On the weak storey behaviour of concentrically braced steel frames

D.B. Merczel 1,2, H. Somja 1, J.-M. Aribert 1, M. Hjijaj 1, J. Lőgő 2
1 Institut National des Sciences Appliquées de Rennes, Laboratory of Structural Mechanics, 20 Av. des Buttes de Coesmes
35000 Rennes, France
2 Budapest University of Technology and Economics, Department of Structural Mechanics, Műegyetem rkp. 3-9.
1111 Budapest, Hungary
email: dmerczel@insa-rennes.fr, hsomja@insa-rennes.fr, aribert@insa-rennes.fr, mhjijaj@insa-rennes.fr, logo@ep-mech.me.bme.hu

ABSTRACT: This article deals with concentrically braced frames that are prone to exhibit weak storey behavior in the case of a seismic event. After the introduction of the key features of seismic design of braced frames according to Eurocode 8 the causes of the weak storey behaviour are presented. This is followed by the description of new design criteria that aim to prevent the occurrence of weak stories. The suitability of the new method is proved by numerical examples.

KEY WORDS: concentrically braced frame; seismic design; weak storey; wavelet transform; plastic analysis

1 INTRODUCTION
Concentrically braced frames (hereinafter referred to as CBF-s) are among the most common structural systems for resisting lateral forces. They represent an economical structural form for providing lateral resistance for various types of actions. For wind effects, low return period seismic actions or for seismic actions in buildings of a high importance class the design behaviour is elastic. Due to their geometry the CBF-s counteract the horizontal forces by truss action that entails primarily axial forces in the members. The large resistance and stiffness limits the drifts and vibrations in the braced buildings, therefore provides occupancy comfort and impedes damages in the non-structural parts. Therefore CBF-s may be favoured over moment resisting frames [1][2] as MRF-s exhibit larger deformations that implies second order effect problems and excess damages.

On the other hand CBF-s are generally considered to have a worse performance than MRF-s [1][3] in case of a more destructive seismic event, where the return period of the earthquake is significantly larger than the expected life cycle of the building. In CBF-s the large majority of the connections and splices are pinned as the structure is stiff and stable without moment resisting connections. Consequently the level of statical indeterminacy is low, thus only a limited number of members, mostly the braces, can be dissipative members in a CBF. Furthermore it has to be ensured that the braces undergo plastic deformations on each floor in order to utilize the maximum available capacity of dissipation. Because of these adversities the allowed behaviour factor used to be small. The current seismic design practice and codes penalize the non-dissipative members with high overstrength demand, in compliance with the principles of capacity design, to safeguard the occurrence of plasticity in the braces only.

In the following the structure specific requirements of Eurocode 8 and several multi-storey CBF structures designed respecting these provisions are presented. This is followed by the analysis of the dynamic behaviour of CBF-s when subjected to earthquakes. Upon the description of the behaviour a redesign method is elaborated, the results of which are presented at the end of the article.

2 SEISMIC DESIGN OF CBF-S BY EUROCODE 8
2.1 Eurocode 8 design criteria for CBF-s
In the design of building structures Eurocode 8 [4], provides two basic concepts for the seismic analysis. Earthquake resistant buildings can either have a low-dissipative or a dissipative behaviour. In a dissipative structure controlled inelastic deformations are expected to dissipate considerable energy and damp the response of the structure. The behaviour factor that accounts for this effect is 4 for CBF-s, which is the lower boundary of high dissipation class (DCH) structures.

In Eurocode 8 the criteria for the verification of dissipative structures follow the capacity design philosophy. Plastic deformations are strictly required to occur in dissipative members that are to be designed with sufficient ductility, whereas the yield or collapse of non-dissipative zones is to be evaded. To ensure the elastic behaviour of non-dissipative zones their seismic requisitions are amplified in the design with a sufficient overstrength. In CBF-s the braces are meant to be the dissipative members. Therefore CBF-s are designed so that the yield of the diagonals takes place before the failure of the connections or the columns or beams.

For the sake of homogeneous dissipative behaviour a simultaneous yield of the braces on every floor has to be ensured. Eurocode 8 aims to promote this by a condition given for the overstrength factors, \( \Omega \), realized on the different floors, making them closely uniform. It needs to be verified that the maximum overstrength does not differ from the minimum by more than 25%.

\[
\Omega = \frac{N_{p.l,Rd,i}}{N_{br,i,Ed,j}} \geq 1.25 \tag{1}
\]

For the design of columns and beams Eurocode 8 requires the fulfilment of the following condition:

\[
N_{p.l,Rd} (M_{Ed}) \geq N_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} \tag{3}
\]
where $N_{pl,d}(M_{Ed})$ is the design buckling resistance taking into consideration the simultaneous bending. $N_{Ed,G}$ and $N_{Ed,E}$ are the axial actions from the non seismic and the seismic actions respectively. The global overstrength factor is defined as:

$$\Omega = \min \Omega_i$$

The overstrength factor, $\Omega$, accounts for the reserve in the dissipative members. It ensures that the resistance of the non-dissipative members is adequate until the first plastic deformations occur. The $\gamma_{lo}$ factor accounts for the random variability of the material properties. Its recommended value is 1.25. The factor 1.1 represents the increase of the yield stress of the dissipative members due to strain-hardening. In addition Eurocode 8 defines limitations to the non-dimensional slenderness:

$$\lambda \leq 2.0$$

and for X-braced configurations also:

$$1.3 \leq \lambda$$

where the non-dimensional slenderness is a ratio between the slenderness and the elastic limit slenderness:

$$\lambda = \frac{\lambda_j}{\lambda_j} = \pi \sqrt{\frac{E}{f_y}}$$

The upper bound defined in Eq. (5) is imposed to reduce the plastic out of plane deformation of gusset plates which are prone to low-cycle fatigue failure. Furthermore this limitation evades sudden impact-like loading at the end of the straightening phase of the buckled diagonals. The lower bound assures the buckling of the diagonals.

### 2.2 Design of CBF-s of various heights

With the aim of better understanding the behaviour of CBF-s subjected to seismic action, a series of concentrically braced steel frame structures were designed. The buildings are identical in plan, see Figure 1. The spacing of two adjacent columns is 6 metres in both directions. There are four bays in both directions giving 24 m × 24 m plan layout. Out of the four bays the inner two are braced concentrically on the four facades. The elevation of the buildings differs in terms of storey number. The number of floors is four, six and eight in the buildings denoted by CBF4, CBF6 and CBF8 respectively. The storey height is 3 metres on every floor in each building. The columns in the braced bays are pinned at the base, but otherwise continuous throughout the whole height and they are positioned in a way that their strong axis participates in the lateral resistance. Consequently the lateral load resisting system is the same in both directions. All the other columns are pinned at both ends. The beams are simply supported 1 section beams. The rectangular hollow section braces are placed diagonally and they are hinged at both ends. The loading is distributed between the steel members via concrete floor slabs, but composite action is not taken into consideration. The dead load is a uniform loading of 6.77 kN/m². The buildings are considered to have an office occupation where the imposed loading of the B category in EC1 gives 3 kN/m² evenly distributed loading on the slabs. The $\psi_2$ combination factor was taken to be 0.3 therefore 30% of the live loading is concurrent with the seismic action. The earthquake acceleration response spectrum is considered to be the type one in EC8 assuming B type soil conditions. The design ground acceleration is taken to be 0.25g, where g is the acceleration of gravity. The behaviour factor, $q$, is 4 in agreement with the structure-specific regulations of the standard. To obtain a more realistic and accurate design, wind loading is also taken into consideration in the calculation and the structural members are verified not only in seismic design situation, but in the ultimate and serviceability limit states also.

In the following table the applied cross-sections are presented. The interior and the exterior columns are listed separately. Furthermore the relative slenderness $\lambda$ and the overstrength factor, $\Omega$, are also indicated.

Table 1. Sections of the members, slenderness and overstrength results of the designed CBF-s.

<table>
<thead>
<tr>
<th>Storey</th>
<th>Int. Col.</th>
<th>Ext. Col.</th>
<th>Beam</th>
<th>Brace</th>
<th>$\lambda$</th>
<th>$\Omega$</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>HEA</td>
<td>HEA</td>
<td>IPE</td>
<td>SHS</td>
<td>1.83</td>
<td>1.05</td>
</tr>
<tr>
<td>3</td>
<td>HEB</td>
<td>HEB</td>
<td>IPE</td>
<td>SHS</td>
<td>1.87</td>
<td>1.01</td>
</tr>
<tr>
<td>2</td>
<td>HEB</td>
<td>HEB</td>
<td>300</td>
<td>100×6</td>
<td>1.91</td>
<td>0.98</td>
</tr>
<tr>
<td>1</td>
<td>HEB</td>
<td>HEB</td>
<td>360</td>
<td>100×8</td>
<td>1.95</td>
<td>1.00</td>
</tr>
<tr>
<td>6</td>
<td>HEA</td>
<td>HEA</td>
<td>IPE</td>
<td>SHS</td>
<td>2.07</td>
<td>1.00</td>
</tr>
<tr>
<td>5</td>
<td>HEB</td>
<td>HEB</td>
<td>IPE</td>
<td>SHS</td>
<td>1.88</td>
<td>0.97</td>
</tr>
<tr>
<td>4</td>
<td>HEB</td>
<td>HEB</td>
<td>300</td>
<td>100×6</td>
<td>1.95</td>
<td>1.04</td>
</tr>
<tr>
<td>3</td>
<td>HEB</td>
<td>HEB</td>
<td>300</td>
<td>100×10</td>
<td>2.01</td>
<td>1.04</td>
</tr>
<tr>
<td>2</td>
<td>HEB</td>
<td>HEB</td>
<td>360</td>
<td>100×10</td>
<td>1.95</td>
<td>1.01</td>
</tr>
<tr>
<td>1</td>
<td>HEB</td>
<td>HEB</td>
<td>360</td>
<td>120×10</td>
<td>1.63</td>
<td>1.00</td>
</tr>
<tr>
<td>8</td>
<td>HEA</td>
<td>HEA</td>
<td>IPE</td>
<td>SHS</td>
<td>2.07</td>
<td>0.99</td>
</tr>
<tr>
<td>7</td>
<td>HEB</td>
<td>HEB</td>
<td>IPE</td>
<td>SHS</td>
<td>1.87</td>
<td>1.00</td>
</tr>
<tr>
<td>6</td>
<td>HEB</td>
<td>HEB</td>
<td>IPE</td>
<td>SHS</td>
<td>1.95</td>
<td>0.98</td>
</tr>
<tr>
<td>5</td>
<td>HEB</td>
<td>HEB</td>
<td>IPE</td>
<td>SHS</td>
<td>2.01</td>
<td>1.02</td>
</tr>
<tr>
<td>4</td>
<td>HEB</td>
<td>HEB</td>
<td>IPE</td>
<td>SHS</td>
<td>1.63</td>
<td>1.10</td>
</tr>
<tr>
<td>3</td>
<td>HEB</td>
<td>HEB</td>
<td>IPE</td>
<td>SHS</td>
<td>1.63</td>
<td>1.01</td>
</tr>
<tr>
<td>2</td>
<td>HEM</td>
<td>HEM</td>
<td>IPE</td>
<td>SHS</td>
<td>1.37</td>
<td>1.05</td>
</tr>
<tr>
<td>1</td>
<td>HEM</td>
<td>HEM</td>
<td>IPE</td>
<td>SHS</td>
<td>1.37</td>
<td>0.99</td>
</tr>
</tbody>
</table>
Figure 1. Plan layout and side view of 4-storey CBF.

3 BEHAVIOUR OF ANALYSED BUILDINGS

3.1 Dynamic behaviour of CBF-s under seismic effect

For the verification of the CBF designs a Nonlinear Time History analysis (henceforth NTHA) program has been carried out. For the analysis seven artificial accelerograms were selected. The length of each excitation is 20 seconds and the spectra of the ground motions were fitted to the design spectrum. The used accelerograms are benchmarks in the RFCS OPUS research program conducted with the cooperation of several European universities [5]. This selection of artificial records complies with the corresponding parts of Eurocode 8. The acceleration records were scaled by multiplying the peak ground acceleration by a scale factor ranging from 0.1 up to 2.0. The analysis was conducted on planar beam models with FinelG nonlinear finite element software. The members were divided into multiple beam elements and the applied material model was the Giuffrè – Menegotto – Pinto cyclic law [6]. The masses and simultaneous loads were applied in the intersections of the beams and columns. The initial imperfection of the braces was provided by distributed loading along the diagonals.

The dynamic behaviour can be well investigated by the lateral displacement history and the deformations of the buildings. By simply looking at the deformed shape of the 4-storey CBF model in Figure 2 that we get as a result of an NTHA at the end, we can see that all the braces have a triangular buckled shape. At the mid section of the bracing bars plastic hinges form due to the local buckling of the plates. In repeated cycles the cumulating plastic deformations gradually facilitate the buckling by rotation of the plastic hinge. Eventually this leads to the angular buckled shape. In the meantime plastic elongations increase the network length of the bars. Consequently the diagonals have to be in compression after the seismic action when the building is laterally unloaded. Plastic elongations therefore further amplify the triangularity of the braces. Also we can notice that the magnitude of the triangular imperfection introduced to the system by the plastic strains may vary on the different floors.

The larger triangular-like imperfection implies larger relative lateral drifts on the particular storey which indicates the formation of a weak storey. The weak storey yield mechanism is related to large relative displacement of the adjacent floors, significant plastic elongation and rotation of the plastic hinge. Therefore to identify the occurrence of a weak storey the lateral displacements of the slabs and the interstorey drift results have to be examined. The lateral displacements of the floor slabs as a result of one of the 20 second excitations are depicted in Figure 3. It can be seen that the top floor exhibits disproportionately large displacements compared to the other three storeys. The maximum interstorey drift ratio results presented in Figure 4 are even more convincing that on the top a weak storey developed. The interstorey drift ratio is the interstorey drift divided by the storey height:

\[ IDR_i = \frac{X_i - X_{i-1}}{H_i} \]  \hspace{1cm} (8)

where the lateral displacement of the floor slabs is denoted by \( X_i \), \( H_i \) is the storey height and \( i \) is the storey index.

Figure 2. Residual deformations after seismic action.

Figure 3. Horizontal displacements of floor slabs.
Figure 4. Maximum interstorey drift ratios.

In the presented time series the weak storey phenomenon can be identified on the top floor, therefore we further analyse its displacements. The main objective of the wavelet transformation is to decompose an arbitrary function into elementary contributions [7]. The wavelet transform measures the similarity of the analysed signal function and a series of wavelets with different translation in time and resolution in the frequency domain. The wavelet transform of the signal $x(t)$ by definition is the inner product in the Hilbert space of the function and a family of wavelets:

$$W(a,b) = \frac{1}{\sqrt{a}} \int g^*(\frac{t-b}{a}) \cdot x(t) \, dt$$  \hspace{1cm} (9)

where $g^*(t)$ is the mother wavelet function which generates the family of the wavelets. The mother wavelet is dilated by parameter $a$ and translated by $b$ which vary continuously so that a series of son wavelets are created. One possible choice as the mother wavelet is the Morlet function which is a modified version of the Gabor wavelet.

$$g(t) = \exp\left(\frac{-\beta^2 (t-b)^2}{a^2}\right) \cdot \cos\left(\frac{\pi (t-b)}{a}\right)$$  \hspace{1cm} (10)

This expression defines a harmonic function that decays exponentially on both sides over a time interval around $t=b$. The parameter $\beta$ defines the length of the envelope wave. By evaluating Eq. (9) on a series of son wavelets in a fixed time and frequency domain we can map the instantaneous frequency components of the analysed signal. The diagram depicting the results is called a scalogram which is analogue with the spectrogram created by Fourier transformation but the time and frequency axes are both variables, therefore the results are represented by a colormap. In our case the signal is the horizontal displacement of a selected floor slab coming from an earthquake excitation.

The wavelet transform of the displacements does not show sharp margins in Figure 5, the shape is more cloud-like. This is due to the fact that the excitation and the displacement both miss any kind of harmonic regularity. Furthermore the displacement diagram only has a few waves over the 20 second time series and this leads to a bad resolution. Nonetheless the local maxima are denoted by a white line on the wavelet transform diagram and this line shows that after the first second of featureless noise the characteristic frequency decreases from about 0,77-0,80 Hz to 0,45-0,50Hz throughout the series.

3.2 Causes of the observed behaviour

So far it has been shown that under a seismic excitation the diagonals of a CBF are prone to gradual deterioration that leads to a triangular imperfect shape. In the meantime the displacements of the floor slabs become disproportional, even unsynchronized when the CBF exhibits weak storey behaviour. It can be assumed that the irreversible deterioration of the braces that are the main horizontal load bearing elements results in a significant alteration of the dynamic behaviour of the structure. To verify this assumption a series of natural frequency analyses were conducted on the elastic model of the four-storey braced frame [8]. The perfect model with two straight bracing bars was altered. At the midpoint of two braces on the same floor different magnitudes of imperfection were introduced. The midpoints of the diagonals were pulled perpendicularly to the bar by 10 centimetres at each step up to 50 centimetres. This resulted triangular shape braces. The triangular alteration of the diagonals was done at every floor separately to analyse the effect of developed weak storeys. Also the same imperfections were applied on every floor simultaneously which corresponds to the development of the global yield mechanism. Furthermore the natural frequency analysis was run on the perfect model with one diagonal on every floor, neglecting the buckled bar in compression, in accordance with Eurocode 8. In Table 2 the natural frequency results for the more relevant first and second modes are listed. The rows of the table correspond to the different levels of the introduced imperfection while the columns differ in the storey of the imperfection. For the sake of comparison it is noted that the frequencies of the perfect model with only one bar yielded 0.84 Hz and 2.26 Hz respectively.

Table 2. Natural frequencies of imperfect models.

<table>
<thead>
<tr>
<th>Mode</th>
<th>1 (cm)</th>
<th>2 (cm)</th>
<th>3 (cm)</th>
<th>4 (cm)</th>
<th>All</th>
</tr>
</thead>
<tbody>
<tr>
<td>1\textsuperscript{st} mode</td>
<td>10</td>
<td>0.95</td>
<td>0.99</td>
<td>1.03</td>
<td>1.11</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>0.67</td>
<td>0.87</td>
<td>0.78</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>0.54</td>
<td>0.76</td>
<td>0.65</td>
<td>0.67</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>0.47</td>
<td>0.59</td>
<td>0.58</td>
<td>0.56</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>0.43</td>
<td>0.57</td>
<td>0.54</td>
<td>0.50</td>
</tr>
<tr>
<td>2\textsuperscript{nd} mode</td>
<td>10</td>
<td>2.75</td>
<td>3.12</td>
<td>2.95</td>
<td>2.36</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>2.52</td>
<td>3.07</td>
<td>2.71</td>
<td>1.90</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>2.46</td>
<td>3.03</td>
<td>2.62</td>
<td>1.79</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>2.43</td>
<td>2.99</td>
<td>2.58</td>
<td>1.75</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>2.42</td>
<td>2.98</td>
<td>2.56</td>
<td>1.73</td>
</tr>
</tbody>
</table>
One can see in the results of the table that with the concurrent deformation of all the braces the decrease of the frequencies in the two modes is roughly proportional. However this is not the case when the triangular modification is done to the braces of one floor only. Consequently it is not only the frequency of the modes that changes. The introduced imperfection of the braces modifies the stiffness matrix of the structure. It decreases the stiffness values corresponding to the end nodes of the altered braces only. Hence the new stiffness matrix of the structure is not proportional to the original, while the mass matrix remains the same. Therefore the mode shapes have to change also. Figure 6 depicts the first two modes of vibration corresponding to the 4-storey CBF with 40 cm brace imperfection on the third floor.

![Figure 6. Modes of vibration of imperfect model.](image)

The first mode exhibits a quasi rigid bodily movement above the weak storey and negligible displacements below. This fundamental mode also has a large mass participation, almost 60%, therefore it governs the response of the building. Consequently the plastic deformation caused imperfection of the braces results an adverse rearrangement of the horizontal inertia forces in a way that this inflicts an increase in the storey shear on the floor already subjected to plastic strains. This increase of the storey shear causes a recurring cycle as it will lead to the occurrence of further and cumulating plastic deformation on the floor under consideration, which is concatenated with the growth of brace imperfection and the change of the dynamic response. Hence the development of the weak storey mechanism can be regarded as a gradual, auto exciting and amplifying phenomenon in case of cyclic seismic action.

In the presented numerical example the weak storey occurred on the fourth floor. The wavelet transformation indicated that in the beginning the motion of this storey can mostly be described by a 0.77-0.80 Hz vibration and after gradual decrease this frequency becomes about 0.45-0.5 Hz. These result are in good agreement with the modal analysis results of the 4-storey model, as the frequency of the fundamental mode of the perfect model is 0.84 Hz and the imperfect model with 50 cm brace imperfection on the top floor yielded 0.50 Hz for the first mode.

### 3.3 Contradiction of the behaviour and EC8 hypotheses

The Eurocode procedure somehpw reduces the MDOF structure to an equivalent SDOF problem and assumes that the extrapolation of the elastic behaviour of this SDOF system gives an upper bound approximation of the actual nonlinear behaviour. This approach is visualized in Figure 7, where the displacement, $\delta$, is plotted against the seismic effect, $a_{gr}$ denoting peak ground acceleration. According to Ballio and Setti [9][10] the $q$ factor is the ratio of the seismic effect corresponding to first yield in the structure i.e. the end of the elastic stage at point B, and the intersection of the curves representing the linear and the nonlinear post-yield behaviour, point C. A design in accordance with Eurocode 8 is elastic therefore it corresponds to a seismic effect at point A, lower than the elastic limit, $a_{el,u}$. Consequently the actual design effect, which is $q$-times larger than the elastic effect, is smaller than the intersection at $q$ times $a_{el,u}$ beyond which dynamic instability can be expected. In this case the linear extrapolation of the displacements obtained by linear analysis certainly gives a safe estimation, point D, of the real values, point E. This approach can safeguard the adequacy of the design. Conversely the verification of a displacement obtained by the analysis of an equivalent SDOF system may not mean that the displacements are under a required limit on every floor. If the behaviour of the building is global with equal drifts on all the floors the curve number one on Figure 7 corresponds to the behaviour of all the storey drifts.

![Figure 7. Generalized displacement – acceleration curve.](image)

If this perfect behaviour is altered however by effects not taken into consideration in design and the curves of different floors deviate, it is possible that certain floors follow the curve number two for example that causes early failure. Unfortunately reality is the second scenario, see the numerical example above, where the interstorey drifts of different floors are not equal, not even closely related. In the authors opinion the uniformity condition imposed to the overstrength factors of every floor by Eq. (2) is not able to evade local mechanisms. However the design shall only be modified by the introduction of new criteria that prevent the occurrence of excess storey drift differences and with this condition the original idea of the design with the $q$ behaviour factor can be successfully applied.

The uniformity condition imposed to the overstrength factors of every floor by Eq. (2) is an attempt in Eurocode to evade the occurrence of localized dissipation on a weak storey by promoting the simultaneous yielding of the braces on multiple, or if possible, on every floor. Both a simplified design and the modal response analysis of a CBF eventuates
that the design is made to a lateral load pattern that is inverted triangular and therefore corresponds to the global yield mechanism. In design the internal forces are calculated by the use of this loading and from the normal forces in the diagonals the formation of the supposed behaviour is verified. Briefly, the development of the global yield mechanism is verified by the use of internal force results that already suppose its presence. In the authors opinion the eligibility of providing proof to the formation of one phenomenon by the analysis of its effects is questionable. Conversely it is more desirable to prove that the structure is susceptible to exhibit plastic behaviour on multiple floors even if subjected to various, more random-like lateral load patterns as this entails the development of a distributed dissipative behaviour eventually. The criterion of Eurocode 8 is necessary but not sufficient to prevent the development of weak storeys as it proves the global plastic mechanism by supposing it.

3.4 Performance of the designs

The main objective of the verification of the presented designs by NTHA is to see whether the designs are prone to weak storey behaviour or not. For the analysis each building is loaded by the seven acceleration records scaled from 0.1 to 2.0, and for each scale factor the average of the seven different results is calculated at the end. This complies with the regulations of Eurocode 8. Previously we have seen that the maximum interstorey drift well indicates the existence of a weak storey. By plotting the maximum interstorey drifts against the scale factor we can observe the development of the weak storey phenomenon. In Figure 8 the maximum interstorey drift results of the 6-storey CBF are presented. For the sake of comparability of all the buildings the maxima and the minima are distinguished for three scale factors and these are depicted by bar charts in Figure 9. The selected scales are 0.5 which is the half strength, 1.0 which is design strength and 1.25. In Figure 9 we can observe that the maximum interstorey drift is tendentiously multiple times larger than the minimum. The difference is large even at half strength and though in the case of the smaller, 4-storey building the maximum – minimum ratio becomes smaller for the higher scale factors, in the larger buildings the difference escalates.

The large differences indicate that in every case the manifold of the curves corresponding to the different floors is rather broad just like in Figure 8. The buildings therefore do not exhibit a distributed dissipation, there are storeys that undergo lot larger plastic excursions than others. As the results indicate an adverse behaviour in every building we can conclude that the CBF-s designed according to Eurocode 8 are susceptible to develop weak storeys even when subjected to seismic actions that are smaller than the design level.

![Figure 8. Maximum IDR diagram of 6-storey CBF.](image)

![Figure 9. Maximum IDR results of designed CBF-s.](image)

4 PROPOSED REDESIGN METHOD

4.1 Definition of the method

In order to establish an appropriate redesign criterion instead of using a perfect and elastic model exhibiting global behaviour, a plastic model penalized with imperfections and their consequential effects should be used. First and foremost we assume that the structure exhibits weak storey phenomenon on one floor. Moreover we suppose that the bars are deteriorated by tensile yield and the deformation of the mid section in a way that the lateral stiffness on the weak storey decreases. A great loss of stiffness significantly changes the fundamental mode so that the displacements above the weak storey are closely rigid bodily, and under negligibly small. The accelerations coming from such a displaced shape can be well approximated by constant acceleration above the weak storey and nothing under. The horizontal loads determined from this acceleration are therefore proportional to the masses of the slabs. If the masses are equal on every storey, the equivalent load pattern that we can use for the analysis is constant above and zero under the weak storey, see Figure 10.

![Figure 10. Equivalent horizontal loading of imperfect CBF model and plastic collapse mechanisms.](image)
As the structural model is in the non-elastic range we shall conduct plastic analysis of the structure to calculate the magnitudes of the loading [11], defined by the load parameter \( \lambda \), that correspond to different plastic collapse mechanisms. Our aim is to provide a condition that can ensure that the structure is ultimately driven towards a distributed dissipation on every floor if possible instead of a weak storey collapse, even in the presence of a developed weak storey. The basic idea of the new criterion is that if the load parameter corresponding to a local storey collapse mechanism is larger than the load parameter of the global collapse mechanism, the weak storey collapse on the analysed floor may not occur as the plastic yield of multiple diagonals precedes it. For the complete analysis of the whole building we need to verify this condition in an n-storey CBF n times, which gives n load parameter pairs. On these pairs the following inequality has to be met to prevent the development of weak storeys:

\[ \lambda_{loc,i} \geq \lambda_{glob,i} \quad (11) \]

This condition does not necessarily ensure the hard-to-realize simultaneous plasticity of every diagonal. Conversely it defines a barrier within which plastic deformations can be realized in the diagonals of multiple storeys. It assures that the CBF tends to exhibit distributed dissipation regardless of the loading history and any instantaneous plasticity scheme. Furthermore the comparison of the load parameters defines an internal condition in the structure that can be evaluated without the ground acceleration or any other parameter that describes the magnitude of the design seismic effect. Therefore the introduced criterion is loading independent, it shows whether the structure is prone to weak storey behaviour or not. The structure that fulfils the condition is deemed to be robust in terms of being able to exhibit a controlled dissipative behaviour under any seismic effect within its strength range of design. For this reason the authors have chosen to call the redesign procedure as the Robust Seismic Brace Design (henceforth RSBD) method.

As the method is independent of the loading, a CBF has to be designed first according to the provisions of Eurocode 8. Then the design can be verified or modified by evaluating Eq. (11) and not respecting the uniformity yield condition or the requirement for columns i.e. Eq. (2) and (3) any longer.

The introduced criterion is necessary to evade weak storey collapse but not sufficient to keep the differences of interstorey drifts on all the floors within an acceptable limit. The reason is the following. The inequality condition defined in Eq. (11) can be satisfied by enlarging either the brace or the column sections as they are both involved in the plastic resistance of a certain storey. If in an extreme case the resistance is only given by the brace and the columns are negligible, the load of the first yield on the storey is equal to the ultimate plastic resistance. As the RSBD method aims to prevent the attainment of the ultimate resistance, the first yield will not be reached either and the storey remains elastic throughout the seismic effect. Conversely, if all the resistance comes from the plastic hinges on the top and on the bottom of the columns and the braces are neglected, the lateral displacement on this certain floor will be a lot larger than of all the other floors as the bent floor is a lot more flexible than the ones stiffened by the braces. Briefly, the local load parameter can be modified by both the braces and the columns. As these two exhibit different behaviour in terms of displacement an additional condition has to be introduced for the sake of a good selection of brace and column sections.

To resolve this problem the Brace Participation Ratio (henceforth BPR) is introduced. The BPR is shortly the product of the plastic overstrength i.e. the ratio of the local and the global load parameters, and an expression describing the involvement of the brace in limiting the lateral displacements by the use of the plastic work of the braces and the columns.

\[
\text{BPR} = \frac{\lambda_{loc,i}}{\lambda_{glob,i}} \cdot \frac{W_{pl,br}}{W_{pl,br,\tau} + \tau \cdot W_{pl,col}} \quad (12)
\]

where \( W_{pl,br} \) is the plastic work of the braces and \( W_{pl,col} \) is the work in the column cross sections in local plastic mechanism.

To take into consideration that columns with two continuous ends or with one or two pinned ends do not limit lateral displacement the same way, the plastic work of columns is multiplied by \( \tau \) parameter. This parameter is 1.0 for continuous columns, 0.6 for one-pinned-end columns. The BPR has to be determined for every floor independently. To obtain a distributed dissipative behaviour in which every floor participates, a uniformity condition is imposed to the BPR similarly to the Eurocode requirement for storey overstrength factors. The maximum and the minimum BPR has to be in a 10% range.

### 4.2 Performance of redesigned structures

In Table 3 the modifications to the previously introduced designs are presented. The changed brace or column cross sections are highlighted and also the storey overstrength factor, the plastic load parameters of the RSBD method and the BPR results are listed.

In Figure 11 the maximum interstorey drift ratio diagram of the redesigned 6-storey CBF is presented. Figure 12 depicts the same bar chart as Figure 9, but for the redesigned buildings.

Comparing Table 1 and 3 we can see that as a result of the RSBD redesign method the columns that are connected to the braces had to be significantly reinforced. This need is not realized in the Eurocode design as the analysis of the perfect structure does not yield significant bending of the columns, therefore by Eq. (3) the required resistance is underestimated. In Table 3 we can also see that due to the reinforcement of certain braces Eq. (2) is not respected anymore. Nevertheless Figure 11 depicts a favourable narrow manifold for CBF-6 and Figure 12 shows that the behaviour is similar for all the redesigned buildings. Comparing Figure 12 to Figure 9 we can see that the minimum drift grows while the maximum significantly decreases in the redesigned buildings. Consequently the RSBD redesign method provides CBF-s that exhibit a lot more distributed dissipative behaviour in a seismic design situation than the ones designed respecting Eurocode 8 provisions. Moreover by decreasing the maximum drift the RSBD method decreases the damages also and provides a more resistant structure that may lead to a larger allowable q factor.
Table 3. Results of redesign by the RSBD method.

<table>
<thead>
<tr>
<th>Storey</th>
<th>Ext. Col.</th>
<th>Brace</th>
<th>Ω</th>
<th>λ_{loc}</th>
<th>λ_{glob}</th>
<th>BPR</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>HEA</td>
<td>SHS</td>
<td>1.30</td>
<td>498</td>
<td>504</td>
<td>78%</td>
</tr>
<tr>
<td>3</td>
<td>HEA</td>
<td>SHS</td>
<td>1.00</td>
<td>358</td>
<td>291</td>
<td>76%</td>
</tr>
<tr>
<td>2</td>
<td>HEB</td>
<td>SHS</td>
<td>0.97</td>
<td>316</td>
<td>227</td>
<td>80%</td>
</tr>
<tr>
<td>1</td>
<td>HEB</td>
<td>SHS</td>
<td>0.99</td>
<td>220</td>
<td>204</td>
<td>84%</td>
</tr>
<tr>
<td>6</td>
<td>HEA</td>
<td>SHS</td>
<td>1.14</td>
<td>525</td>
<td>545</td>
<td>73%</td>
</tr>
<tr>
<td>5</td>
<td>HEA</td>
<td>SHS</td>
<td>1.00</td>
<td>369</td>
<td>303</td>
<td>73%</td>
</tr>
<tr>
<td>4</td>
<td>HEB</td>
<td>SHS</td>
<td>1.00</td>
<td>260</td>
<td>222</td>
<td>75%</td>
</tr>
<tr>
<td>3</td>
<td>HEB</td>
<td>SHS</td>
<td>1.02</td>
<td>238</td>
<td>185</td>
<td>80%</td>
</tr>
<tr>
<td>2</td>
<td>HEB</td>
<td>SHS</td>
<td>0.97</td>
<td>233</td>
<td>166</td>
<td>76%</td>
</tr>
<tr>
<td>1</td>
<td>HEB</td>
<td>SHS</td>
<td>0.97</td>
<td>163</td>
<td>159</td>
<td>80%</td>
</tr>
</tbody>
</table>

Figure 11. Maximum IDR diagram redesigned CBF-6.

Figure 12. Maximum IDR results of redesigned CBF-s.

5 CONCLUSIONS

In the article three CBF-s were presented that comply with the seismic design requirements of Eurocode 8. For the sake of verifying the seismic behaviour and resistance of these designs NTHA calculations were conducted. By the analysis of obtained displacement data series the dynamic behaviour of CBF-s was examined. An explanation was given for the observed behaviour and inconsistencies of the Eurocode hypotheses and the actual behaviour were pointed out. By the analysis of the three buildings it was proved that Eurocode designs are prone to exhibit weak storey behaviour. In the last section a new redesign method was proposed that is based on the observed behaviour of the buildings. By analysis of the redesigned buildings it was proved that the RSBD method successfully prevents the occurrence of the weak storey phenomenon and by it, significantly enhances the performance of braced frames.

REFERENCES