

# Design check of BRBF system according to Eurocode 8 *Use of pushover analysis*



This report presents a simple computer-based push-over analysis for a steel structure with Buckling Restrained Braced Frame system subjected to earthquake loading. The proposed analysis technique is illustrated for a building framework example in accordance with Eurocode.

## In general

As per Eurocode 8 (EC8) Part 1, section 4.3.3.4.2.1, in general, as an alternative to the behavior-factor linear elastic design, seismic no-collapse requirement check can be performed by non-linear static (pushover) analysis.

Since Buckling Restrained Braced Frame (BRBF) systems are not included and regulated in the current version of EC8, designer shall use pushover analysis (or time-history analysis) for the design check.

Note: For information, the new Romanian Seismic Design Code P100-1/2011 (this version is currently under public review) already includes behavior factors and design rules for BRBF system.

## Requirements for the analysis and design

Pushover analysis means non-linear static analysis, typically with geometrical and material non-linearity included. The analysis is carried out with constant gravity loads and monotonically increasing lateral loads.

The pushover analysis shall meet the following criteria, as required by EC:

- Material non-linearity (for the dissipative members) and geometrical non-linearity in general should be normally included.
- Real material/element behavior shall be represented in the numerical model and analysis. This includes:
  - mean values of material properties (i.e. element resistances),
  - realistic post-elastic behavior; yielding, hardening, strength/stiffness degradation, etc.
- Structural performance shall be evaluated at the so-called target displacement level (this displacement is expected to develop under the design earthquake effect).
- Capacity curve (relation of the base shear to the control node – normally the roof – displacement) shall be determined in a range of 0% to 150% of the target displacement.

## Software requirements

The applied software shall be capable of:

- performing material and geometrical (large displacements) non-linear analysis,
- material model for BRB members: at least bilinear force-elongation relation (multi-linear relation is preferred).

Capable commercial softwares are e.g. SAP, Ansys, ETABS, etc.

## Data required from Star Seismic Europe

Star Seismic Europe should provide the following information:

- mean yield strength of the core material,
- yield length of the brace member,
- force – elongation characteristic curves for the BRB elements.

Note that preliminary general design shall precede the final analysis. For correct mathematical modeling and for connection detailing also consult with Star Seismic Europe.

## Example: Design check of BRBF systems

Note that the following example is limited for a general introductory illustration of the method, and thus it does not constitute full adherence to design check requirements insofar, as additional investigations may be required. Also note that for specific data related to the BRB elements and BRBF system parameters one should consult with Star Seismic Europe.

### 1 Building

#### 1.1 Global geometry

- building width:  $a = 3 \times 6 = 18\text{m}$
- building length:  $b = 3 \times 6 = 18\text{m}$
- story height:  $h = 3\text{m}$
- number of stories:  $n_s = 5$
- building height:  $H = 15\text{m}$

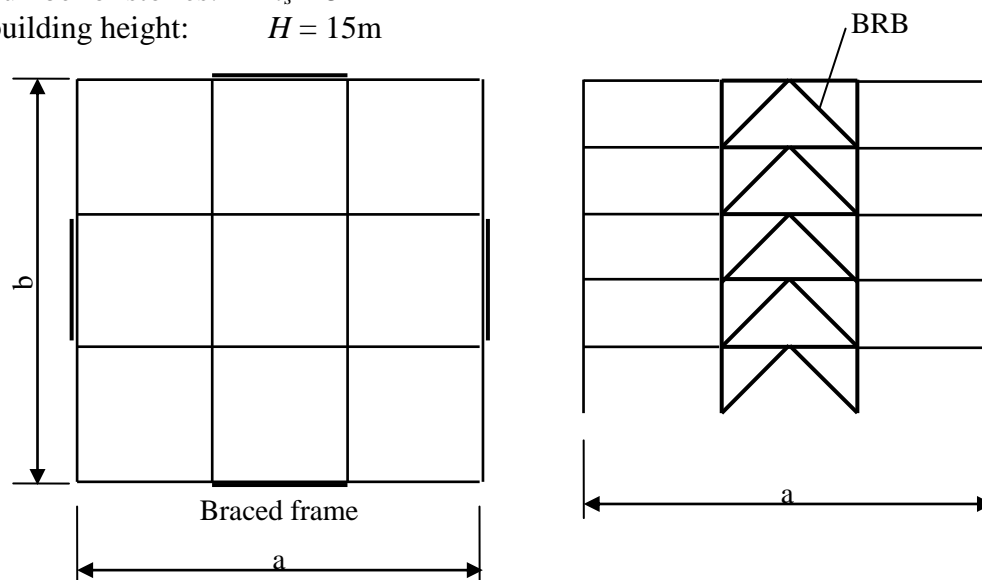


Figure 1. Building geometry

Story #	$A_{sc}$ ( $\text{cm}^2$ )	$f_y$ ( $\text{N/mm}^2$ )	$F_{pl,Rd}$ (kN)	$L_{tot}$ (m)	$L_y$ (m)
Roof	5.6	235	131.6	4.24	2.97
4	16.8		395		
3	25.2		592		
2	30.8		724		
1	33.6		790		

Table 1: BRB elements

## 1.2 Bracing system

Chevron bracing (inverted V) configuration is applied (Figure 1). Data for the BRB elements are summarized in Table 1. Columns have sections HEA450,  $f_y = 235\text{N/mm}^2$ .

## 1.3 Design loads and seismic actions

### 1.3.1 Dead loads:

- roof:  $g_{roof} = 3\text{kN/m}^2$
- floor:  $g_{floor} = 8\text{kN/m}^2$

### 1.3.2 Live loads (imposed loads):

- on roof:  $q_{roof} = 1\text{kN/m}^2$
- on floor:  $q_{floor} = 2.5\text{kN/m}^2$
- combination factor:  $\psi_{2,i} = 0.3$

In this example it is assumed that the seismic design situation is dominant, and thus additional (e.g. meteorological) loads are excluded ( $\psi_2 = 0$ ). Note that designer has to check if e.g. ULS controls the design of the bracing system.

### 1.3.3 Seismic parameters:

Design spectrum parameters:

- peak ground acceleration (PGA):  $a_{gR} = 0.3g$
- spectrum: Type 1
- ground type: D  
 $S = 1.35; T_B = 0.2\text{s}; T_C = 0.8\text{s}; T_D = 2\text{s}; \beta = 0.2$
- importance class: normal  $\rightarrow \gamma_I = 1.0$
- design PGA:  $a_g = \gamma_I a_{gR} = 0.3g$

Total weight in seismic action:

- on roof:  $G_k + \psi_{E,i} Q_{k,i}$   
 $q_{d,roof} = g_{roof} + \psi_{2,i} q_{roof} = 3.3\text{kN/m}^2$
- on floor:  $q_{d,floor} = g_{floor} + \psi_{2,i} q_{floor} = 8.75\text{kN/m}^2$

### 1.3.4 Load combinations:

Seismic design situation:

$$\sum G_k + E_{Ed} + \sum \phi \cdot \Phi_{2,i} \cdot Q_{k,i}$$

where:

- seismic action:  $E_{Ed} = \pm E_{Edx} \pm 0.3E_{Edy} = \pm E_{Edx}$   
(assume: braced frames in the two direction do not interact with each other)
- assume  $\phi = 1$

## 2 Numerical modeling technique

### 2.1 Geometrical model

Due to the regularity of the building (both in elevation and plan), simple 2D frame analysis can be performed. Note that accidental torsional effects shall be considered: building regularity allows the simplified procedure as given in Section 4.3.3.2.4 of EC8-1.

For simplification, in this example assume that the total horizontal load is resisted by the bracing system, i.e. contribution of other structural elements (e.g. continuous column, frame effect) to the lateral load resistance is neglected. Also, beam-to-column connections are pinned.

Consequently, only the braced frame part is to be modeled. Columns are represented by pin-ended spar elements. Beams are continuous between columns. Leaning column (additional pinned columns, connected to the frame with pin-ended rigid links at each floor) is used to consider the whole mass tributary to the bracing system, primarily for inclusion of second-order (P- $\Delta$ ) effects.

Note that, in general, analysis should consider the actual connection conditions (i.e. connections pinned/rigid/semi-rigid; column splices, etc.) with certain modeling simplification allowed.

The BRB elements are modeled with non-linear springs (link with non-linear force-deflection characteristics associated). The element extends within the hinge-to-hinge distance. As the pinned connection is located eccentrically to the model workpoints (node in intersection of beam and column axis), additional eccentricity element (rigid body element) is necessary.

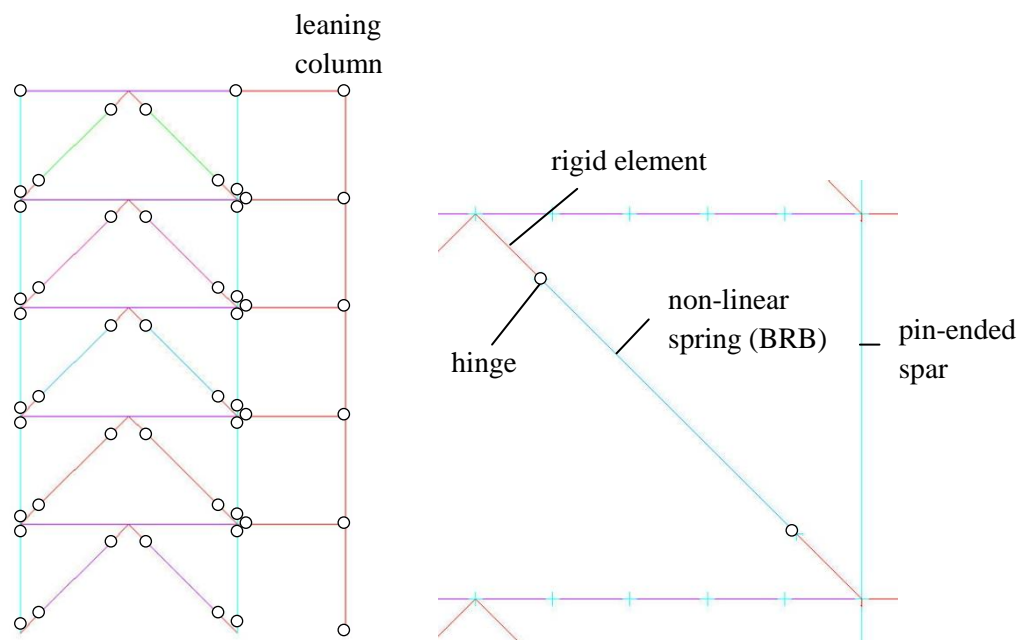


Figure 2. Geometry model

Story #	$A_{sc}$ (cm <sup>2</sup> )	$L_{tot}$ (m)	$L_y$ (m)
<b>Roof</b>	5.6	4.24	2.97
<b>4</b>	16.8		
<b>3</b>	25.2		
<b>2</b>	30.8		
<b>1</b>	33.6		

$A_{sc}$ : steel core area;  $L_{tot}$ : workpoint-to-workpoint distance;  $L_y$ : yield length

Table 2: BRB elements

## 2.2 Material models

Linear elastic material ( $E = 210\text{GPa}$ ) is applied for all members except for BRB elements. (In the intended plastic mechanism, plasticity develops in the BRB elements, while the rest of the structure remains elastic.)

Special attention shall be paid for the BRB material model / element characteristics. Three aspects will influence the choice of modeling:

- 1) Reliable post-elastic representation of the element behavior is necessary, including strength/stiffness degradation effects, etc.
- 2) As per EC8-1, Section 4.3.3.4.1, “element properties should be based on mean values of the properties of the materials”.
- 3) The resulting model (material + geometry model) shall capture the actual element behavior (not material behavior), i.e. calibrated to the envelope of the cyclic response characteristic curves.

Accordingly, non-linear spring characteristic is associated with the BRB link element. Consult with Star Seismic Europe for details.

In this example, assume the following characteristics:

- mean value of yield strength of the core material:  $245\text{N/mm}^2$ , which leads to the mean element resistances listed in Table 3.
- post-elastic behavior shown in Figure 3.

Story #	$A_{sc}$ (cm <sup>2</sup> )	$f_y^{mean}$ (N/mm <sup>2</sup> )	$F_y$ (kN)
Roof	5.6	245	137
4	16.8		412
3	25.2		617
2	30.8		755
1	33.6		823

$A_{sc}$ : steel core area;  $f_y^{mean}$ : mean value of yield strength;  
 $F_y$ : element strength at first yield

Table 3: BRB elements

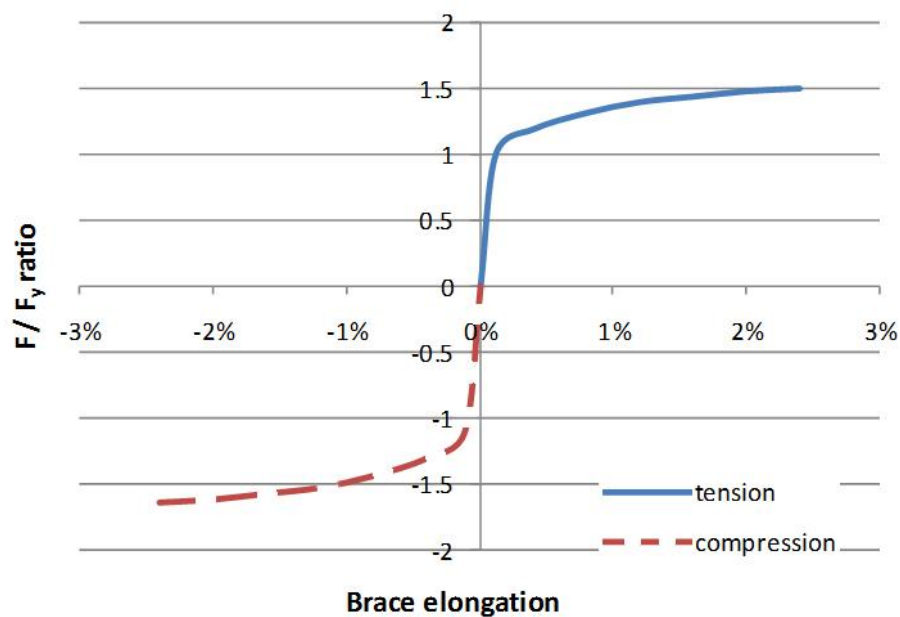


Figure 3. BRB characteristics

### 2.3 Load patterns

Gravity loading is reduced to the column nodes at each floor, as shown in Figure 4. Vertical masses that do not directly act on the brace columns, but tributary to the braced frame in a horizontal sense, are applied on the leaning column.

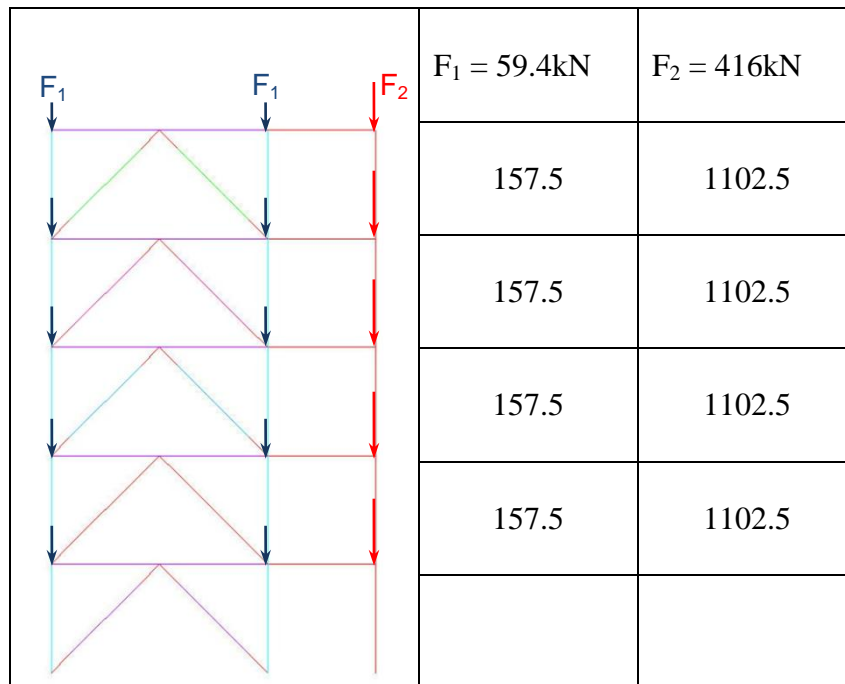


Figure 4. Gravity loading

EC8-1 requires the application of two different lateral load patterns (Figure 5):

- “uniform” pattern (uniform acceleration pattern), proportional to mass regardless of elevation,
- “modal” pattern, proportional to lateral forces consistent with the lateral force distribution used in elastic analysis (i.e. dominant vibration mode).

In this example, for illustration purposes the latter one (modal pattern) will only be analyzed. The same procedure shall be repeated for the uniform pattern.

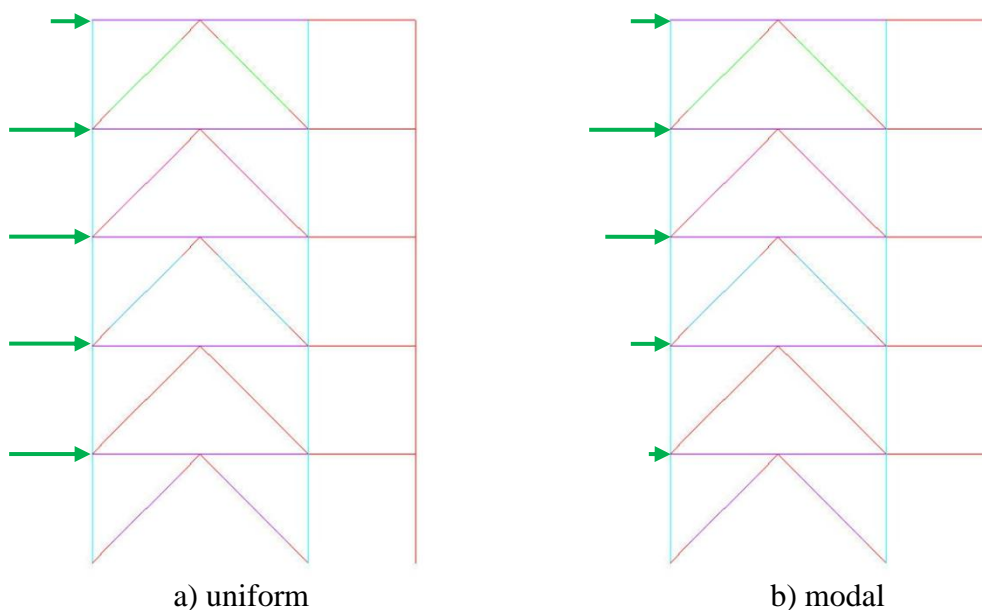


Figure 5. Horizontal load pattern

## 2.4 Analysis procedure

Geometrical and material non-linear analysis is performed (i.e. large displacements and plasticity effects included) in two steps:

- Step 1: gravity loading is applied (with no lateral loads),
- Step 2: at constant vertical load, gradually increasing one-parameter lateral loads.

Roof node shall be selected as the control node. The analysis shall continue up to a certain deformation level: as EC8 requires, the capacity curve shall be determined up to 150% of the control node displacement.

## 3 Results of pushover analysis

Major outcome of the pushover analysis is the so-called capacity curve – the relation of the base shear force to the control node displacement. Figure 6 shows the capacity curve in case of the modal load pattern.

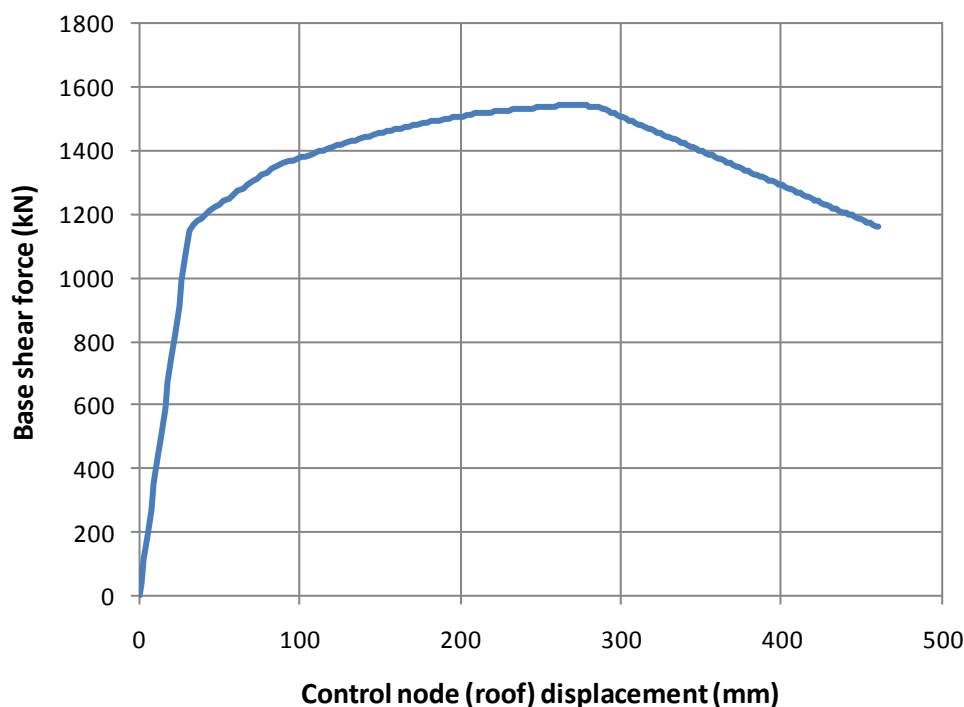


Figure 6. Capacity curve

## 4 Determination of target displacements

As per Annex B of EC8 Part 1, the target displacement is determined from the elastic response spectrum, based on a generalized SDOF system equivalence.

The method consists of the following steps:

- transformation of the MDOF system to an equivalent SDOF system,
- determination of an equivalent idealized elasto-perfectly plastic system,
- determination of the target displacement for the equivalent system,
- transformation to the MDOF system.

## 4.1 Transformation to single degree of freedom system

Table 4 summarizes the tributary masses and normalized displacements of each story. Normalization is completed in such a way that displacement of the control node (roof level) is unit.

Story #	$m_i$ (t)	$\Phi_i$ (-)
Roof	54.5	1.00
4	144.4	0.77
3	144.4	0.54
2	144.4	0.34
1	144.4	0.16

Table 4. Tributary mass ( $m_i$ ) and normalized displacements ( $\Phi_i$ ) per floor

The equivalent mass of the generalized SDOF system:  $m^* = \sum m_i \Phi_i = 316.6 \text{ t}$

The transformation factor:  $\Gamma = \frac{m^*}{\sum m_i \Phi_i^2} = 1.56$

Transformation of forces and displacements:  $F^* = \frac{F_b}{\Gamma}; \quad d^* = \frac{d_n}{\Gamma}$

The resulting capacity curve is illustrated in Figure 7.

## 4.2 Idealized elasto-plastic system

In determination of an equivalent idealized elasto-perfectly plastic system, it is assumed that the yield force equals to the base shear force at formation of plastic mechanism. The initial stiffness is determined on the bases of equal deformation energy (Figure 8):

$$F_y^* = F_m^* = 994 \text{ kN}$$

$$d_y^* = 2 \left( d_m^* - \frac{E_m^*}{F_y^*} \right) = 49.1 \text{ mm}$$

Where  $E_m^*$  is the actual deformation energy up to the formation of the plastic mechanism.

The period of the system:  $T^* = 2\pi \sqrt{\frac{m^* d_y^*}{F_y^*}} = 0.786 \text{ s}$

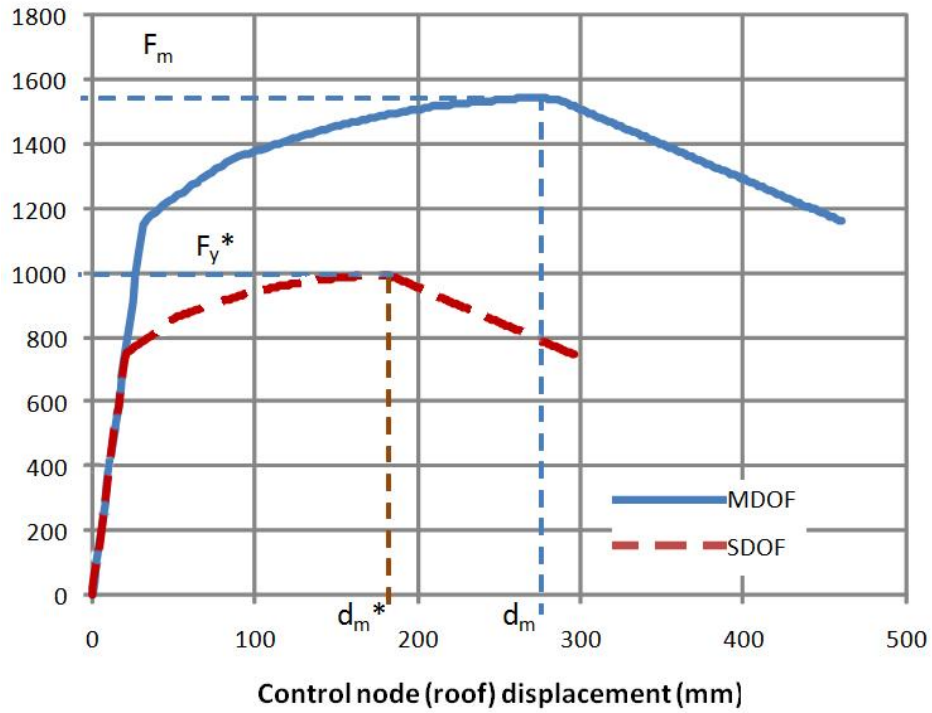


Figure 7. Capacity curve of the equivalent SDOF system

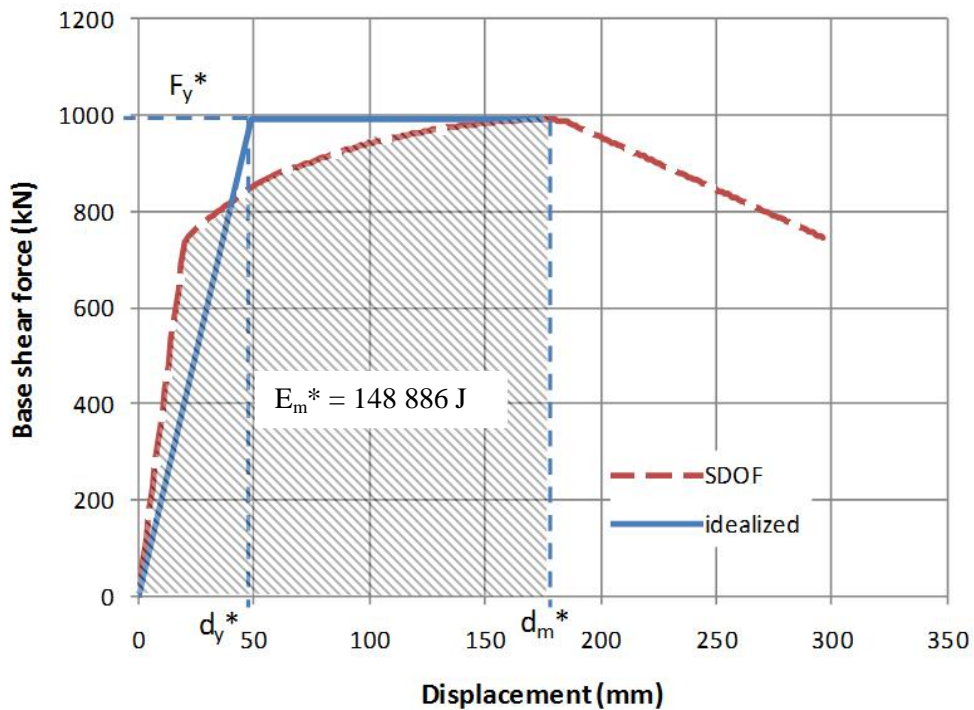


Figure 8. Idealized elasto-plastic system

### 4.3 Target displacement of the SDOF system

In the medium and long period range ( $T^* \geq T_C$ ), the target displacement of the plastic system equals to the target displacement of the elastic system with period  $T^*$  (equal displacement rule):

$$d_t^* = d_{et}^* = S_e(T^*) \left[ \frac{T^*}{2\pi} \right]^2 = 158 \text{ mm}$$

### 4.4 Target displacement of the MDOF system

As for the MDOF system, the target displacement of the roof node:

$$d_t = \Gamma d_t^* = 245.9 \text{ mm}$$

## 5 Design check

### 5.1 Criteria

The target displacement should be used as the basis of the design. In short, at the target displacement level, the structure shall remain stable. Important aspect is to check whether the intended failure mechanism is actually developing in the structure, so the behavior is controlled, i.e. plasticity is concentrated in members designed for energy dissipation (i.e. dissipative members: core of BRB elements), while the rest of the structure remains elastic.

Assuming that the controlled behavior is confirmed, the check practically leads to:

- 1) local ductility criteria of BRB elements: whether deformations belonging to the target displacement level can actually develop;
- 2) strength criteria of non-dissipative parts: under the internal loads belonging to the target displacement level members remain elastic.

### 5.2 Results at the target displacement level

For the target displacement level, Figure 9 and 10 illustrate the plastic elongations of BRB elements and the developing internal axial loads, respectively.

### 5.3 Ductility check of BRB elements

For each BRB member, the total computed elongation shall be limited. As Figure 9 proves, the maximum plastic elongation is 1.54%, the elastic elongation is 0.12% = total of 1.66%. The criteria of local ductility are met, as BRB member ductility exceeds this minimum value.

Consult with Star Seismic Europe for allowed elongations.

## 5.4 Strength check of non-dissipative members

The non-dissipative members (column, beam, connections, BRB ending, foundation, etc.) shall be checked for the internal forces developed at the target displacement level.

For instance, the check of column member on the first floor:

The maximum design load (Figure 10) in the column is 2540kN. The buckling resistance of the HEA450 column is 3810kN, thus appropriate.

## 5.5 Damage limitation

Different performance levels typically termed by lateral deformations, interstory drifts may be needed to be investigated. The pushover analysis results corresponding to the different displacement levels will allow the designer to check these performance criteria.

Accordingly, the “limited damage” check required by EC8 can also be completed by checking the corresponding results at the displacement level of 95-year return period seismic event.

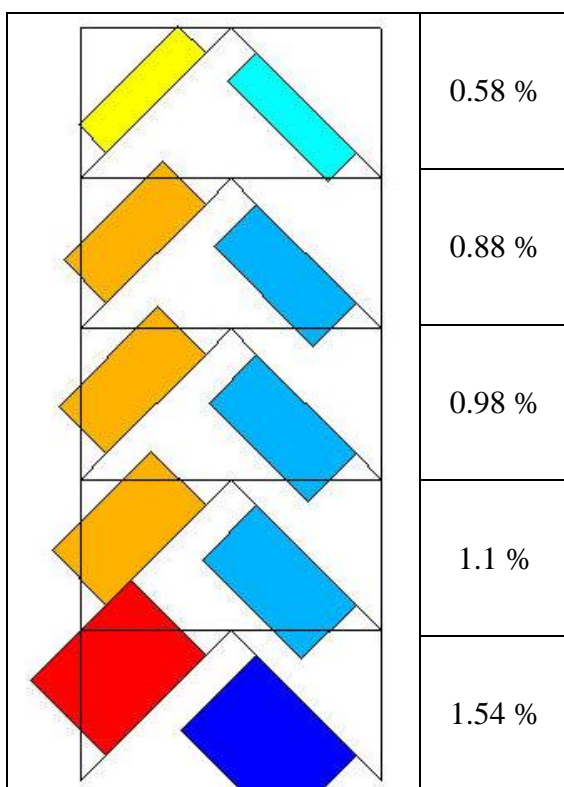


Figure 9. Plastic elongations

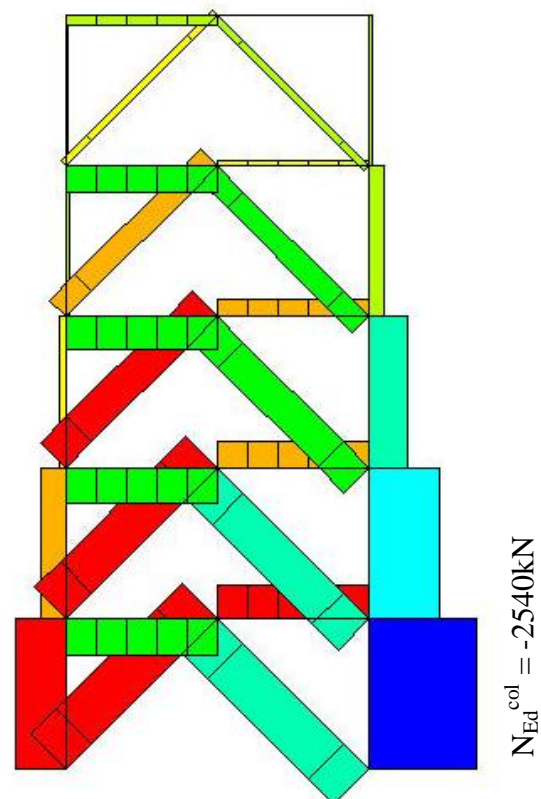


Figure 10. Axial loads in members

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