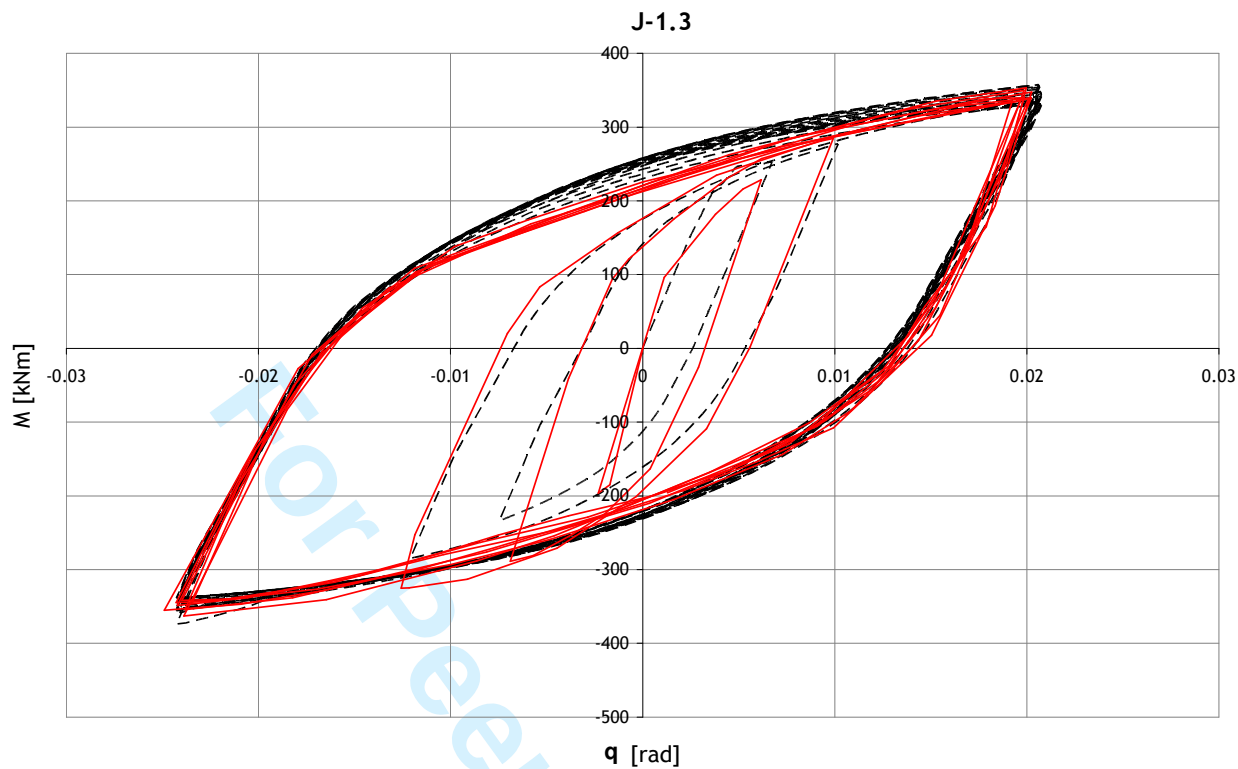


Fig. 14: Theoretical-Experimental comparison for specimen J-1.3

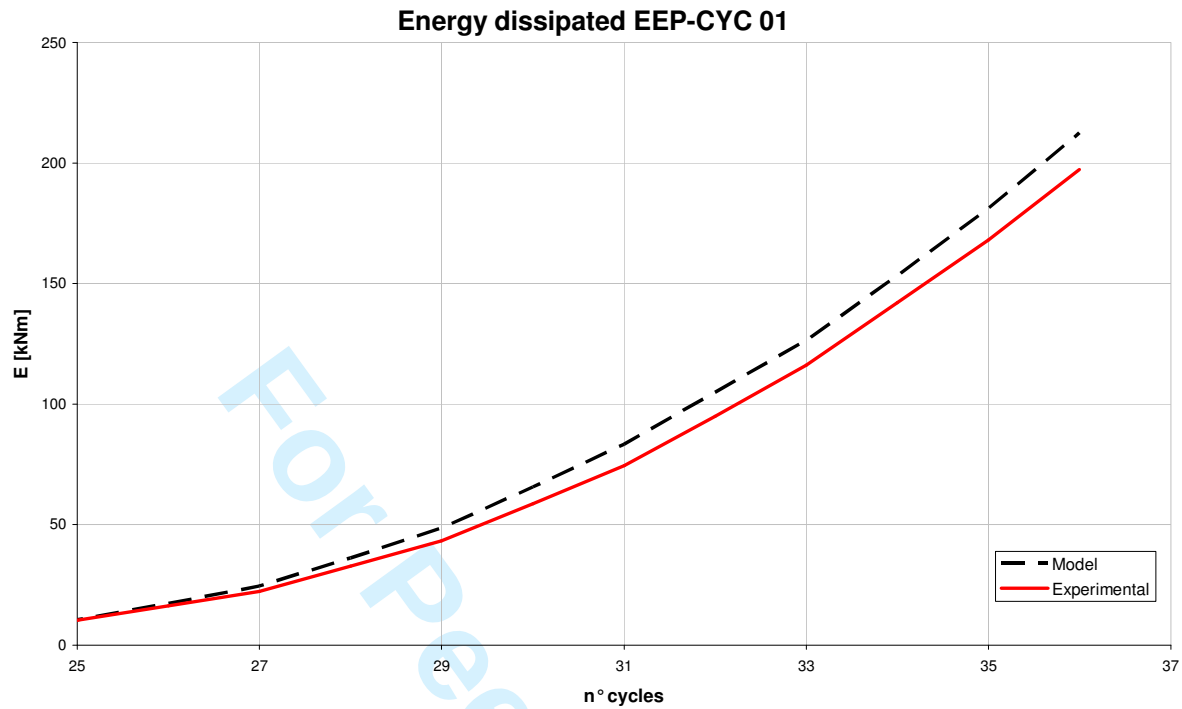


Fig.15: Theoretical-Experimental comparison of the energy dissipated for specimen EEP-CYC 01

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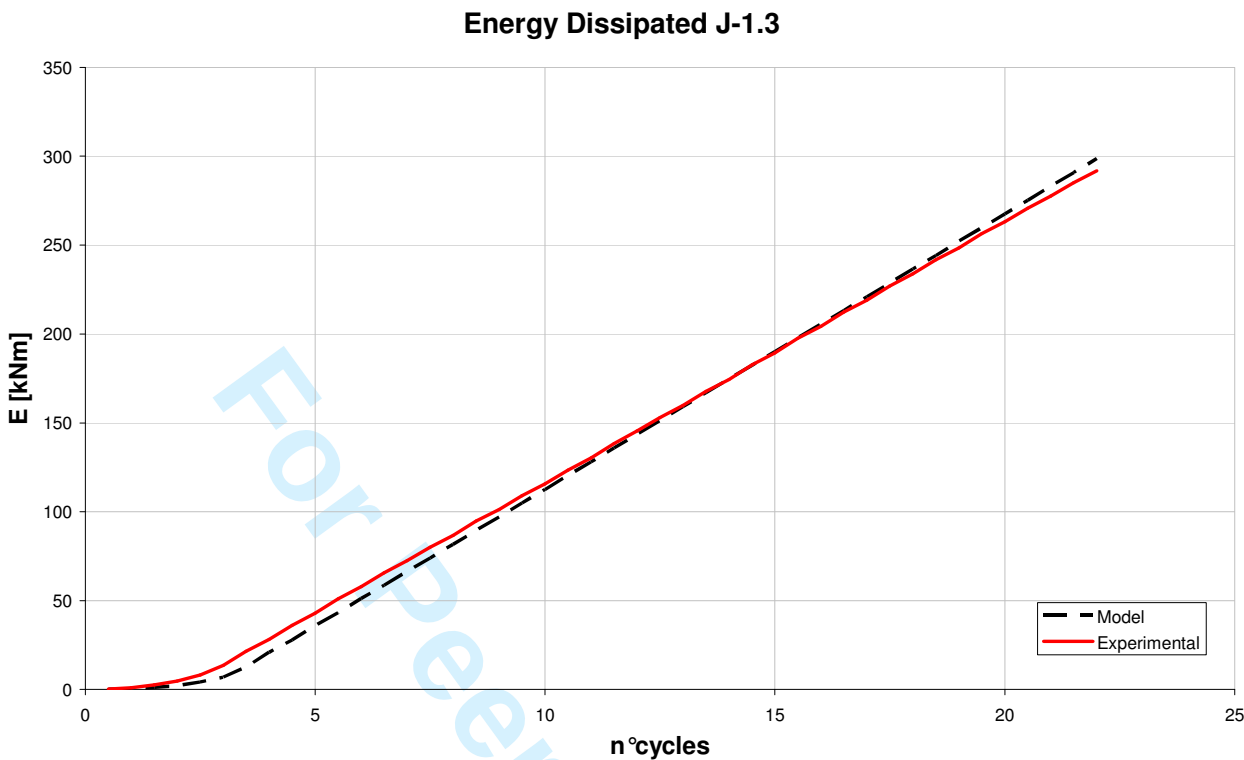


Fig. 16: Theoretical-Experimental comparison of the energy dissipated for specimen J-1.3

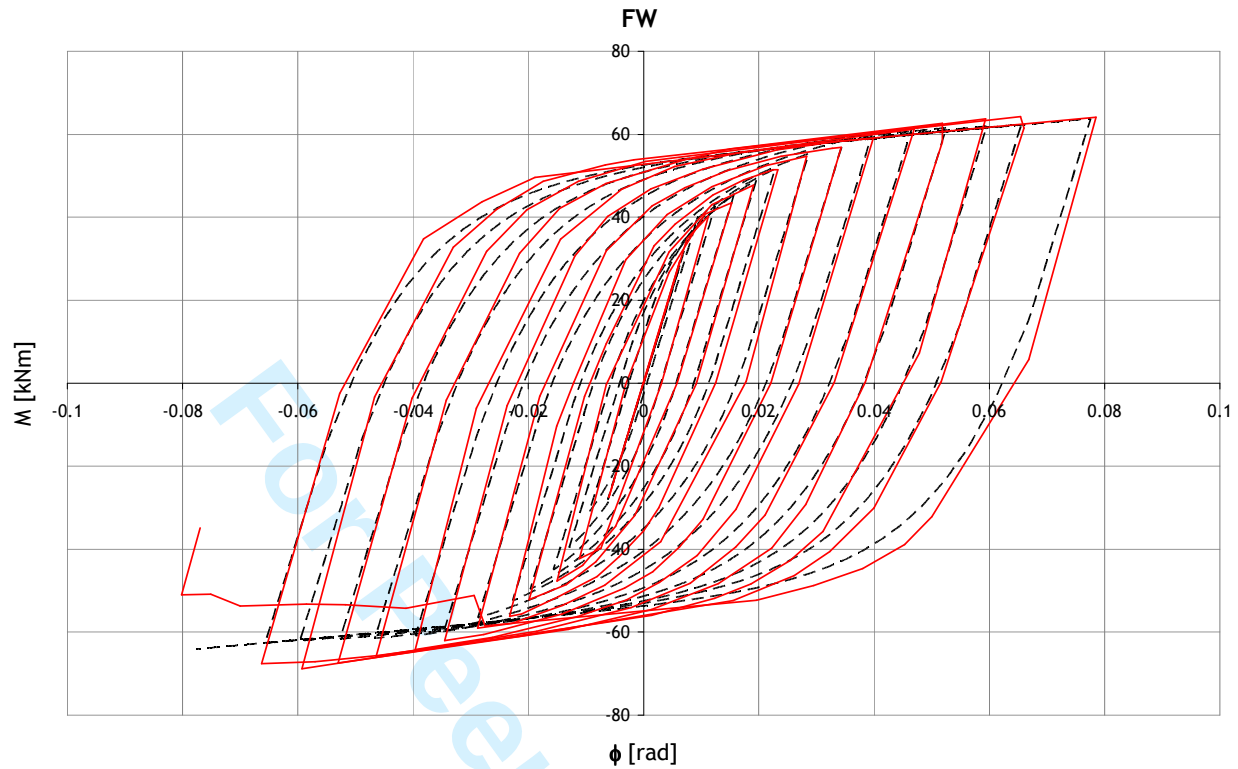


Fig. 17: Theoretical-Experimental comparison for specimen FW

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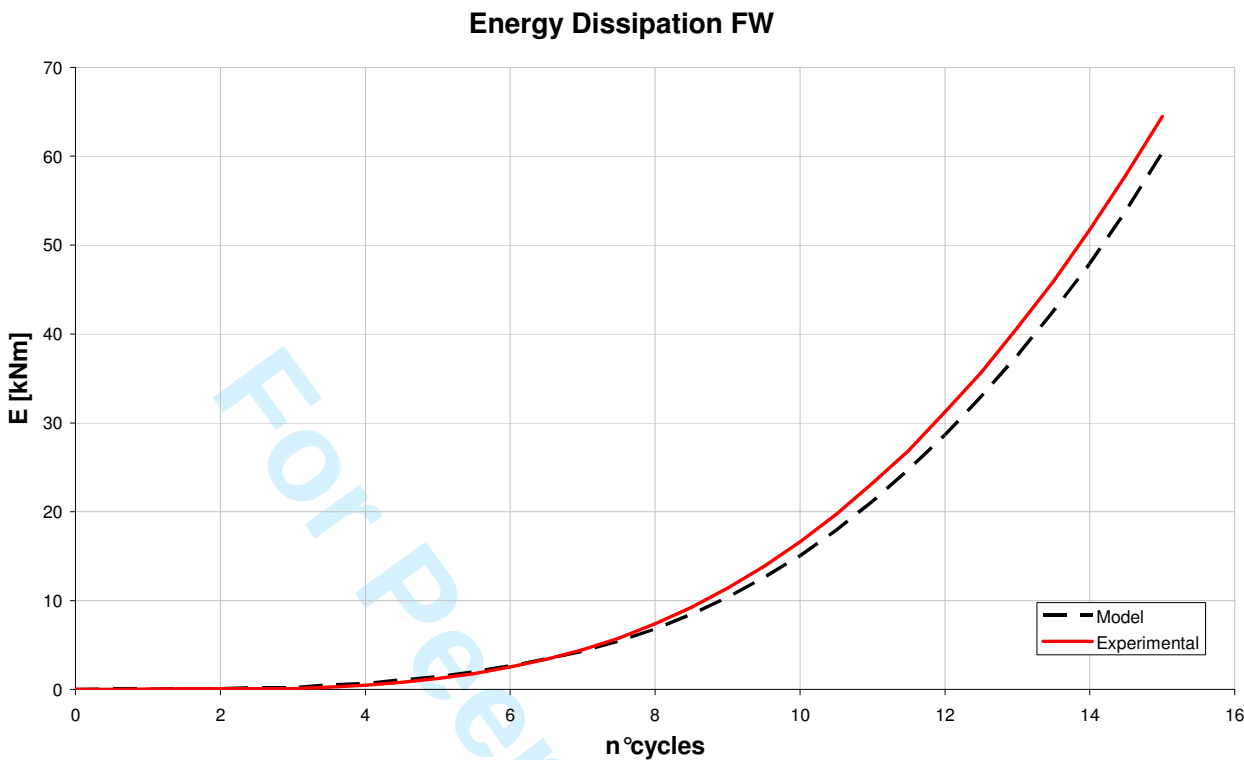


Fig. 18: Theoretical-Experimental comparison of dissipated energy for specimen FW

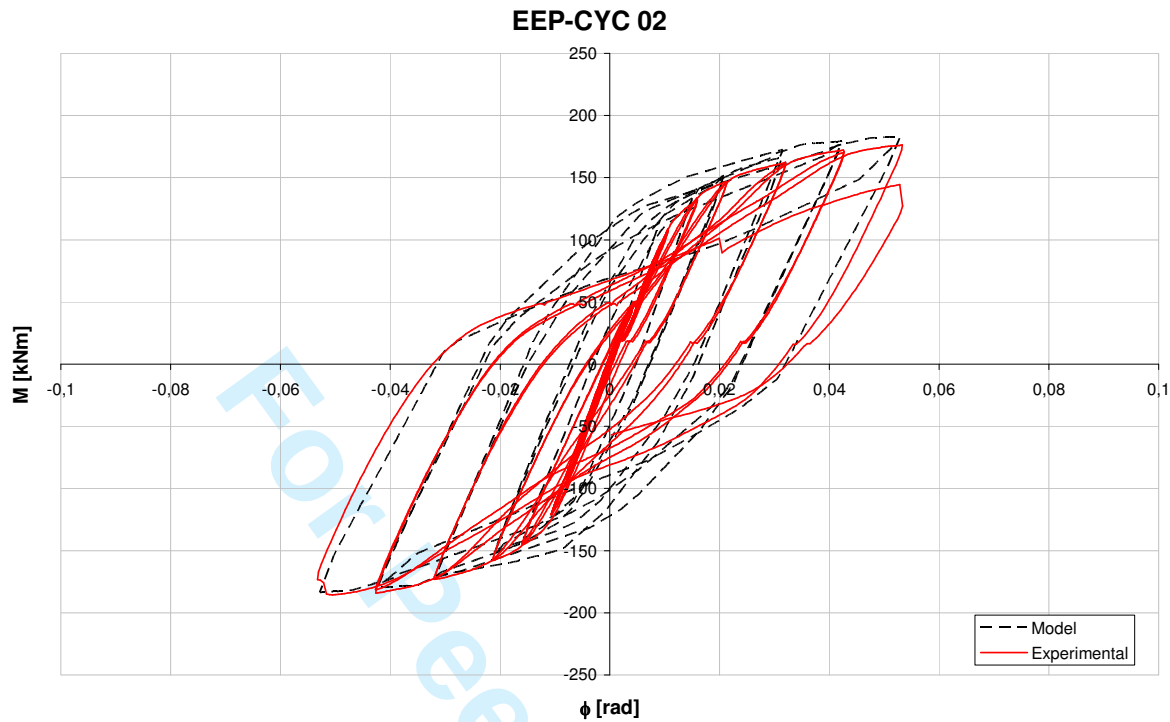


Fig. 19: Theoretical-Experimental comparison for specimen EEP-CYC 02

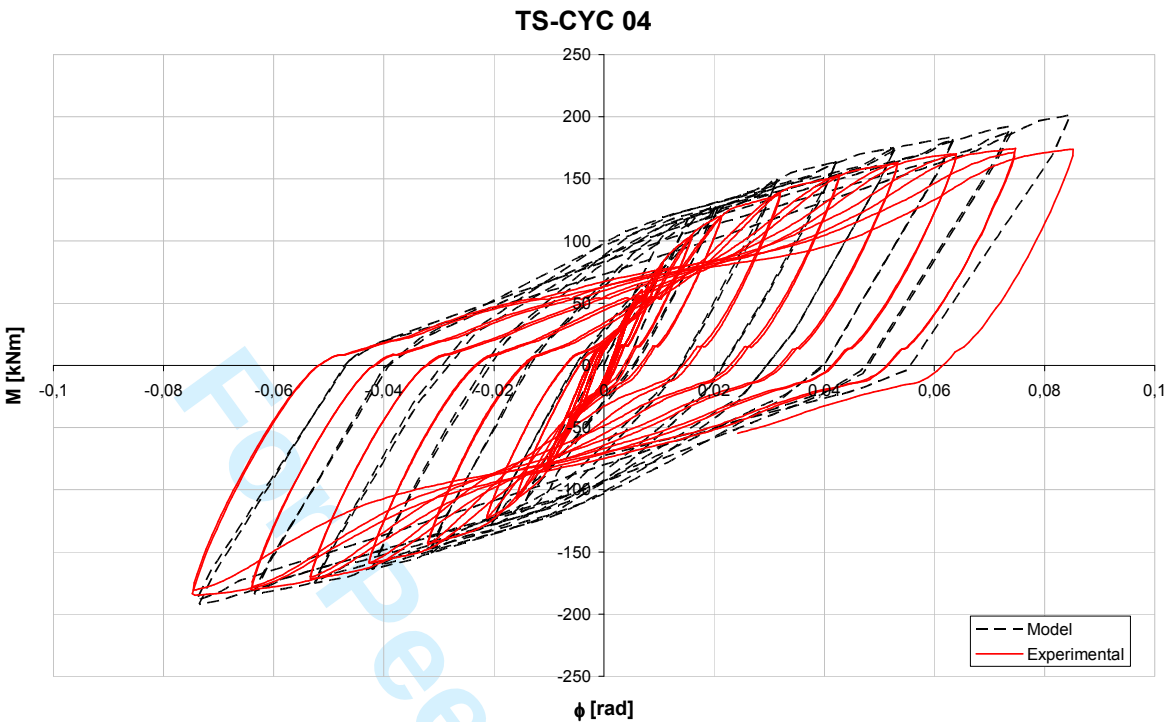


Fig. 20: Theoretical-Experimental comparison for specimen TS-CYC 04

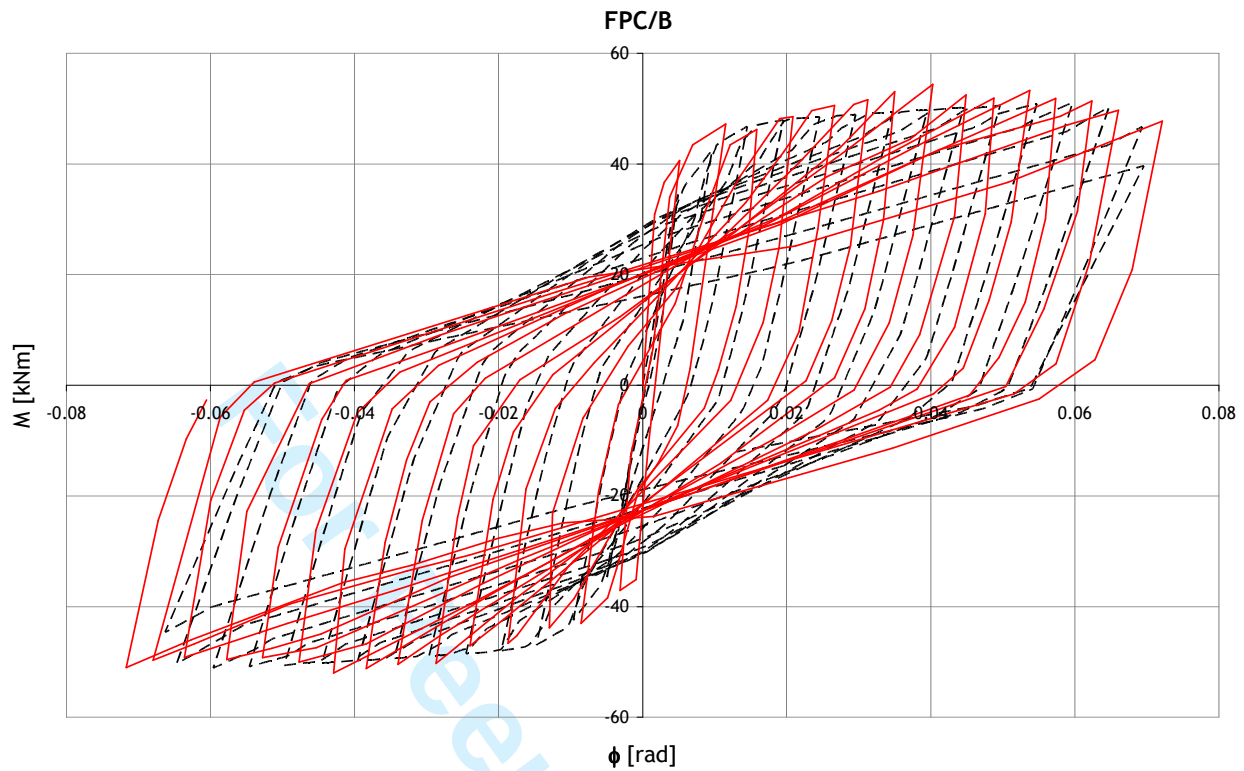


Fig. 21: Theoretical-Experimental comparison for specimen FPC/B

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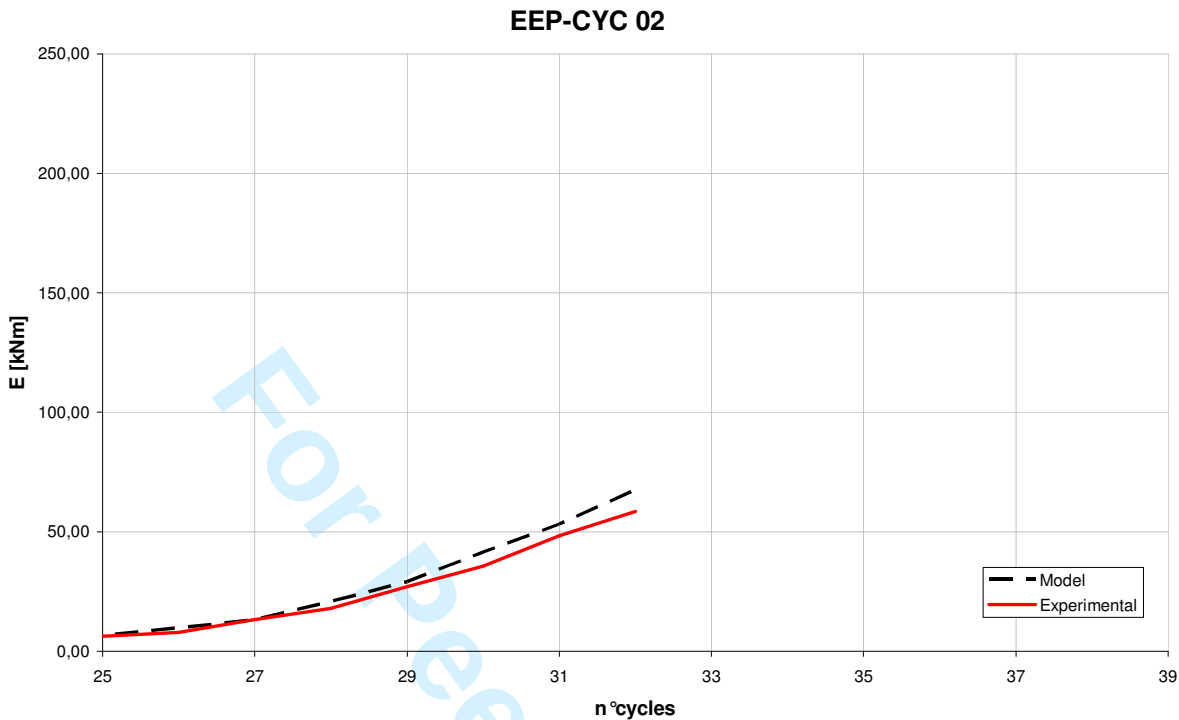


Fig. 22: Theoretical-Experimental comparison of the energy dissipated for specimen EEP-CYC 02

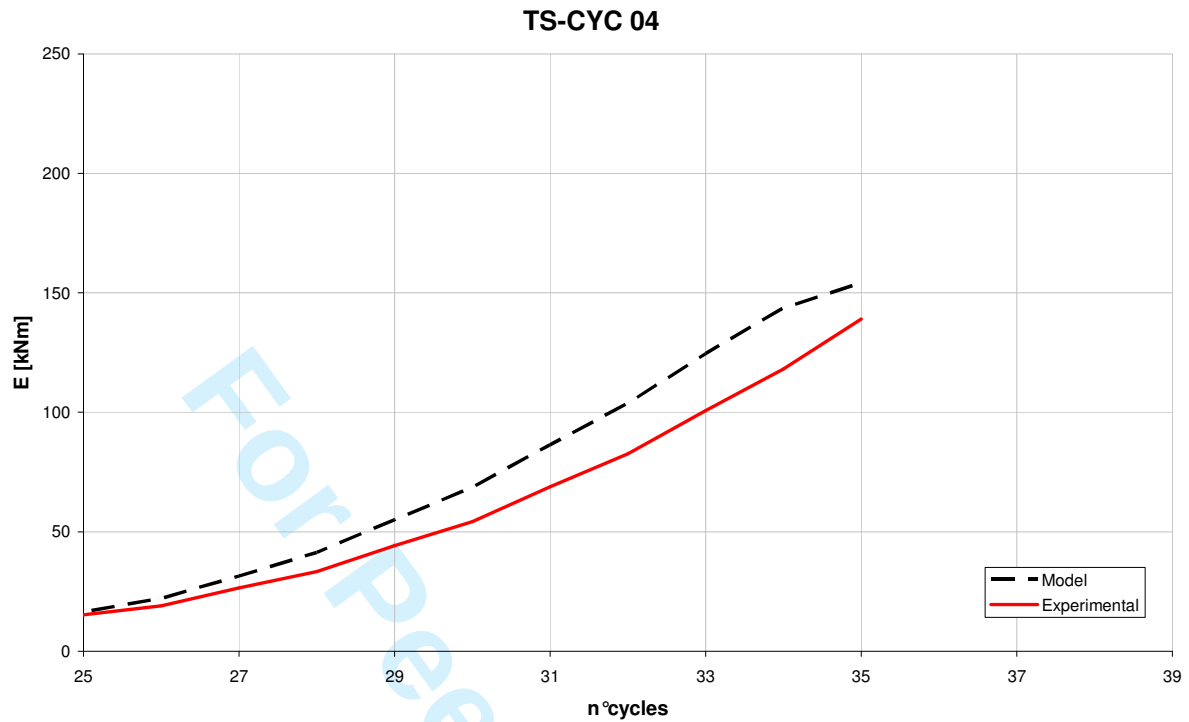


Fig. 23: Theoretical-Experimental comparison of the energy dissipated for specimen TS-CYC 04

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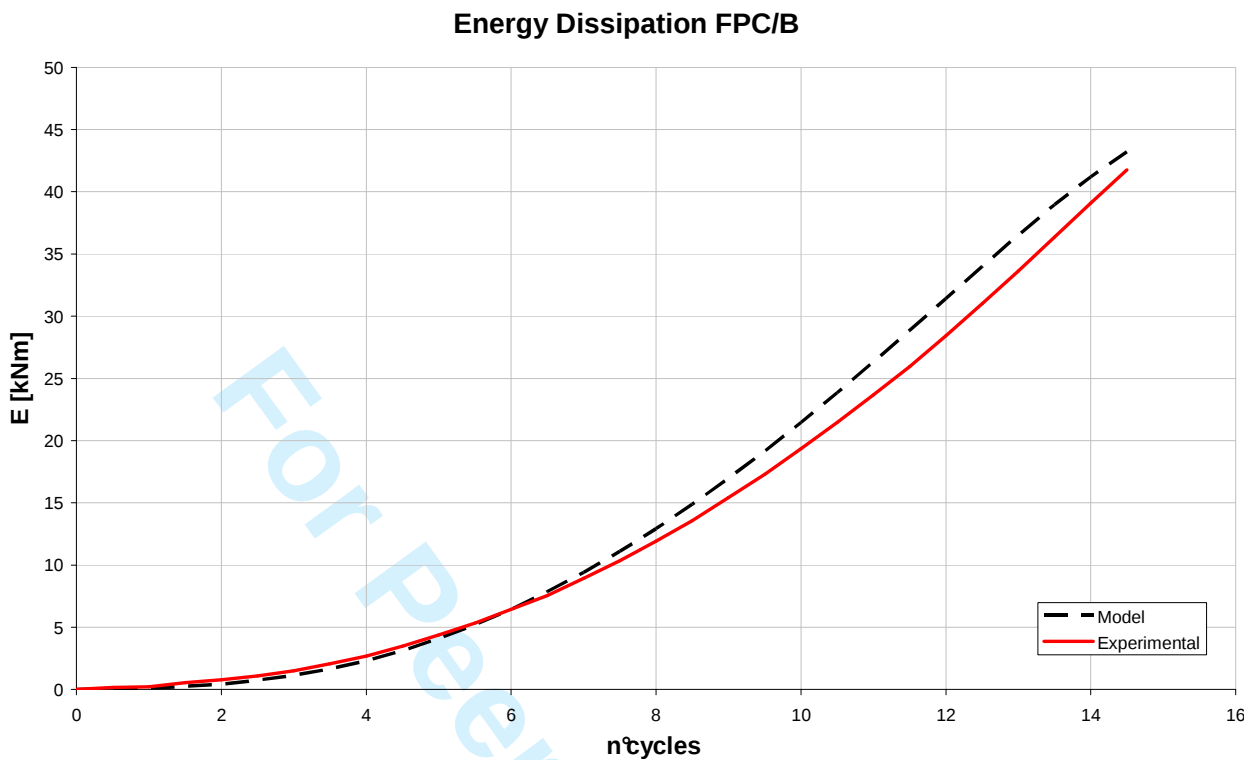


Fig. 24: Theoretical-Experimental comparison of dissipated energy for specimen FPC/B

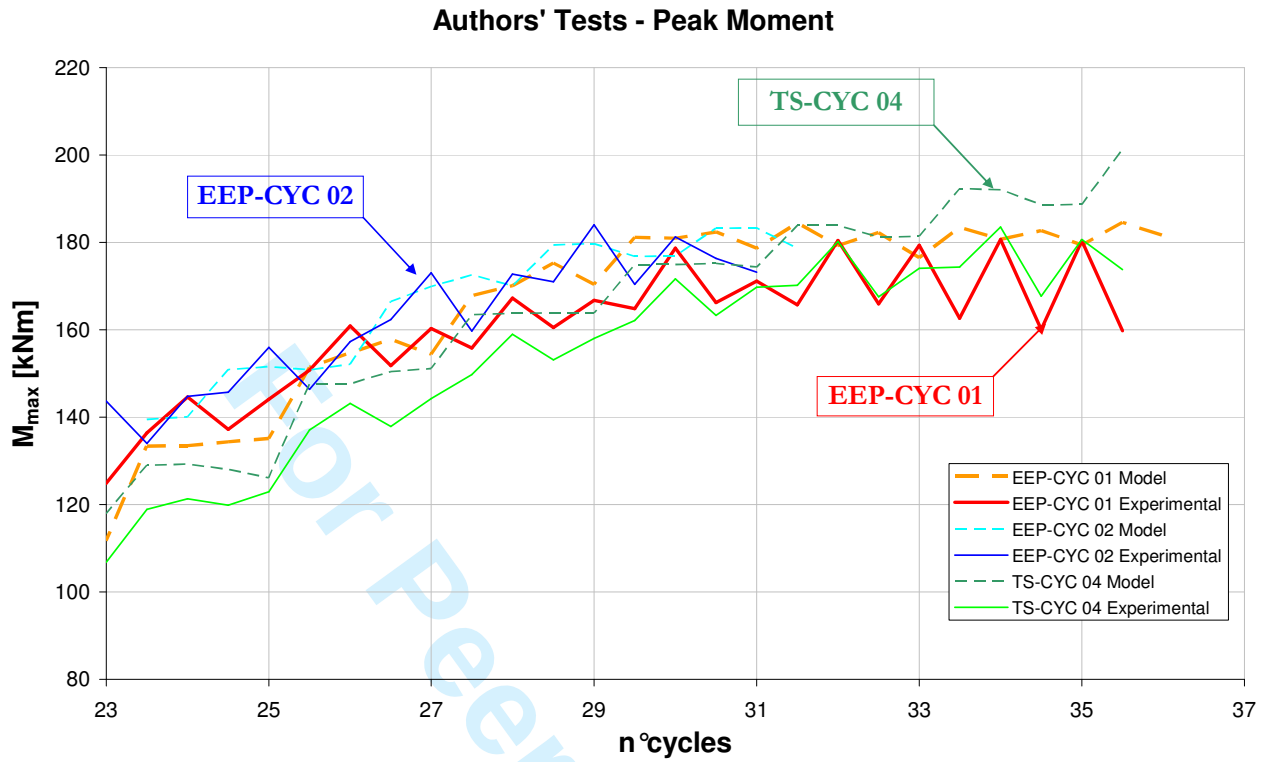


Fig. 25: Model performance in terms of Peak Moment for Authors' Tests

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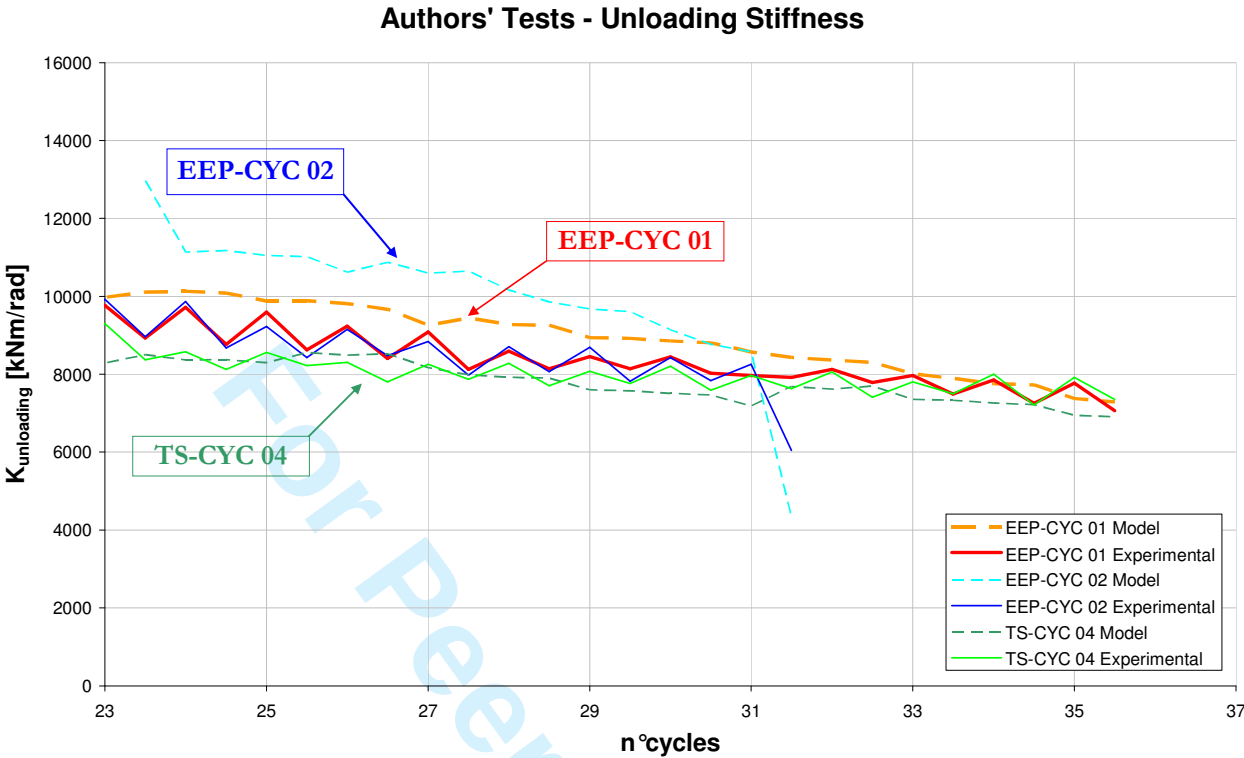


Fig. 25: Model performance in terms of Unloading Stiffness for Authors' Tests

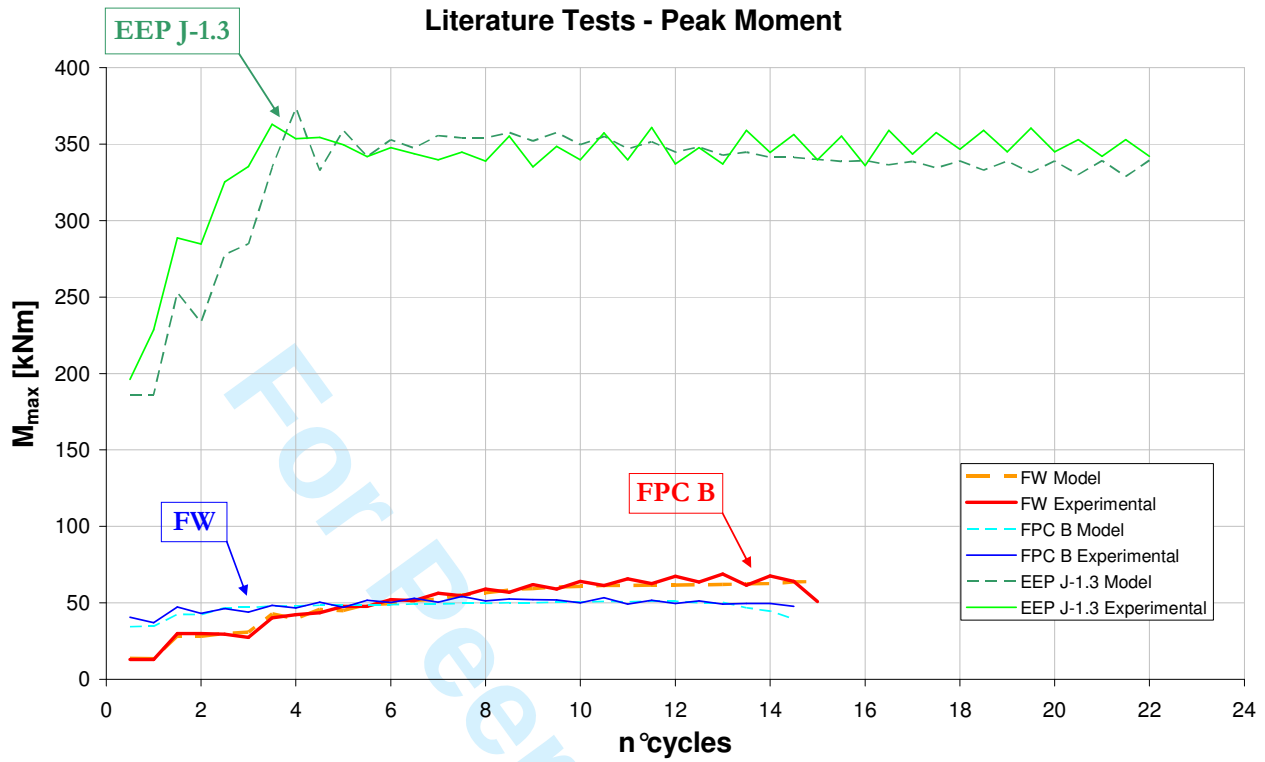


Fig. 27: Model performance in terms of Peak Moment for Literature Tests

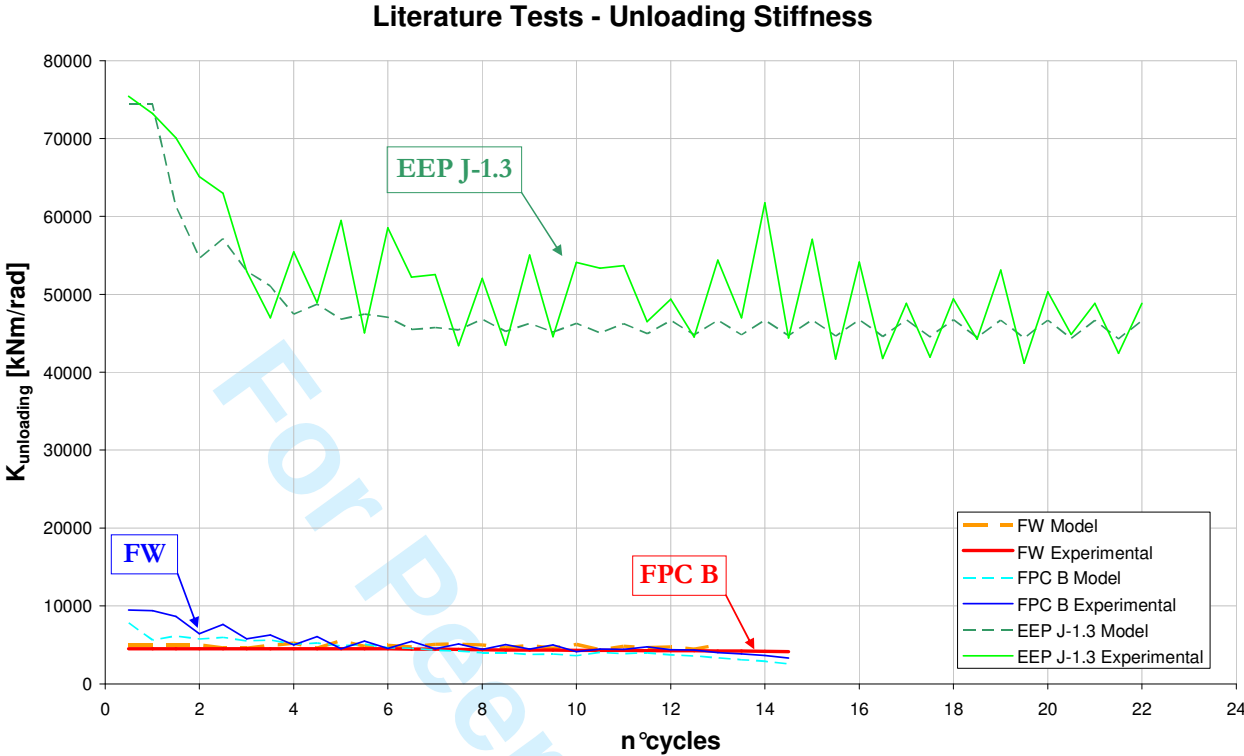


Fig. 28: Model performance in terms of unloading stiffness for Literature Tests

Table 1. Empirical parameters for definition of stiffness and strength degradation laws

a_0	b_0	a_1	a_2	a_3	b_1	b_2	b_3
1.1	1.212	0.345	0.158	3.595	0.849	0.053	0.137

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Table 2. Empirical parameters for the application of Kim and Engelhardt model

ξ	c	F_H	F_S	F_R
1.1	7	0.45	0.07	0.05

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Table 3. Empirical parameters for the application of Krawinkler et al. model

K	n	K'	n'	h	F_H	F_S	F_R
0.51	0.23	0.9	0.19	45	0.45	0.07	0.05

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Table 4. Comparison between model and experimental dissipated energy

Joint	Typology	$E_{diss,mod}$ [kNm]	$E_{diss,exp}$ [kNm]	$E_{diss,mod} / E_{diss,exp}$ [kNm]
EEP-CYC 01 (Iannone et al., 2010)	Extended end-plate	210	197	1.07
J-1.3 (Kim & Yang, 2006)	Extended end-plate	299	292	1.02
FW (Nogueiro et al., 2006)	Fully welded	60	64	0.94
EEP-CYC 02 (Iannone et al., 2010)	Extended end-plate	67	58	1.15
TS-CYC 04 (Iannone et al., 2010)	Extended end-plate	154	139	1.10
FPC/B (Bernuzzi et al., 1996)	Flush end-plate	43	42	1.04