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THE "MEAKADO" PROJECT: DESIGNING STEEL AND COMPOSITE STRUCTURES FOR OPTIMIZED PERFORMANCES IN MODERATE EARTHQUAKE AREAS

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Abstract. *The purpose of the present contribution is to summarize the current state of the developments carried out within the running research project MEAKADO, the final aim of which is to develop specific design procedures for steel and steel-concrete composite structures in regions characterized by a low to moderate seismic activity, with an appropriate reliability level. The intention is to find an optimal balance between safety and economical concerns.*

1 INTRODUCTION

Even in the most advanced seismic design methods like performance-based design, the general philosophy is always based on the assumption of global and fully developed plastic mechanisms whatever the seismicity level, together with the use of corresponding capacity design principles. The strict application of these principles for designing steel and steel-concrete composite structures in regions of low to moderate seismicity is however clearly leading to solutions that are on one hand rather difficult to implement and on the other hand often resulting in a significant increase of the building costs. As consequence, for economy reasons, it is often decided to design on the base of q=1,5 only (DCL design) and to neglect any further provisions aiming at enhancing the seismic performance, a practice which, from a safety point of view, cannot always lead to satisfactory structural solutions.

The aim of the research program Meakado is therefore to study design options with requirements proportioned to the actual seismic context of constructions in areas characterized by a low or moderate seismic hazard, contrary to most researches aiming at maximizing the seismic performances. More precisely the objective is to propose design rules that are optimised for the actual seismic action, providing the necessary safety level without imposing excessive requirements, and thus limiting the incremental complexity and costs associated with anti-seismic design.

The research Meakado has chosen to focus essentially on concentrically braced frames (CBF) and moment resisting frames (MRF), as being the most relevant typologies in the European construction market. Frames with dissipative eccentric bracing or other anti-seismic configurations (damping devices, isolators,…) are of scarce relevance for low-to-medium seismicity areas.

Having these fundamentals in mind, an extensive literature review [1] actually shows out that, by studying in detail the results and achievements of previous works related to seismic performance of steel and composite structures, three types of research can be identified:

1. Fundamental research aiming at the improvement of background knowledge on material, members and structures behavior in cyclic/seismic conditions. The main outcomes are models and relations allowing an improved prediction of the seismic performance in detail and globally;

2. Experimental and numerical studies on various structural typologies aiming at the verification and validation of behavior factors. The focus is set on the optimization of structures targeting the highest achievable behavior factors;

3. Investigation of the seismic behavior of particular products or application fields (e.g. cold formed profiles, sheeting, low rise residential or commercial buildings). The outcomes are performance characteristics evaluated having in mind the objective of maximizing seismic resistance for each particular solutions.

In practice, the present proposal is targeting three main structural configurations, identified as the most common in regions where earthquake action is not the parameter governing the design, namely steel and composite moment resisting frames and concentrically braced frames. Therefore, a number of the above references can serve as initial input for the present proposal, since providing information on structural elements or structural solutions of interest for the Meakado context.

It is however evidenced from the research projects referenced above and from most publications in the domain that research in earthquake engineering is essentially carried out for applications in regions exhibiting a very significant seismicity level, with the double aim of preventing brittle collapses, which remains certainly relevant for the Meakado context, and of at the same time maximizing the capacities in terms of energy dissipation. Many outcomes of these researches have been included in the most recent seismic design codes following the concept of q-factors related to the highest achievable ductility.

These observations evidence the need for a specific research line dedicated to optimizing seismic design rules for low to moderate active seismic areas.

2 GENERAL METHODOLOGY

As said above, the objective of the proposed research action is to study an intermediate way of design in which reduced but controlled amount of ductility is accounted for, providing thus the necessary safety with respect to uncertainties on the seismic action, but where the local ductility and structural homogeneity requirements are less stringent than for DCM in order to focus on intermediate values of behavior factors. These requirements should be tuned according to the actual seismicity level of the area. Two main directions of investigation are to be followed to target the announced objectives.

The first one consists in the exploitation of phenomena that are known to contribute to energy dissipation in steel structures subjected to earthquake, but whose knowledge is not yet sufficient to quantify them as sources of controlled energy dissipation in the definition of the q-factors. So phenomena like

- Slip in bolted connections,
- Plastic ovalization of bolt holes,
- Post-buckling strength of diagonals in compression,
- Energy dissipation capacity in beams with class 3 and 4 cross-sections

will be considered in the research and investigated either by means of experimental tests or numerical simulations in order to quantify the energy dissipation that they can provide and to adjust consequently the values of corresponding q factors.

The second direction consists in an investigation on the possibilities of tuning EC8 design rules given for DCM to the actual seismicity zone and to the targeted behavior factor. The research will focus on moment resisting frames and concentrically braced frames, as being the most common configurations in practice.

The following DCM-EC8 design rules are planned to be re-considered, mainly on the base of numerical simulation tools calibrated and validated by the experimental campaigns.

- For DCM, if q is higher than 2, currently only class 1 and 2 profiles may be used in the dissipative zones of MRF. The possible use of class 3 (and even class 4) could be considered if the ductility demand is limited and an appropriate resistance level considering post-critical behaviour is considered.

- Rotation capacity of moment connections in MRF must currently be higher than 25 mrad for DCM. Smaller values could be allowed in case of limited overall ductility demand.

- In MRF, the ratio of the sum of column resistances to the sum of beam resistances at any structural node must always be higher than 1.3. This criterion will be reconsidered either regarding the value of the limit or the number and location of nodes where it has to be fulfilled.

- For braced structures, slenderness of diagonals must remain within a limited interval, limiting consequently the type of profiles that can be used for seismic bracings. Boundaries of this interval will be reconsidered. Another possibility could be to impose these restrictions only on a limited number of storeys.

- The overstrength coefficient (ratio between seismic demand and cross-section resistance) of diagonal bracings must not vary of more than 25% all over the height of the entire building. The limit of 25% as well as the number of storeys on which the variation has to be limited will be reconsidered.

- Horizontal displacements of the structure under earthquake action must be limited in such a way that the so-called sensitivity coefficient $\theta = P \cdot dr / V \cdot h$ (P-Δ-effects) remains smaller than 0.2 (or 0.3 if non-linear analysis is used). For MRF, this limitation is generally severe; for low seismic actions V (which is the denominator) the condition becomes even much more rigorous. In the case of limited ductility demand, higher values could be allowed in particular when major part of the structure remain elastic which under dynamic action means an immediate reversing of the deformations. It will be investigated if the limitation of the sensitivity factor is really relevant for the collapse limit state.

All the above parameters are planned to be studied first separately then in combination.

The main expected deliverable is a set of recommendations that could be included in the next revision of Eurocode 8 and that would allow better tuned design of steel structures in low-to-moderate seismicity areas, aiming at ensuring the adequate reliability level, improving the economic competitiveness and simplifying the design practice where possible.

The research work carried out within the proposed project is intended to focus exclusively on the two main structural types for ordinary steel structures, i.e. concentrically braced frames (CBF) and moment resisting frame (MRF). A very special focus will be put on CBF as being the very most commonly realized in practice (80 to 90% of the steel structures really built are laterally braced by CBF at least in one direction) and as the most constrained by current Eurocode 8 rules due to the combined requirements on diagonal truss bars slenderness and homogeneity of overstrength all over the height of the structure.

The objective of the present paper is to give a broad overview of the current state of the developments carried out in the frame of this research program. Detailed information can be found in the latest versions of the progress report of the project.

3 ASSESSMENT OF ENERGY DISSIPATION IN BOLTED SHEAR CONNECTIONS

One of the most generally used systems employed to resist an earthquake is the concentrically braced frame (CBF), done by X, N or even V systems, as described in EC8. The horizontal forces induced by inertia when the base motion occurs are absorbed by means of diagonal bracing elements that works in tension and compression, depending on the sense of the load at each time. This cyclic loading could induce local plasticity in connection regions, inducing fatigue but also dissipating energy by hysteresis. The control of this hysteresis behavior is likely to enhance the energy dissipation in a building in a way allowing taking it into account from the very beginning in design stage, letting the structural designer to do a safe relaxation of structural requirements and avoiding unnecessary waste of steel resources.

Considering this, a first Task is defined in the Meakado project to test some of the most typical CBF bracing connections trying to understand the variables conditioning their energy absorption under cyclic loads. To this respect, five very relevant parameters have been identified, whose participation in hysteretic behavior could be foreseen.

- The first parameter is the use of prestressed bolts to execute the connection, this enables energy dissipation by friction, thus, in principle, prestressed bolt connections should improve the connection performance regarding energy dissipation, but the quantification of this effect remains unsolved.

- The second relevant parameter is the geometry of the bolt connection, the amount of bolts, the bolt-rows, the metric, the distances to edges and spacing between bolts etc. Initially, this should have influence in the development of local plasticity that leads to energy absorption.

- The third relevant parameter is the gusset thickness, the more thickness, the more rigidity in relation to profile, the less energy absorption developed in the gusset by plastic deformation

- The fourth being the steel profile. Typically both channels and angles are used in practice for CBF bracing (i.e. UPN and LPN profiles). The profile type will influence in the maximum allowable load, the amount of bolts and the web/flange thicknesses, etc.

- The last relevant parameter to be assessed is the material properties, whose variations depend on the hot rolling process producing changes in the yield and maximum resistance of profiles changing its dissipative behavior. The more ductility and plasticity margin, the better energy absorption.

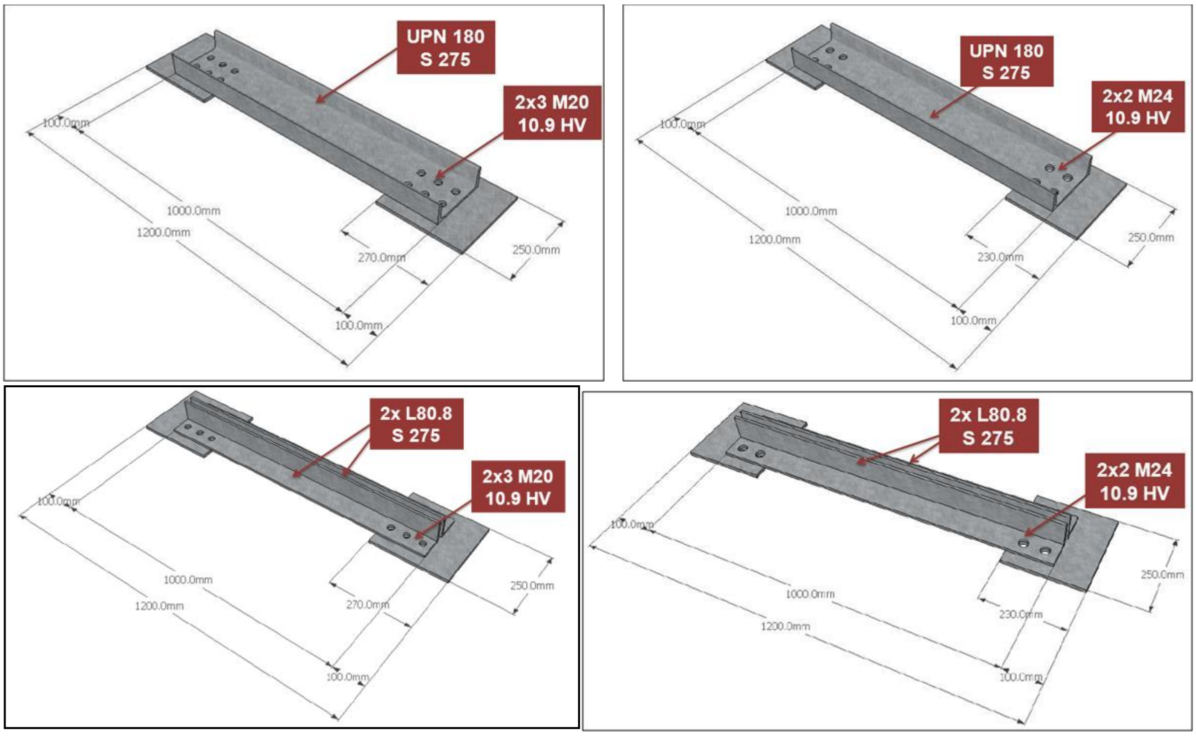

Figure 1: tested specimens.

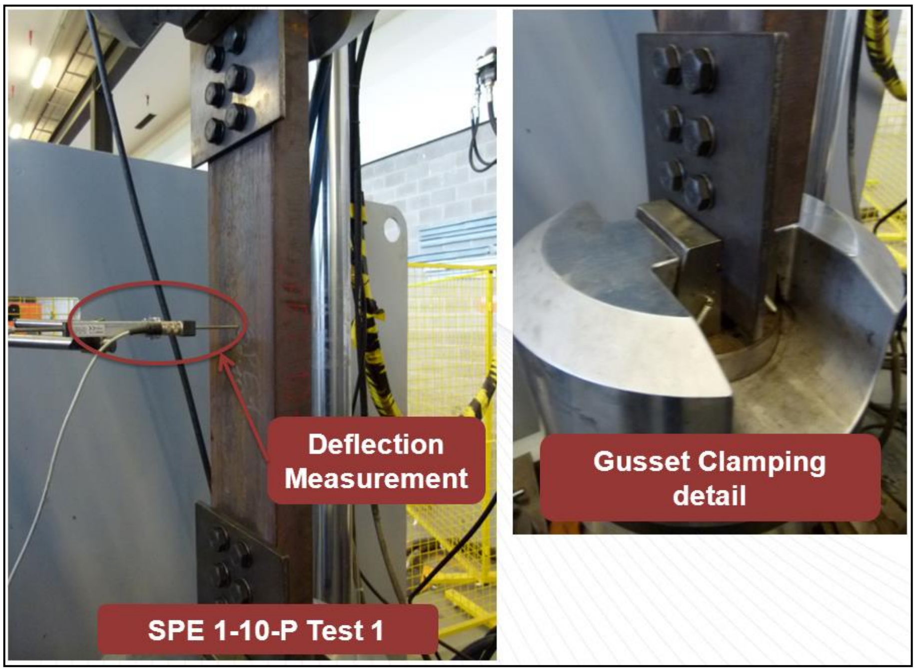

Figure 2: test setup.

An interesting first observation in non prestressed specimens is illustrated in Figure 3 (left) with monotonic tests of SPE1-10-NPT. The two graphs exhibit an initial region with high displacements at constant low force associated with the adjustment of all bolts in their holes until their complete load-share. As second interesting observation, the curve in tension reaches a maximum force of about 700 kN while in compression it barely reaches 500 kN because of the buckling in compression. It is noteworthy that, during monotonic tests, failure comes from the hole ovalization and then block shearing failure of profile web, while in compression the failure comes from the permanent buckling of the profile, not of the connection. In comparison, monotonic tests carried out on prestressed connections, such as illustrated in Figure 3 (right), show out a well fit from the very beginning, with two clear force drops in force-displacement curves corresponding to the points where bolt slip occurred. The ultimate force is approximately the same as for the non-prestressed configuration, but without initial displacement zone without force augmentation. Such a region is, instead, delayed to the point where bolt slip happens and is extended until the point where the specimen recovers its bearing capacity. Once again, the failure of the element in tension occurs by block shearing (Figure 4) and the failure in compression by unstable buckling.

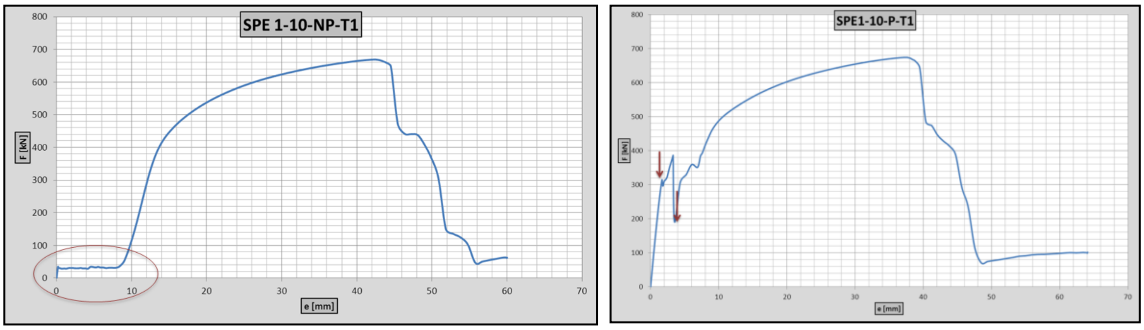

Figure 3: Illustrative monotonic curves (comparison prestressed / non-prestressed connection).

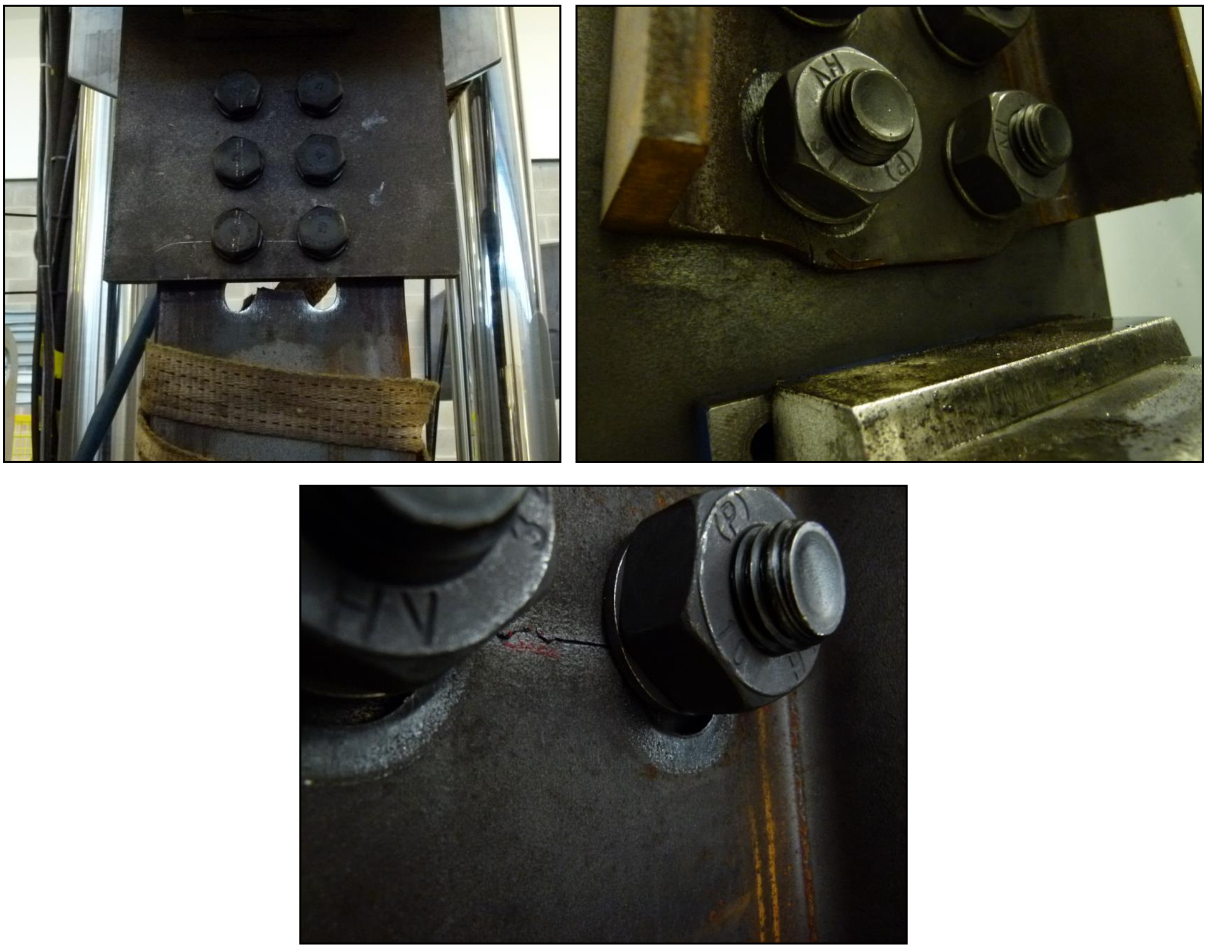

Figure 4: Typical failure mode (Channel connection in tension).

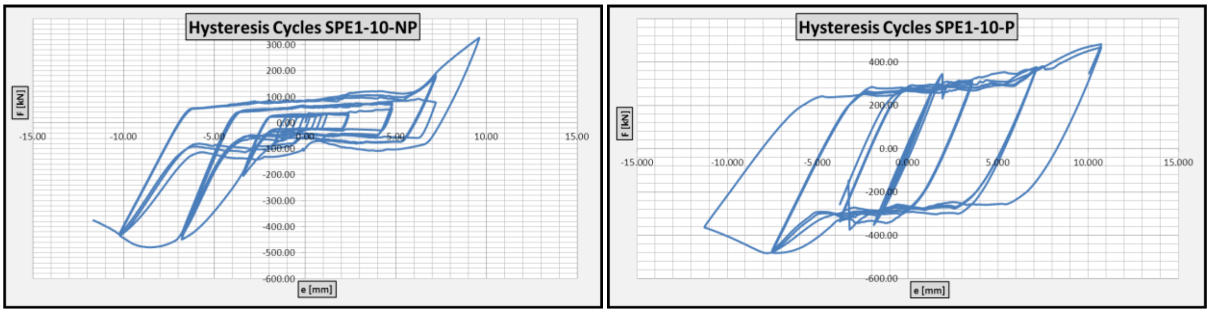

Figure 5: Typical cyclic curve (comparison prestressed / non-prestressed connection).

At the time of finalizing the present paper, all tests have been carried out but the test results are still being processed in order to compare the different configurations and evaluate and quantify the influence of the different parameters both on the monotonic and cyclic behavior of shear-bolted connections.

Moreover, beside these experimental testing activities, specific numerical models have also been implemented to allow the extension of the scope of the test results. Numerical simulations and test results show out to be in reasonably good agreement, as illustrated in Figures 6 and 7.

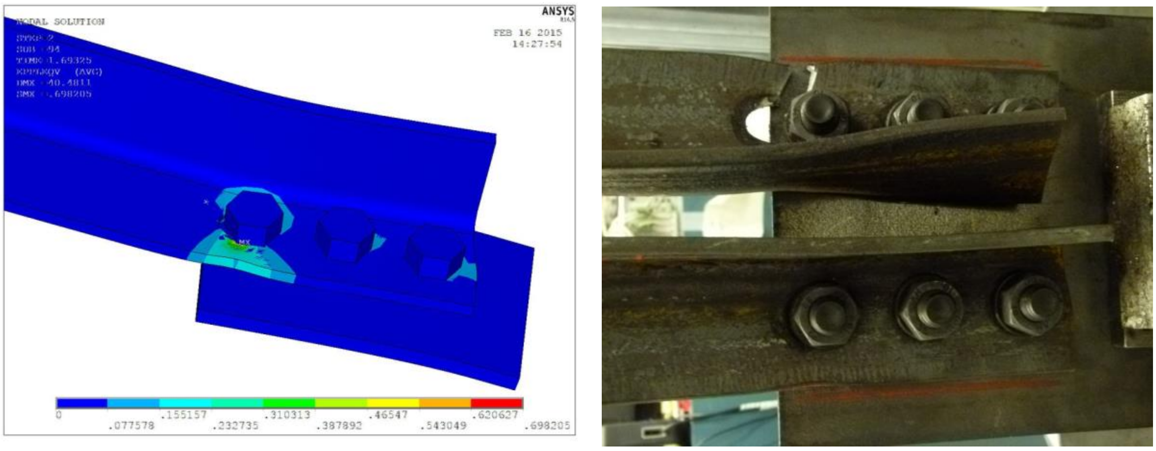

Figure 6: Failure mode of angle specimens in tension.

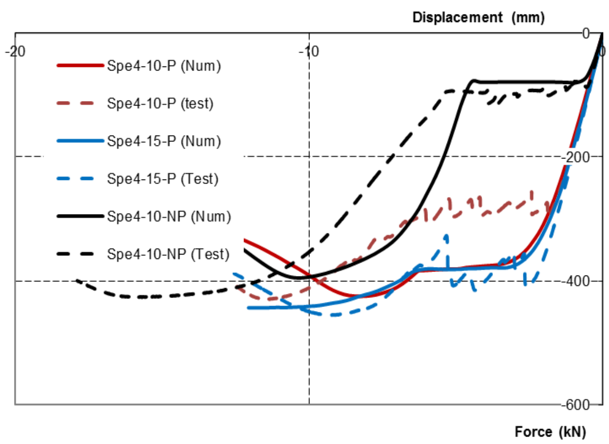

Figure 7: Comparison of numerical (block lines) and experimental (dash line) results.

4 DESIGN AND PERFORMANCES OF CBF WITH RELAXED DESIGN CRITERIA

When considering a CBF structure (concentrically braced frame) designed according to ductility classes DCM or DCH of EN 1998-1, stringent requirements are to be considered regarding the bracing slenderness, the overstrength homogeneity and the overstrength of the joints.

A first step in a global evaluation procedure consists in designing a set of case studies according to these requirements. In order to be consistent with the objectives of the Meakado project, these case studies are evaluated considering a low to medium seismicity. 15 configurations have been designed for 3 structural typologies (Figure 8) and a number or levels ranging from 4 to 12.

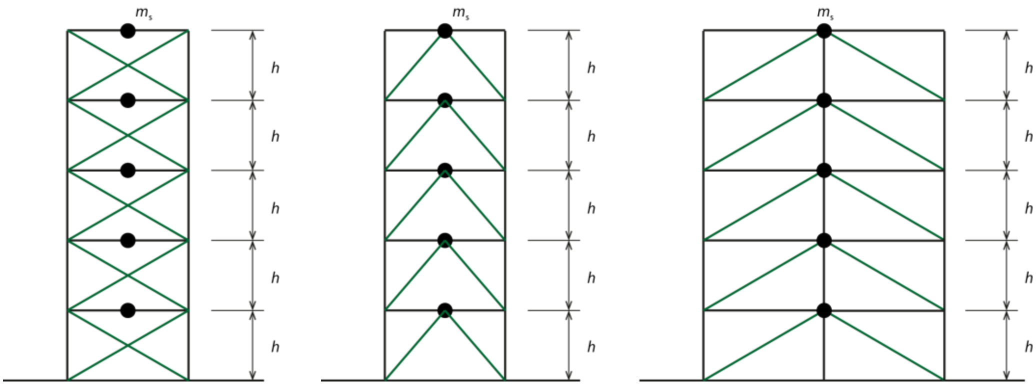

Figure 8: Structural typologies.

After selection of a reduced set of configurations, for 4 and 8 levels, the steel structures have been re-designed considering successively the variation of following parameters (as summarized in Figure 9):

- ***peak ground acceleration*** of 0.25 g and 0.35 g;
- ***relaxed overstrength criterion for 0.15 g***, with $\Omega_i \leq 1{,}50\ \Omega_{max}$ (instead of 1.25 Ω_{max});
- ***relaxed slenderness criterion for 0.15 g***, with $\bar{\lambda} \leq 2{,}5$ (instead of 2);
- ***simultaneous reduction on criteria for overstrength and slenderness***.

It can then be showed that relaxation of the EC8 rules permits the adoption of lighter profiles, more in the case of relaxing slenderness than overstrength homogeneity rule.

EN1998-1 rules	$\Omega_{max} < \mathbf{1{,}25}\ \Omega_{min}$ $\mathbf{1{,}3} < \lambda < \mathbf{2}$
Relaxed slenderness (**λ**) **rule**	$\Omega_{max} < \mathbf{1{,}25}\ \Omega_{min}$ $\mathbf{1{,}3} < \lambda < \mathbf{2{,}5}$
Relaxed overstrength homogeneity (**Ω**) rule	$\Omega_{max} < \mathbf{1{,}50}\ \Omega_{min}$ $\mathbf{1{,}3} < \lambda < \mathbf{2}$

Figure 9: Relaxed design parameters.

Incremental dynamic time-history analyses are then used to assess the performances of the structures designed strictly to Eurocode 8 or with relaxed criteria. Figures 10 and 11 illustrate results of such analyses in terms of maximum base shear and of distribution of the yielding within the brace elements.

Based on results considering a limited number of configurations and a limited number of input time-histories, it appears that even ignoring completely the code limits on slenderness and overstrength does not cause the occurrence of soft-storey mechanism and provide a ductility of the structure sufficient to resist the targeted acceleration level with a sufficient safety margin. The conclusions need however to be further elaborated in a broader range of parameters.

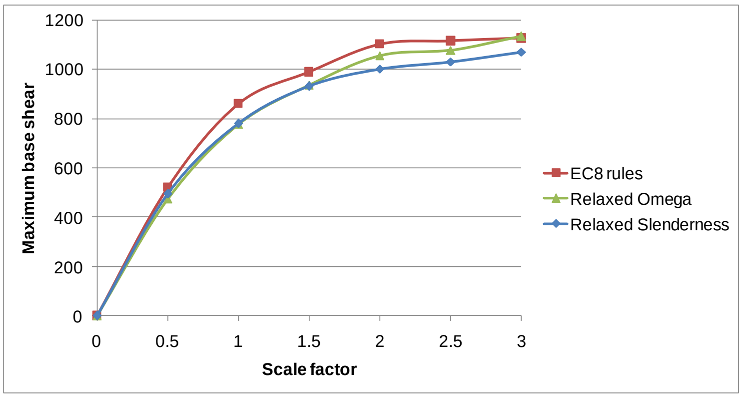

Figure 10: Example of Incremental Dynamic Analyses curves for a 4-storey N-configuration.

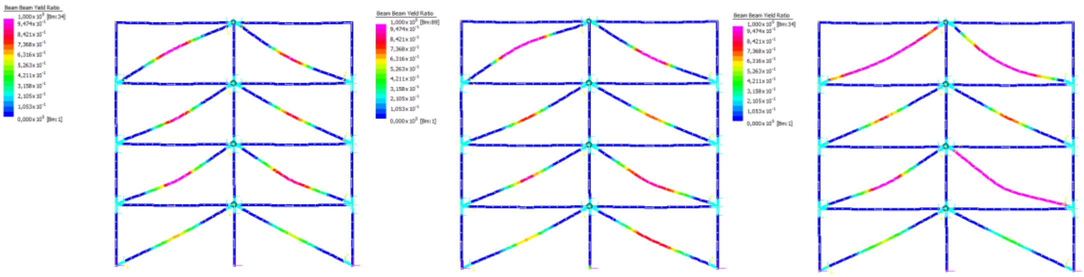

Figure 11: Distribution of yielding for a 4-storey N-configuration. Left = EC8 design; Middle = relaxed slenderness; Right = relaxed overstrength homogeneity

5 CYCLIC BEHAVIOUR OF CLASS 3 AND 4 STEEL CROSS-SECTIONS

A specific test program has been carried out in order to evaluate the dissipative capacity of a set of class-4 built-up steel sections.

The test program comprises in total 6 cyclic tests. It has been decided to test only frame corners as subsystems of a complete MRF and due to the test setup the length of the beams and columns were limited. All specimens were made as class 3 cross sections and in order to reach plastic hinges in the beams and columns the panel zones were reinforced. One additional test on a frame corner without reinforcement and one additional test on a frame corner made of class 1 cross sections with comparable monotonic resistance and stiffness were carried out. The 6 different specimens that were finally decided on were:

- S1 - Welded frame corner with constant girder
- S2 - Bolted frame corner with constant girder
- S3 - Welded frame corner with haunched girder
- S4 - Bolted frame corner with haunched girder
- S5 - Welded frame corner with constant girder - class 1

- S6 - Welded frame corner with constant girder - unreinforced panel zone

Figure 12 shows the above mentioned specimens or types of frame corners respectively. The main dimensions (e.g. length of column and beam, cross-sectional height and width) were equal for all specimens. The boundary conditions of the test setup were designed to represent the cut-out from a complete MRF; the test setup is shown in Figure 13.

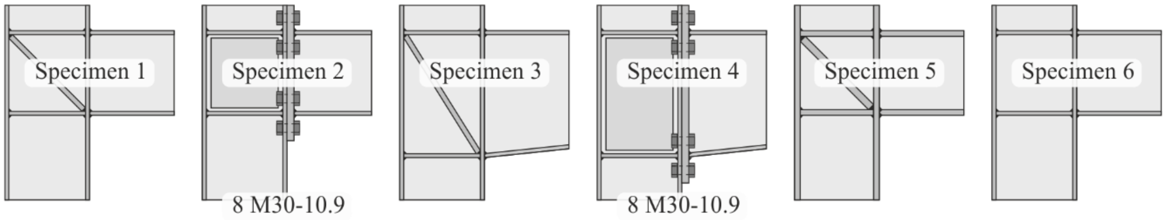

Figure 12: Overview of test specimens – types of frame corners.

Figure 13: Test set-up with test specimen S2.

In parallel with the physical testing, numerical simulations using shell FE models were also implemented with the aim of:

- Validating of the numerical model with regard to its accordance with the experimental behaviour and thus its applicability for parametric studies;
- Identifying the damage and formulate of an equivalent damage criterion transferrable from damages observed in the tests to the numerical model in the perspective of further parameter studies.

The experimental and numerical load-displacement curves show a very good agreement confirming the capability of the FE-models to represent the overall performance of the investigated frame corners, as illustrated in Figures 14 and 15 for specimen S1. For the identification of failures (cracks) in the numerical FE-models an approach has been selected, in which the accumulated plastic strains $\overline{\varepsilon}_p$ were compared to a limit value $\overline{\varepsilon}_{p,limit}$. The limiting values have been calibrated by the test results such that for each tested specimen respectively its FE-models the location and the accumulated plastic strains have been determined from the simulations at the cyclic deformation stage of the tested specimen corresponding to the first visually detected crack. From this evaluation the number of cycles until failure and the location (point of failure) can be obtained.

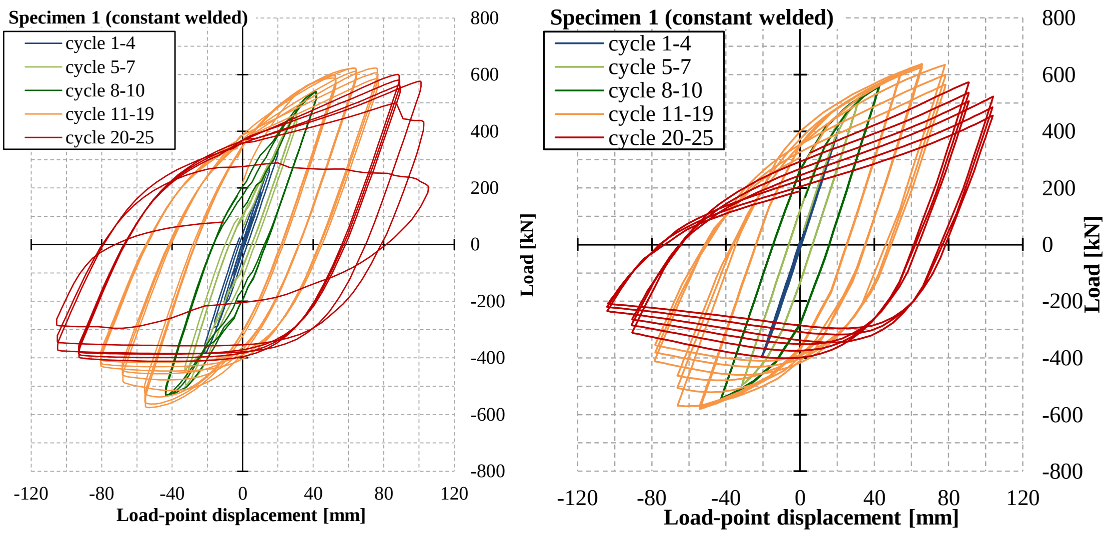

Figure 14: Cyclic curve for test S1 – Left: numerical simulation; Right: test results.

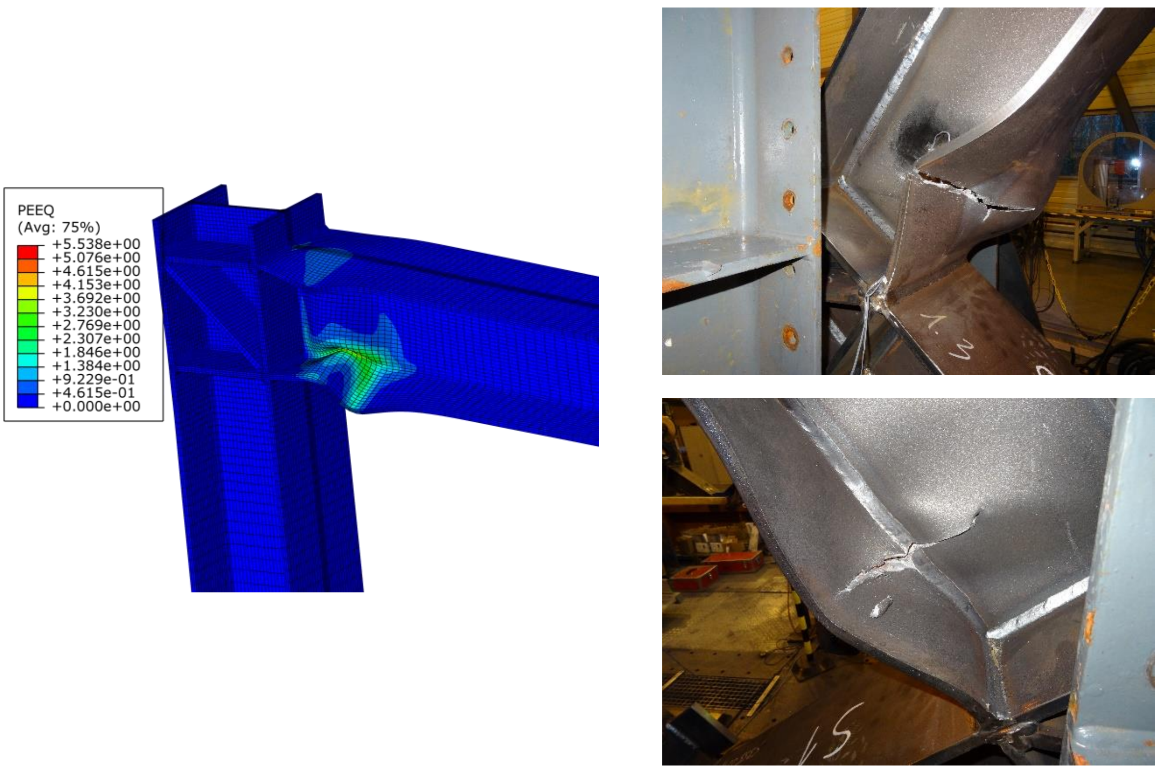

Figure 15: Failure mode of specimens S1 – Left: numerical model; Right: test observations.

The next step in the research process has to be the evaluation of the global behavior of MRF account taken for the local behavior identified from the test results. To this purpose, a set of structures has been designed according to Eurocode 8 rules [2] while a specific non-linear spring model is designed and calibrated with respect to test results, with the aim of allowing a an effective modelling of the global behavior frames. The development of the equivalent spring and the design procedure of the reference MRF structures can be found in [1].

6 CONCLUSIONS AND PERSPECTIVES

As stated in the description of its general methodology, the final objective of the Meakado research project is to study an optimized way of designing steel and composite structures in which reduced but controlled amount of ductility is accounted for (intermediate between DCL and DCM), providing thus the necessary safety with respect to uncertainties on the seismic action, but where the local ductility and structural homogeneity requirements are less stringent than for DCM. So the seismic risk can be tackled in a reasonable manner in regions with low or moderate seismic action.

The development of this optimized approach is to be based on a deeper understanding of a number of specific mechanisms. During the first part of the project, described in the present paper, specific investigations (experimental and numerical) have been carried out in the direction of such a deeper understanding:

- Quantification of the energy dissipation in classical bolted shear connections, investigated through tests and numerical simulations;
- Energy dissipation in beams with class 3 and 4 cross-sections, investigated through tests and numerical simulations;
- Post-buckling strength of diagonals in compression, investigated through literature review and complementary numerical simulations.

The following development of the project will then investigate the consequences of these identified local characteristics on the global performances of typical CBF and MRF structures in view of finally deriving practical design recommendations.

Moreover, beside a better control of the local characteristics, the project also focuses on the possibility to release some rules recommended by Eurocode 8 and felt as too stringent by most designers in low seismic country where earthquake is generally considered as a non-critical issue. To this respect, some parameter studies have been initiated and can already show out interesting trends. These investigations will continue all along the second part of the project and be complemented by validation tests, also in view of finally deriving practical design recommendations.

ACKNOWLEDGEMENTS

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