SEISMIC PERFORMANCE OF MULTI-STOREY FRAMES WITH SLIM-FLOOR BEAM-TO-COLUMN JOINTS

PhD Dissertation – Extended abstract

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1. Introduction

The slim-floor is an alternative flooring system characterized by the integration of more components into one structural element. This means that the main structural member, the asymmetric steel beam, and other components are incorporated into the concrete slab. Since their introduction to the construction market in the 1990s, slim-floor systems have been efficiently applied to mid- to high-rise steel and composite structures of buildings throughout Western and Nordic European countries and even in China [1]. In most cases, the functions of these buildings are office, residential or medical care and recovery units [2] [3]. According to a slim-floor solutions developer [4], requirements at the Ultimate Limit State (i.e., resistance, stability) and at the Serviceability Limit State (i.e., vibration) could be met. Thus, the application to multi-storey buildings is no longer a novelty in civil engineering. However, due to the current typology of the slim-floor beam-to-column connections, e.g., pinned end connections, and to the fact that the slim-floor beams are designed exclusively for gravity loads in the elastic range, the technical solution is incompatible with the seismic design of frame systems. A further issue resides in the actual design of the slim-floor system, which is predominantly regulated by technical approvals instead of code provisions. Until the release of the new version of the composite European code, prEN 1994-1-1 [5], in which some rules are included, and a range of application is defined, the design of slim-floors is not easily approachable, and applications are rather limited. The current study is developed with the aim to provide a technical solution for slim-floor beam-to-column joints, which would make the shallow flooring system applicable to structures designed not only for medium, but also for high seismicity. In order to apply capacity design principles to the slim-floor beam-to-column joints, thus to meet Ductility Class 3 resistance, stiffness and rotation capacity criteria according to prEN 1998-1-2 [6], a structured procedure entailing the following main steps is presented in the current study:

- **Design of joint assemblies**
  - estimation of demand on seismic-resistant structures with slim-floor (SF) beam-to-column joints;
  - design based on Finite Element Method (FEM) of the SF beam-to-column joints;
- **Experimental investigations**
  - experimental campaign with monotonic and cyclic tests on joints;
  - interpretation and evaluation of experimental results;
- **Numerical investigations**
  - calibration of numerical model and development of parametric study with FEM;

- interpretation and evaluation of numerical results;
- **Development of design procedure and detailing**
  - evaluation of mechanical characteristics of the SF beam;
  - capacity design of the SF beam-to-column joint;
  - classification of stiffness and resistance of the SF beam-to-column joint according to prEN 1993-1-8 [7];
  - detailing rules for SF beam-to-column joints;
- **Structural analyses and seismic performance evaluations**
  - development of a simplified numerical SF beam-to-column joint model for integration into MRF, CBF and D-CBF structures;
  - validation of SF beam-to-column joint model against experimental data;
  - calculation of plastic rotation demand on SF beams, interstorey drift demand on SF beam-to-column joints and generally, seismic performance evaluation of Moment-Resisting Frames, Concentrically-Braced Frames and Dual Concentrically-Braced Frames with slim-floor beam-to-column joints with nonlinear static and dynamic analyses.

2. Experimental Program

The experimental program on slim-floor beam-to-column joint assemblies consisted of monotonic and cyclic tests. The main components of the joint specimens were, as follows: ▪ slim-floor beam consisting of an asymmetric steel cross section with a *Reduced Flange Section RFS* (S355), ▪ end-plate bolted connection with 8*M36 bolts, grade 10.9, ▪ H-profile steel column (S355) and ▪ reinforced concrete slab (class C30/37; B500B reinforcing steel bars and ▪ steel rebars for the concrete dowels. The main objective of the experimental tests was to attain a joint rotation capacity of ± 40 mrad at Significant Damage, as required by AISC 341-16 [8] for joints of unbraced frames designed to resist seismic loads. The novelty of the tests resided in the following:

- the use of a bolted moment-resisting connection (extended two ways beam-to-column connection with high strength bolts and full-penetration groove welds) for SF beams;
- the application of a *RFS* to the lower flange of the SF beam as one measure to ensure member ductility and to prevent the brittle failure of the bolted connection;
- SF beam-to-column joints subjected to cyclic experimental tests.

2.1 Monotonic Experimental Test

Joint specimen *SF J-M* was tested under a monotonic load applied by an incremental increase of the load under hogging bending, followed by a reversed load under sagging bending. After reaching a considerable bending moment of *- 907 kNm* and a rotation of *- 93 mrad* under hogging bending, a weld fracture occurred at load reversal. The fractured weld was between one of the stiffeners and the column flange. The fracture of this weld rapidly led to multiple weld failures under reverse load. While the value of the bending moment at the time of the first failure was high (i.e., *730 kNm*), the failure mechanism was not expected. A post-test inspection of the welds revealed some fabrication faults, which led to the strengthening of the stiffeners-to-column flange welds prior to the cyclic test.

2.2 Cyclic Experimental Test

The cyclic test was performed on joint specimen *SF J-C* by applying the loading protocol of AISC 341-16 [8]. During the cyclic test, high plastic deformations were sustained by the dissipative zone of the SF beam – observed on-site by the cracking and flaking of the whitewash, but also supported by the measurements recorded by the measuring system. The results confirm the development of a plastic hinge in the dissipative zone with *RFS*. The test was stopped after the first cycle of ± *60 mrad*, when the weld between the lower flange of the SF beam and the end-plate fractured. This specimen evidenced a symmetric and relatively stable hysteretic behaviour with a low degradation of stiffness and resistance. This behaviour can be considered adequate for beam-to-column joints of Ductility Class 3 structures. The evaluation of the seismic performance was carried out by using the provisions of FEMA P-795 [9] for the construction of the envelope curves. Furthermore, the ECCS procedure [10] was used to calculate joint rotations corresponding to Damage Limitation (DL), Significant Damage (SD) and Near Collapse (NC). The following joint rotations were obtained: ± *16 mrad* at DL, ± *45.4 mrad* at SD and ± *60.5 mrad* at NC. As the achieved rotation at SD is ± *45.4 mrad*, meaning with *13 %* more than required rotation of ± *40 mrad*, the AISC 341-16 [8] was fulfilled.

The implementation of capacity design principles to the SF beam-to-column joint, according to which the overstrength of the end-plate connection and of the adjoining welds should be ensured in dissipative joints, led to the use of an extended end-plate connection with high strength bolts and the application of a *RFS* to the lower flange of the steel beam. Thus, the bolted connection developed higher bending moment resistances than those of the connected member, i.e., the reduced section of the SF beam. According to the resistance classification of prEN 1993-1-8 [7], the joint is categorised as full-strength. Considering the stiffness criterion of prEN 1993-1-8 [7], the joint is classified as semi-rigid when used in unbraced frames and rigid if used in braced structural systems.

3. Numerical Program

3.1 Reference Numerical Model

The reference numerical model (*RM*) was created with both solid and beam elements using Abaqus v19 [11]. Regarding the interaction law that defines the way in which components interfere with each other, the following criteria were applied: "*tie constraint*" (modelling of: welds); "*embedded constraint*" (modelling of: rebars-to-concrete slab interaction); and "*contact interaction*" (modelling of: interaction of the different model components such as between concrete slab, steel SF beam and column, respectively between bolts, end-plate and column flange). As for the "*contact interaction*" - a contact law was defined considering both normal and tangential properties. The normal contact was defined as a "*normal hard contact*" that allowed separation; the tangential contact was defined as a "*friction / penalty contact*" with a friction coefficient of $\mu = 0.6$. The value $\mu = 0.6$ of the friction coefficient resulted from the calibration process of the finite element (FE) joint model, as the use of other values of μ generated different results from the experimental ones. The material stress-strain relationships considered for the main components of the FE model of the *RM* were calibrated against the data obtained from material test samples. The type of analysis that was used in the numerical program was *"dynamic, explicit"*, also accounting for geometrical nonlinearity.

The results of the reference FE model, as well as of the FE models from the parametric study, were obtained by performing advanced finite element analyses (FEA) and have led to important conclusions. The first of these conclusions is related to the results of the calibration of the reference numerical model *RM*, which evidenced a high accuracy in reproducing the experimental test curve. To support the high correlation of the numerical curve to the experimental one, deviations or aberrations were calculated, and found to be in the range of *0.1 ÷ 3.2 %*. A further conclusion is related to the accuracy of the *RM* in reproducing the development of the failure mechanism. Under maximum hogging bending, most of the plastic deformation was sustained by the dissipative zone of the SF beam. In this case, deformations of the dissipative zone of the SF beam occurred under compression. Other components of the *RM* sustained localised inelastic deformations. Thus, the results of the *RM* under hogging bending supported the observations made during the experimental tests: ductile behaviour of the SF beam and a mainly elastic response of the bolted end plate connection. Under sagging bending, the dissipative zone of the SF beam exhibited a ductile behaviour, sustaining high plastic deformations. Overall, in the maximum bending points indicated on the moment-rotation curve corresponding to the *RM*, the highest values of plastic deformations were developed in the dissipative zone of the SF beam. Compared to the dissipative zone of the beam, the equivalent plastic strain within the bolts were low and the distribution limited (localised phenomena), which supports the conclusion that the failure mechanism was ductile. This observation is consistent with the conclusions of the experimental study.

3.2 Parametric Study

The second part of the numerical program was dedicated to the development of a parametric study. The parametric study was focused on isolating the influence of parameters that were part of the joint solution (e.g., *RFS*, reinforced concrete slab, etc.) in order to better assess the sensitivity of the FE model to them, but also included new parameters (e.g., backing plates, reinforced concrete ribs, trapezoidal steel sheets, etc.). The influence of the following parameters was analysed within the parametric study:

- reduced flange section *RFS* in model M_1;
- reinforced concrete slab in model M_2;
- concrete dowels in model M_3;
- concrete dowels plus "*frictionless contact*" between components in model M_4;
- longitudinal reinforcement ratio in model M_5;
- backing plates in model M_6;
- concrete class in models M_7 and M_8;
- reinforced concrete ribs in model M_9;
- reinforced concrete ribs and trapezoidal steel sheets in model M_{10};
- rib stiffener welded on the top flange of the SF beam in model M_{11};
- decoupled dissipative zone of SF beam from concrete in model M_{12}.

Influence of the *RFS*. The application of a *RFS* ensures member ductility which is manifested through to a balanced or symmetric response the SF beam-to-column joint. Thus, another important conclusion drawn from the numerical program is that the shape of the SF beam in the dissipative zone has a significant influence on the failure mechanism of the SF beam-to-column joint. Through the application of the *RFS*, the stresses and strains are more evenly distributed on the height of the dissipative zone, which eventually leads to a ductile failure mode in the dissipative zone. If the *RFS* is removed, the lower bolt rows fail under sagging bending. Prior to sudden drop in resistance on the moment-rotation curve, the components which sustained the highest plastic strain values were the bolts (e.g., *0.119 mm/mm*). Simultaneously, the maximum value of plastic strain within the dissipative zone was half of that in the bolts (e.g., *0.055 mm/mm*). A similar conclusion was reached in the study of Plumier [12], in which the failure mechanism of connections with *Reduced Beam Section* (*RBS*) and without *RBS* was investigated experimentally. In the previously mentioned study, it was concluded that that the investigated specimens, which did not include a *RBS*, sustained bolt failure.

Influence of the concrete slab. The presence of the reinforced concrete slab influenced the bending resistance, stiffness and rotation. For example, when the concrete slab was removed from model M_2, the bending resistance, initial stiffness and rotation at maximum bending moments were lower than in the *RM*.

Influence of concrete dowels. The shear interaction was assured by rebars and concrete dowels. Although the concrete dowels were removed from numerical model M_3, the stresses were transferred through inclined rebars and friction, so the moment-rotation curve remained similar to that of the *RM*. Followingly, apart from the removal of the concrete dowels the friction between the components of model M_4 was eliminated. The effect of the latter produced a series of changes, the most important of which being a high value of relative slip, e.g., *17.6 mm*. Considering this, it was concluded that the SF beam-to-column joint solution should include concrete dowels. The conclusion is even more important in seismic regions, where multiple loading cycles could diminish friction and lead to undesired consequences.

Influence of increased reinforcement ratio. A higher longitudinal reinforcement ratio of the concrete slab led to a different failure mechanism. The development of plastic strain in the portions of the concrete slab under compression began at values of the bending moment equal to *450 kNm*. At *940 kNm*, portions of the concrete slab under compression experienced severe cracking and one of the transverse rebars (located behind the column) fractured leading to the end of the analysis. The parameter could be further investigated, but the increase in longitudinal reinforcement ratio should be accompanied by an increase in concrete class.

Influence of backing plates, rib stiffener and decoupled dissipative zone. Considering the investigated parameters, it was concluded that measures could be employed to reduce the development of plastic strain within the beam-to-column connection. In this sense, the addition of backing plates in model M_6, the addition of a rib stiffener in model M_{11} and the decoupling of the dissipative zone from the reinforced concrete in model M_{12} proved efficiency:

- model M_6: *17.6 %* less plastic strain within bolts under sagging bending;
- model M_{11}: *32 %* less plastic strain within bolts under sagging bending, and *24 %* less under hogging bending;
- model M_{12}: *24 %* less plastic strain within bolts under sagging bending, and *6.7 %* less under hogging bending.

Influence of increased concrete class. The increase in the concrete class as a stand-alone parameter in models M_7 and M_8 was found not to be not influential on the initial stiffness. In both of the analysed FE models, the value of initial stiffness remained almost identical with those of the *RM*. A delayed initiation of the cracking or even reduced cracking of the concrete slab could not be demonstrated.

Influence of reinforced concrete ribs and trapezoidal steel sheets. Neither the addition of reinforced concrete ribs nor that of trapezoidal steel sheets to models M_9, M_{10} prevented the development of a plastic hinge in the dissipative zone of the SF beam. The presence of steel sheets was included in the experimental program of Wang et al. [13], who concluded that the addition of this parameter did not modify the previously obtained failure mechanism.

Results of the FEA underlined the central roles of the *RFS* in the dissipative zone of the SF beam as the “weaker” component and of the end-plate connection as the resistant component in obtaining an adequate seismic performance of the SF joint. However, the numerical program should be extended with additional analyses to help establish a range of application for SF beam-to-column joints. For instance, each of the investigated parameters could be further parametric analysed.

4. Design and detailing procedure

Applications of slim-floor beam-to-column joints are not covered by the European codes, although due to the increasing interest and push in the direction of efficiency and sustainability, this might soon change. Therefore, provisions for slim-floor systems are needed. The current design procedure is based on new rules and on existing design provisions for steel and steel-concrete composite structural members, that were proposed to be extended to SF beam-to-column joints. The design procedure that is proposed within this study is addressed to SF beam-to-column joints of seismic-resistant structures developed to meet DC3 criteria. Considering the resistance and stiffness classifications of prEN 1993-1-8 [7], the beam-to-column joints of Moment-Resisting Frames should be classified as full-strength and rigid or semi-rigid, according to Landolfo et al. [14]. To obtain an adequate response of the SF joint, several strategies were employed:

- ensuring the ductility of the main fuse (dissipative zone of steel SF beam);
- ensuring the overstrength of the end-plate connection of the web panel;
- ensuring the overstrength of the welds;
- ensuring the overstrength of the adjacent member (column).

The slim-floor beam should be obtained from half of a steel I-profile [1]. A wide steel plate should be welded on the remaining half of the steel profile, forming an asymmetric steel SF beam. Ductility at member level is obtained through the application of the *RFS*. As the SF beam is the main component to dissipate seismic energy, the following rules and techniques were proposed for application in the design procedure to ensure the ductility of the SF beam:

- material requirements of the current and of the pre-normative versions of the Eurocodes (i.e., prEN 1998-1-2 [6], EN 1993-1-1 [15], prEN 1993-1-1 [14]) for dissipative steel-concrete composite and steel structural elements;
- material requirements of the *National Technical Approval Z-26.4-59* for *CoSFB* [17];
- rules for the section class for composite shallow flooring systems of the pre-normative version of prEN 1994-1-1 [5];
- application of *RFS* to the lower flange of the SF beam.

Consequently, the steel grade of the SF beam profile should be in the range of S355 ÷ S420 [17], and the material should ensure minimum ductility, i.e., $f_u / f_y \geq 1.10$, elongation higher than *15 %*, in accordance with prEN 1998-1-2 [6], EN 1993-1-1 [15]. According to the pre-normative version of Eurocode 4 [5], the cross section class of the SF beam should be 1. The *RFS* should be applied to the lower flange of the SF beam and the type of the trimming should be radius cut. The proposed dimensioning tools of the *RFS* were based on AISC 358-16 [8] for *RBS* connections. However, due to the larger width of the lower beam flange and to the partial concrete encasement of the steel SF beam– both of which set apart the slim-floor systems from downstand configurations – the dimensions of the *RFS* were adapted to particularities of shallow flooring systems.

The bolted end-plate connection should be kept within the elastic response range. Thus, the bolted beam-to-column connection should be designed to develop higher resistance than the dissipative zone with *RFS* of the SF beam under both hogging and sagging bending. To achieve this, it was proposed that the type of the end-plate connection should be extended above and below the flanges of the SF beam. Moreover, high strength bolts are recommended, e.g., grade 10.9. The verification of the bolted connection should be performed in accordance with prEN 1998-1-2 [6] and EN 1993-1-8 [7] including the effects of the material overstrength γ_{rm} and of the strain hardening factor γ_{sh}. The bending moment and the shear demand for the bolted connection should be calculated considering internal forces from the dissipative zone with *RFS* projected to the column face and multiplied by the strain hardening and the material overstrength factors. The welds adjoining the bolted beam-to-column connection should be designed to develop higher resistance than the dissipative zone of the SF beam. In accordance with the provisions of the European seismic code [6], the following should be respected:

- critical welds should be performed with full penetration groove welds and reinforcing fillet welds; the following welds should be considered critical:
    - welds between the SF beam flanges and the end-plate;
    - welds between the stiffeners and the column flange;

- fillet welds should have a minimum thickness of $0.8 \cdot t_{min}$, where t_{min} is the minimum thickness of the welded components.

The column should be fabricated either in a steel or in a steel-concrete composite solution from an H-profile. Stiffeners and supplementary web plates could be used for the strengthening of the web panel in accordance with prEN 1998-1-2 [6] and prEN 1993-1-8 [7]. The shear resistance $V_{wp,Rd}$ of the web panel should be taken as the elastic shear resistance of the web panel without a surplus of resistance provided by continuity plates, in accordance with EN 1998-1 [18].

The current design procedure promotes the application of certain existing code rules and others which will be included in the upcoming version of prEN 1998-1-2 [6] and prEN 1994-1-1 [5] for steel and steel-concrete composite elements designed as dissipative structural elements, as well as the rules of the National Technical Approval for *CoSFB* [17]. In addition to these, the use of a *Reduced Flange Section* (*RFS*) was introduced in the current design procedure together with the corresponding dimensions. Essentially, the objective of the proposed design procedure is to obtain an adequate seismic performance of the SF beam-to-column joint (i.e., full-strength and rigid or semi-rigid joints, joint rotation at SD of ± *40 mrad*, ductile failure mechanism). As proved by experimental and numerical means, adequate seismic performance can be achieved as long as the plastic hinge development is directed to the dissipative zone of the SF beam. However, plastic hinge development in the SF beam is only possible if the beam is a ductile structural element, whereas the end-plate connection, the welds and the web panel have sufficient overstrength compared to the dissipative zone with *RFS*.

5. Structural analysis

5.1 Moment-Resisting Frame

The evaluation of the seismic performance of a Moment-Resisting Frame with slim-floor beam-to-column joints, *MRF-SF*, with nonlinear static and dynamic analyses was performed with SAP2000 [19]. In parallel, a reference Moment-Resisting Frame with regular composite beams with partial shear interaction, i.e., *MRF-RF*, was developed. The aim of the structural analyses was (i) to verify the rotation demand resulted from the seismic design situation on the SF beam-to-column joint, (ii) to compare it the experimental rotation capacity and (iii) to assess the seismic performance of the *MRF-SF* with SF beam-to-column joints. In accordance with the aims, the following objectives were established:

- development of a structural model for the tested SF beam-to-column joint;
- application of structural analyses in the nonlinear range, e.g., *Pushover* with the *N2 method* [20] and *Response-History Analysis* with a set of 7 accelerograms selected from [21];
- monitoring of the structural damage at the three Limit States, with particular interest at DL and SD, in terms of interstorey drifts and plastic rotation in the plastic hinges of the SF beams.

The modelling of the SF beam-to-column joint should be performed in detail in order to get the most realistic results. The conclusion was drawn from iterations on the modelling approach of the SF beam-to-column joint, which were validated against the experimental results. However, the results of other less demanding SF beam-to-column joints models were also explored and found to be adequate. In the case of the *MRF-SF*, should the modelling of the beam-to-column connection with link elements be replaced with a rigid connection, then the elastic stiffness of the frame would be slightly higher and the fundamental period would decrease with roughly *2 %*. Nevertheless, this idealisation of the connection produced very similar results in the nonlinear range to the recommended herein, yet more demanding modelling procedure.

The optimal modelling approach entailed the following steps:

- modelling of “full” section of the SF beam with:
    - tested geometry and material;
    - equivalent moment of inertia, $I_{eq,full\ section}$;
- modelling of dissipative zone with *RFS* of the SF beam:
    - tested geometry and material;
    - equivalent moment of inertia, $I_{eq,RFS}$;
    - plastic hinge model based on processed experimental data, which contained the plastic rotation of the dissipative zone of the SF beam;
- modelling of the beam-to-column connection:
    - linear elastic link;
    - contained the elastic stiffness of the connection;
- modelling of the panel zone:
    - rotational spring;
    - contained the stiffness of the web panel.

The contribution of the reinforced concrete slab to the resistance and stiffness of the SF beam-to-column joint was included in the structural model through the equivalent moment of inertia of the “full” section and of the dissipative zone with *RFS*. In addition, the plastic hinge model of the SF beams contained the plastic rotation of the dissipative zone, which included the reinforced concrete slab component. The envelope curve of the dissipative zone of the SF beam was obtained from the cyclic curve by following the provisions of FEMA P-795 [9] and by considering the first cycle of each amplitude. The response parameters and the acceptance criteria corresponding to the plastic hinge model of the dissipative zone were calculated based on Landolfo et al. [14].

The seismic performance was evaluated by performing nonlinear static and nonlinear dynamic analyses. The maximum interstorey drifts obtained from *Pushover* analyses on *MRF-SF* were: *7.8 mrad* at DL, *16.8 mrad* at SD and *24.3 mrad* at NC. The average interstorey drifts obtained from *Response-History Analyses* on *MRF-SF* were: *6.5 mrad* at DL, *13.1 mrad* at SD and *19.4 mrad* at NC. Finally, the maximum interstorey drifts obtained from *Response-History Analyses* with the most unfavourable accelerogram *#A1* on *MRF-SF* were: *6.8 mrad* at DL, *13.7 mrad* at SD and *21.1 mrad* at NC. Although the application of *Pushover* analyses on the frames led to more conservative results than those obtained with *Response-History Analyses*, the outcomes of nonlinear analyses revealed an overall acceptable seismic performance of *MRF-SF*, as the frame evidenced desirable behaviour at each Limit State:

- <u>at DL</u>: elastic response; interstorey drifts within the imposed limit of *7.5 mrad* on MRFs;
- <u>at SD</u>: development of plastic hinges at SF beam ends with deformations corresponding to pre-DL; interstorey drifts within the imposed limit of *20 mrad* on MRFs;
- <u>at NC</u>: plastic hinges in the SF beams end reached deformations corresponding to DL; development of plastic hinges at the 1st storey column bases with deformations corresponding to DL.

The average rotation demand at SD from *Response-History Analyses* was *13.1 mrad.* The maximum rotation demand at SD that resulted from applying *Response-History Analyses* with accelerogram *#A1* was *13.7 mrad.* Considering that the SF beam-to-column joint specimens attained a rotation of ± *45.35 mrad* at SD, and that both the average and the maximum rotation demands were *13.1 mrad*, *13.7 mrad*, respectively, an adequate seismic performance of the *MRF-SF* was proven. The rotation demand is lower than the available rotation of the SF beam-to-column joint.

Maximum plastic rotations within the plastic hinges of the SF beams at SD were obtained from

Pushover and *Response-History Analyses*. Thus, a plastic rotation of *7.7 mrad* was obtained from *Pushover*, and a value of *4.53 mrad* from *RHA* with accelerogram *#A1*. Considering the joint rotations that were determined on the envelope curve, the following rotations resulted: *16 mrad* at DL, *45.35 mrad* at SD and *60.5 mrad* at NC. If the elastic joint rotation of *16 mrad* was subtracted from the rotations at SD and NC, the plastic joint rotations would be *29.4 mrad* at SD and *44.5 mrad* at NC. As the maximum plastic rotation within the plastic hinges of the SF beam was *7.7 mrad*, which is significantly smaller than the plastic rotation capacity at SD of *29.4 mrad*, it is concluded that the plastic rotation demand is smaller than the plastic rotation capacity.

Followingly, a simplified evaluation of the steel use per frame accounting for the beams, columns and secondary beams on the longitudinal direction (corresponding to one bay of 6 m) was performed. In the case of *MRF-SF*, the steel use is *22.3 tons* or *51.5* kg/m^2. Oppositely, a steel use of *23.2 tons* or *53.7* kg/m^2 is obtained in the case of the *MRF-RF*. Overall, the steel use is *3.9 %* lower in the case of the *MRF-SF*. However, as this evaluation was performed on 2D frames, ignoring the rebars, the concrete and the steel decking, a more realistic approach would need to account for these aspects.

Ultimately, as the results of the nonlinear structural analyses evidenced, the integration of the SF beam into a lateral load-resisting system as the MRF was possible, and led to good results in the inelastic range. In order to develop an accurate model of the SF joint, the geometrical and mechanical characteristics need to be considered in the elastic domain. In the nonlinear range, the backbone curve should be constructed from the cyclic curve following the relevant provisions of the codes. Compared to the *MRF-RF*, a vertical space gain of *0.20 m* per story was gained in the *MRF-SF* due to the reduced height of the flooring system. Thus, for relatively the same seismic performance, a total of *0.80 m* in vertical space – corresponding to 4 storeys - were gained in the *MRF-SF*.

5.2 Concentrically-Braced Frame

The evaluation of the seismic performance of a 16-storey Concentrically-Braced Frame with slim-floor beam-to-column joints (*CBF-SF*) was performed. The seismic performance was assessed by means of nonlinear static and nonlinear dynamic analyses (e.g., *Pushover* with the *N2 method* [20], *RHA* with 7 accelerograms) with SAP2000 [19]. The aim of the case study on the *CBF-SF* was (i) to verify the rotation demand resulted from the seismic load on the SF beam-to-column joint, (ii) to compare the joint demand to the experimental rotation capacity and (iii) to assess the seismic performance of the *CBF-SF* with SF beam-to-column joints. In accordance with the aims, the following objectives were established:

- adaptation of the developed numerical model for the SF beam-to-column joint to the braced structural system;
- application of structural analyses in the nonlinear range, e.g., *Pushover* and *Response-History Analysis* to the subject frame;
- monitoring of the structural damage at the three Limit States, with particular interest at DL and SD, in terms of interstorey drifts and plastic rotation within the plastic hinges of the SF beams.

In general, the inelastic response of the SF beams from the central span of the *CBF-SF* was defined in a similar manner as in the study case on Moment-Resisting Frame with slim-floor beam-to-column joints. In comparison to the beam-to-column joint model used for the *MRF-SF*, the joint model for the *CBF-SF* was considered as rigid due to the integration in a braced frame. The stiffness of the joint was verified in accordance with prEN 1993-1-8 [7], the calculation allowing for the rigid classification. A rigorous approach was also taken to the modelling of braces. Based on information from the literature (e.g., D'Aniello et al. [22] [23], Dicleli and Calik [24]) and on several iterations, two models of the “X” braces were developed, e.g., phenomenological model (*P hinge*) for *Pushover* analyses and physical theory model (*P-M2-M3 fibre plastic hinge*) for *RHA*. Comparisons to an experimental force-deformation curve of a brace with the same cross section (brace specimen *SP59-1* from [25]) provided information on the reliability of the developed models, which was proven to be adequate.

The seismic performance of the *CBF-SF* was assessed based on the following criteria:

- development of global mechanism, which includes the history of plastic hinges, the location of plastic hinges and the value of the deformations sustained by plastic hinges in relation to the acceptance criteria of prEN 1998-1-2 [6];
- interstorey drifts values at DL and SD.

Depending on the type of nonlinear analysis, the seismic performance of the *CBF-SF* could be characterised differently. Based on the outcomes, the *Pushover* analyses provided more conservative results, which in terms of interstorey drifts, are similar to the ones obtained from applying *RHA* with accelerogram *#A4*, which was the most unfavourable. A good example of this is the interstorey drift at SD, which according to the results of the *Pushover* analyses, exceeded the limit of the code [6] (e.g., *18.5 mrad* interstorey drift > *15 mrad* limit at SD) and was similar to the maximum value resulted from *RHA* with *#A4*, e g., *19.7 mrad*. Judging solely by the *Pushover* results, the seismic performance of the *CBF-SF* could be improved at SD. However, the average results obtained by applying *RHA* with a set of 7 accelerograms, proved the contrary. The average interstorey drift values at DL and SD (e.g., *6.6 mrad* and *13.8 mrad*, respectively) obtained from *RHA*, which were within the imposed limits of the code, can be considered indicators of adequate seismic performance.

Another indicator of adequate seismic performance is the development of the global mechanism. In braced frames, the development of plastic hinges needs to occur in braces prior to other structural elements, as required by the seismic code. This condition is principally satisfied regardless of the nonlinear analysis applied to the frame. By the time plastic hinges within the dissipative zone of the SF beams attain pre-DL and DL deformations, extensive structural damage is already sustained by most of the braces.

The demands in terms of interstorey drifts on SF beam-to-column joints, as resulted from nonlinear static and dynamic analyses applied to the *CBF-SF*, are as follows: *18.5 mrad* at SD from *Pushover*; *13.8 mrad* at SD from *RHA* (average of 7 accelerograms). In this context, the available experimental rotation capacity of the SF beam-to-column joint of ± *45.35 mrad* at SD is higher than the interstorey drift demand that resulted from analyses on the *CBF-SF*. Plastic rotations within the plastic hinges of the SF beams at SD were obtained from *Pushover* and *Response-History Analyses*, as follows: *8.62 mrad* from *Pushover* and *8.82 mrad* from *RHA* with accelerogram *#A4*. Considering the experimental plastic rotation capacity of the dissipative zone of the SF beam of *29.4 mrad* at SD, the demand is considerably lower than the available rotation.

5.3 Dual Concentrically-Braced Frame

In the current case study, the evaluation of the seismic performance of a 16-storey Dual Concentrically-Braced Frame with slim-floor beam-to-column joints (*D-CBF*) was presented. The seismic performance evaluation was performed with nonlinear static and dynamic analyses, i.e., *Pushover* with *N2 method* and *Response-History Analysis*. The aim of the case study on the *D-CBF* was (i) to verify the rotation demand resulted from the seismic situation on the SF beam-to-column joint, (ii) to compare the joint demand to the experimental rotation capacity, (iii) to assess the seismic performance of the Dual Frame with SF beam-to-column joints and (iv) to verify whether the *D-CBF* can be re-centred. In accordance with the aims, the following objectives were established:

- implementation of the developed numerical model for the SF beam-to-column joint to the structural system;
- application of structural analyses in the nonlinear range, e.g., *Pushover* and *Response-History Analysis* to the subject frame;
- ensuring an adequate contribution of the MRF sub-systems to the total resistance of the Dual Frame;
- monitoring of the structural damage at the three Limit States, with particular interest to DL and SD, in terms of interstorey drifts and plastic rotation within the plastic hinges of the SF beams;
- assessment of the re-centring potential of the MRF sub-systems from the *D-CBF* following seismic events up to SD intensities.

The seismic performance of the *D-CBF* was assessed based on the following criteria:

- contribution of the MRF sub-systems to the total resistance of the D-CBF;
- development of global mechanism, which includes the history of plastic hinges, the location of plastic hinges and the value of the deformations sustained by plastic hinges;
- interstorey drifts at DL and SD.

The *25 %* contribution of the MRF sub-systems to the total resistance of the Dual Frame was verified using 3 methods, all of which having evidenced a sufficient strength / capacity of the unbraced spans. This allowed for the use of the upper limit value of the behaviour factor for Dual Frames with a CBF sub-system, i.e., *4.8*. The first method was based on existing formulae from different studies. As part of the second used method, a different approach to the evaluation of a CBF sub-system with rigidly-connected beams was proposed. In the proposed analytic approach, apart from the resistance of the braces, the resistance of the beams from the CBF sub-system can also be taken into consideration. The third method consisted of an individual resistance assessment of the sub-systems of the Dual Frame by means of nonlinear analyses. The obtained results are as follows:

- using method 1 (analytical): *43 %* contribution of two MRF sub-systems;
- using method 2 (analytical) : *31 %* contribution of two MRF sub-systems;
- using method 3 (nonlinear static analyses): *25 %* at DL, *35 %* at SD and *36 %* at NC contribution of two MRF sub-systems.

Although generally the *Pushover* results were more conservative than the average *RHA* results, both the results of nonlinear static and dynamic analyses evidenced an adequate seismic performance of the Dual Frame *D-CBF*. Similar to the case study on the *CBF-SF*, the *Pushover* analysis provided results that were comparable to those obtained by applying the most unfavourable accelerogram for *RHA*, which in this case was accelerogram was *#A3*.

Indicators of adequate seismic performance are the transitory interstorey drifts at DL and SD. As the obtained values were smaller than the seismic code limits [6], e.g., *7.5 mrad* at DL and *20.0 mrad* at SD, both the interstorey drift criteria were satisfied. The development of the global mechanism of the Dual Frame could be broadly characterised as follows:

- <u>at DL</u>: elastic except for some braces;
- <u>at SD</u>: DL and SD plastic hinges were developed in almost all braces, thereby satisfying the hierarchy of resistances required by the seismic code [6]; plastic hinges with deformations corresponding to pre-DL were developed in the SF beams of the MRF sub-systems and in a few SF beams of the CBF sub-system;
- <u>at NC</u>: half of the plastic hinges in the braces attained deformations corresponding to SD, NC or post-NC; plastic hinges were developed in most of the SF beams of the MRF

sub-systems, with deformations matching pre-DL and DL; plastic hinges were developed in some of the SF beams of the CBF sub-system; pre-DL and DL plastic hinges were evidenced in two column bases, but only by applying the most unfavourable accelerogram, *#A3*.

As the development of plastic hinges occurs in braces prior to other structural elements, the condition of the seismic code can be considered satisfied.

The demands in interstorey drifts on the SF beam-to-column joints, as resulted from nonlinear static and dynamic analyses applied to the *D-CBF*, were as follows: *17.0 mrad* at SD from *Pushover*; *11.7 mrad* at SD from *RHA* (average of seven accelerograms). Considering these results and the experimental rotation capacity of the SF beam-to-column joint of ± *45.35 mrad* at SD, it is evident that the demand is lower than the available rotation capacity. According to the results of both the nonlinear static and dynamic analyses, the highest plastic rotation demand on the SF beams resulted from the MRF sub-systems of the *D-CBF*. As the experimental plastic rotation capacity of the dissipative zone of the SF beam was *29.4 mrad* at SD, the maximum demand at SD was *9.54 mrad* (obtained from *RHA* with accelerogram *#A3*) – a value considerably lower than the available rotation capacity.

The re-centring capability of the Dual Frame *D-CBF* at DL and up to SD was verified by employing nonlinear structural analyses. The approach used for the verification allowed for the determination of the interstorey drifts at which: (i) the yielding of the SF beams from the MRF sub-systems is initiated and (ii) the ultimate deformation of the braces from the CBF sub-system is attained. As the yielding of the SF beams $\Delta_{y,MRF}$ occurs after the braces attain their corresponding ultimate deformation $\Delta_{u,CBF}$, this was considered to satisfy the requirement of prEN 1998-1-2 [6] at DL. In addition, residual interstorey drifts were calculated and compared to the acceptance criteria of FEMA 356 [26] for permanent drifts of braced structural systems. According to the obtained results, the permanent interstorey drifts corresponding to the *D-CBF* were within the limits of FEMA 356 [26].

6. Conclusions

The slim-floor is an alternative flooring system characterized by the integration of several components into one structural element. This means that the main structural member, the asymmetric steel beam, and other components are incorporated into the concrete slab. Due to the current typology of the slim-floor beam-to-column connections, e.g., pinned end connections, and to the fact that the slim-floor beams are designed exclusively for gravity loads in the elastic range, the flooring system is incompatible with the seismic design of frame systems. The current study is developed with the aim to provide a technical solution for slim-floor beam-to-column joints, which would make the shallow flooring system applicable to structures designed not only for medium, but also for high seismicity.

Main conclusions on the experimental program. Taking into consideration the full-strength and semi-rigid classifications of the joint, the joint rotation capacity of ± *45.4 mrad* at SD, the dissipative zone of the SF beam as the main source of energy dissipation, the stable and symmetric hysteretic response with low degradation of stiffness and capacity under cyclic loads, and the ductile failure mechanism, an adequate seismic performance was provided by the SF beam-to-column joint.

Main conclusions on the numerical program. The calibrated reference FE model of the SF beam-to-column joint, referred to as the *RM*, proved good compatibility with the monotonic curve (e.g., differences in the range of *0.1 ÷ 3.2 %*), supported the experimental findings in terms of the failure mechanism and main source of energy dissipation, and allowed for the development of a parametric study. Based on the results of the calibrated model, the *RM*, the ductile failure mechanism consisted of the development of a plastic hinge in the dissipative zone of the SF beam regardless of the bending direction. Although other joint components, such as the continuity plates and some of the bolts, had sustained localised plastic deformations, most of the phenomenon occurred in the dissipative zone of the SF beam. Thus, considering that the plastic deformation in the bolted connection was significantly lower and limited to a few finite elements, the response of the component was characterised as mainly elastic.

Main conclusions on the design and detailing procedure. The basis of the proposed design procedure is represented by the following: (i) some design rules from the pre-normative version of prEN 1994-1-1 for slim-floors and by (ii) the main principles of capacity design for steel and composite joints of MRF (DC3). In addition to these, a method for ensuring ductility of the SF beam through the implementation of a *Reduced Flange Section* was proposed herein. An adequate seismic performance of SF beam-to-column joints with similar cross section to the one tested in the current study, can be achieved by full-strength and rigid or semi-rigid joints, that can develop a joint rotation capacity of ± *40 mrad* at SD (according to AISC 341-16) or a joint plastic rotation of ± *30 mrad* (according to prEN 1998-1-2). Although the web panel is considered in the literature to be a source of plastic deformation, limitation of its contribution to the overall joint rotation is also advised.

Main conclusions on the structural modelling. The section dedicated to structural modelling and analysis is comprehensive, consisting of individual case studies on multistorey braced and unbraced frames with slim-floor beam-to-column joints. Considering the intended application of SF beam-to-column joints to the seismic design of DC3 frame systems, nonlinear static and dynamic analyses, e.g., *Pushover* with *N2*, *Response-History Analysis* with a set of seven accelerograms, were applied to four storeys unbraced and sixteen mid-rise braced frames using a software for structural analysis. The aim was to assess the seismic performance based on criteria such as: the plastic rotation demand on SF beams, the interstorey drift demand, an adequate contribution of the MRF sub-systems to the total resistance of the Dual Frame, the development of global ductile plastic mechanism, the re-centring potential of the Dual Frame following seismic events below SD intensity.

For reasons of reliability of the developed SF beam-to-column joint model, the modelling procedure was validated against the experimental data. Iterations on the modelling approach of the SF beam-to-column joint have shown that a rigorous approach, involving an accurate geometrical ("full" section, dissipative zone with *RFS*), material (tested materials) and mechanical ($I_{eq,full\ section}$, $I_{eq,RFS}$) modelling leads to the most realistic results. The optimal modelling procedure of the SF beam-to-column joint consisted in the definition of: (i) the inelastic response of the SF beam through a plastic hinge, (ii) of the bolted connection through a link element containing the stiffness of the tested bolted connection and (iii) of the web panel through a spring (containing the stiffness of the web panel) combined with end-length properties. To capture the rotation capacity of the composite SF beam, a plastic hinge model was defined based on the experimental plastic rotation of the SF beam and the surrounding concrete.

As the bolted beam-to-column connection was semi-rigid in the case study on the *MRF-SF* (Moment-Resisting Frame with SF beam-to-column joints), the component was explicitly modelled by a link element containing its stiffness. Considering the stiffness classification for joints of braced frames, a recalculation was made in the study cases on braced frames, the result allowing for the modelling of the connection as rigid. In the elastic range, the explicit modelling of the bolted connection and of the web panel influences the elastic stiffness of the SF beam-to-column joint, though to a limited extent. By comparison to a *MRF* of four storeys, in which the definitions of the connection and web panel were disregarded, only a decrease of approximately *2 %* in elastic stiffness could be determined.

Experimental data was used to check the reliability of the brace model, which was used in the case studies on braced frames, e.g., Concentrically-Braced Frame *CBF-SF* and Dual Concentrically-Braced Frame *D-CBF*. Iterations have shown that the brace model had to be adapted to the type of the nonlinear analysis. In consequence, a phenomenological model was used for the *Pushover* analysis and a physical theory model for the *Response-History Analysis* – both of which were previously validated against the experimental data corresponding to a brace with the same cross section.

Main conclusions on the seismic performance of the *MRF-SF*. The seismic performance of the *MRF-SF* was confirmed by the results of the nonlinear static and nonlinear dynamic analyses. On a macro or structural level, acceptable demands in terms of interstorey drifts (*PO*: *7.8 mrad* / average *RHA*: *6.5 mrad < 7.5 mrad* code limit) were obtained at DL. Doubled by satisfactory interstorey drifts at SD from *Pushover* and *RHA* (*PO*: *16.8 mrad* / average *RHA*: *13.1 mrad < 20 mrad* code limit), the criteria of the European seismic code for MRFs was fulfilled. In comparison to the experimental joint rotation capacity of ± *45.4 mrad* at SD, the average demand from *RHA*, e.g., *13.1 mrad*, is smaller. In conclusion, the demand was lower than the available rotation capacity.

Main conclusions on the seismic performance of the *CBF-SF*. An adequate seismic performance of the *CBF-SF* was demonstrated. In terms of interstorey drifts at DL and SD, both the limits of prEN 1998-1-2 were exceeded (*PO* at DL: *7.6 mrad > 7.5 mrad* code limit; *PO* at SD: *18.5 mrad > 15.0 mrad* code limit), thus accentuating the need for improvement in seismic performance. However, taking into consideration the average *RHA* results in terms of interstorey drifts, the need for improvement was contradicted, as neither of the code limits was exceeded (*RHA* at DL: *6.6 mrad < 7.5 mrad* code limit; *RHA* at SD: *13.8 mrad < 15.0 mrad* code limit). Based on the *RHA*'s enhanced ability to assess the seismic performance of relatively high structures, the criteria of prEN 1998-1-2 can be considered as fulfilled by the *CBF-SF*.

Main conclusions on the seismic performance of the *D-CBF*. The seismic performance evaluation of the Dual Frame, consisting of a central CBF sub-system adjoined by two outer MRF sub-systems, provided results which revealed an adequate response of the *D-CBF*. Consistent with observations on the *MRF-SF* and *CBF-SF*, the *Pushover* analysis led to more conservative results on the *D-CBF* than the average ones from *RHA*. The contribution of the MRF sub-systems to the overall resistance of the Dual Frame was verified and found to fulfil the *25 %* requirement of the European seismic code prEN 1998-1-2. In terms of interstorey drifts, both the *Pushover* and the average *RHA* results at DL and SD were within the acceptable limits (i.e., *7.5 mrad* at DL and *20.0 mrad* at SD) of prEN 1998-1-2. The average interstorey drift demands at DL and SD were *7.1 mrad* and *11.7 mrad*, respectively. Considering the available rotation capacity of the SF beam-to-column joint of ± *45.4 mrad* at SD, the resulted demand of *11.7 mrad* was considerably lower.

Taking into consideration the experimental results, the FEA results of the reference model and of the parametric study, the outcomes of the structural nonlinear static and nonlinear dynamic analyses on the *MRF-SF*, *CBF-SF* and *D-CBF*, the possibility to adapt the slim-floor beam-to-column joints to the requirements of the European seismic code [6] for Ductility Class 3 frame systems is confirmed. Capacity design principles are applicable to the slim-floor beam-to-column joints and adequate seismic performance is achievable if the seismic energy dissipation is directed to the ends of the beams, while the bolted connection, the adjoining welds and the web panel provide overstrength. For this purpose, full-strength and rigid / semi-rigid joint classifications should be met together with an adequate joint rotation capacity of ± *40 mrad* at Significant Damage.

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