Business from technology

Advanced earthquake modeling of high-strength steel frames

Vilho Jussila, Ludovic Fülöp

VTT Technical Research Centre of Finland

ludovic.fulop@vtt.fi

Outline

- The challenge.
- Design targets for ordinary buildings in strong earthquake.
- The design process. How design targets are followed.
- Advanced modeling techniques in the non-linear behavior range.
- Interpretation of the performance targets.
- Some result interpretation.

The challenge

- The wider acceptability of high-strength steel (HSS) in building structures for seismic applications is hindered by reservations about the available ductility:
    - of members
    - and especially of connections.

- Currently research efforts are being directed towards developing connection configurations for HSS with sufficient ductility supply (*HSS_SERF/RFCS project – Universitatea "Politehnica" of Timisoara, RIVA Acciaio, VTT, University of Liege, Universität Stuttgart, University of Naples "Federico II“ Italy, Univerza v Ljubljani, GIPAC, Rautaruukki, Consorzio Pisa Ricerche*)

- Also to qualify global behavior of selected frame typologies to strong earthquake.

Definition of "goals" and "tools" in design codes

- At ultimate limit state (ULS) EN1998 aims at "no-collapse requirement". The structure shall withstand the design seismic actions (DSA) without local or global collapse and must retain structural integrity and a residual load bearing capacity. *The primary aim is to save the lives of the occupants*.

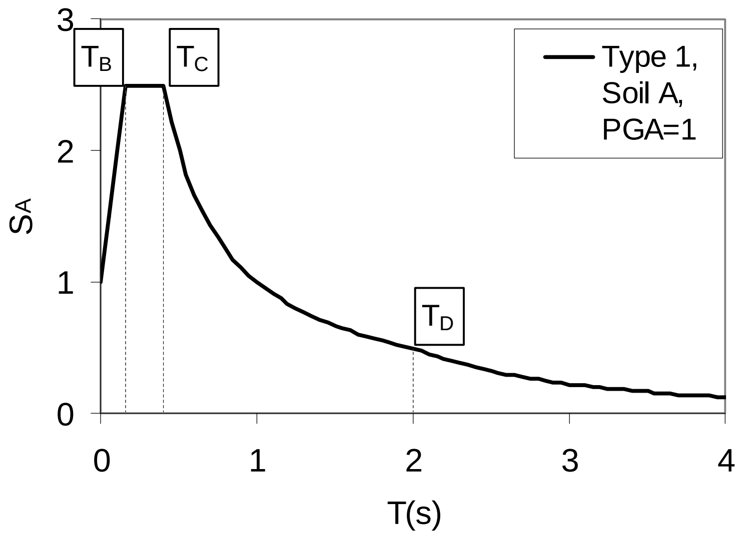
- DSA defined by a reference seismic action with probability of exceedence of 10 % in 50 years, or the reference return period of 475 years.

- However, in the basic case, even EN1998 provides the designer ways to calculate a structure supposing elastic behavior of the elements (i.e. elastic design like Response Spectrum Analysis).

Bridging the gap in EN1998 – The q factor

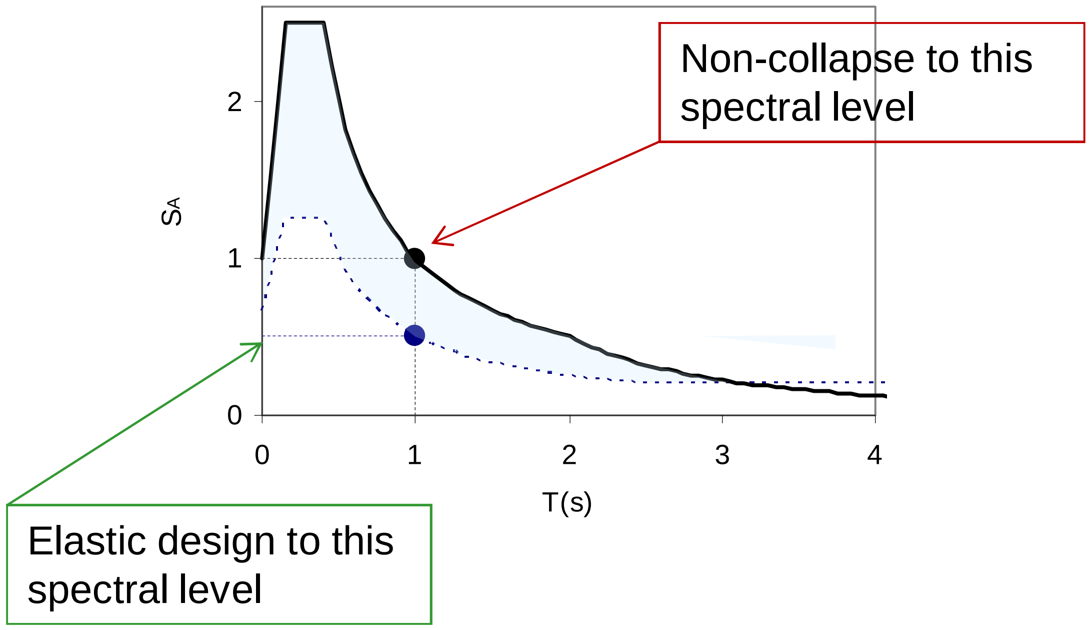
Elastic spectra - S_A:

$$S_e(T) = S_A = \begin{cases} a_g \cdot S \cdot \left[1 + \frac{T}{T_B}(2.5\cdot\eta - 1)\right] & 0 \le T \le T_B \\ a_g \cdot S \cdot \eta \cdot 2.5 & T_B \le T \le T_C \\ a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left[\frac{T_C}{T}\right] & T_C \le T \le T_D \\ a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left[\frac{T_C \cdot T_D}{T^2}\right], & T_D \le T \end{cases}$$



Design spectra - S_d:

$$S_d(T) = \begin{cases} a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B}\cdot\left(\frac{2.5}{\boxed{q}} - \frac{2}{3}\right)\right] & 0 \le T \le T_B \\ a_g \cdot S \cdot \left[\frac{2.5}{\boxed{q}}\right] & T_B \le T \le T_C \\ a_g \cdot S \cdot \left[\frac{2.5}{\boxed{q}}\cdot\frac{T_C}{T}\right], > \beta \cdot a_g & T_C \le T \le T_D \\ a_g \cdot S \cdot \left[\frac{2.5}{\boxed{q}}\cdot\frac{T_C \cdot T_D}{T^2}\right], > \beta \cdot a_g & T_D \le T \end{cases}$$



CBF's according to Eurocode

Table 6.2: Upper limit of reference values of behaviour factors for systems regular in elevation

STRUCTURAL TYPE	Ductility Class	
	DCM	DCH
a) Moment resisting frames	4	$5\alpha_u/\alpha_1$
b) Frame with concentric bracings		
Diagonal bracings	4	4
V-bracings	2	2,5
c) Frame with eccentric bracings	4	$5\alpha_u/\alpha_1$
d) Inverted pendulum	2	$2\alpha_u/\alpha_1$
e) Structures with concrete cores or concrete walls	See section **5**	
f) Moment resisting frame with concentric bracing	4	$4\alpha_u/\alpha_1$
g) Moment resisting frames with infills		
Unconnected concrete or masonry infills, in contact with the frame	2	2
Connected reinforced concrete infills	See section **7**	
Infills isolated from moment frame (see moment frames)	4	$5\alpha_u/\alpha_1$

EN1998-1 Table 6.2

What happens between the elastic design level and DSA?!

- Above the elastic load level, the structure will deviate from the elastic range of response. At DSA the structure is non-linear.
- The reserve up to ”collapse” depends on the structure’s ability to accommodate (1) in a stable way (2) non-linear deformations up to (3) DSA load levels.
- Regularity and ability to undergo repeated plastic deformations, without loosing capacity is a precondition of good earthquake performance.
- Difficult to satisfy, because the requirements are related to constructive and detailing conditions (usually backed by tests). And here is where HSS is lacking the empirical/testing background.

Design/Guiding principles

WHAT?

1. Structural simplicity
2. Uniformity, symmetry
3. Redundancy
4. Diaphragm behavior at floor level
5. Bi-directional resistance and stiffness
6. Torsional resistance and stiffness
7. Adequate foundation
8. Global collapse mechanism of vertical systems
9. Ductility in dissipative elements
10. Strength in non-dissipative elements

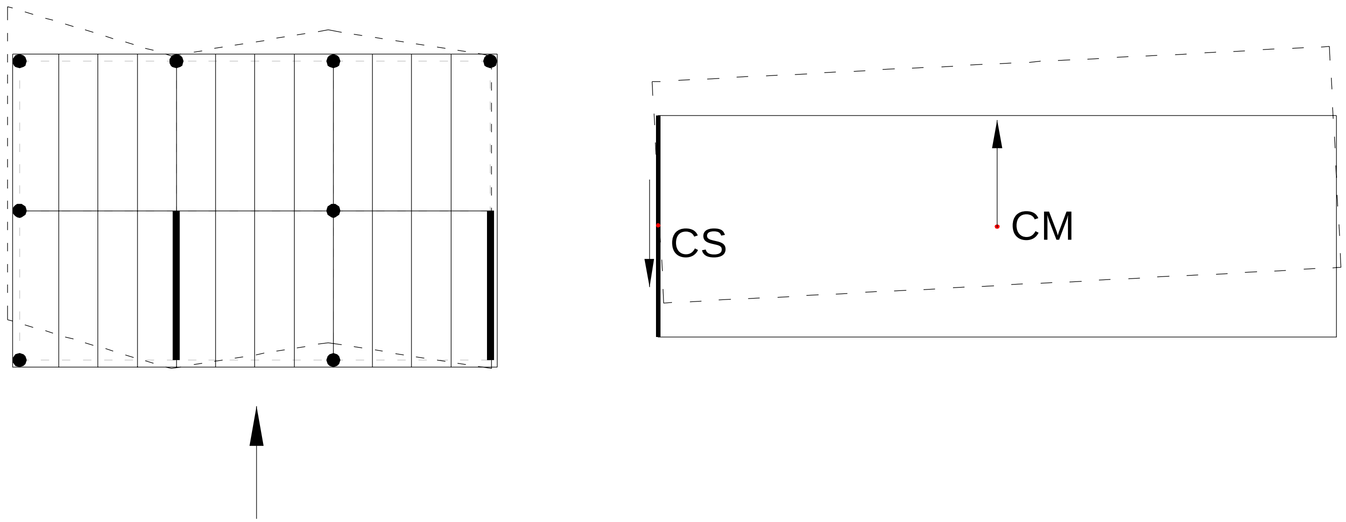
WHY?

1. To *reduce uncertainty of modeling, analysis, dimensioning, detailing and construction*
2. To *reduce uncertainty*
3. To *have alternative load paths in case of local plasticization*
4. To *distribute floor loads evenly to vertical systems, to be able to exploit redundancy of the vertical systems*
5. Because *earthquake load in horizontal directions is identical*
6. To *separate response in the two directions (reduce uncertainty) and avoid loading external vertical frames excessively*
7. Because *foundations transmit loads, for out of phase loading on foundations most analytical tools are invalid*
8. To *exploit redundancy within the vertical system, spread damage evenly (do not concentrate damage), maximally exploit energy dissipation*
9. Because *elements deforming in non-ductile way fail right after yielding*
10. To *keep dissipative element stable while they deform*

Global plastic collapse mechanism – role of slabs

- Regularity conditions in plan and elevation.

- Provisions for concrete diaphragms to play the role of a diaphragm:
  - (1) solid **reinforced concrete slab** considered a diaphragm;
  - (2) **cast-in-place topping on a precast floor** or roof system can be considered diaphragm in certain condition (concrete toping is strong & stiff enough).

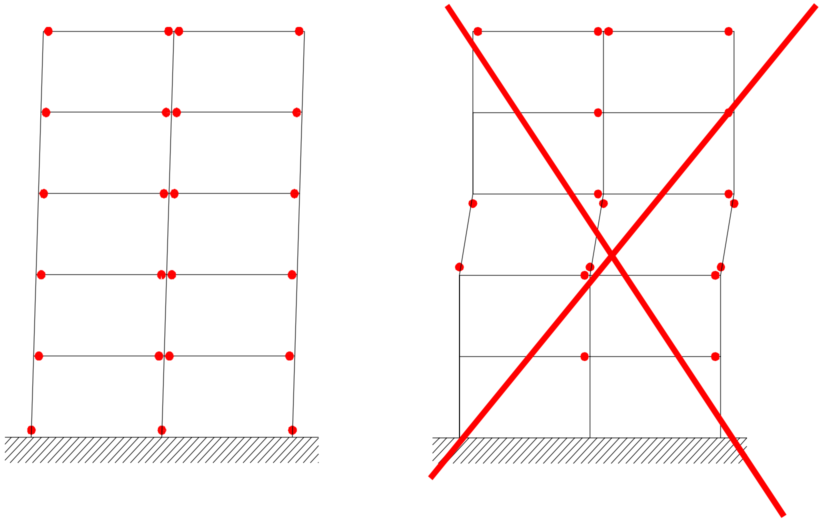


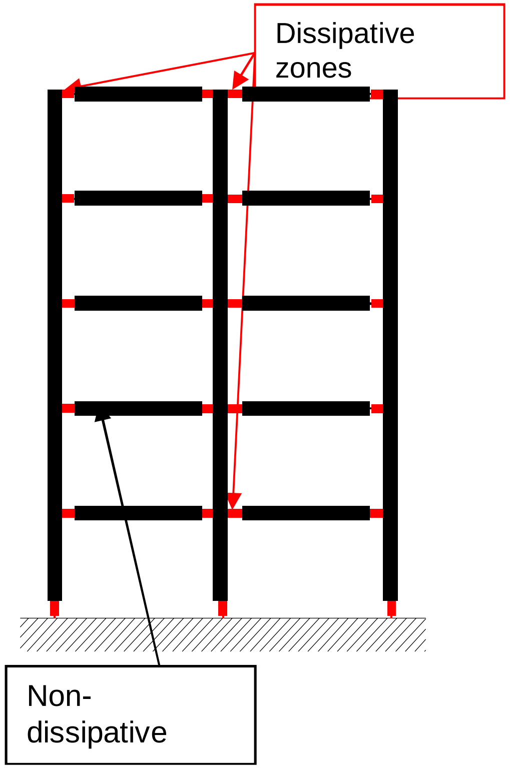
Global plastic collapse mechanism – vertical frames

- Promote global mechanism of vertical frames:
  - (1) the desired source of inelastic deformations e.g. **rotation in plastic hinges at beams-ends** (curvature ductility μ_φ) can be related to the displacement ductility factor (μ_δ) for the entire frame: $\mu_\phi = 2\mu_\delta - 1$
  - and further to the q factor:

$$\mu_\phi = 2q_o - 1 \qquad \text{if } T_1 \geq T_C$$

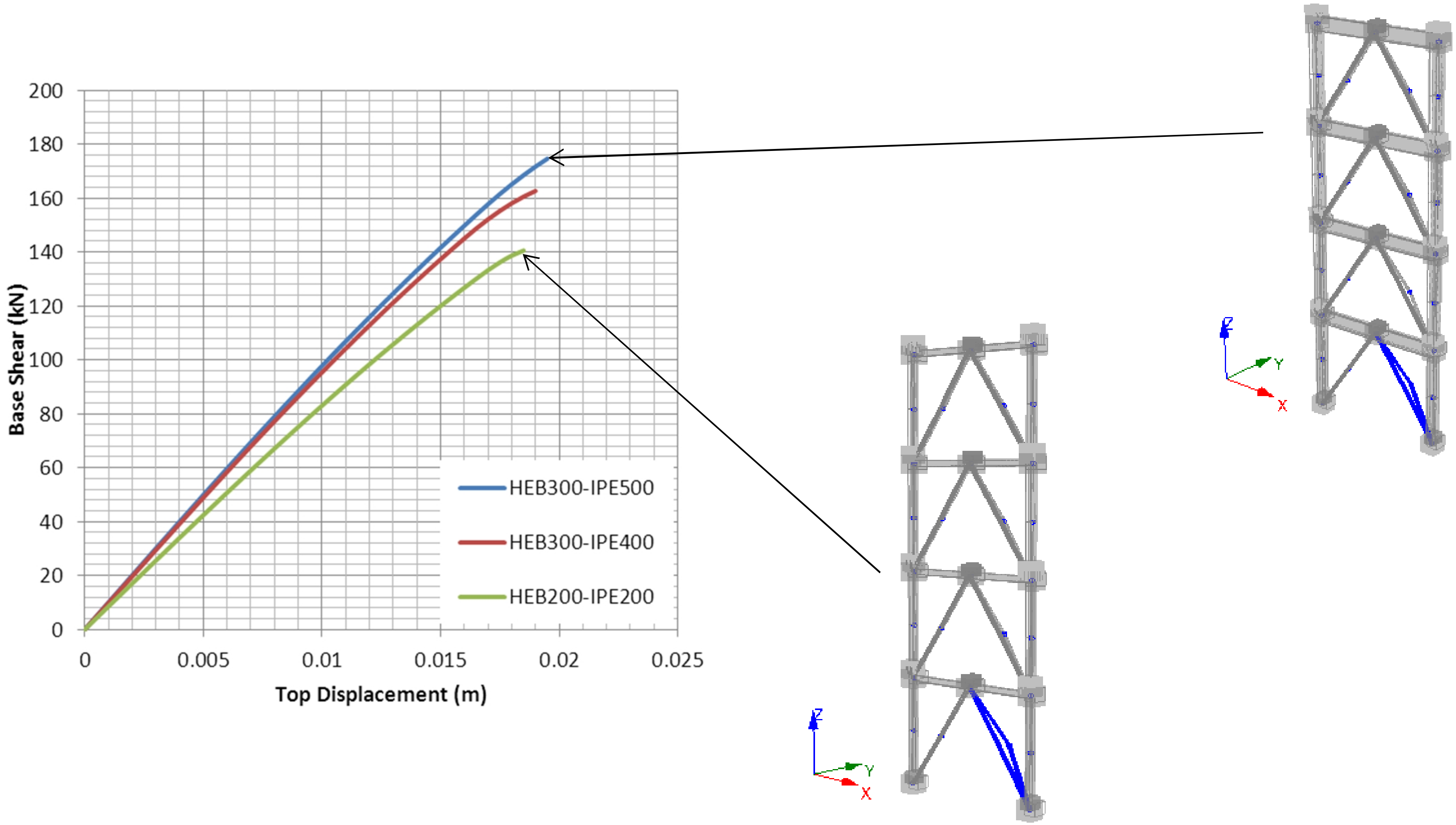
$$\mu_\phi = 1 + 2(q_o - 1)T_C/T_1 \quad \text{if } T_1 < T_C$$



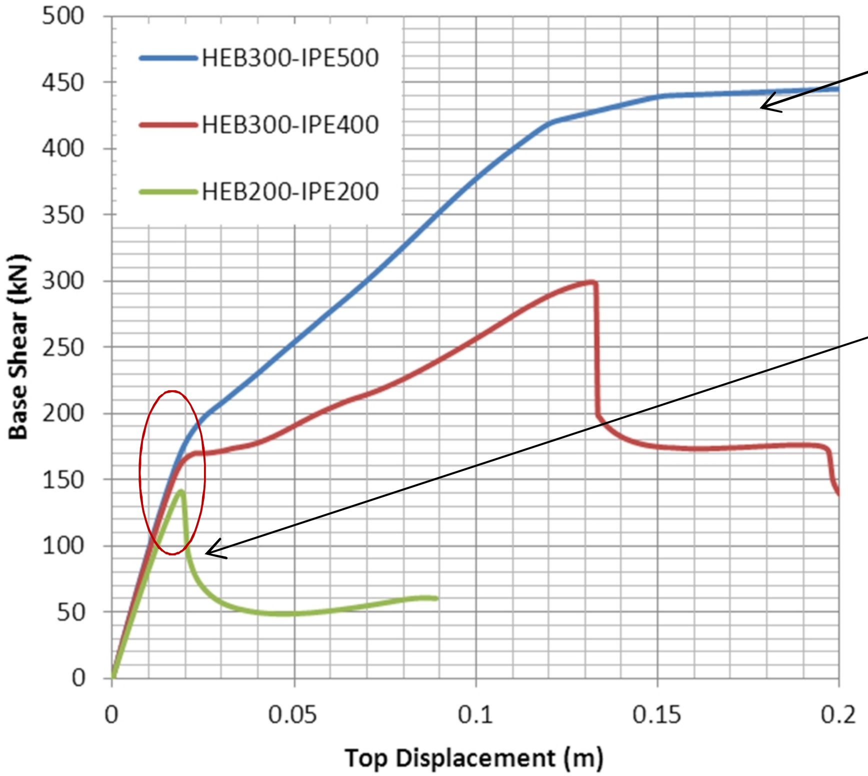


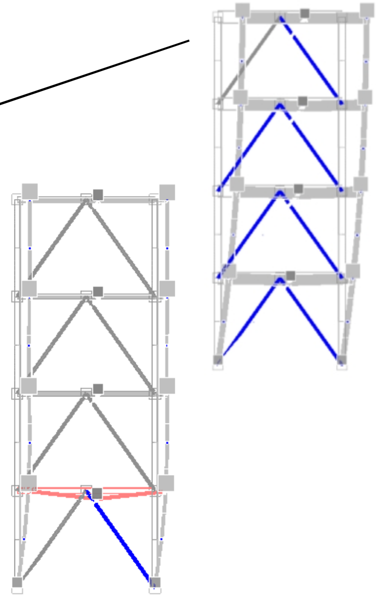
- Detailing of dissipative zones need to supply curvature ductility/local ductility μ_φ; over-strength rules need to be sufficient to concentrate plasticity in dissipative zones only.

Global plastic collapse mechanism – V braced frames (1)



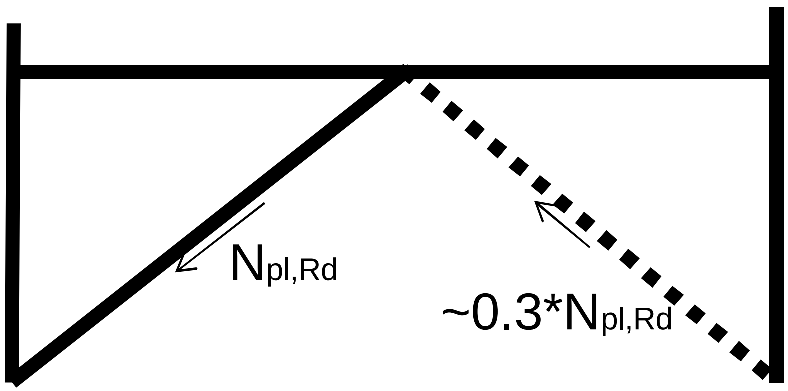
Global plastic collapse mechanism – V braced frames(2)





- (1) Not much difference in the elastic range – catch, designer calculations are in elastic only;
- (2) But huge difference in the inelastic range

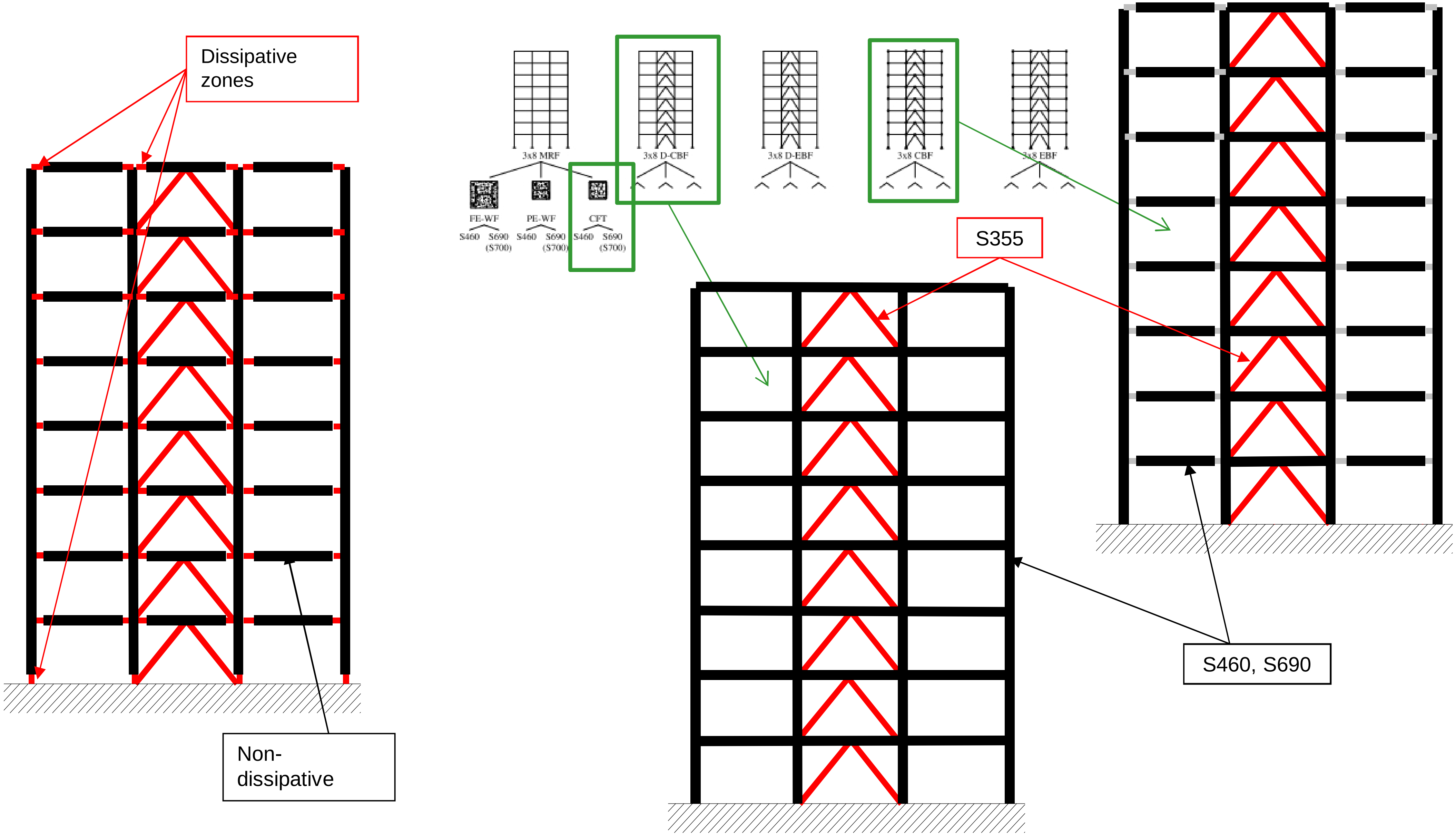
- Clearly, only configurations with strong horizontal beams should be acceptable:



- Clearly, only configurations with strong horizontal beams should be acceptable – so EN1998-1 states:

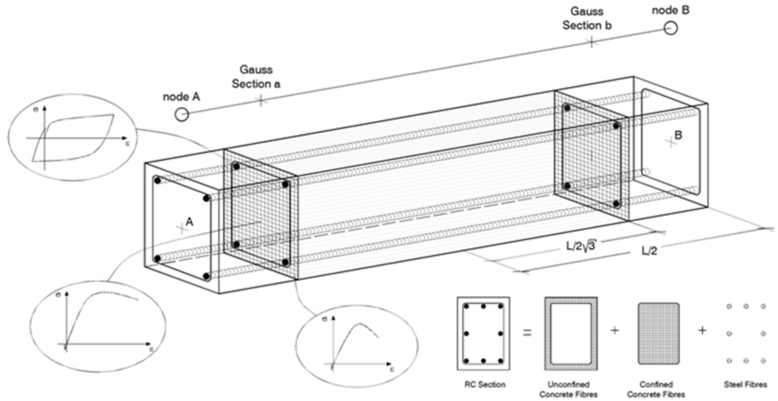
> – the unbalanced vertical seismic action effect applied to the beam by the braces after buckling of the compression diagonal. This action effect is calculated using $N_{pl,Rd}$ for the brace in tension and $\gamma_{pb}\ N_{pl,Rd}$ for the brace in compression.
>
> NOTE 1 The factor γ_{pb} is used for the estimation of the post buckling resistance of diagonals in compression.
>
> NOTE 2 The value ascribed to γ_{pb} for use in a country may be found in its National Annex to this document. The recommended value is 0,3.

RFCS project to study the posible use of HSS

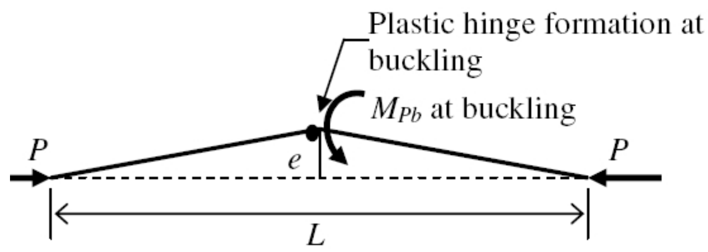


Particularities of the modeling

- Structures strictly designed to EN1998-1;
- Incremental dynamic analysis is used with 7 acceleration records selected to match the elastic spectra (EN1998-1);
- Material strength in the models correspond to mean strength (f_y=1.25*355=443N/mm^2, f_y=1.1*460=506N/mm^2). Only this can lead to the formation of most likely collapse mechanism.



- Members modeled as fiber elements. Takes into account bending and axial loads, shear deformation and failure needs to be modeled separately;



- Buckling of braces modeled using geometric imperfections calibrated to reach buckling to EN1993-1.

Result intrepretation

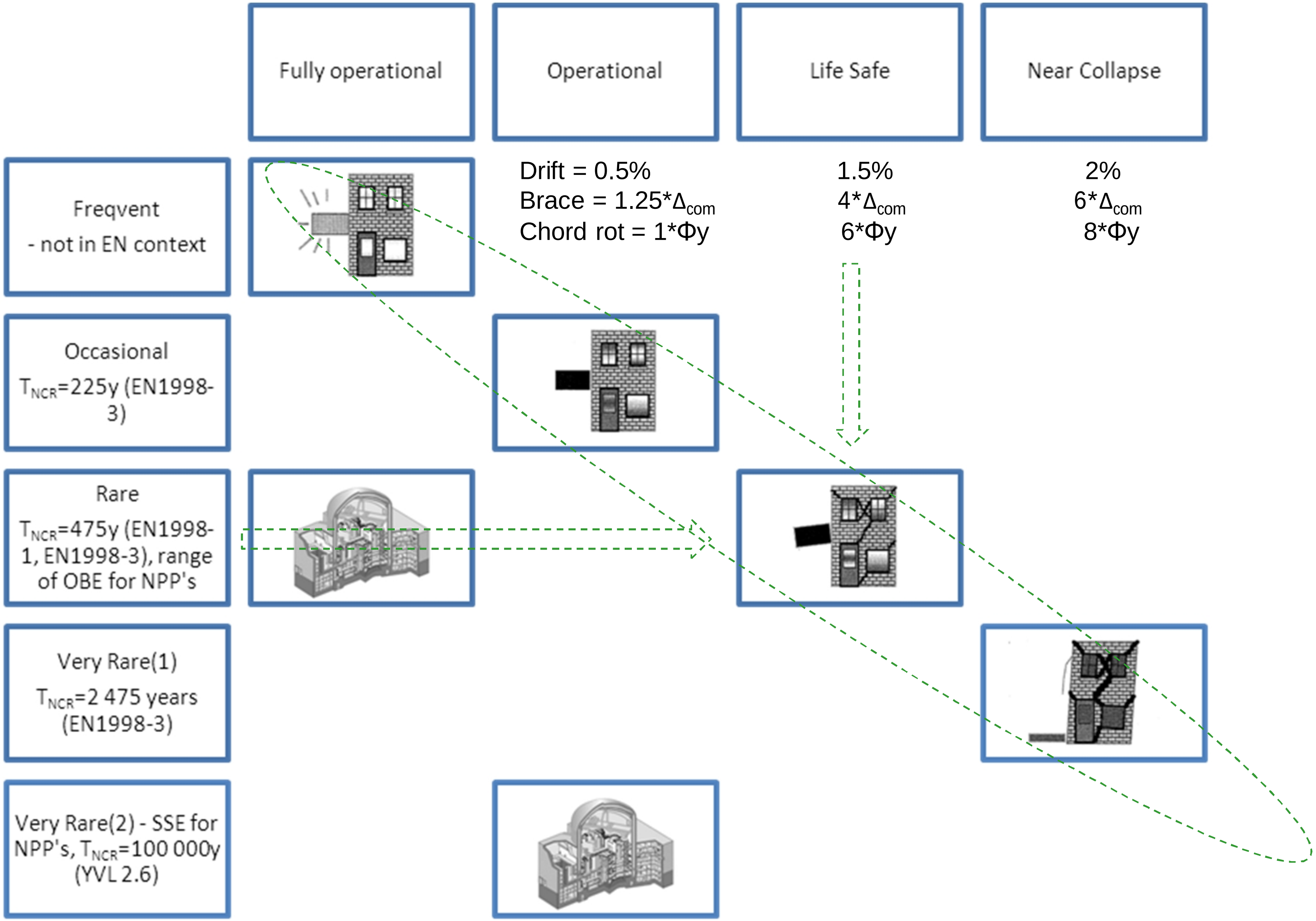
- Interpreted in the framework of performance design targets, as much as possible using Eurocodes:
  - Inter-story drift ratio based limits (FEMA);
  - Member deformation based limits (EN1998-1-3);

- Drift limits – 0.5%, 1.5% and 2% for damage limitation/immediate occupancy limit state (DL/IO), significant damage/life safety (SD/LS) and near collapse/collapse prevention (NC/CP)

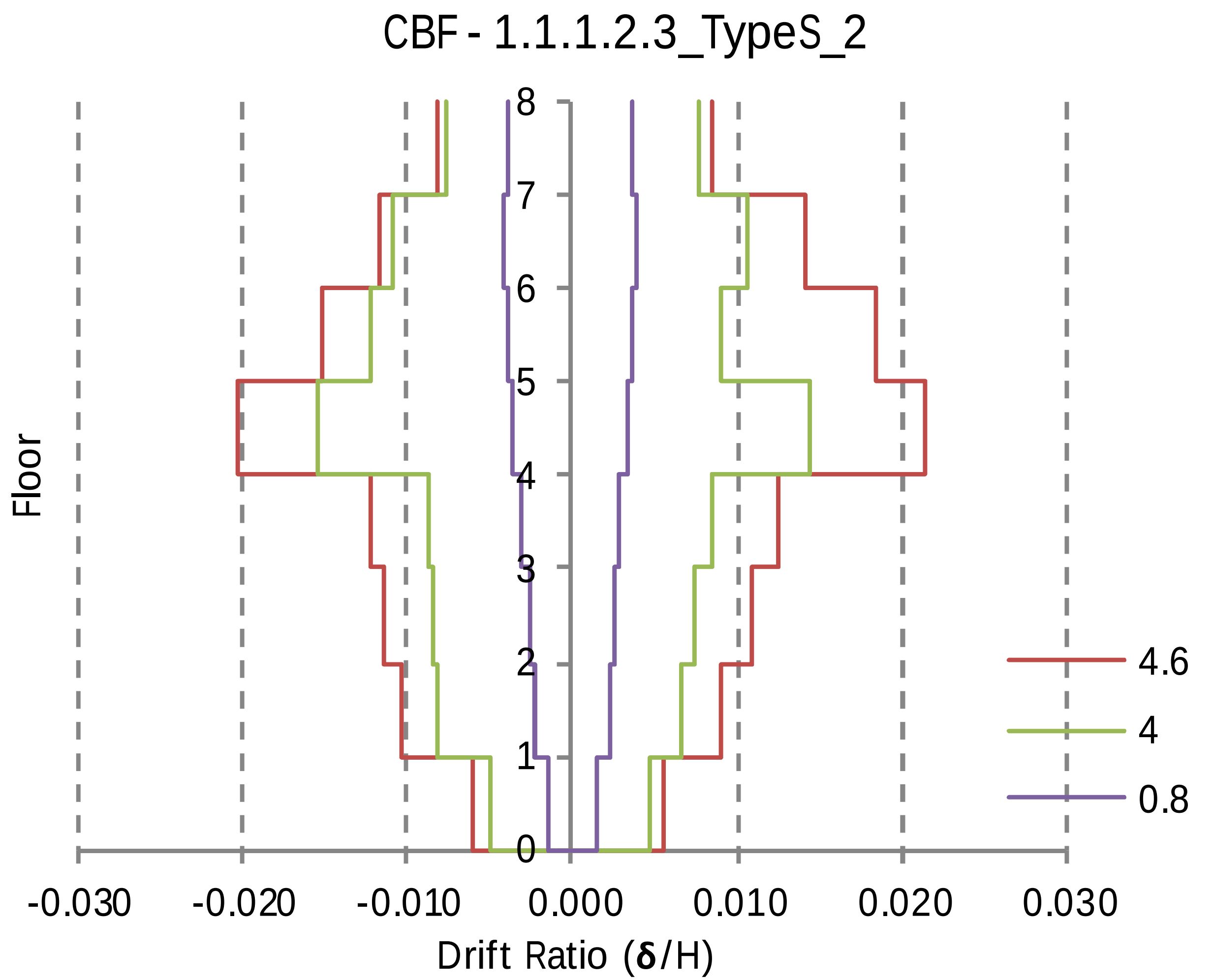
- Brace shortening – $1.25*\Delta_{com}$, $4*\Delta_{com}$, $6*\Delta_{com}$ – EN1998-3, Table B.2;

- Beams/Columns: – chord rotations corresponding to $1*\Phi y$, $6*\Phi y$, $8*\Phi y$ – EN1998-3, Table B.1.;

Performance based context for results

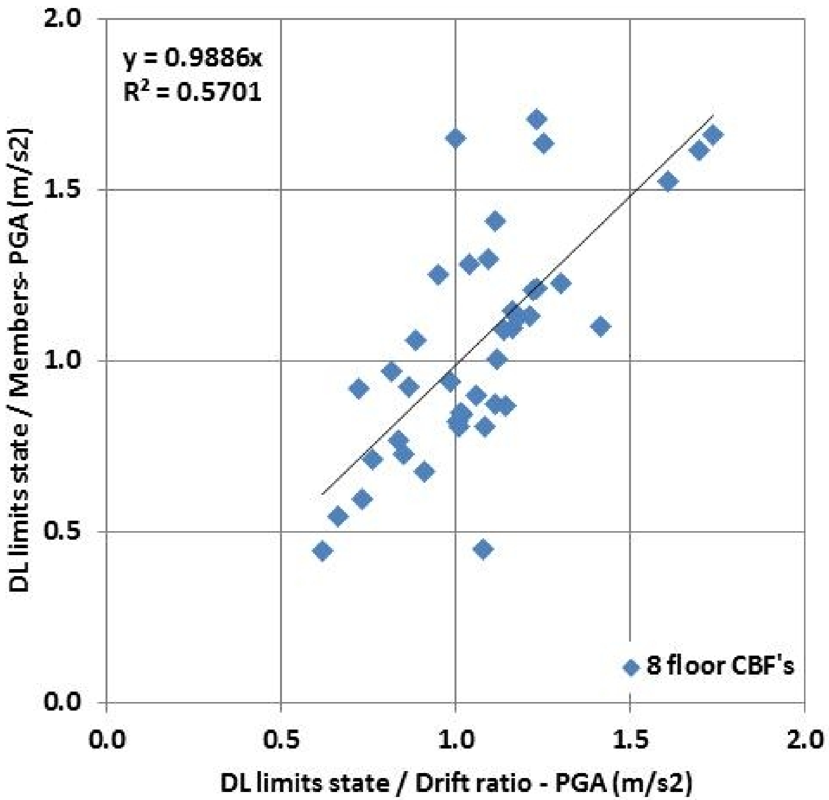


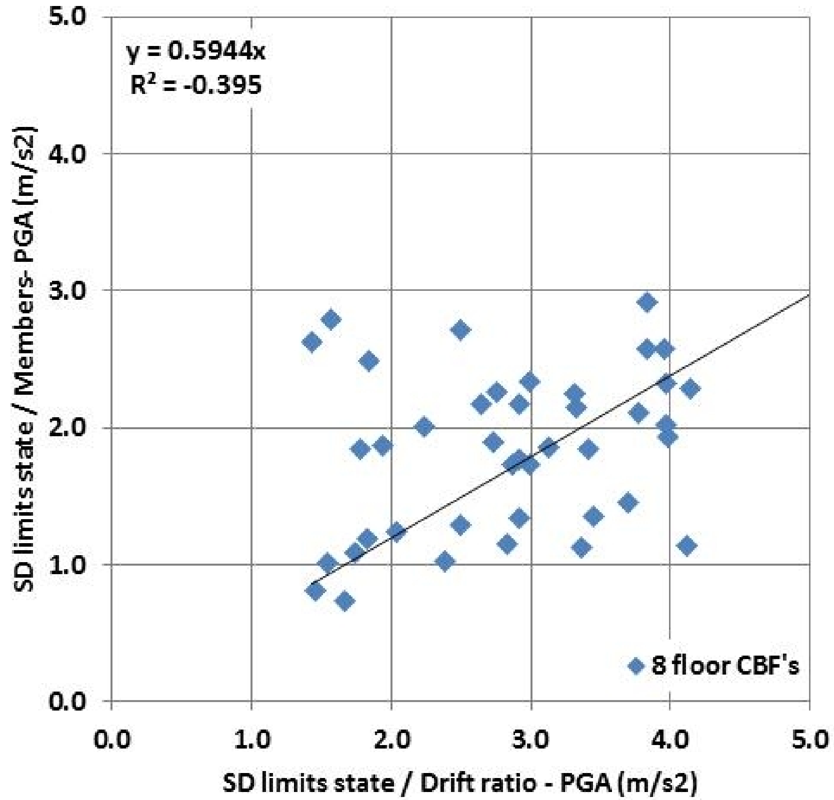
Drift based results



- DL limit state is reached at acceleration $0.8 m/s^2$ = $0.25 * PGA_{DSA}$) by drift ratio in floor 7;
- At SD limit state ($4m/s^2$ = $1.27 * PGA_{DSA}$), governed by 5^{th} floor drift, all floors except the ground floor has drifts suggesting presence of damage – frame beams work in redistributing forces between floors;
- The 5^{th} floor attracts significant damage in SD limit state due to the column change located in that floor;

Drift based limits compared to member based limits





Discussion

- Work is ongoing. Results are being centralized and interpreted for all configurations (CBF, DCB-F; EBF, DEB-F).

- Some preliminary conclusions on CBF and D-CBF:
  - The frames using HSS have comparable performance with traditional frames;
  - CBF and D-CBF frame behavior seem to be consistent with the design targets – braces control the performance, damage at SD limit state is spread in almost all floors;
  - The weakening of columns in floor 5 does attract a damage concentration. Maybe section change is too steep?! (*Go back to check the design.*)
  - Member (Brace) based performance limits is not consistent with Drift based limits – the brace criteria in EN1993-3 is stricter. (*Can not dispute it based on result from this project.*)

VTT creates business from technology