2019 SECED Conference
Earthquake risk and engineering towards a resilient world
9-10 September 2019, Greenwich, London

OPTIMIZED DESIGN OF MOMENT RESISTING FRAMES WITH SLENDER STEEL AND COMPOSITE SECTIONS IN MODERATE SEISMIC AREAS

Hervé DEGEE[1], Yves DUCHENE[2] & Benno HOFFMEISTER[3]

Abstract: *This paper aims at defining 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 optimized 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. Specific investigations have been carried out regarding typical beam profiles commonly used for steel and composite frames. In a first stage, experimental tests on class-3 and class-4 built up steel profiles and composite beam-to-column nodes were performed. The measurement results were evaluated with regard to the development of the hysteretic behaviour with particular emphasis on the cyclic degradation. These test results have been used as reference for the calibration and validation of numerical model aiming at extending the scope of the experimental outcomes through appropriate parametric variations regarding the behaviour of nodal connections as well as towards the global analysis and behaviour of structures made of class 3 and 4 profiles. Based on the outcomes of these investigations, practical design recommendations are finally derived for moment resisting frames located in low and moderate seismicity regions.*

Introduction

According to current version of Eurocode 8 only cross-sectional classes 1 or 2 are permitted for steel or composite structures when a behaviour factor higher than 1.5 (or 2.0) is intended to be taken into account. Within moment resisting frames almost all members are affected by this limitation:

Single-bay – single-storey frames: Such structures are part of a highly competitive market requiring a strong optimisation in terms of material requirement and ease of construction. Light weight steel frames are an optimum solution so far, with the exception of seismic regions where the above mentioned restrictions strongly limit their applicability despite their low masses. On the other hand such structures may develop plastic hinges alternatively in the columns or beams and thus offering the possibility to adjust the static stiffness and resistance e.g. by means of haunches. In order to remain competitive however both columns and beams need to be kept slender yielding cross-sectional classes 3 or 4. Such cross-sections however can develop significant cyclic bending capacity which, although at a lower level than the elastic one, can be sufficient in order to resist seismic actions.

Multi-bay – multi-storey composite buildings: Due to the high position of neutral axis of composite cross-sections in negative bending the web of steel profiles is very often to be classified as class 3 or 4 section, although almost all rolled steel profiles may be classified as class 1 or 2 cross-sections. On the other hand in such kind of buildings under seismic actions there are always both positive and negative moments developing at the corners of the frame and thus providing a significant resistance and dissipation capability when the entire system is considered.

The above mentioned aspects are of particular importance in countries with moderate seismicity regions. Hence an extension of the applicability of the afore mentioned structural types to

[1] Professor, Hasselt University, Hasselt, Belgium, herve.degee@uhasselt.be
[2] Doctor, Design Office Greisch, Liège, Belgium
[3] Professor, RWTH, Aachen, Belgium

moderate seismic regions would largely enhance their competitiveness. This leads to the following issues to be addressed in this paper:

1. Extension of the applicability of class 3 or 4 steel cross-sections within single-bay – single-storey frames to moderate seismic actions.
2. Extension of the applicability of class 3 (web) composite sections to moderate seismic actions.

Experimental investigations

Tests on steel portal frame corners

The test program considered typical welded profiles typically used for light weight frames of small industrial or storage halls. In total 6 cyclic tests on frame corners with welded and bolted connections as well as haunched and constant depth girders were carried out. The test specimens were designed such that for constant depth girders the plastic hinges developed in the beams whereas for the haunched girders the plastic hinges developed in the top of the column.

The tests were performed according to the ECCS testing procedure with increasing amplitudes of deformation cycles and were executed until collapse (e.g. rupture due to low cycle fatigue or extensive local buckling). The measurements were evaluated with regard to the development of the hysteretic behaviour with particular emphasis on the degradation.

Different examples for moment resisting frames (MRF) have been compiled from literature as well as from direct dialogue with manufacturers to mirror the state-of-the-art. Following these examples the geometries, plate thicknesses and joint details have been assigned. Furthermore 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, whereby one additional test on a frame corner without a 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 was 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 1 shows the above mentioned specimens or types of frame corners respectively.

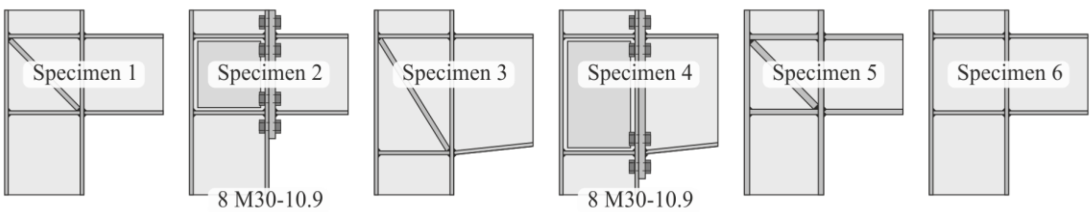

Figure 1. Overview of the specimens – types of frame corners

The boundary conditions of the test setup were designed to represent the cut-out from a complete MRF; the boundary conditions have been particularly focused on a best possible representation of the real load-deformation behaviour of a complete MRF. The test setup is shown schematically in Figure 2. The detailed geometrical and material data, instrumentation and loading procedure are given in Degee et al. (2018), as well as the detailed experimental measurements. All results in terms of measured values of the displacement and number of cycles at failure are summarized in Table 1. As a matter of illustration, Figure 3 shows the results in terms of cyclic curves for specimens S1.

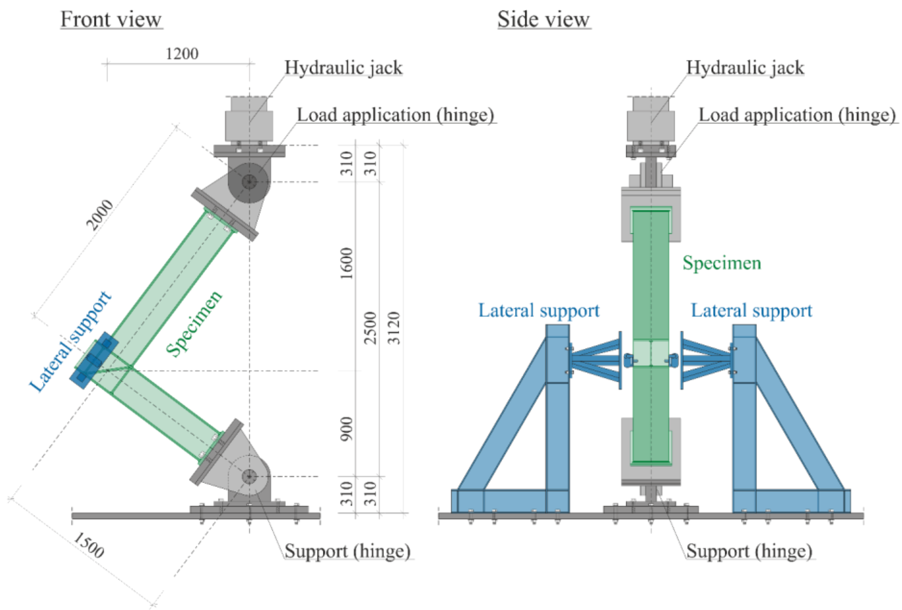

Figure 2. Schematic representation of the test setup

Specimen	e_i in [mm] for cycle												N_f
	4	5-7	8-10	11-13	14-16	17-19	20-22	23-25	26-28	29-31	32-34	35-36	[-]
S1	20.7	31.4	42.7	54.1	66.3	78.6	91.0	104.1	-	-	-	-	23.3
S2	20.8	31.9	43.5	55.1	67.0	79.6	-	-	-	-	-	-	18.3
S3	15.9	24.2	33.2	42.7	52.5	62.6	73.2	86.3	-	-	-	-	22.3
S4	16.1	24.8	33.8	43.4	53.1	63.5	77.2	-	-	-	-	-	18.3
S5	20.4	31.1	42.6	54.2	66.0	77.6	89.6	101.9	-	-	-	-	22.8
S6	13.9	21.4	28.9	36.7	44.5	52.3	60.2	68.1	76.1	84.0	92.2	67.4	35.3

Table 1. Measured values of displacements and number of cycles at failure.

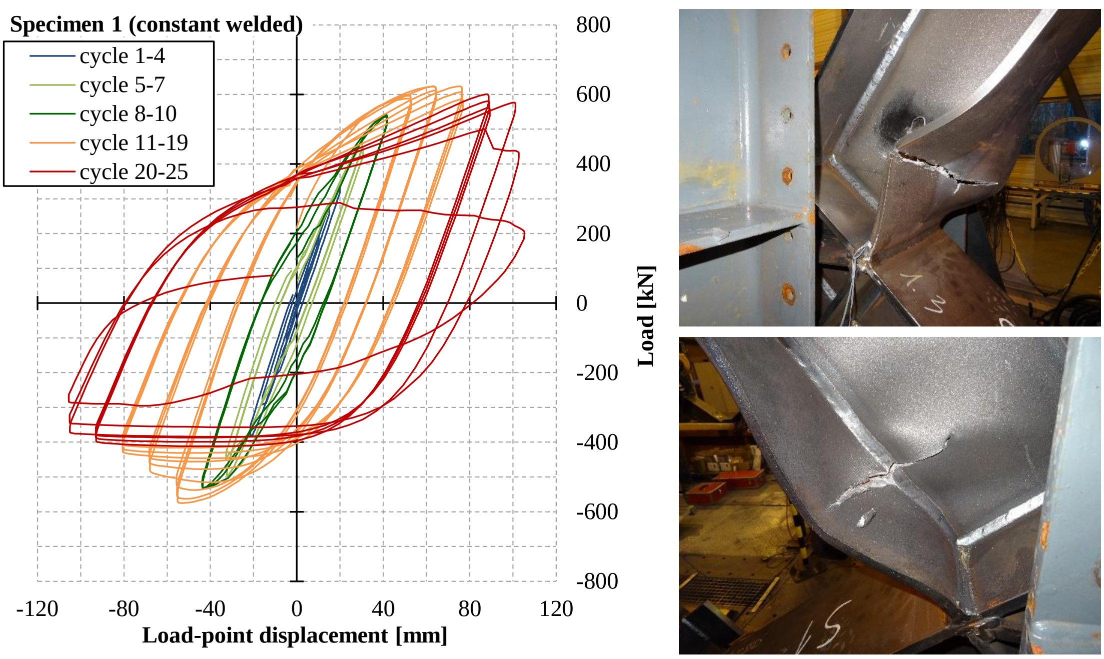

Figure 3. Cyclic load-displacement curve and failure mode (specimen S1).

The main conclusions from these tests can be summarised as follows:

- The presence of class 3 cross-sections (slender flanges) did not influence the cyclic rotational capacity;
- The evaluation of the absorbed energy did not exhibit a significant, negative influence of the flange slenderness on the performance.
- The achieved plastic moments exceeded the full resistance of the beam or column (whichever was governing) calculated as compact section; additionally, the material over-strength and strain hardening contributed to higher resistances;
- The formation of plastic hinges in column or in the beam did not significantly influence the cyclic performance;
- The detailing of bolted connections needs to take into account the full plastic resistance of the governing member, local effects need to be considered (e.g. by local strengthening of the column flanges). In the tested cases the bolted specimens higher susceptibility to local damage due to local effects from the bolts on flanges of the columns.
- Unstiffened web panels may provide significant dissipation; however, it may lead to strain concentrations in complex weld-zones, in particular for welded (not hot rolled) members.

Tests on composite portal frame corners

The test program considered typical steel composite beam profiles commonly used for multi-bay - multi-storey composite frames. In total three cyclic test on frame corners with welded connections and constant depth girder were carried out. The test specimens were designed such that the plastic hinges developed in the beams.

The large scale test specimens represent a section of an exterior corner of a composite frame. The main dimensions in length of column and beam were equal for all specimens and were chosen so, that the resulting moment distribution is comparable to that of a complete MRF loaded by vertical and horizontal loads (hinges were set at points of zero moment). All columns consisted of a HEB 360, Class 1 profile. The girder was joined to the column with double side full penetration butt welds. The effective width of the concrete slab was determined according to EN 1998-1. Specimen S01 and S02 were classified as Class 3 Beams under negative bending. The only difference between these specimens was an additional vertical stiffener in the beam. The third test consisted of a class 1 Beam, with comparable monotonic bending resistance and stiffness. The reinforcement arrangements were the same for each specimen, as well as the arrangement of the headed shear studs. The 3 different specimens that was decided on were:

- Specimen 1 (S01) IPE 450 – Class 3 Beam
- Specimen 2 (S02) IPE 450 – Class 3 Beam, reinforced
- Specimen 3 (S03) HEA 360 – Class 1 Beam

Specimens during fabrication are illustrated in Figure 4. Table 2 gives the measured values of the displacement and the number of cycles at failure. The results in terms of cyclic curves and failure modes are illustrated in Figure 5 for specimen S01.

All three specimen failed in a similar sequence. First cracks appeared near the L-profile at the bottom side of the concrete slab, as well as at the overhang. During testing concrete spalling at the slab overhang and concrete crushing on the top around the column were observed. It is worth noting that crack re-closing could established upon load reversal. Furthermore, it must also be noted, that all 3 specimens failed by brittle fracture. And there was not observed significant local buckling in the flanges or webs of the girders during the whole loading history. While for specimen S01 and S02 crack initiation get started in the bottom flange of the girder near the welds in the heat-affected zone (HAZ), for specimen S03 crack initiation occurred in the upper flange of the girder. Figure 6 documents the damage evolution during the tests.

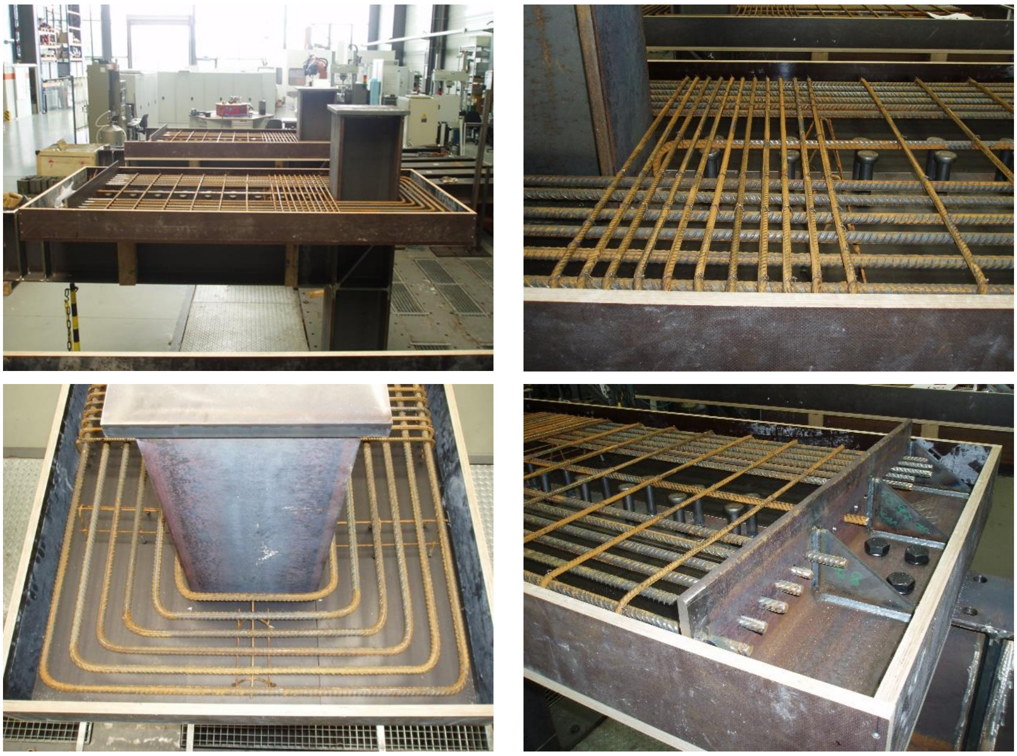

Figure 4. Fabrication of the composite specimens.

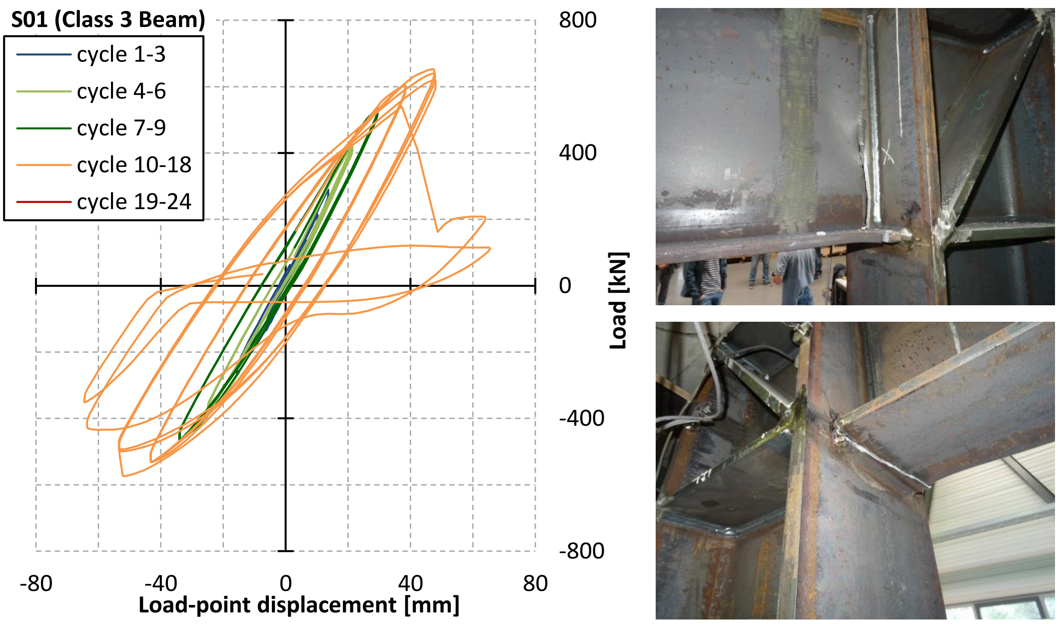

Figure 5. Cyclic load-displacement curve and failure mode (specimen S01).

Specimen	e_i in [mm] for cylce								N_f
	3	4-6	7-9	10-12	13-15	16-18	19-21	22-24	[-]
S01	14.6	23.0	31.7	40.8	50.7	64.4	-	-	15.1
S02	14.9	23.1	36.2	50.0	59.8	71.6	-	-	15.9
S03	15.2	23.6	32.2	41.2	50.3	60.0	70.4	74.5	20.6

Table 2. Measured values of displacement and number of cycles until failure.

Figure 6. Damage evolution of steel-concrete specimens.

The main conclusions from these tests can be summarised as follows:

- The presence of class 3 cross-sections (slender web in compression) did not show any significant influence on the resistance and on the cyclic rotational performance. The observed differences in the performance were within the expected scattering of the properties;
- No stability phenomena in the web, only slightly developed local buckles in the flanges were observed;
- The achieved ductility and number of cycles was limited by the formation of cracks in the steel cross-section in tension; the cracks started apparently from the weld details;
- The composite action was fully achieved, leading to a steep gradient of strains towards the lower steel flange which led to a rather early failure of the specimens.

Numerical investigations

Model calibration

The test results presented above have been used as reference for the calibration and validation of numerical model aiming at extending the scope of the experimental outcomes through appropriate parametric variations regarding the behaviour of nodal connections as well as towards the global analysis and behaviour of structures made of class 3 and 4 profiles.

Two types of numerical models have been calibrated. First, detailed FE models using solid elements have been calibrated at local level. Then these detailed local models have been used to calibrate equivalent spring models of the nodes and/or of the cross-sections in order to be used in the context of a model resorting to one-dimensional beam elements. These calibration phases are detailed respectively in Degee et al (2018). The corresponding models are illustrated in Figure 7 and 8

Analyses at node level

Additional simulations were then carried out in order to determine the performance of slender sections in the plastic range and to compare them to the performance of similar compact sections.

A total of 45 parametric variations of the steel test specimens and of 4 parametric variations of the composite specimens have been carried out.

Regarding the steel nodes, the extension of the tests by numerical simulations included the following parameters:

- Flange thickness (class 1 … 4), constant depth and width of profiles
- Web thickness (4 … 12 mm, the web remained class 1), constant depth and width of profiles
- Profile depth, constant width and plate thickness, web (6 mm) class 1 … 4

Regarding the composite nodes, the extension of the tests by numerical simulations included the following parameters:

- Variation of the longitudinal reinforcement in the slab (decreasing from 26 cm² to 6,5 cm²)
- Variation of the concrete strength (decreasing from 54 N/mm² to 28 N/mm²)

A detailed analysis of the outcomes of the numerical parameter studies is provided in Degée et al. (2019). These results constitutes the main background of the recommendations concluding this paper.

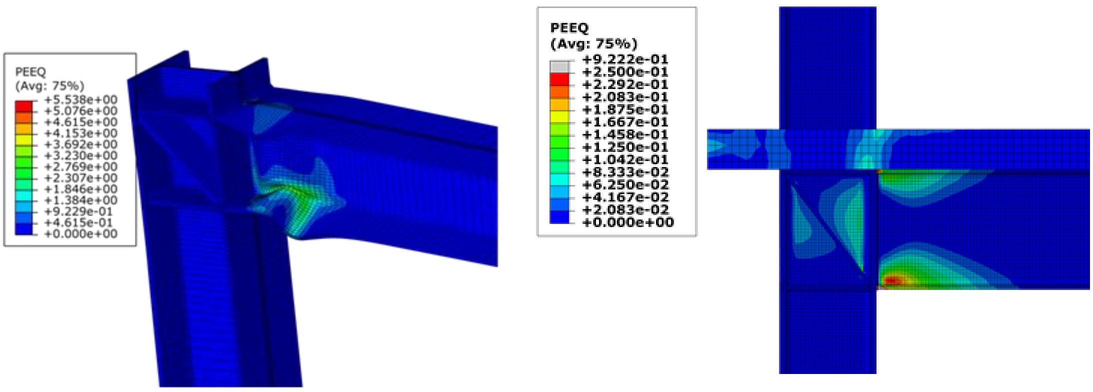

Figure 7. FE models for detailed analysis at node level.

Analyses at structural level

The aim of these analyses is to evaluate the global performances of steel and composite moment resisting frames (MRF) designed by seismic actions in moderate seismic regions for different number of storeys and bays.

Case studies are designed according to DCM requirements (weak beam-strong column rule, interstorey drift limitation, second order effects), except the requirements on the cross-section limitation in dissipative zones, assuming however a behaviour factor q equal to 4.

The numerical investigations are performed by making use of non-linear time-history analysis of the entire frame structures, including thus second order effects and inelastic cyclic behaviour of beams. The analysis of the results is specifically focusing on the comparison of the rotation capacity of the slender section with the actual rotation demand imposed by a moderate intensity earthquake.

Results are given in Figure 9 for a typical one-bay – multi-storey frame example. It illustrates the general observation that the rotation capacity is in no cases fully exploited (in the range of 18 to 35%)

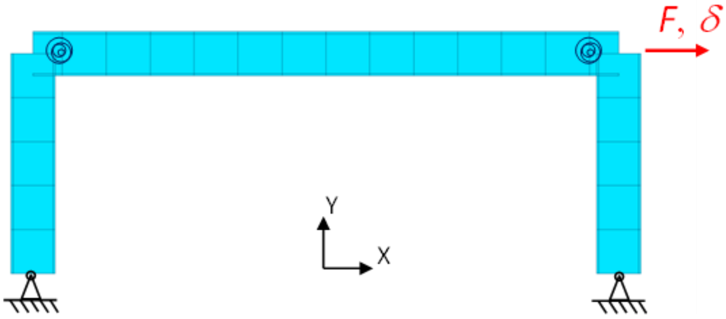

Figure 8. Combined Beam FE - spring models for analysis at structural level

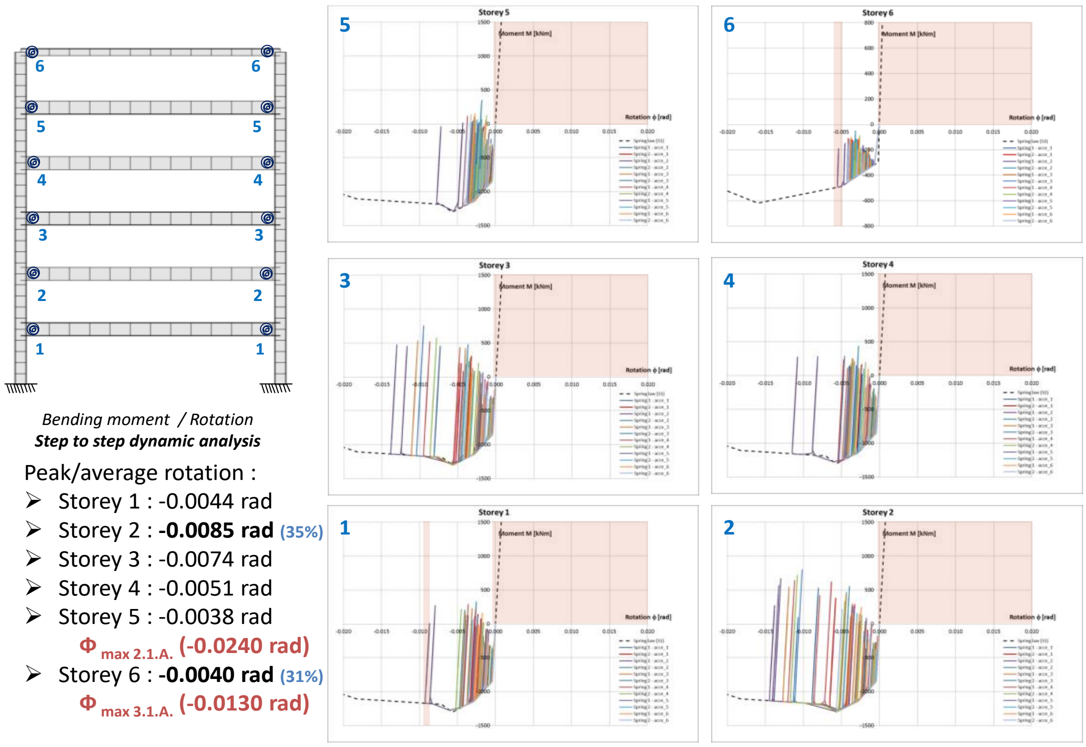

Figure 9. Rotation demand in corner springs (illustration on a 1 bay – 6 storey configuration)

Conclusions and recommendations

The research concludes by making recommendations based on the outcomes of the tests and simulations at local and global levels. The main recommendations are as follows:

Recommendations for frames with slender steel profiles

The scope of these recommendations are single storey portal frames.

- Single storey, single bay moment resting frames may be designed for energy dissipation with profiles belonging to the cross-sectional class 3 or 4.
- The frames may have hinged or fixed column bases.
- The slender cross-sections may be used for columns and rafters.
- The preferable dissipating mechanism is the formation of plastic hinges in bending in the frame eaves. The hinges may form in the rafter or in the column.

- In order to allow for the development of plastic hinges capacity design rules need to be applied to the eaves connections. In particular, the following requirements shall be fulfilled:
    - Shear buckling of flanges shall be prevented;
    - For plastic hinge in a profile with class 3 or 4 web the full plastic moment of the relevant cross-section must be transferred by the connection;
    - For plastic hinge in a profile with class 3 flanges the full plastic moment must be transferred by the connection;
    - For plastic hinge in a profile with class 4 flanges the bending moment determined with fy at the extreme fibres must be transferred by the connection;
    - Overstrength of the material must be considered.
- Haunched rafters are permitted, when the plastic hinges shall form at the top of the column.
- The frame must be prevented from out-of-plane instability.
- Additionally, the following verifications are required for the seismic design situation:
    - Buckling stability of the columns under the assumption of a hinged-hinged system:
    - The rafter shall be capable to resist the gravity loads (seismic design situation) under the assumption of a simply supported beam system.
- The two above requirements shall prevent the global collapse of the frame, in case of significant degradation of the profiles due to cyclic plastification.
- Portal frames satisfying the requirements (4) to (8) may be designed using a behaviour factor
    - q = 2,5 for cross-sections with class 4 flanges
    - q = 3,5 for cross-sections with class 3 flanges
    - q = 4 for cross-sections with class 3 web (shear buckling and plastic shear failure excluded)
    - q = 3 for cross-sections with class 4 web (shear buckling and plastic shear failure excluded)
    - for cross-sections with web and flange belonging to class 3 or 4 the lowest of the above values applies
- The design resistance for safety verifications shall be determined according to Eurocode 3 under consideration of the web or flange slenderness.

Recommendations for frames with slender steel-concrete section

The scope of these recommendations are multi-storey, multi-bay moment resisting frames with type b (full composite action) beam-to column connections.

- Composite frames with class 3 web under negative bending (concrete slab in tension, steel profile in compression, assumption of a fully yielded cross-section) may be designed for energy dissipation.
- The dissipating mechanism must be the formation of plastic hinges in the beams.
- Capacity design and detailing rules according to ductility class DCM shall be applied under the assumption of a fully yielded beam cross-section with class 2 web.
- The nominal resistance for the verification in the seismic design situation shall be determined according to EC 3 and EC 4, considering the slenderness of the web.
- Shear buckling and yielding of the web in shear must be excluded.

- Particular attention should be paid to the design and execution of the weld seams in welded beam-to column connections.
- If the above requirements are satisfied, composite frames with class 3 web in the beams under negative moment may be designed using the behaviour factor q =4.

Acknowledgement

This research has been carried out with the support of the Research Fund for Coal and Steel (RFCS) of the European Commission under the grant agreement RFSR-CT-2013-00022.

References

Eurocode 8 (2009). Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings; EN 1998-1:2004 + AC:2009.

Kato B (1989). Rotation Capacity of H-Section Members as Determined by Local Buckling, *Journal of Constructional Steel Research* 13 (1989), 95-109.

Sedlacek, G. and Feldmann, M (1995). *Background Document 5.09 for chapter 5 of Eurocode 3, Part 1.1, The b/t-ratios controlling the applicability of analysis models in Eurocode 3, Part 1.1.*, 1995.

Elgahzouli, A (2010). Assessment of European seismic design procedures for steel framed structures, *Bulletin of Earthquake Engineering*, 8, N° 1 (2010), 65-89.

Degée, H. et al (2018). *Meakado - Design of steel and composite structures with limited ductility requirements for optimized performances in moderate earthquake areas* – Final report. RFCS publications (2018).

ECCS (1986). *Recommended Testing Procedure for Assessing the Behaviour of Structural Steel Elements under Cyclic Loading*, ECCS - Technical Committee 1 - Structural Safety and Loadings, Technical Working Group 1.3 - Seismic Design, Brussels 1986, Belgium.

Bisch P, Carvalho E, Degee H, Fajfar P, Fardis M, Franchin P, Kreslin M, Pecker A, Pinto P, Plumier A, Somja H, Tsionis G. (2012). *Eurocode 8: Seismic Design of Buildings worked examples*, JRC technical report. Publications Office of the European Union/Joint Research Centre, Luxembourg.

Degée, H., Duchene, Y and Hoffmeister B (2019). *Performances of moment resisting frames with steel and composite sections in low to moderate seismic areas*. Proceedings of the SDSS 2019 conference (Prague).