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# DESIGN CRITERIA AND MODELLING OF RE-CENTRING DUAL ECCENTRICALLY BRACED FRAMES

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**Abstract.** *Conventional seismic design philosophy is based on dissipative response, which implicitly accepts damage of the structure under the design earthquake and leads to significant economic losses. Repair of the structure is often impeded by the permanent (residual) drifts of the structure. The repair costs and downtime of a structure hit by an earthquake can be significantly reduced by adopting removable dissipative members and providing the structure with re-centring capability. The paper presents application of this concept to a dual structure, obtained by combining steel eccentrically braced frames (EBFs) with removable bolted links and moment resisting frames (MRFs). The solution was validated experimentally using component and system tests. Structural design of re-centring eccentrically braced frames can be performed using conventional code-based approach, but some additional criteria need to be considered. Among these are (1) providing moment resisting frames with a minimum strength in order to obtain a dual frame; (2) checking the re-centring capability by keeping the moment-resisting frames in the elastic range and constraining plastic deformations to replaceable dissipative members; (3) choosing an appropriate behavior factor and (4) designing the connections of replaceable links to allow re-placement. Moreover, due to limitations of simplified code-based approach, nonlinear pushover or time-history analyses are recommended to validate the intended structural performance. Modelling approach of the system for nonlinear static and dynamic analysis is presented, and the results of seismic performance assessment are presented.*

# 1 INTRODUCTION

Conventional seismic design philosophy is based on dissipative response, which implicitly accepts damage of the structure under the design earthquake and leads to significant economic losses. Repair of the structure is often impeded by the permanent (residual) drifts of the structure. The repair costs and downtime of a structure hit by an earthquake can be significantly reduced by adopting removable dissipative members and providing the structure with re-centring capability. The paper presents application of this concept to a dual structure, obtained by combining steel EBFs with removable bolted links and MRFs.

Re-centring dual EBFs with replaceable bolted links were previously studied and developed by Politehnica University Timisoara (UPT) in the frame of several research projects.

# 2 SYSTEM DESCRIPTION

Most of the structures designed to modern codes would experience inelastic deformations even under moderate seismic action, with permanent (residual) displacements after an earthquake. Repair is difficult in such cases. Solutions providing self-centring of the structure exist, but are technically demanding (post-tensioned strands, shape memory alloy devices, etc.). An alternative solution is the one that provides re-centring capability (as opposed to self-centring), through removable dissipative members and dual (rigid-flexible) structural configuration.

Application of the concept of removable dissipative members to EBFs, where links act as dissipative zones, is presented in Figure 1. The link to beam connection is realized by a flush end-plate and high-strength friction grip bolts. The main advantage over other dissipative devices is that removable links can be designed using methods readily available to structural engineers and can be fabricated and erected using procedures standard to the profession.

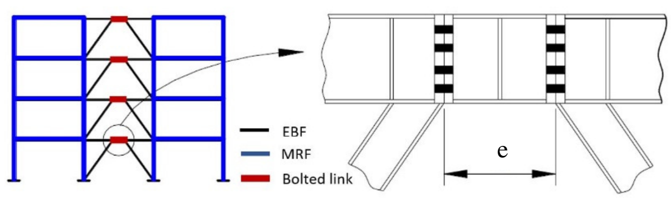



Figure 1. Replaceable link concept

The re-centring of the system is attained by designing the structure as a combination of EBFs and MRFs. The elastic response of the flexible subsystem (MRF) provides the restoring forces, once the links damaged during an earthquake are removed. For this principle to be efficient the flexible subsystem should remain in the elastic range.

The solution was validated experimentally using component and system tests. An experimental program was carried out at UPT, CEMSIG Research Centre, to determine cyclic performance of isolated bolted links [1-2] and another at the European Laboratory for Structural Assessment (ELSA) of the Joint Research Centre (JRC) in Ispra, Italy, to validate the proposed solution through a pseudo-dynamic testing campaign of a full-scale model of a dual EBF structure [3].

# 3 EXPERIMENTAL VALIDATION OF TECHNICAL SOLUTION

## 3.1 Component tests

An experimental program was carried out at UPT, CEMSIG Research Centre, to determine cyclic performance of bolted links [1-2], for which the experimental set-up of only isolated link is presented in Figure 2.

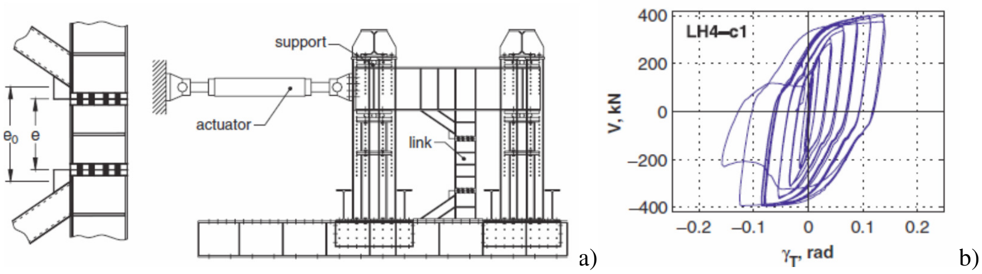



Figure 2. a) Experimental set-up and b) force–total deformation relationship $V$–$\gamma_T$ for specimen LH4-c1 [1].

The removable link was fabricated from IPE240 profile of S235 grade steel, while the rest of the structure – from S355 grade steel. Four link lengths were considered: $e_0$=400 (denoted with 4), 500 (denoted with 5), 600 (denoted with 6) and 700 (denoted with 7) mm, with "rare" (L) and "close" (H) spacing of stiffeners, and all links were classified as short ones according to AISC [4] and EN1998-1-1 [5]. During this experimental program, small height section links were investigated, the same as the ones from the DUAREM Project (240 mm section height). The complete ECCS 1985 [6] loading procedure was then applied, consisting of one monotonic (m) and two cyclic (c1 and c2) tests for each specimen. Meanwhile, for future investigations on links, AISC [4] has a dedicated loading protocol that is recommended.

The strategy adopted for the design of the flush-end plate connections was to provide sufficient over-strength of the connection over the link shear resistance. A reduction in total initial stiffness of the bolted link in comparison with the classical solution, as a result of both the semi-rigid end-plate and slip in the connection, was observed. Therefore, it was concluded that either explicit modelling of the semi-rigid connection behavior or consideration of equivalent link stiffness is necessary for global analysis of frames with bolted links.

| Specimen | LL7 | LL6 | LL5 | LL4 | LH7 | LH6 | LH5 | LH4 |
|---|---|---|---|---|---|---|---|---|
| m | 0.155 | 0.273 | 0.360 | 0.395 | 0.235 | 0.278 | 0.345 | 0.420 |
| c1 | 0.097 | 0.129 | 0.106 | 0.101 | 0.114 | 0.143 | 0.17 | 0.126 |
| c1 | 0.092 | 0.133 | 0.156 | 0.112 | 0.109 | 0.136 | 0.182 | 0.125 |

Table 1. Ultimate deformation $\gamma_{Tu}$.

Table 1 shows that cyclic loading reduced by 40% to 70% rotation capacity, with the maximum reduction for short links. Rotation capacity increases slightly for shorter links, with the exception of LL4 and LH4 specimens.

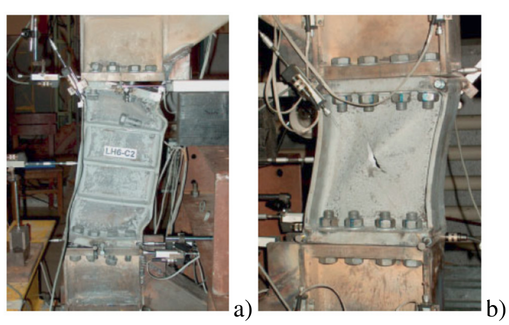



Figure 3. Failure by connection degradation at the LH6-c2 specimen (a) and plastic web buckling at the LL4-c1 specimen (b) [1].

The behavior of long specimens was much influenced by the response of the bolted connection (see Figure 3a), characterized by a gradual reduction in strength due to bolt thread stripping and a pinching cyclic response. The latter effect reduced the energy dissipated in the group of cycles of constant amplitude. Full bolt preloading reduced partially this effect. Response of short specimens was controlled by the shear of the link web (see Figure 3b), characterized by important hardening and energy dissipation capacity, but a more rapid degradation in strength after web tearing. Stiffener spacing had maximum importance for short links. Their effect was to limit plastic local buckling of the web, increasing the maximum force and deformation capacity, and providing a more stable cyclic response. However, after the attainment of ultimate deformation, failure of LH4 specimens was more rapid in comparison with LL4 specimens.

Therefore, choosing the link's length is of high importance, since in case of longer lengths ($e<1.6M_{p,link}/V_{p,link}$, where $M_{p,link}$ is the moment resistance of the link and $V_{p,link}$ is the shear resistance of the link) is difficult to dimension an elastic flush end-plate connection that might get damaged and make the replacement procedure more problematic, as opposed to using very short links ($e<0.8M_{p,link}/V_{p,link}$), as was the case of both JRC and UPT tests.

From available tests, bolted links specimens with rare stiffeners showed a stable deformation capacity of at least 0.09 rad, while the ones with close stiffeners showed a stable deformation capacity of at least 0.11 rad. In case of LH5 specimens, with a length $e=0.8M_{p,link}/V_{p,link}$, the ultimate deformation capacity reached a value of at least 0.17 rad.

### 3.2 System tests

The validation of the proposed solution was realized through a pseudo-dynamic testing campaign of a full-scale model of a dual EBF structure at the European Laboratory for Structural Assessment (ELSA) of the Joint Research Centre (JRC) in Ispra, Italy within FP7 SERIES DUAREM Project ("Full-scale experimental validation of dual eccentrically braced frame with removable links").

The test specimen in presented in Figure 4. There are 2 central EBFs and 4 MRFs on test direction that represent the lateral load resisting system.

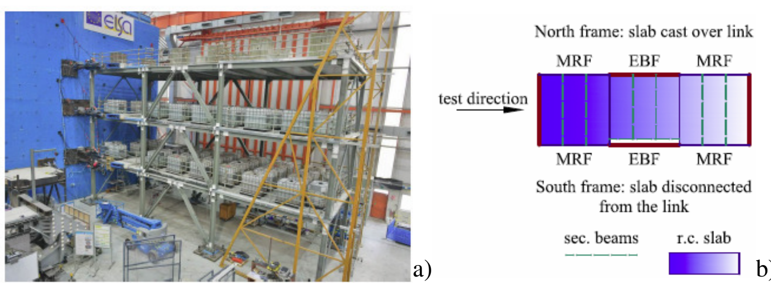



Figure 4. The test specimen: a) 3D view; b) plan layout [3]

Steel structural components were designed in S355 grade steel, with two exceptions. Grade S460 steel was used for columns and links were designed from S235 steel grade.

The testing sequence on the mock-up in the reaction wall facility of ELSA consisted in pseudo-dynamic (PsD) tests, together with some monotonic and link replacement tests [3].

One ground motion record was chosen (from seven selected by matching the elastic response spectrum used in design) to be used in the pseudo-dynamic tests in order to evaluate the structural performance of the test structure, applied with several input levels (see Table 2, where $a_{gr}$ is the reference peak ground acceleration and $a_g$ represents the peak ground acceleration for a specific earthquake level):

| Limit state | Performed PsD tests | Return period, years | Probability of exceedance | $a_g/a_{gr}$ | $a_g/g$ | Additional monotonic tests |
|---|---|---|---|---|---|---|
| Full Operation | FO1, FO2, FO3 | - | - | 0.062 | 0.020 | |
| Damage Limitation | DL | 95 | 10% / 10 years | 0.59 | 0.191 | |
| Significant Damage | SD | 475 | 10% / 50 years | 1.00 | 0.324 | PO1 |
| Near Collapse | NC | 2475 | 2% / 50 years | 1.72 | 0.557 | PO2, PO3 |

Table 2. Limit states and corresponding scaling factors for seismic input.

The testing program was completed with two link replacement tests:

• First link replacement (LR1) – after DL test, where links were removed from the structure by unscrewing the bolts;

• Second link replacement (LR2) – after PO1 test, where links were removed by means of flame cutting with a torch;

FO tests were performed in order to assess the elastic response of the structure with each new set of links, before the main DL, SD and NC PsD tests, the selected seismic record being scaled to have the PGA of 0.02g. During these tests, the structure manifested an elastic response.

DL test was performed, in order to simulate a moderate earthquake, causing moderate structural damage, the selected seismic record being scaled to have the PGA of 0.191g. SD test was performed in order to simulate a stronger earthquake, causing larger structural damage, the selected seismic record being scaled to have the PGA of 0.324g. During these tests, no yielding was observed in the elements outside links and small to moderate maximum plastic deformations occurred in links. Minor to moderate cracks were observed in the concrete slab. The structure exhibited low residual top displacement. Also low residual inter-story drifts were observed.

PO1 test (a monotonic pushover test until an additional displacement of 55 mm) starting from the end of the SD test position was necessary. This was done to obtain larger residual displacements that were necessary in order to validate the feasibility of the link removal process and re-centring of the structure. During this test, no yielding was observed in the elements outside links. Higher maximum plastic deformations occurred in links (see Fig. 3). More visible cracks were observed in the concrete slab (see Figure 5). After this test, the structure exhibited significant larger residual top displacement. Larger residual inter-story drifts amounting were observed.

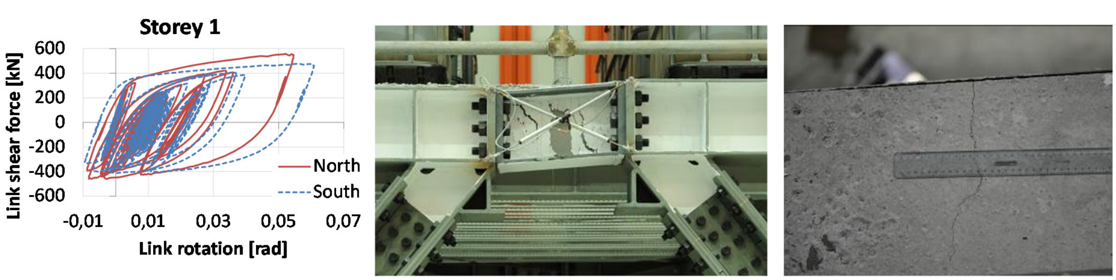



Figure 5. PO1 test results [3]

Because after the DL test the structure exhibited low residual top displacement and low residual drifts were observed, the decision was to remove the first set of damaged links, by removing the bolts, on a level by level basis, starting from the lower level to the upper one. The low value of the residual top displacement from the end of the DL test decreased after the elimination of the damaged links. A new set of unused links was then mounted into the structure.

Because after the PO1 test the structure exhibited significant larger residual top displacement and larger residual drifts were observed, the decision was to remove the second set of damaged links, by flame cutting both the web and flanges of the links, from the top story downwards. The value of the residual top displacement from the end of the PO1 test was decreased after the elimination of the damaged links. A new set of unused links was then mounted into the structure.

NC test was proposed in order to simulate a much stronger earthquake and to obtain extensive damage throughout the structure, the re-centring capability being lost due to yielding in other members apart from the links, the selected seismic record being applied with a scaling factor of 0.557. This test was prematurely stopped because the available actuator capacity (1000 kN per frame at every floor) was not enough to carry it out with the imposed null torsion at every floor.

Another cyclic pushover test (PO2) with maximum displacement amplitudes of 150 mm was further proposed after the actuators' release of force from the NC test and afterwards a final cyclic pushover test (PO3) with maximum displacement amplitude of 400 mm. The last three tests brought extensive plastic behavior throughout the entire structure (see Figure 6).

Figure 6. State of the specimen after the last test [3]

The maximum link demand after PO1 test is of 0.075 rad (see Table 3), smaller than the acceptable criterion for this limit state adopted by FEMA356 [7], which is of 0.11 rad.

| Test | DL | SD | PO1 |
|---|---|---|---|
| Maximum link rotation [rad] | 0.032 | 0.061 | 0.075 |
| Residual link rotation [rad] | 0.014 | 0.022 | 0.066 |

Table 3. Deformation demands for links.

After the DL test, the structure exhibited a low residual top displacement of 5 mm (0.05%), the maximum top displacement being 32 mm. Also, a low residual inter-story drift amounting to a maximum of 3 mm (less than 0.1%) was observed. At the end of the LR1 procedure, a very small residual drift (H/5250 for both frames) that is lower than the erection tolerance (H/300) was observed, the structure being almost re-centred.

After the completion of PO1 test, the structure exhibited a significantly larger residual top displacement of 45 mm (0.43%), the maximum top displacement being 68 mm. Larger residual inter-story drift amounting to a maximum of 18 mm (0.5%) was observed. At the end of the LR2 procedure, a small residual drift (H/5250 for the south frame and H/1750 for the north frame) that was lower than erection tolerance was observed.

## 4 DESIGN PROCEDURE FOR EBFS WITH REPLACEABLE LINKS

A flowchart that briefly illustrates the design of EBFs with removable links and re-centring capability is shown in Figure 7.

Structural design of re-centring eccentrically braced frames can be performed using conventional code-based approach, but some additional criteria need to be considered.

Firstly, a capacity design can be performed, according to Eurocodes. The dissipative behavior concept is recommended, A global dissipative behavior of the structure should be achieved, checking that the individual values of the ratios Ωi for each dissipative member not to exceed the minimum value Ω by more than 25%.

Additional to current design provisions according to EN1998-1 [5], the investigated structural system has to meet the following requirements.

(1) the structure should be dual (providing moment resisting frames with a minimum strength) and possess re-centring capability (pre-designed using approximate formulas and confirmed through nonlinear analyses) after link elimination;
(2) the links should be replaceable, using bolted elastic connections that must possess an over-strength with respect to links;
(3) re-centring capability should be checked by keeping the moment-resisting frames in the elastic range and constraining plastic deformations to replaceable dissipative members until reaching the ultimate shear deformation in links;
(4) an appropriate behavior factor should be chosen.

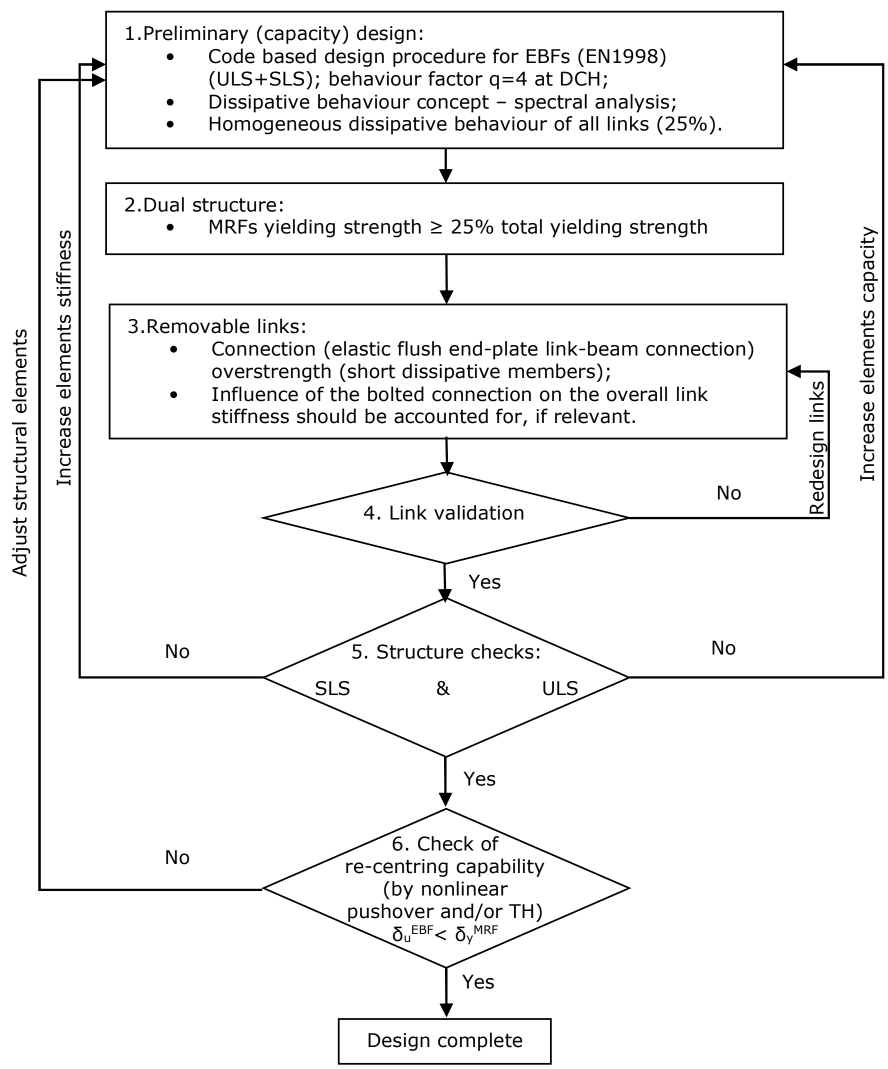


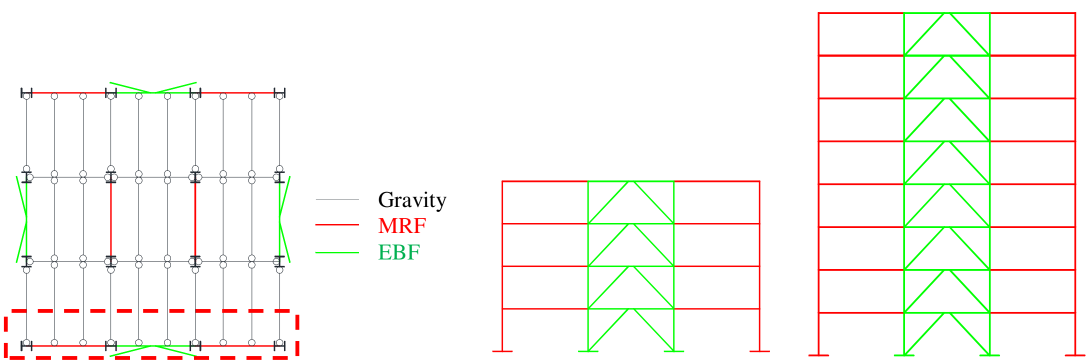

Figure 7. Design flowchart for EBFs with removable links and re-centring capacity

All the additional design requirements presented above lead to the necessity of short removable links prequalification (validation) [8]. Since there's only a limited number of experimental tests on short bolted links involving this type of connection (flush-end plate) and link section (I section with 240 mm height), when using other types of sections and connections it is recommended to confirm the links performance through experimental validation and/or numerical testing.

## 5 CASE STUDY

### 5.1 Description of examined building frames

The case study presented hereafter consists of designing and analyzing 2 four-story and 2 eight-story buildings. The common plan view for the buildings is presented in Figure 8a. The number of bays in both directions is 3, with a span length of 8m. The height of each story is 4m. The main lateral load resisting system is composed of four MRFs and two EBFs on transversal direction and two MRFs and two EBFs on longitudinal direction. The marginal frames on transversal direction consist of dual steel frames, combining two moment resisting frames (MRFs) (which provide the necessary re-centering capability to the structure, assuring the restoring forces after an earthquake) with one central eccentrically braced frame (EBF) with replaceable bolted links (which are intended to provide the energy dissipation capacity and to be easily replaceable) (Figure 8b-c). All the other frames are gravitational loads resisting systems (with pinned HE200A composite steel-concrete beams). The main beams, columns and braces are made of European I-sections (IPE, HEA, HEB and HEM type), while the removable links are made of welded I-sections. The material used for structural elements is S355 steel.




a) b) c)

Figure 8: Structures description: a) plane configuration, b) 4-story frame elevation and c) 8-story frame elevation.

The gravity loads were applied as uniform distributed loads on the secondary beams and reduced to concentrated loads on the main frames. The dead load takes into account the composite slab and steel sheeting, resulting 2.75 kN/m$^2$. There were considered some superimposed loads from services, ceilings and raised floors. A 4.0 kN/m$^2$ was taken into account for perimeter walls. The live load takes into account the destination of the buildings (offices - class B) and movable partition walls, resulting 3.8 kN/m$^2$. All gravitational loads assigned to the analyzed frames correspond to half the bay (4m). Two different design cases are taken into account: moderate (considering DCM) and high (considering DCH) seismicity cases. Type 1-C spectrum was selected for design considering two peak ground accelerations: 0.3g for high seismicity case and 0.15g for moderate seismicity case.

A behavior factor q=4 was adopted for DCH. In case of DCM a behavior factor of 2.5 has been considered.

## 5.2 Structural analysis

The modelling, analysis and design of the buildings, was performed with the finite element program SAP2000 [9]. The structural model was a linear-elastic 2D model with beam elements of the perimeter frame.

Rigid diaphragms were assigned at each level to account for the effect of reinforced concrete slabs.

The structural masses (in tons) considered from half of the total bay of the structure (12m) were assigned in the frame's structural nodes, since only the marginal frames resist to lateral loads (Fig. 6.3)

Multi-modal response spectrum analysis was performed. The first two modes, for 4-story buildings and the first three modes, for 8-story buildings, activated more than 90% of the mass.

Global imperfections were considered in the structural analysis, according to EN1993-1-1 [10], through equivalent lateral forces $H_i$, from combination $1.35 \cdot G + 1.5 \cdot Q$. These forces were computed based on total gravitational loads and initial global imperfection $\phi$, level by level, and considered in every load combination further on. Small lateral equivalent forces were obtained, of 8.7 kN for current levels and 8.9 kN for roof level.

Second order effects were not accounted for in design because the inter-story drift sensitivity coefficient $\theta$ was computed according to EN1998-1-1 and found to be smaller than 0.1.

## 5.3 Conventional code-based design

MRFs were designed from fundamental Ultimate Limit State (ULS) design load combination $1.35 \cdot G+1.5 \cdot Q$. IPE330 sections were obtained for beams, HE160B sections for columns of 4-story buildings and HE200B sections for columns of 8-story buildings.

Beams deflections were checked from fundamental Serviceability Limit State (SLS) design load combination $1.0 \cdot G+1.0 \cdot Q$. They had to be increased to IPE360 to have deflections less than $L/350$ (22.86mm).

Further on are presented conditions that need to be fulfilled for seismic design in accordance with rules described in EN 1998-1-1 [5].

Shear links are the dissipative elements of the system. They are designed from welded (h x b x tf x tw) class 1 I-sections and dimensioned from the following governing seismic load combination: $1.0 \cdot G+0.3 \cdot Q+1.0 \cdot S$.

For each building, a homogeneous dissipative behavior was ensured between links (25%). The structural over-strength was computed as:

$$\Omega=\gamma_{ov}\ \Omega_i \tag{1}$$

$$\Omega_i = \gamma_{sh} \frac{V_{p,link,i}}{V_{Ed,i}}$$

where $\gamma_{ov}$ is 1.25 and $\gamma_{sh}$ was adopted 1.8 for DCH (according to DUAREM project [3] results) and 1.5 for DCM.

EBFs columns, braces and beams are the non-dissipative elements of the system and were designed from the seismic load combination that provides over-strength ($\Omega$) to these elements with respect to dissipative ones: $1.0 \cdot G+0.3 \cdot Q+\Omega \cdot S$.

Columns should satisfy the "weak beam-strong column" condition:

$$\sum M_{Rc} \geq 1.3 \sum M_{Rb} \tag{2}$$

where: $\Sigma M_{Rc}$ is the sum of upper and lower columns moment resistance and $\Sigma M_{Rb}$ is the moment resistance of the MRF beam.

Inter-story drifts were checked from the seismic SLS combination: $1.0 \cdot G + 0.3 \cdot Q + \nu \cdot q \cdot S$ (where $\nu$=0.5). The check is verified for all stories, with values much lower than the limit value 30mm (0.0075$h$, where $h$ is the story height).

### 5.4 Additional design provisions

The links should be designed as removable. This can be done by using a flush end-plate link-beam connection that should be kept elastic. This means that the connection should have a design shear force $V_{j,Ed}$ and bending moment $M_{j,Ed}$ corresponding to a fully yielded and strain hardened link, computed as follows:

$$V_{j,Ed} = \gamma_{sh} \gamma_{ov} V_{p,link} \tag{3}$$

$$M_{j,Ed} = \frac{V_{j,Ed} e}{2}$$

In order to achieve the connection over-strength, very short dissipative members were adopted (with length e as small as $0.8 M_{p,link}/V_{p,link}$). Therefore, links have lengths of 0.5 m in case of 4-story buildings and 0.9 m in case of 8-story buildings.

It was considered that the bolted connection has no influence on the overall link stiffness because of preloading of bolts.

The consequence of using a non-dissipative flush end-plate bolted connection is represented by the necessity of having very short links (as short as $e = 0.8 M_{p,link}/V_{p,link}$). This leads to larger deformation demands in links under the design seismic motion. As a result, EBF frames with very short links fail to fulfil the performance requirements at ULS when designed for a behavior factor q=6 at DCH. In order to reduce these requirements and obtain acceptable performance is necessary to limit the behavior factor q at 4 at DCH.

The duality of the structure should be checked by verifying that the MRFs should be able to resist at least 25% of the total seismic force [11-12]:

$$F_y^{MRF} > 0.25\, (F_y^{MRF} + F_y^{EBF}) \tag{4}$$

$$F_y^{EBF} = \frac{L}{H} V_{p,link}$$

$$F_y^{MRF} = \frac{4 M_{pl,b}}{H}$$

where: $F_y^{MRF}$ is the yield strength of MRF, $F_y^{EBF}$ is the yield strength of EBF, $L$ is the frame span, $H$ is the frame story height, $V_{p,link}$ is the shear strength of the link and $M_{pl,b}$ is the beam plastic moment.

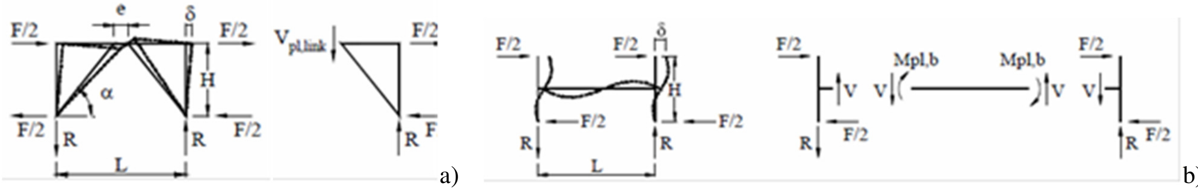



Figure 9: Basic one-story a) EBF and b) MRF components [13].

In order to have dual frames, the MRFs beams were increased. MRFs beams from transversal plane are larger because there are only 2 MRFs on that direction (Figure 8d).

Final sections from code-based design and additional recommendations are presented in the following tables:

| Ductility class | Story | Links | Braces | Beams | Columns | MRFs beams | MRFs columns |
|---|---|---|---|---|---|---|---|
| DCH | 1 | 350x190x18x9 | HE280B | HE360A | HE320B | IPE400 | HE240B |
| | 2 | 350x190x18x9 | HE280B | HE360A | HE320B | IPE400 | HE240B |
| | 3 | 290x190x16x8 | HE240B | HE300A | HE300B | IPE360 | HE220B |
| | 4 | 230x140x16x6 | HE200B | HE240A | HE300B | IPE360 | HE220B |
| DCM | 1 | 350x130x18x6 | HE240B | HE360A | HE260B | IPE400 | HE240B |
| | 2 | 350x130x18x6 | HE220B | HE360A | HE260B | IPE400 | HE240B |
| | 3 | 290x140x16x6 | HE220B | HE300A | HE240B | IPE360 | HE220B |
| | 4 | 230x120x16x5 | HE180B | HE240A | HE240B | IPE360 | HE220B |

Table 4. 4-storey frame sections.

| Ductility class | Story | Links | Braces | Beams | Columns | MRFs beams | MRFs columns |
|---|---|---|---|---|---|---|---|
| DCH | 1 | 490x260x20x8 | HE320B | HE500A | HE340M | IPE450 | HE260B |
| | 2 | 490x260x20x8 | HE320B | HE500A | HE340M | IPE450 | HE260B |
| | 3 | 440x230x20x7 | HE300B | HE450A | HE300M | IPE400 | HE240B |
| | 4 | 440x230x20x7 | HE280B | HE450A | HE300M | IPE400 | HE240B |
| | 5 | 390x200x20x6 | HE280B | HE400A | HE300B | IPE360 | HE220B |
| | 6 | 390x200x20x6 | HE260B | HE400A | HE300B | IPE360 | HE220B |
| | 7 | 330x210x16x5 | HE240B | HE340A | HE280B | IPE360 | HE220B |
| | 8 | 250x190x14x4 | HE200B | HE260A | HE280B | IPE360 | HE220B |
| DCM | 1 | 440x230x20x7 | HE260B | HE450A | HE300M | IPE400 | HE240B |
| | 2 | 440x230x20x7 | HE260B | HE450A | HE300M | IPE400 | HE240B |
| | 3 | 390x220x18x6 | HE260B | HE400A | HE280M | IPE360 | HE220B |
| | 4 | 390x220x18x6 | HE240B | HE400A | HE280M | IPE360 | HE220B |
| | 5 | 350x220x18x6 | HE220B | HE360A | HE280B | IPE360 | HE220B |
| | 6 | 330x210x16x5 | HE220B | HE340A | HE280B | IPE360 | HE220B |
| | 7 | 290x210x16x5 | HE200B | HE300A | HE260B | IPE360 | HE220B |
| | 8 | 210x190x14x4 | HE180B | HE220A | HE260B | IPE360 | HE220B |

Table 5. 8-storey frame sections.

### 5.5 Seismic performance

In order to verify the re-centring capability of EBFs with removable links, the ultimate displacement of the EBFs ($\delta_u^{EBF}$) at ultimate limit state (ULS) (corresponding to the plastic deformation capacity of the link) must be smaller than the yield displacement of the MRFs ($\delta_y^{MRF}$), meaning the yielding in MRFs is prevented up to the attainment of ultimate deformation capacity in the EBFs with removable links. Nonlinear static and/or dynamic analyses are recommended in order to check the re-centring capability.

Pushover (PO) analyses were performed on both 4-story and 8-story frames, considering a modal distribution of lateral forces. P–Delta effects were also included in PO analyses.

A leaning column was modelled in order to account for the gravitational loads from the remaining half of structure (8m) that were not considered on the analyzed frames (Figure 10).

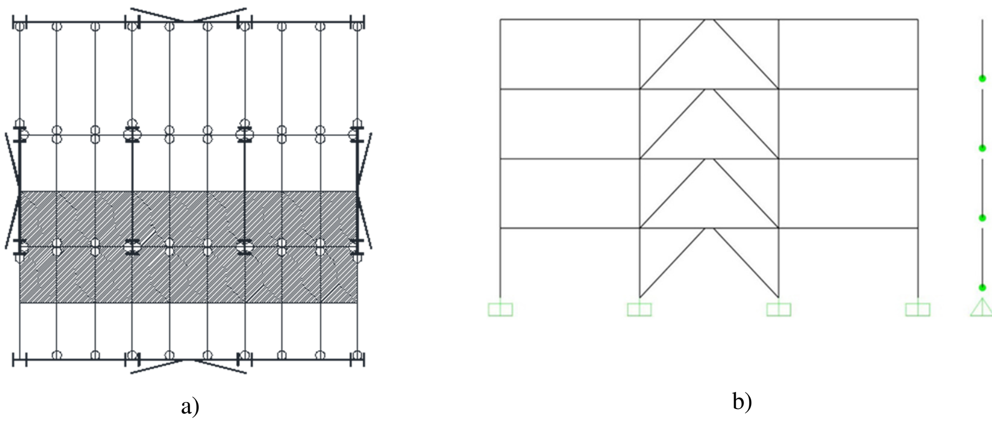



Figure 10: Leaning column approach: a) not yet considered gravitational loads and b) leaning column modelling.

Expected (based on $\gamma_{ov}$=1.25) material properties were used for dissipative elements and nominal material properties for non-dissipative elements.

Nonlinear plastic hinges of bending type M3 were assigned at the ends of the MRFs beams and of bending with axial force type P-M3 at the ends of columns and EBFs beams. For braces, nonlinear plastic hinges of axial type P were used, being assigned at the middle of members. These properties were calculated according to ASCE41-13 [14].

In order to account for the short links nonlinear behavior in shear (V2), MultiLinear Plastic Link elements were defined, with nonlinear behavior on 2-direction, described by the following backbone curve (Figure 11).

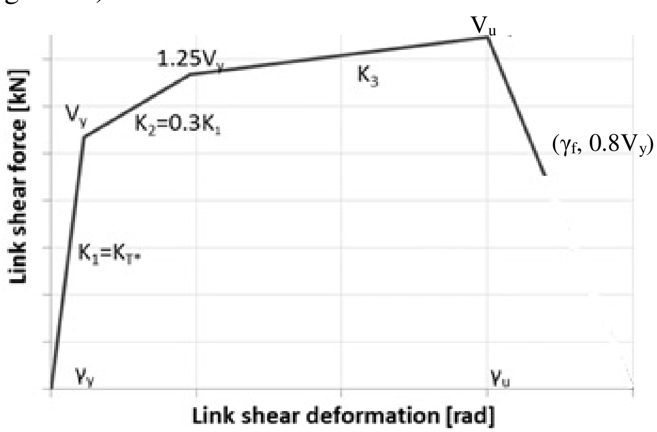



Figure 11: Shear links nonlinear behavior.

where: $K_1$ is the initial (elastic) stiffness of the link (accounting for shear and bending stiffness), $V_y$ is the shear resistance of links ($V_{p,link}$), $V_u$ is considered 1.8$V_y$ in case of DCH and 1.5$V_y$ in case of DCM, $\gamma_u$ is the ultimate shear rotation considered 0.15 rad for DCH and 0.1 rad for DCH and $\gamma_f$ was considered 0.17 rad for DCH and 0.11 rad.

2-Joint Link elements were drawn between end-joints of every link and the short links bars were pinned at the ends.

After running the PO analyses on elastic designed frames, in case of 4-story and 8-story buildings, at DCH, yielding was observed in MRFs before the attainment of ultimate deformation capacity in the EBFs with removable links. Therefore, some sections were replaced as follows: for the 4-story frame, only EBF columns sections were changed (from HE320B to HE280M and from HE300B to HE280B) and for the 8-story frame, MRFs were made from

S690 steel (therefore sections were slightly decreased: for EBFs columns from HE340M to HE300M, from HE300M to HE260M, from HE300B to HE260B and from HE280B to HE240B and all beams to IPE360 and lateral columns to HE220B).

It was observed that no yielding in any other structural elements appears before reaching 0.15 rad in links at DCH and 0.1 rad at DCM. For DCH frames, when peak link rotation reaches 0.15 rad, full plastic mechanism is attained with plastic rotations in other links ranging between 0.102 rad and 0.128 rad for 4-story frame and between 0.066 rad and 0.149 rad for 8-story frame. For DCM frames, when peak link rotation reaches 0.1 rad, other links show deformations ranging between 0.061 rad and 0.094 rad for 4-story frame and between 0.024 rad and 0.095 rad for 8-story frame.

Pushover curves for all frames are presented in Figure 12.

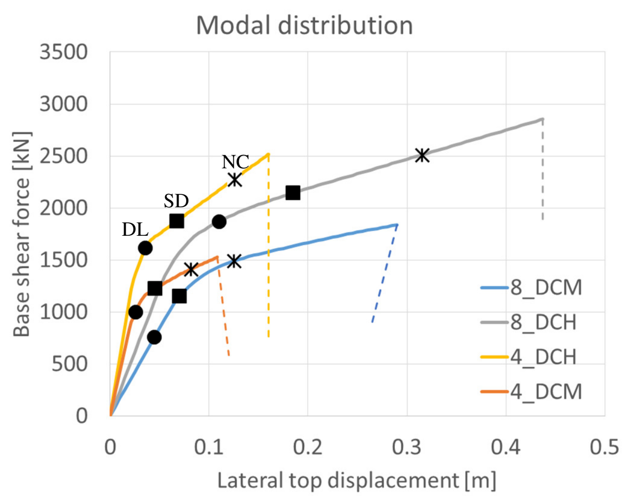



Figure 12: Pushover curves.

The curves are represented until the maximum capacity (when the links reach their ultimate shear deformation), because convergence was not attained in case of using the adopted approach for link modelling.

The frames designed assuming DCH, have a larger capacity and ductility than the ones designed assuming DCM. The 8-story frames are more ductile than the 4-story frames and were designed to resist similar seismic forces (within the same ductility class).

Seismic performance of the studied frames was assessed using the N2 method [15], with the bilinear idealization of the equivalent single degree of freedom system to match the initial stiffness of the system (P100 [12] approach).

Structural performance was evaluated for the limit states shown in Table 2 (DL, SD and NC). Target displacements ($D_t$) were computed for each limit state and corresponding link rotations are presented below.

| Ductility class | Limit State | $D_t$ [mm] | Link rotation at $D_t$ [rad] | Acceptance rotation [rad] | Corresponding top displacement [mm] |
|---|---|---|---|---|---|
| | DL | 36.8 | 0.016 | 0.005 | 23.5 |
| DCH | SD | 69.7 | 0.053 | 0.14 | 151.5 |
| | NC | 127.4 | 0.115 | 0.16 | - |
| | DL | 26.5 | 0.009 | 0.003 | 19.5 |
| DCM | SD | 46.3 | 0.037 | 0.09 | 99.5 |
| | NC | 81.3 | 0.074 | 0.11 | - |

Table 6. Target displacements for 4-storey frame.

| Ductility class | Limit State | $D_t$ [mm] | Link rotation at $D_t$ [rad] | Acceptance rotation [rad] | Corresponding top displacement [mm] |
|---|---|---|---|---|---|
| | DL | 107.3 | 0.031 | 0.005 | 59.9 |
| DCH | SD | 182.2 | 0.062 | 0.14 | 404.9 |
| | NC | 313.3 | 0.109 | 0.16 | - |
| | DL | 41.9 | 0.003 | 0.003 | 44.9 |
| DCM | SD | 71.2 | 0.009 | 0.09 | 259.9 |
| | NC | 122.4 | 0.036 | 0.11 | - |

Table 7. Target displacements for 8-storey frame sections.

The performance objectives are accomplished for SD and NC limit states. Although the performance objectives are not satisfied for DL using the N2 approach, the objective of having no yielding in the MRFs before the attainment of the SD deformation in the removable links (0.14 rad) of the EBFs is accomplished, representing the basic design requirement for dual frames with removable dissipative members. MRFs provide the re-centring of the specimen until the links ultimate deformation (0.15 rad).

## 6 CONCLUSIONS

The dual eccentrically braced structure showed an excellent performance at the SLS and ULS earthquakes within experimental program. Small permanent deformations were recorded for both seismic intensity levels, which are within the erection tolerance limits defined in EN 1090. This behavior occurs mostly due to the large post-elastic stiffness of the system, provided by the MRFs. Small permanent deformations effectively mean that the structure is self-centring to a certain degree.

A design procedure is presented and provides recommendations for re-centring dual EBFs with replaceable links. Additionally to existing current code specifications for designing this system, specific design provisions were applied in this paper in order to ensure the re-centring capability and duality of the case study frames.

Short removable links validation is necessary. Since there’s only a limited number of experimental tests on short bolted links involving this type of connection (flush-end plate) and link section (I section with 240 mm height), when using other types of sections and connections it is recommended to confirm the links performance through experimental validation and/or numerical testing based on parametric studies.

Nonlinear static and/or dynamic analyses are recommended in order to check the re-centring capability of the system. Seismic performance of the studied frames was assessed using the N2 method. Performance objectives are accomplished for SD and NC limit states. Although the performance objectives are not satisfied for DL using the N2 approach, the objective of having no yielding in the MRFs before the attainment of the SD deformation in the removable links (0.14 rad) of the EBFs is accomplished, representing the basic design requirement for dual frames with removable dissipative members.

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