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DESIGNING STEEL AND COMPOSITE STRUCTURES IN A LOW-TO-MODERATE SEISMIC AREA

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Abstract. *This contribution summarizes the results achieved in the frame of the research project MEAKADO funded by the European Commission. The goal of this project is to develop specific design procedures for steel and steel-concrete composite structures in regions characterised by a low to moderate seismic activity, aiming at an appropriate balance between safety level and economical concerns. The research program focuses on moment-resisting and concentrically braced frames. The targeted design methodology consists in providing the structures with limited but controlled ductility through structural requirements that are less stringent than designing according to Eurocode 8 – Ductility Class Medium principle, focusing on intermediate values of the behaviour factor.*

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 [2]. The strict application of these principles for designing steel and steel-concrete composite structures in regions of low to moderate seismicity [3] is however clearly leading to solutions that are on the one hand rather unconventional for countries that are not used to seismic design, and thus difficult to implement in the daily practice, and on the other hand often resulting in a significant increase of the building costs. As a consequence, for economy reasons, it is often decided to design on the base of a behaviour factor q equal to 1,5 only (DCL design) and to neglect any further provisions aiming at enhancing the seismic performances, a practice which, from a safety point of view, cannot always lead to satisfactory structural solutions (i.e. no control is done on the hierarchy of failure modes, making the always structure likely to fail in a brittle way).

The aim of the research program *Meakado* [1] 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 indeed of limited relevance for low-to-medium seismicity areas.

Meakado investigates an intermediate way of designing steel structures in which reduced but controlled amount of ductility is accounted for, providing the necessary safety with respect to uncertainties on the seismic action, but where the local ductility and structural homogeneity requirements are less stringent than required for instance by Eurocode 8 for a Ductility Class Medium (DCM) design. It targets on intermediate values of behaviour factors, typically 2 to 3, that are often sufficient to bring the horizontal seismic forces down to the level of horizontal wind loads. The specific requirements are thus clearly meant to be tuned according to the actual seismicity level of the area.

Two main directions of investigation are followed to reach these objectives. The first one consists in the exploitation of phenomena that are known to contribute to energy dissipation in steel structures subjected to earthquake action, 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;

are taken into consideration 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 some EC8 design rules given for DCM to the actual seismicity level and to the targeted behaviour factor. The following DCM-EC8 design rules are for instance being reconsidered, mainly on the base of numerical simulation tools calibrated and validated against some specifically dedicated experimental campaigns:

- Slenderness of diagonal bracing in CBF's must remain within a given limited interval, limiting consequently the flexibility in terms of profiles that can be used for seismic bracings. Boundaries of this interval are hereby reconsidered. The possibility to impose these restrictions only on a limited number of storeys is also investigated.
- Except for low-rise buildings, the overstrength coefficient Ω (i.e. ratio between seismic demand and cross-section resistance) of diagonal bracings must not vary with more than 25% all over the height of the entire building. The limit of 25% as well as the number of stories on which the variation has to be limited is reconsidered as well.

The main expected outcome of this research program is a set of recommendations that could hopefully be included in the upcoming revision of Eurocode 8 and that would allow a 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 present contribution summarizes the main findings of the research program, to be officially ended in December 2016. For more details, the interested readers are invited to refer to the available detailed research reports and associated journal and conference publications.

2 CONCENTRICALLY BRACED FRAMES

2.1 Characterisation of shear connections

In case of seismically resistant CBF structures, the horizontal inertial forces induced by the earthquake are resisted by the diagonal bracing elements working in tension or compression depending on the direction of the action. In a conventional DCM design, the connections of these bars are designed

according to the capacity design principle in such a way that they remain perfectly undamaged until the yielding of the tension diagonal [4]. However, an alternative design procedure could consider some permanent deformation of the connection itself, inducing a risk of fatigue failure but also a possibility to dissipate a certain amount of energy through a hysteretic behaviour. The control of this hysteretic behaviour is likely to enhance the energy dissipation in a building in a way allowing taking it into account from the very beginning of the design process, giving the possibility to the designer of relaxing an unnecessary and costly overdesign of the connections.

Based on this reasoning, a first task of the Meakado project consisted in characterizing typical shear connections of CBF bracings in view of identifying the parameters conditioning their energy absorption capacity under cyclic loading. To this respect, experimental tests have been carried out considering a variation of the following parameters:

- Prestressing or not of the bolts;
- Geometry of the bolted connection (number, size and position of the bolts);
- Gusset thickness;
- Cross-section of the brace.

Figures 1 and 2 illustrate respectively the test setup for a channel cross-section and a comparison of typical cyclic curves for non-prestressed and prestressed connections.

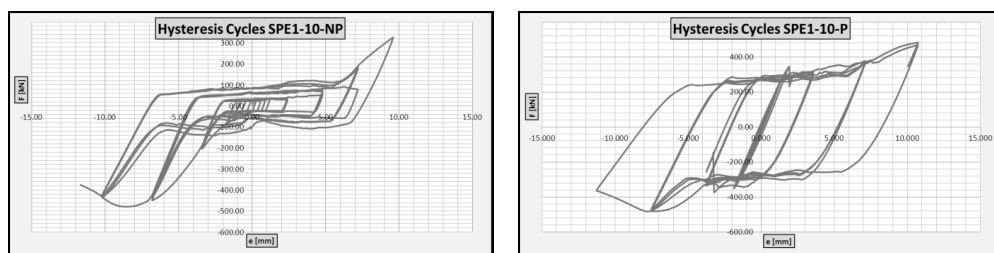Figure 1. Test setup and example of failure mode.

Figure 2. Cyclic curve – typical comparison of non-prestressed and prestressed connections.

Experimental results have been obtained on channel and angle cross-sections then extended with duly calibrated advanced numerical models (using software package ANSYS) to double-channels and circular hollow sections with different gusset configurations. The detailed results are available in [1]. These results are being processed first to calibrate equivalent springs to be used in global structural models as well as to sort between suitable and less suitable configurations (see also § 2.4 of the present contribution for more details about these conclusions).

2.2 Design according to Eurocode 8 with relaxed rules

When considering a CBF structure designed according to DCM or DCH principle of EN 1998-1 [2], stringent requirements have to be met regarding bracing slenderness, homogeneity of the overstrength and overstrength of the joints.

In order to assess this procedure and in particular to investigate a possible relaxation of the slenderness and homogeneity requirements, a first step consisted in designing a set of case studies according to a strict application of Eurocode 8 for a moderate level of the seismic action (ground acceleration a_g equal to 0,15g, EC8 type 2 spectrum and ground type B). 15 configurations are designed corresponding to 3 structural typologies (figures 3.a to 3.c) and a number of levels ranging from 4 to 12, using a behaviour factor q of 3 for configurations (a) and (c) and a q of 2 for configuration (b). A selection of these configurations are then redesigned based on the following variation of parameters:

- *Relaxed slenderness criteria* (λ up to 2,5 instead of 2,0 as requested by EN 1998-1);
- *Relaxed overstrength criteria* ($\Omega_i \leq 1,5\ \Omega_{max}$ instead of $\Omega_i \leq 1,25\ \Omega_{max}$);
- *Combination of relaxed slenderness and overstrength.*

For these variations, a "worst-case scenario" is implemented with the lowest overstrength at the lower level (i.e. $\Omega_1 = 1,0$ and $\Omega_i = 1,5$ for $i > 1$) and the highest slenderness at the lowest level (i.e. $\lambda_1 = 2,5$ and $\lambda_i < 2,0$ for $i > 1$) in such a way to trigger a possible soft-storey mechanism.
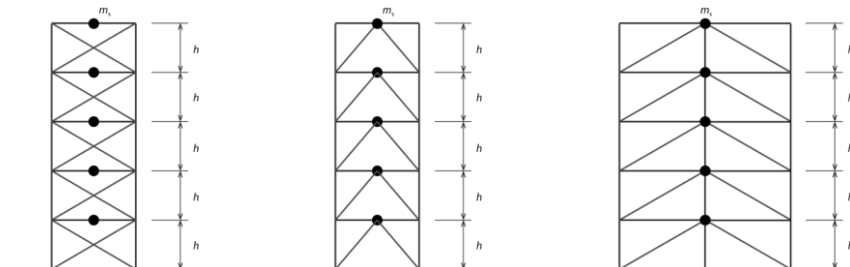

Figure 3. Structural typologies – (a) X bracings, (b) Inverted V bracings, (c) N bracings.

Numerical models duly calibrated against available results from the literature [5] are then used to evaluate the actual performances of the full set of frames. Incremental dynamic analyses are run with Eurocode 8 spectrum-compatible time-histories. Results are available in [1]. They show that, even with relaxed rules, maximum drifts obtained at each floor remain below the damage limits indicated by Eurocode 8 (see illustration in figure 4 for the 4-levels "N" configuration and one arbitrary time-history). Although the general distribution of plasticity remains generally homogeneous, some cases exhibit the initiation of a soft-storey at the ground floor but, given the moderate seismic level, corresponding drifts remain small and do not trigger global structural problems. The seismic performances have also been investigated under natural ground motion time-history record with magnitude 6 (namely Friuli 1976 aftershock), leading to similar conclusions.
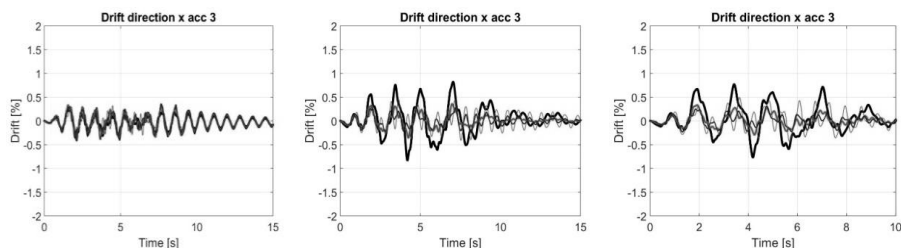

Figure 4. Comparison in terms of interstorey drifts. (a) EC8 design – (b) Relaxed Ω – (c) Relaxed Ω and λ.

2.3 Effect of energy dissipation in shear connection on the brace behaviour

2.3.1 Theoretical approach

Specific numerical investigations have been carried out to evaluate the influence of the cyclic behaviour of bolted shear connections on the global seismic behaviour of CBF's. As a first step, 6 configurations have been selected from the set described in section 2.2. For each of these configurations, 4 different joints have been designed with increasing resistance, ranging from partially resistant joint (i.e. ultimate resistance of the joint equal to 75% and 90% of the nominal resistance of the diagonal) to joints exhibiting a significant overstrength (133% and 187,5% of the nominal resistance of the diagonal). The full force-displacement curves of the connections are then defined based on the outcomes of investigations summarized in section 1.

Figure 5 shows the pushover curves obtained for a given configuration and for the four different joints, compared to a reference case in which the connections are assumed to be pinned and infinitely resistant. The dotted part of the curves corresponds to situations in which the local deformation of the joint exceeds its capacity. It can be observed that the curves for the strongest joint (J4) and the curve for the reference case are almost perfectly similar. For joint J3, a slight difference is observed. In this situation, the joint exhibits a sufficient resistance compared to the resistance of the diagonal but it doesn't remain perfectly stiff (some slippage is observed). For joints J1 and J2, the unfavourable influence of the partial resistance becomes visible.
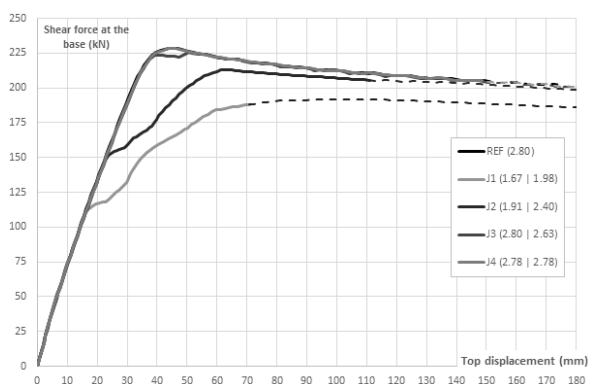

Figure 5. Influence of the joint resistance on the global pushover curves.

Table 1. Assessment of behaviour factor from IDA – mean value (standard deviation).

Configuration	Reference case	Joint J1		Joint J2	
		Cat. B	Cat. C	Cat. B	Cat. C
C1	4,98 (0,90)	2,47 (0,75)	3,92 (1,25)	2,01 (0,46)	3,34 (1,01)
C2	4,29 (0,62)	2,52 (0,59)	4,14 (1,25)	1,88 (0,69)	3,37 (0,91)
C3	3,90 (0,94)	2,21 (0,62)	3,57 (1,02)	1,95 (0,58)	3,95 (1,05)
C4	4,78 (0,85)	2,38 (0,61)	3,54 (0,70)	1,87 (0,76)	3,64 (0,83)
C5	3,72 (0,93)	2,11 (0,61)	3,32 (0,54)	1,81 (0,21)	4,39 (1,57)
C6	4,21 (0,67)	1,94 (0,52)	3,29 (1,04)	1,86 (0,26)	3,81 (1,22)

These results have been confirmed by a set of incremental dynamic analysis (see [1]) which also allows evaluating the influence of the joint resistance on the available behaviour factor. This latter is obtained as the ratio between the multiplier of the seismic action triggering the conventional failure of structure (i.e. reaching of the ultimate displacement of the connection, with an upper bound

corresponding to an interstorey drift of 2%) and the multiplier of the seismic action corresponding to the end of the elastic behaviour of the structure. Results are given in table 1 for the 6 selected configurations. As a direct consequence of Figure 5, only joints J1 and J2 leads to a reduction of the behaviour factor. A distinction is also made between two variants of joints J1 and J2, namely joints designed as category B or C according to the definition of Eurocode 3 part 1-8 (i.e. with slip resistance guaranteed until serviceability limit state – cat. B – or until ultimate limit state – cat. C).

2.3.2 Experimental findings

In order to deeper investigate and validate the results achieved in Meakado in a numerical way and presented in sections 2.2 and 2.3.1, two sets of experimental investigations have additionally been prepared regarding the global behaviour of CBF's. Low slenderness and frame effect contributions are investigated by means of full scale cyclic tests while high slenderness and relaxed homogeneity of diagonals are investigated by means of scaled shake table tests.

The cyclic test program consists of 15 full scale tests with 1 level and 1 bay (2,6 x 4,3 m) representing a single storey frame extracted from a multi-storey structure. Different types of diagonals are tested both in a frame with real beam-to-column connections and in an ideal pinned frame (see figure 6).
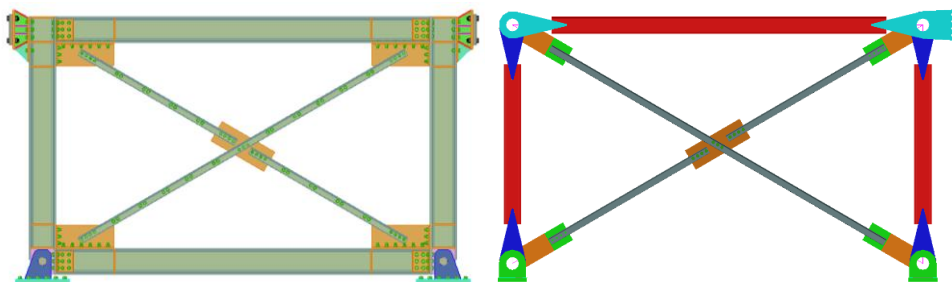

Figure 6. Cyclic tests – test frames.

The dynamic test program embraces a set of 5 structures with 1 or 3 levels aiming at investigating the impact of overstrength homogeneity and high slenderness of the diagonals, as well as deriving conclusions on the way to cope with the compression diagonal in the design. At the moment of finalizing the present contribution, all tests are done and their results are being processed to derive relevant conclusions. Preliminary results can be found in [6].

2.4 Provisional conclusions

In the previous sections, different sets of numerical analysis have been presented. First, a homogenous relaxation of overstrength and slenderness rules allowing designers to choose lighter profile sections from a larger database have been analysed through IDA of case studies designed in purpose. "Worst-case" structures have been designed, considering that the worst situation in seismic design is to have the weak brace at the first level and strong braces at all other levels. Results show that, in case of relaxed design rules, the maximum drifts reached remain below the damage limits of EC8. Slight soft storey shapes have been observed but without a global collapse risk.

An additional set of simulations has then been performed to evaluate the actual effect of the end-connections of the braces on the global behaviour of CBF's. Based on pushover analyses and IDA, the resulting behaviour factors are assessed for each configuration. Although requiring additional extensive validation, it is tentatively observed that important standard deviations are obtained. It is then proposed to retain a final value of the behaviour factor equal to the mean value minus the standard deviation, giving thus the 85%-fractile value. For the reference joint, the final behaviour factor is generally always greater than 3, at least for the configurations fulfilling the capacity design requirements (overstrength homogeneity and slenderness criteria). When weaker joints are used instead of over-strengthen

connections, the value of the behaviour factor logically decreases, but a final value around 2.5 can still be accounted for when considering the category C shear resistance of the joint as the first yielding of the system, even for the less suited configurations of CBF's.

3 MOMENT RESISTING FRAMES

3.1 Behaviour of class 3 and 4 steel and composite cross-sections

As a first step towards a better knowledge of the behaviour of moment resisting steel frames made of slender cross-sections, a specific test program has been carried out in order to evaluate the dissipative capacity of a set of built-up class-3 and class-4 steel and composite sections. The program comprises in total 6 cyclic tests on steel section and 3 on composite sections. Typical frame corners are studied, as illustrated in Figure 7 with one of the steel specimens. The parameters of the study are the type of beam-to-column connection (welded or bolted), the possible tapering of the beam and the presence or not of vertical stiffeners on the web of the beam. Both for the steel and composite specimens, a reference is obtained by testing a class-1 section having the same resistance as the class 3 and 4 specimens. Detailed results are presented in [1]. As a matter of illustration, Figure 8 shows the comparison of a cyclic curve obtained for a class-4 welded frame corner with constant girder and for a class-1 girder having the same theoretical resistance. This comparison shows that although the class-4 specimens exhibit some cyclic degradation, it is however possible to dissipate a significant amount of energy.
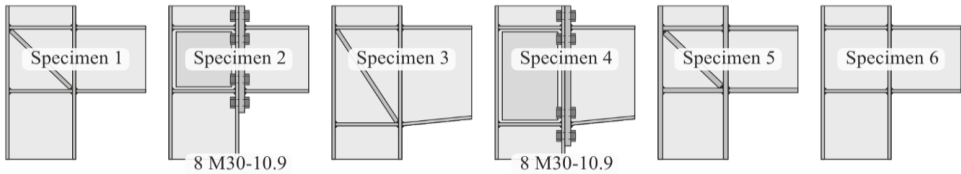

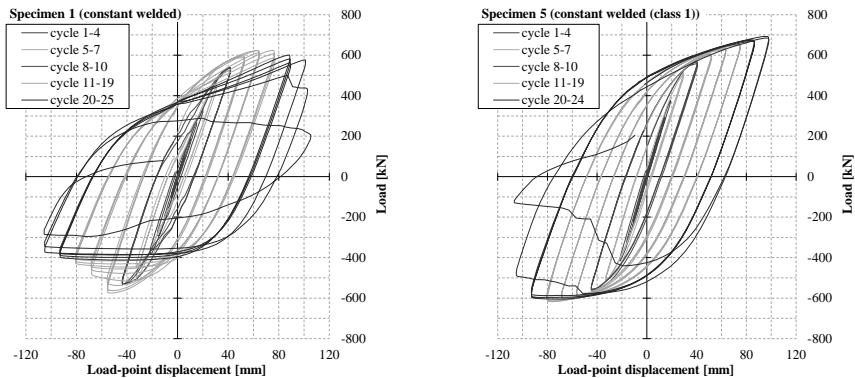Figure 7. Overview of the "steel" test specimens.

Figure 8. Cyclic tests – comparison of a class-1 (left) and class-4 (right) cross-section.

3.2 Global performances of frames

After this experimental part, a classical procedure is followed consisting in calibrating 3D FE models against the experimental results, enlarging the experimental database by additional FE calculations then

calibrating equivalent springs versus this database, in the perspective of using them in global frame models.

At the same time, a set of MRF structures are designed according to EN 1998-1 with requirement on the profiles to choose expressed in terms of bending modulus W and second moment of area I. Solutions are then implemented by designing build-up class-3 or 4 sections matching the required W and I. Systematic evaluation of these structures is then performed via IDA resorting to frame models with connections modelled as springs having the properties calibrated as described above. Detailed evaluations are provided in [1]. The first main observation is that, due to the relatively high influence of the gravity loads with respect to the seismic load associated to the moderate seismicity, the plastic hinges tend to appear at rather unexpected locations and not at the nodes. The second observation is that, for slender cross-section, it remains possible to reach values of the behaviour factor in the range of 2,6 to 3,2. This is less than 4, as normally considered for MRF's, but this is perfectly in line with the targeted values for an efficient design in low or moderate seismic zones.

4   CONCLUSIONS AND PERSPECTIVES

The research project Meakado has achieved significant advances in the direction of a better control of the seismic performances of structures not perfectly fulfilling the EC8-DCM requirements. In particular, progress has been gained regarding the characterization of shear connections not specifically designed as ductile nor capacity-designed with respect to the braces, as well as on their interaction with the behaviour of the braces themselves. Hints have been given on how to implement a "limited ductility" design allowing reaching in a reliable way behaviour factors in the range of 2 to 3. Similar results are achieved for steel and composite moment resisting frames with class 3 or 4 cross sections. These results will then serve as a base for implementing design recommendations at a format that could be easily and directly implemented in design codes.

ACKNOWLEDGEMENT

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