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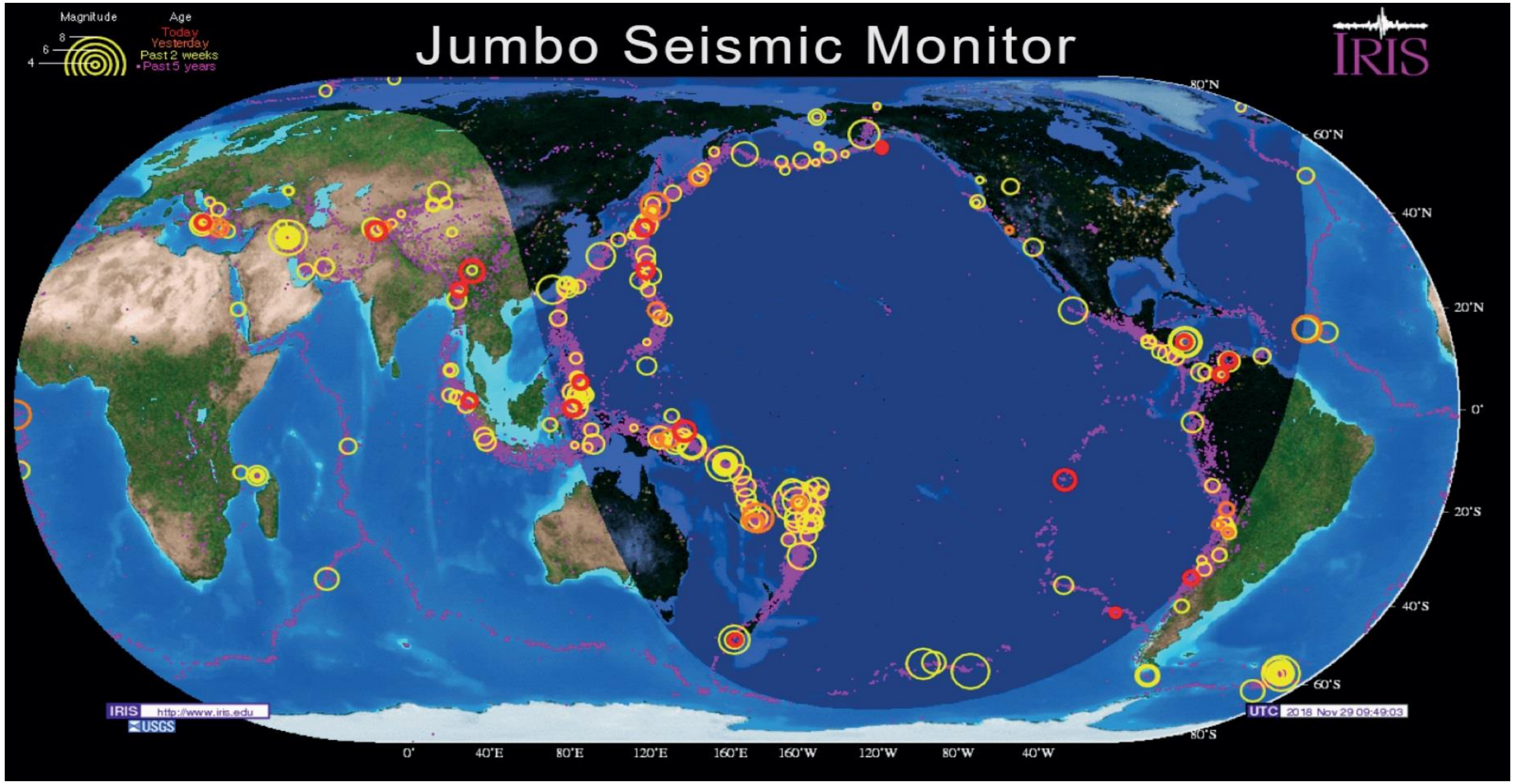
Progettazione sismica e regole di modellazione per le connessioni acciaio/calcestruzzo e componenti

Bologna

3 Ottobre 2019

Seismic events in the world:

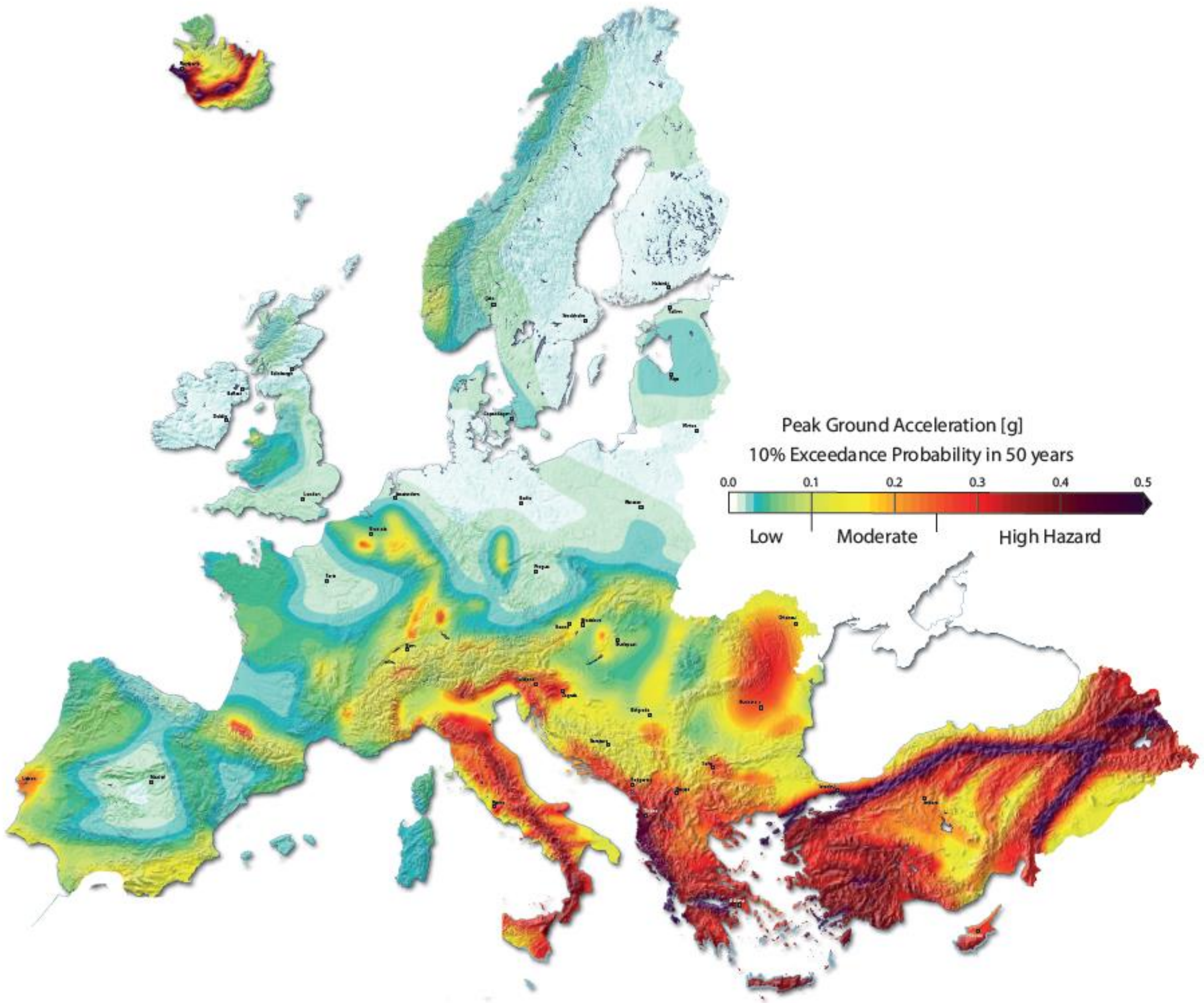
- happen everyday;
- are not possible to predict;
- could occur where a seismic event happened in the past.



Earthquake happened in the last five years till November 2018 - IRIS

Seismic Hazard Maps

- are plotted thanks to data collected about seismic events;
- correlate the hazard level to the maximum PGA (peak groud acceleration) recorded for a specific site;



Seismic hazard map from European Facilities for Earthquake Hazard and Risk EFEHR

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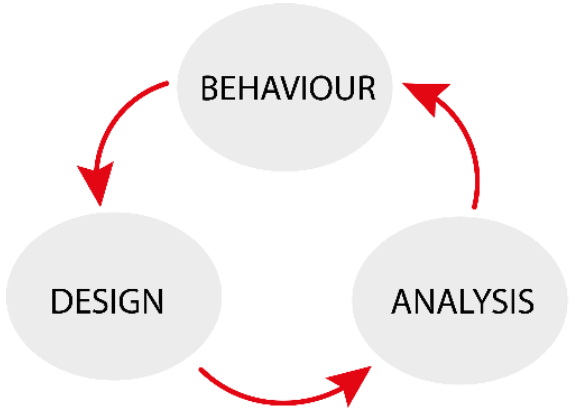
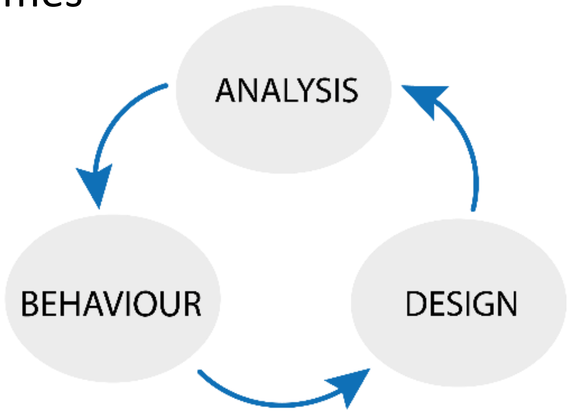
Analysis vs Design

A strong relationship exists between the actual behaviour of buildings and the adopted analysis and design methodologies. Both analysis and design are meant to finally achieve a certain structural behaviour, but the analysis is definitely the main tool to correlate design criteria and the imposed structural behaviour.

New building design starts from design-code, passing through the analysis to control/impose a desirable behaviour.



While for what concerns an **existing building**, the starting point is the analysis of the structural behaviour in order to evaluate the capacity of the structure, and then to eventually design effective retrofit schemes



Elastic design

In the elastic design no hierarchy of resistance is provided, the design is conduced based on the hypothesis that members remain in the elastic field and so no failure occur to the structure subjected to the design earthquake.

No ductile behaviour is expected and for that reason no ductility requirements are provided, in each element of the structure the design action effects have to be lesser than the resistance of the elements.

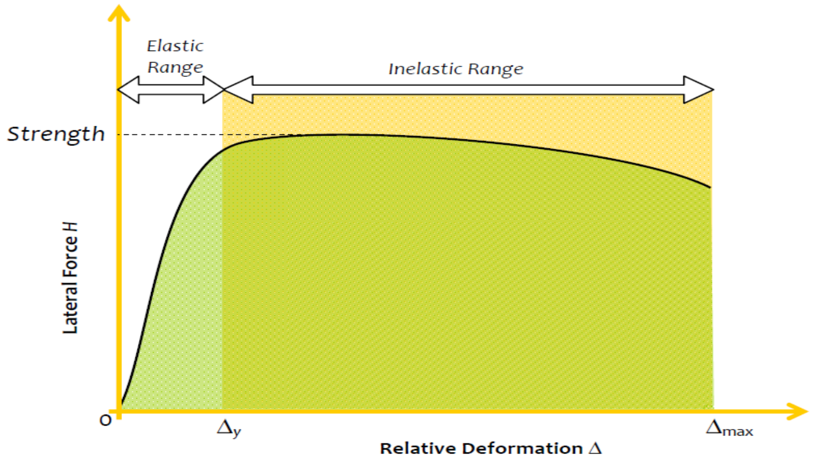
In the elastic design seismic action effects are not reduced, because of the lack ductile dissipation of energy in the building.

Noteworthy if the building is subject to an earthquake which exceed the design action effect, fragile failure of the members could occur

Elastic vs Inelastic behavior

Structures subjected to vertical and wind loads are designed to remain in the elastic regime of their response, while structures subjected to seismic action could undergo inelastic deformation.

In a generic force-deformation curve for a generic building, strength and displacement are directly proportional in the elastic range. Noteworthy is the tangent line to such response curve represents the stiffness of the systems. The latter parameter is constant in the elastic branch, while in the inelastic one the stiffness decreases owing to the occurred damage.



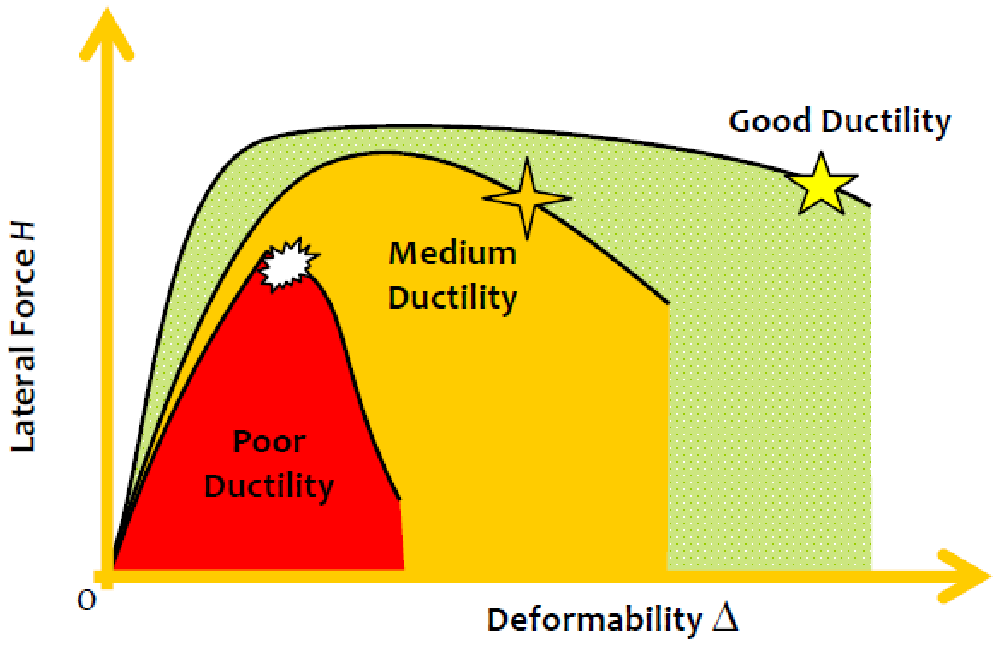
Ductility

Once pushed far away from the peak strength, the structure starts to decrease its stiffness and some of its elements could damage and then fail, making the structure collapse.

Ductility is the **capability to allow inelastic/plastic displacements without any substantial decrease in strength**.

It depends on the properties and construction detailing of the elements.

Ductile structures can accommodate large displacements before reaching failure, while fragile/low ductility structures can't survive large displacements after their strength peak is achieved.



Performance criteria

In Eurocode 8 “Design of structures for earthquake resistance”– Part 1, two different performance requirements are defined, as follows (2.1 of EN 1998-1):

- **No-collapse requirement.** The structure shall be designed and constructed to withstand the design seismic action without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events. The design seismic action is expressed in terms of the reference seismic action associated with a reference probability of exceedance, P_{NCR} of 10% in 50 years, corresponding a return period T_{NCR} of 475 years.
- **Damage limitation requirement**. The structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself. The design seismic action is expressed in terms of the reference seismic action associated with a reference probability of exceedance, P_{NCR} of 10% in 10 years, corresponding a return period T_{NCR} of 95 years.

Compliance criteria

The fundamental requirements shall be met for the corresponding limit states defined by international and national Code. The Eurocode 8 – Part 1 defines the following **design limit states**:

- *ultimate limit states*: they are associated with collapse or with other forms of structural failure which might endanger the safety of people;
- *damage limitation states*: associated with damage beyond which specified service requirements are no longer met.

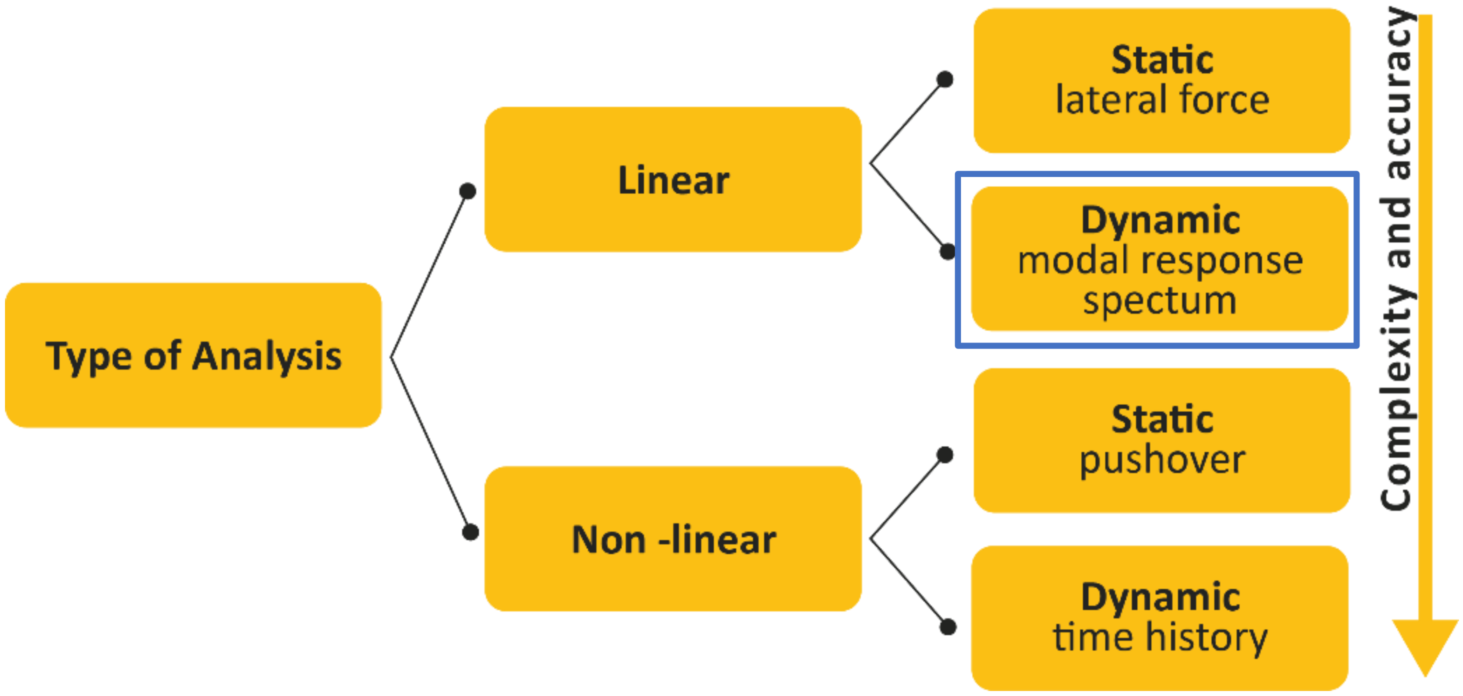
Analisys

Could be divided into two macro-categories:

- **linear**: elements have an elastic behaviour whatever the action imposed is.
- **non-linear**: the actual inelastic behaviour of the elements is taken into account and element’s parameters are set to reproduce the actual stress-strain response or moment-rotation relationships such that the actual nonlinear behaviour of the building can be simulated.

Could be split into other two sub-categories:

- **dynamic**: the analysis takes into account the dynamic nature of seismic-induced actions through the application of an accelerogram at the base of the structure, or it considers at least the dynamic properties of the structure;
- **static**: an equivalent distribution of forces simulated the inertia forces generate during earthquake shaking;



Select the type of the analysis

The type of the analysis should be selected according to:

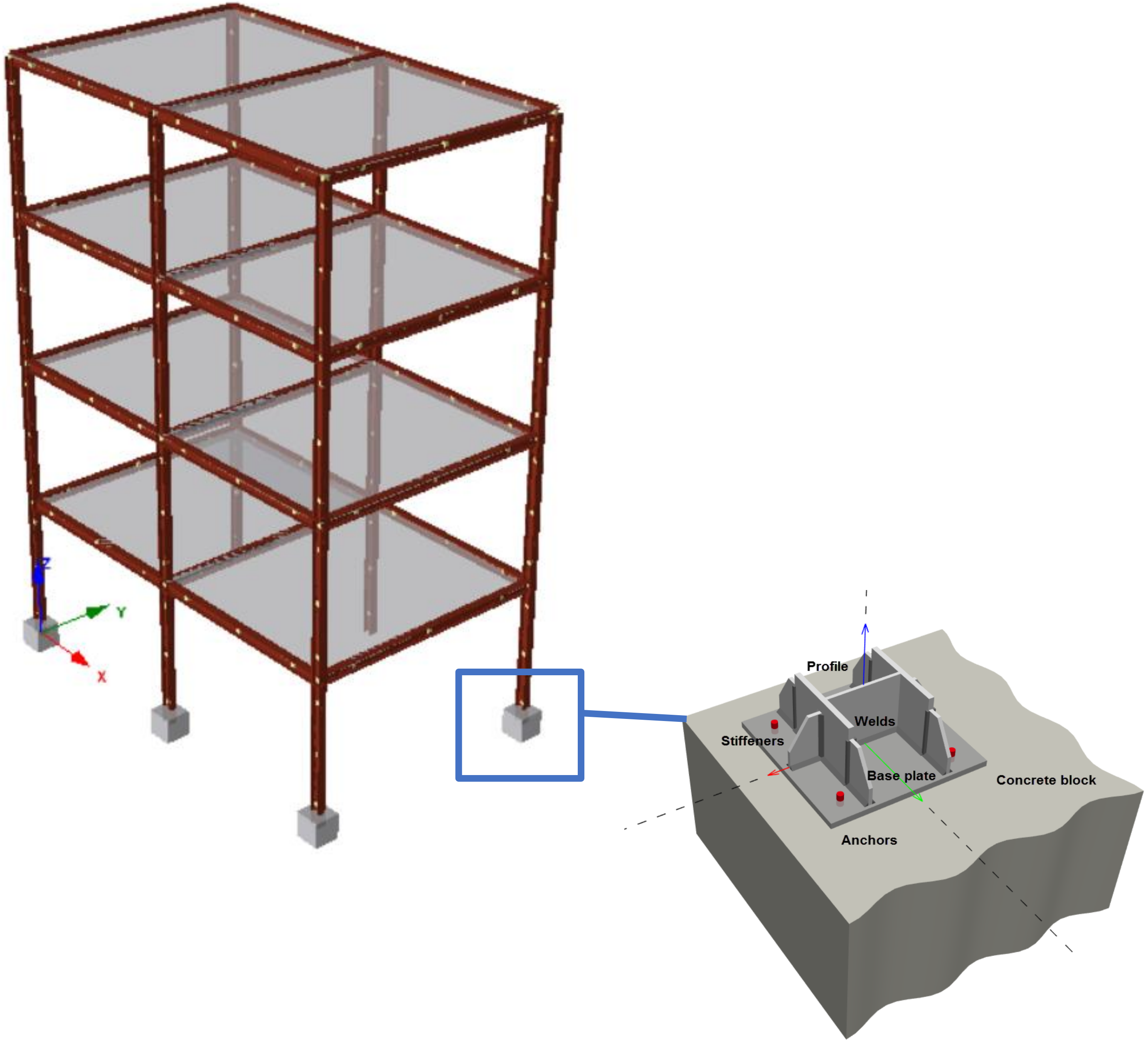
- trueness;
- reliability;
- complexity and precision required;
- duration.

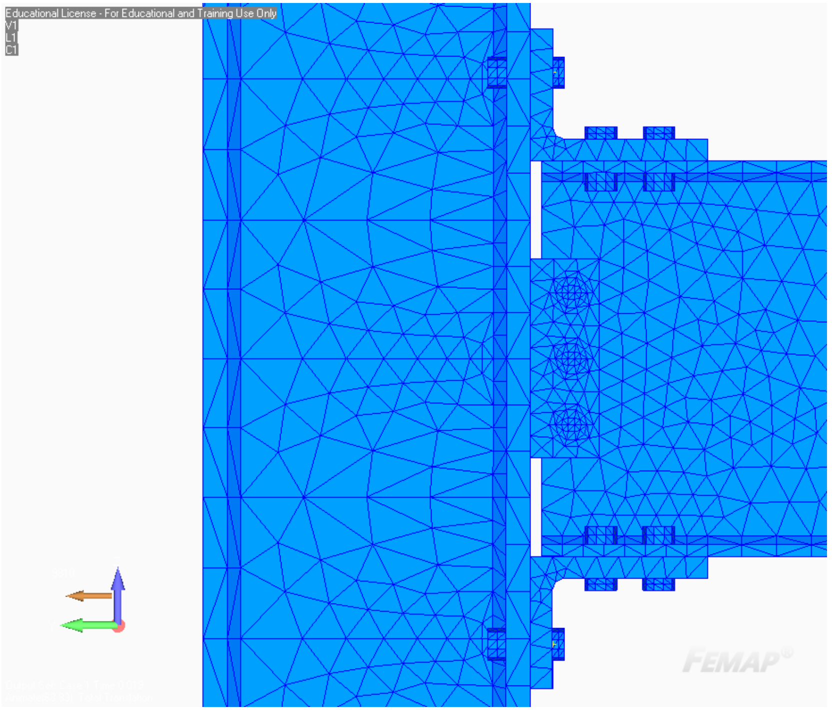
Modal response analysis is typically performed for design purpose

Global vs Local Design

Local/sectional approach starts from the design actions resulting from the global/structural analysis and allows the design of elements and components or connection components that are not explicitly included in the structural model.

Elements that do not change significantly the structural response are not necessarily included in the model. They are often considered in term of mass or they are simplified through springs or ad-hoc boundary conditions.

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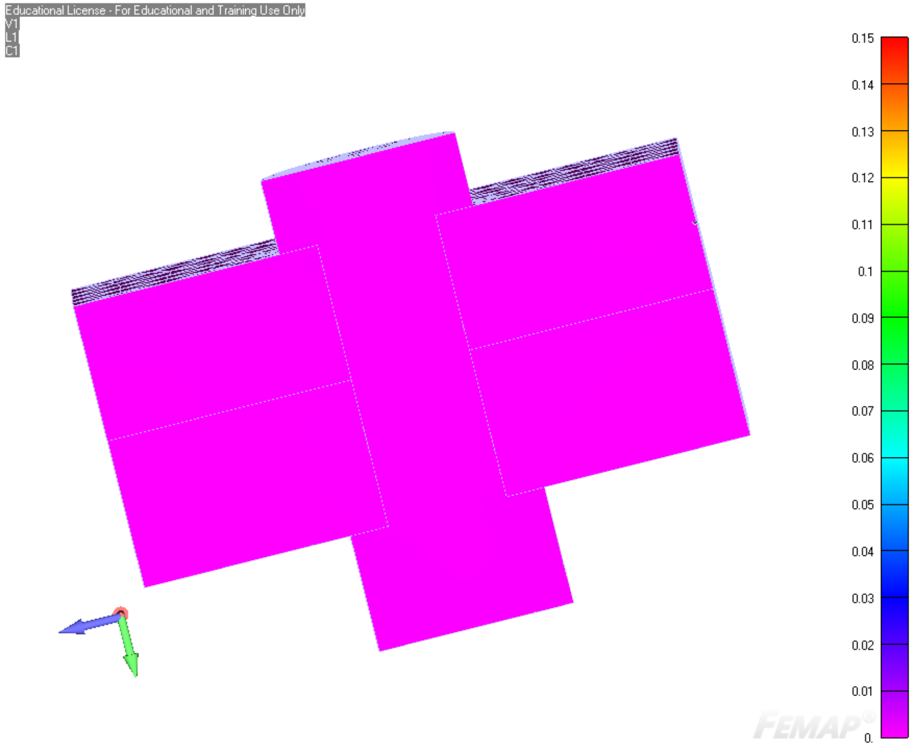
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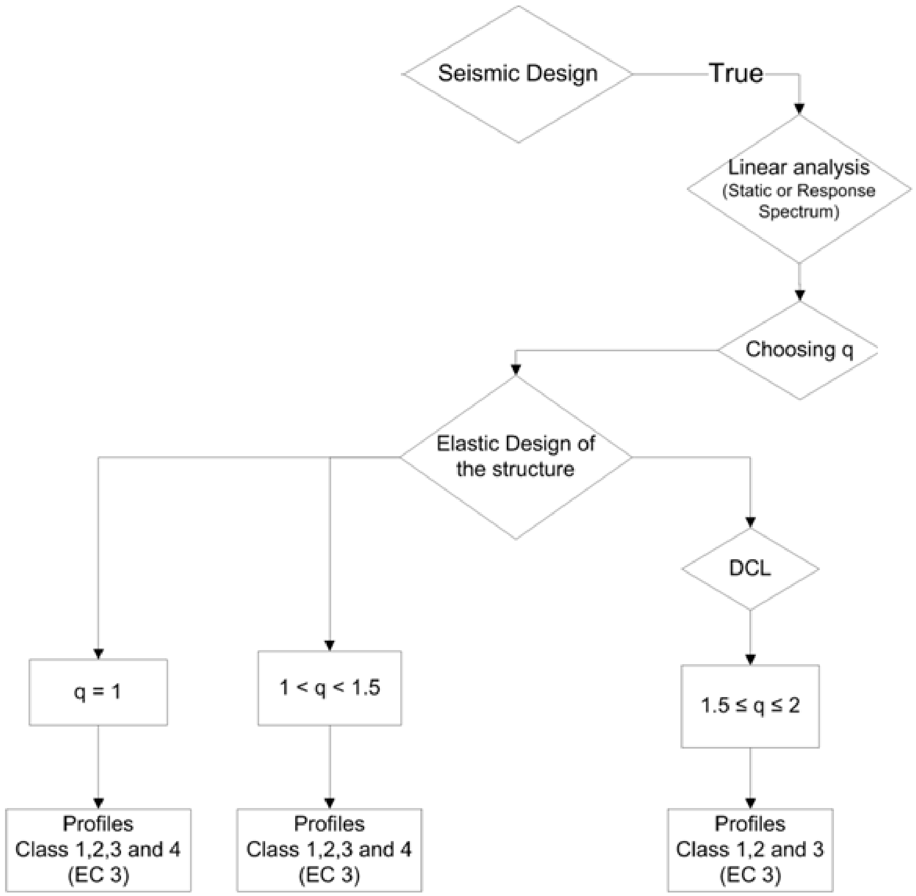
FEMAP

Preliminary Design procedure
in Profis

The action effects used as input in the software have to be outcome from a consistent global design procedure.

The following steps have to be reached in order to start the design of the connection on the software:

- type of analysis (Linear);
- behaviuor factor;
- Elastic design (Elastic/Low ductility DCL);
- limitations on class of the profile depending on the behaviour factor.



Code and technical documents for anchors/fasteners

The main technical documents referred to metal anchors for using in concrete are summarised by the following list:

- **EAD 330232-00-0601**, “Mechanical Fasteners for Use in Concrete” (this document supersedes ETAG001 – Part 1);
- **EOTA Technical Report TR029**, “Design of Bonded Anchors” (this document is superseded by EN 1992-4:2018);
- **EOTA Technical Report TR045**, “Design of Metal Anchors For Use In Concrete Under Seismic Actions”;
- **EOTA Technical Report TR049**, “Post-installed fasteners in concrete under seismic action” (this document supersedes ETAG001 – Part1: Annex E);
- **Eurocode 2 – Design of concrete structures – Part 4**: “Design of fastenings for use in concrete”.

The EAD 330232-00-0601 and the EOTA TR049 are reference documents to reach the technical approval, they focus on tests;

EC2-Part 4 and EOTA TR029 and TR045 are documents related to design of fasteners. In particular EOTA TR045 and chapter 9 of the EC2 – Part4 are dedicated to anchors under seismic action.

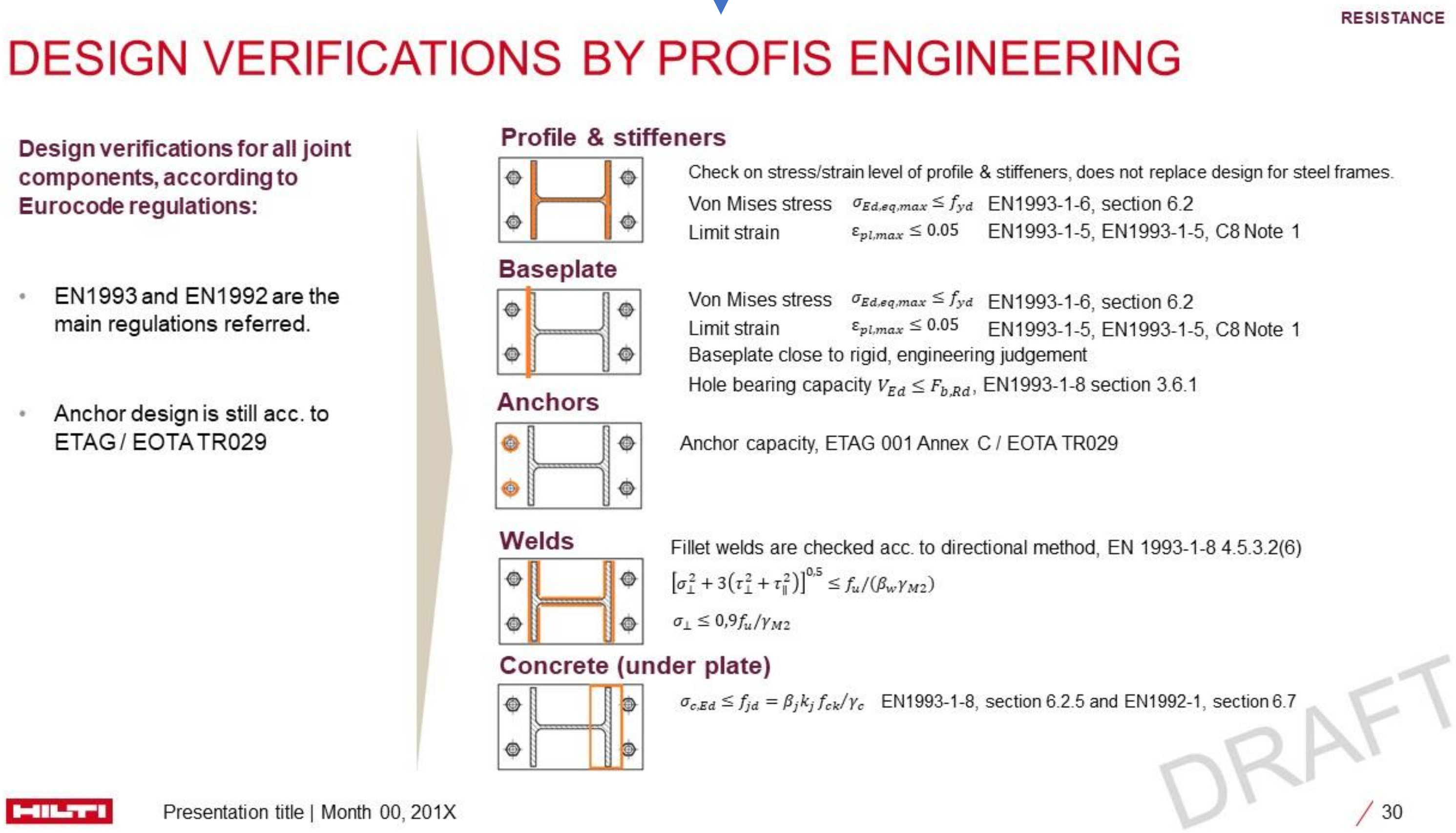
Requirements on base connection are found in EN 1993-1-8 chapter 6 “Structural joints connecting H or I sections”.

The design procedure of the base connection implies that all parts that compose the joint have to be designed in accordance with the corresponding standard code.

Design procedure of the connection

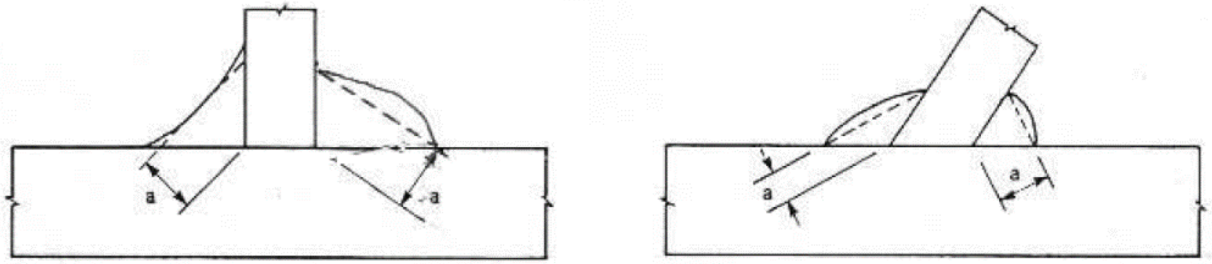
The design steps are the following ones:

- steel profiles (and stiffeners);
- welds;
- base plate;
- anchors;
- concrete under the base plate.



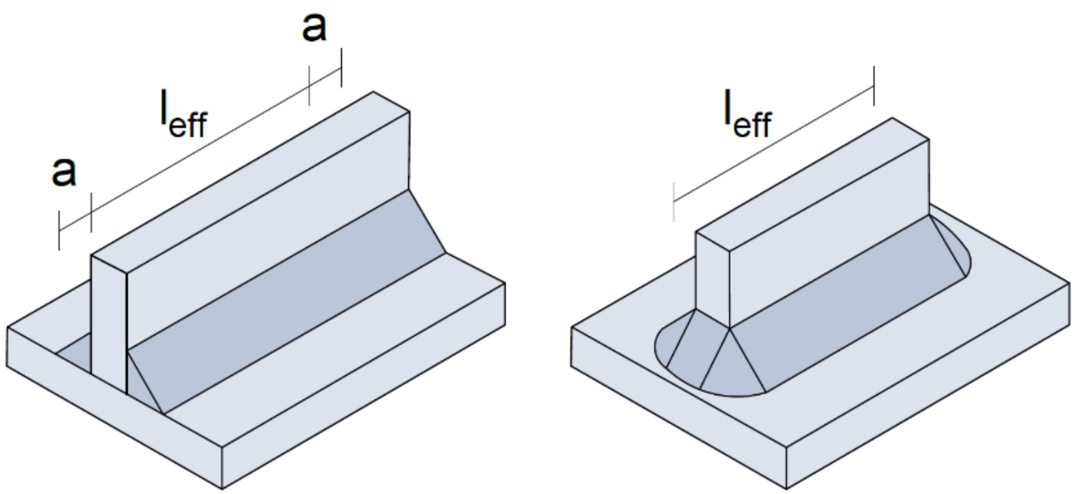
Fillet welds

Fillet welds may be used for connecting parts where the fusion faces form an angle of between 60° and 120°.
The effective length l_{eff} is assumed as the length over which the fillet is full-size.



The design throat area A_w is assumed to be concentrated in the root and it is considered as indicated below:

$$A_w = \sum a\, l_{eff}$$



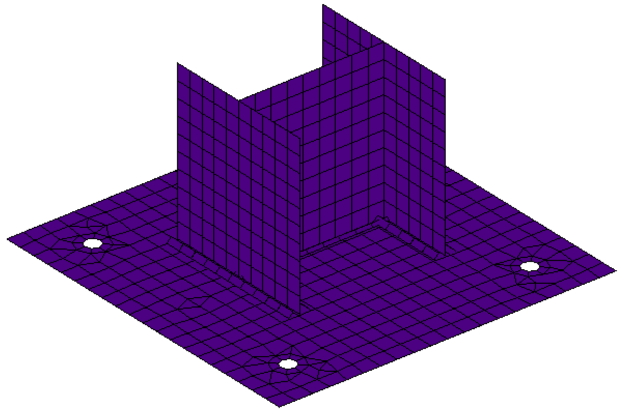
Design

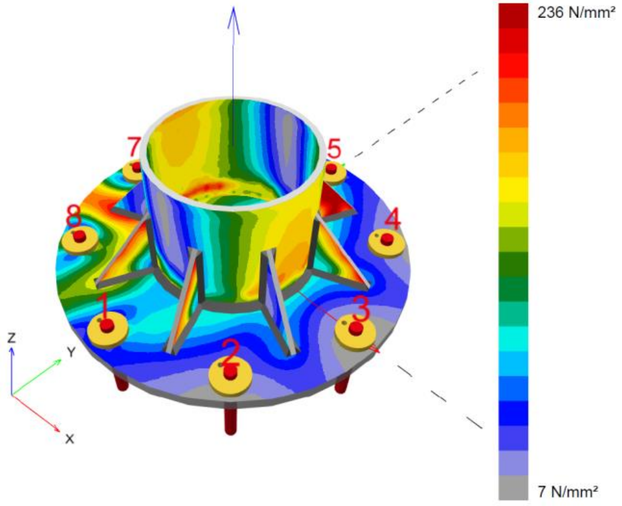
Directional method
(used in Profis)

Simplified method
(not used in Profis)

Base plate

The resistance of the steel base plate (fixture) is evaluated in terms of admissible tension in material.
At each point in the structure the design value of the stress $\sigma_{eq,Ed}$ should be taken as the highest primary stress determined in a structural analysis that considers the laws of equilibrium between imposed design load and internal forces and moments, according to 6.2.1 of EN 1993-1-6.





6.2 Stress design

6.2.1 Design values of stresses

(1) Although stress design is based on an elastic analysis and therefore cannot accurately predict the plastic limit state, it may be used, on the basis of the lower bound theorem, to provide a conservative assessment of the plastic collapse resistance which is used to represent the plastic limit state, see 4.1.1.

Base plate design stresses

The Von Mises equivalent resistance strength corresponds to the yielding strength of the material indicated in equation 6.5 of EN 1993-1-6:

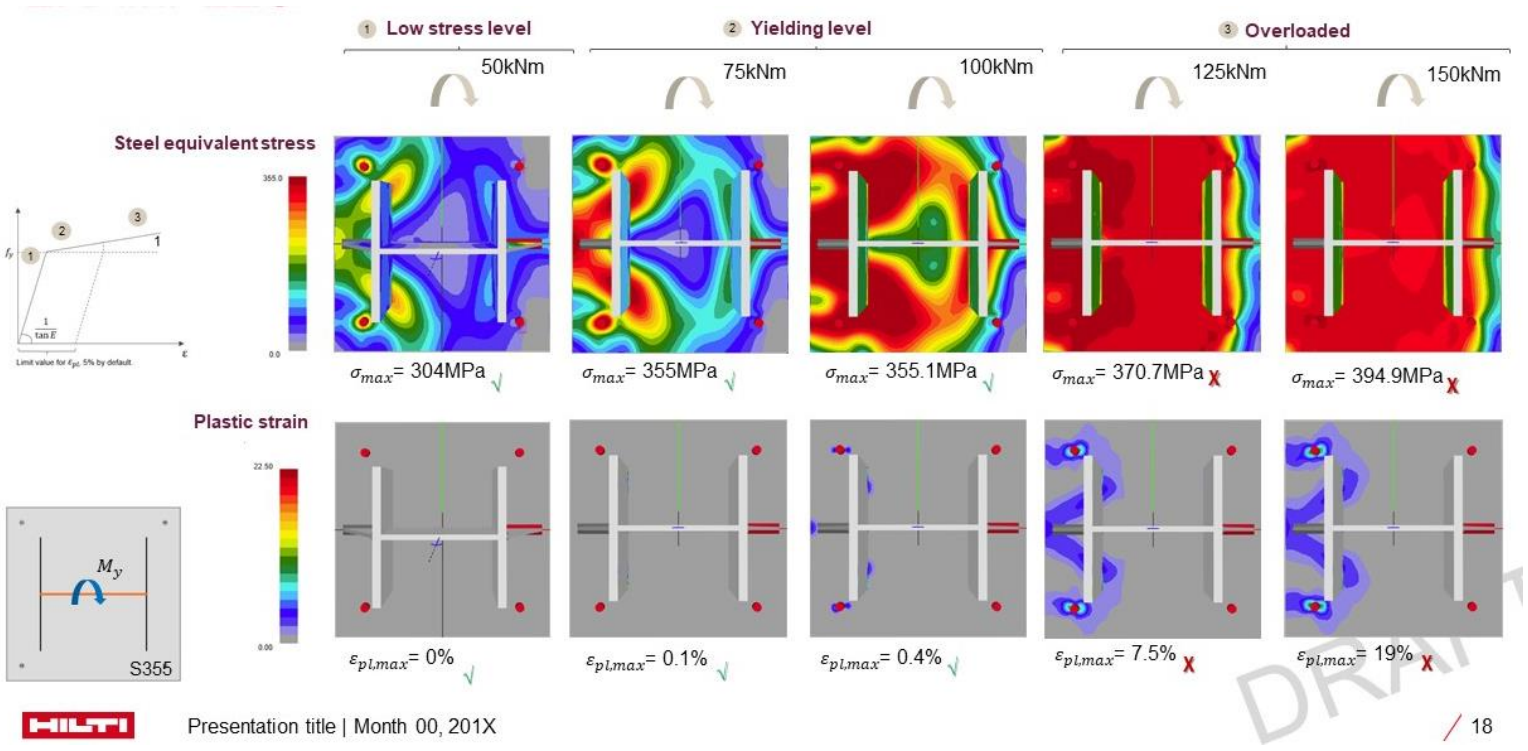
$$f_{eq,Rd} = f_{yd} = f_{yk} / \gamma_{M0}$$

where:

f_{yd}	design yielding strength;
f_{yk}	characteristic yieldingstrength;
γ_{M0}	is the partial factor for resistance of cross-sections whatever the class is, according to EN 1992-1-1;

In every verification the design stresses have to satisfy the equation 6.6 of EN 1993-1-6 in term of Von Mises equivalent stess:

$$\sigma_{eq} \leq f_{eq,Rd}$$



Metal anchors/fastener for use in concrete

Installed anchors have to sustain the design loads in tension, shear and combined tension and shear to which they are subjected for the assumed working life while providing:

- adequate resistance to failure (ultimate limit state);
- adequate resistance to displacements (serviceability limit state).

A minimum diameter of the thread or cross-section 6 mm (M6) is required. A maximum diameter of 60mm is required for shear loading, while no limits are imposed in tension.

The minimum embedment h_{ef} required amounts to 40 mm. For post-installed bonded anchors, it is permitted a maximum depth $h_{ef} \leq 20d$ to be covered by the Code.

Design criteria for fasteners under seismic actions

Only fasteners qualified for cracked concrete and seismic applications shall be used and they have to meet all the requirements provided in static design.

Two types of connections are distinguished according to C1(2) of EN 1992-4:

- **Type "A"**: connection between structural elements of primary and/or secondary seismic members according to EN 1998-1;
- **Type "B":** attachment of non-structural elements.

Requirements on performance of fastener (Table C.1 of EN 1992-4) are related to both seismicity level according to clause 3 of EN 1998-1) and importance class of the building (according to 4.2.5 of EN 1998-1).

Performance categories of fasteners

C1
provides capacities only in term of resistances at ultimate limit state

C2
provides capacities in terms of both resistances at ultimate and displacements at damage limitation state and ultimate state

Table C.1 of EN 1992-4

Seismicity level[a]		Importance Class acc. to EN 1998-1:2004, 4.2.5			
Class	$a_g \cdot S^c$	I	II	III	IV
Very Low[b]	$a_g \cdot S \leq 0{,}05\ g$	No seismic performance category required			
Low[b]	$0{,}05\ g < a_g \cdot S \leq 0{,}1\ g$	C1	C1[d] or C2[e]		C2
> low	$a_g \cdot S > 0{,}1\ g$	C1	C2		

[a] The values defining the seismicity levels are subject to a National Annex. The recommended values are given here.

[b] Definition according to EN 1998-1:2004, 3.2.1.

[c] a_g = design ground acceleration on type A ground (see EN 1998-1:2004, 3.2.1),

S = soil factor (see EN 1998-1:2004, 3.2.2).

[d] C1 for fixing non-structural elements to structures (Type 'B' connections).

[e] C2 for fixing structural elements to structures (Type 'A' connections).

Approaches on design of the fastener

In order to design the fasteners for the base connection different approaches are proposed, depending on the ductility required for the fastener. The selected design approach shall be in accordance with the assumptions made for ductility, for the design of the structure, as specified by EN 1998-1.

In **9.2 of EN 1992-4** three design approaches are listed (options **a1), a2) or b)**), approaches which could be selected.

Fasteners could be verified omitting requirements given in 9.2 of EN 1992-4, if:

the seismic design tension load applied to a single or a group of fasteners, for seismic design situation at the ultimate limit state, is equal to or less than 20% of the total design tensile load for same load combination;

the seismic design shear load applied to a single or a group of fasteners, for seismic design situation at the ultimate limit state, is equal to or less than 20% of the total design shear load for same load combination;

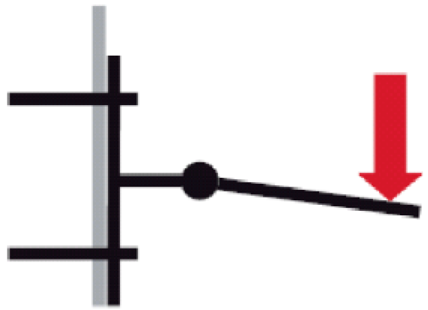
According to C3(2) of EN 1992-4 for the design of anchors according to design option a2) “elastic design” the action effects for connections shall be derived according to EN 1998-1 with a behaviour factor **q = 1,0.**

Approach **a)**

design **without requirements on the ductility** of the fasteners (such as no dissipative elements without ductile hysteretic behaviour). No contribution to the overall ductility is considered:

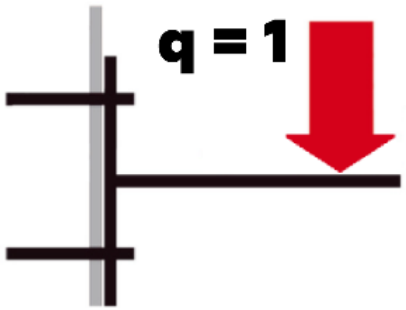
a1) Capacity design

“the fastener or group of them is designed for the maximum tension and/or shear load that can be transmitted to the fastening, based on either the development of a ductile yield mechanism in the fixture or the attached element taking into account strain hardening and material over-strength or the capacity of non-yielding attached element” (see 9.2(3)a1)of EN 1992-4).



a2) Elastic design

“the fastening is design for the maximum load obtained from the design load combinations that include seismic actions E_{Ed} *corresponding to the ultimate limit state (see EN 1998-1) assuming elastic behaviour of the fastening and the structure. Furthermore, uncertainties in the model to derive seismic actions on the fastening shall be taken into account”* (see 9.2(3)a2) of EN 1992-4).



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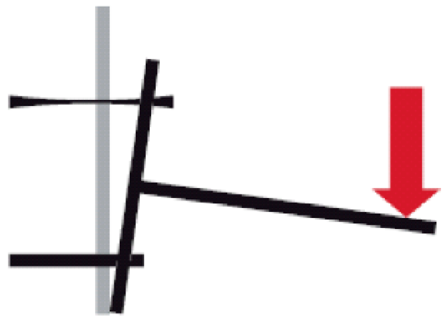
Approach **b)**

b) design **with requirements on ductility**

of the fasteners (applicable only for tension load component of the acting load). *“The tension steel capacity of the fastenings have to be smaller than the tension capacity governed by concrete related failure modes. Sufficient elongation capacity is required. Fasteners should not be accounted for energy dissipation in capacity of the structure* […] *is not addressed within this standard”* (see 9.2(3)b) of EN 1992-4).

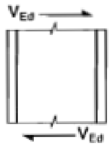
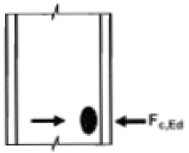
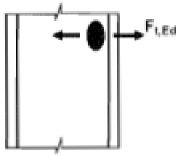
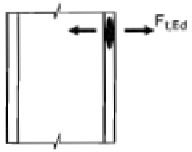
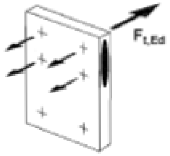
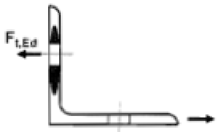


Design **with requirements on ductility** is not permitted for the fastening of primary seismic members, so it is not possible to apply to the base connection of columns (Hilti portfolio N/A).
It is possible to consider, in option b), the energy dissipation only performing a dynamic non-linear time history analysis (NLTHA) and providing the hysteretic behaviour from a European Technical Product Specification

Steel to concrete base connection EN 1993-1-8

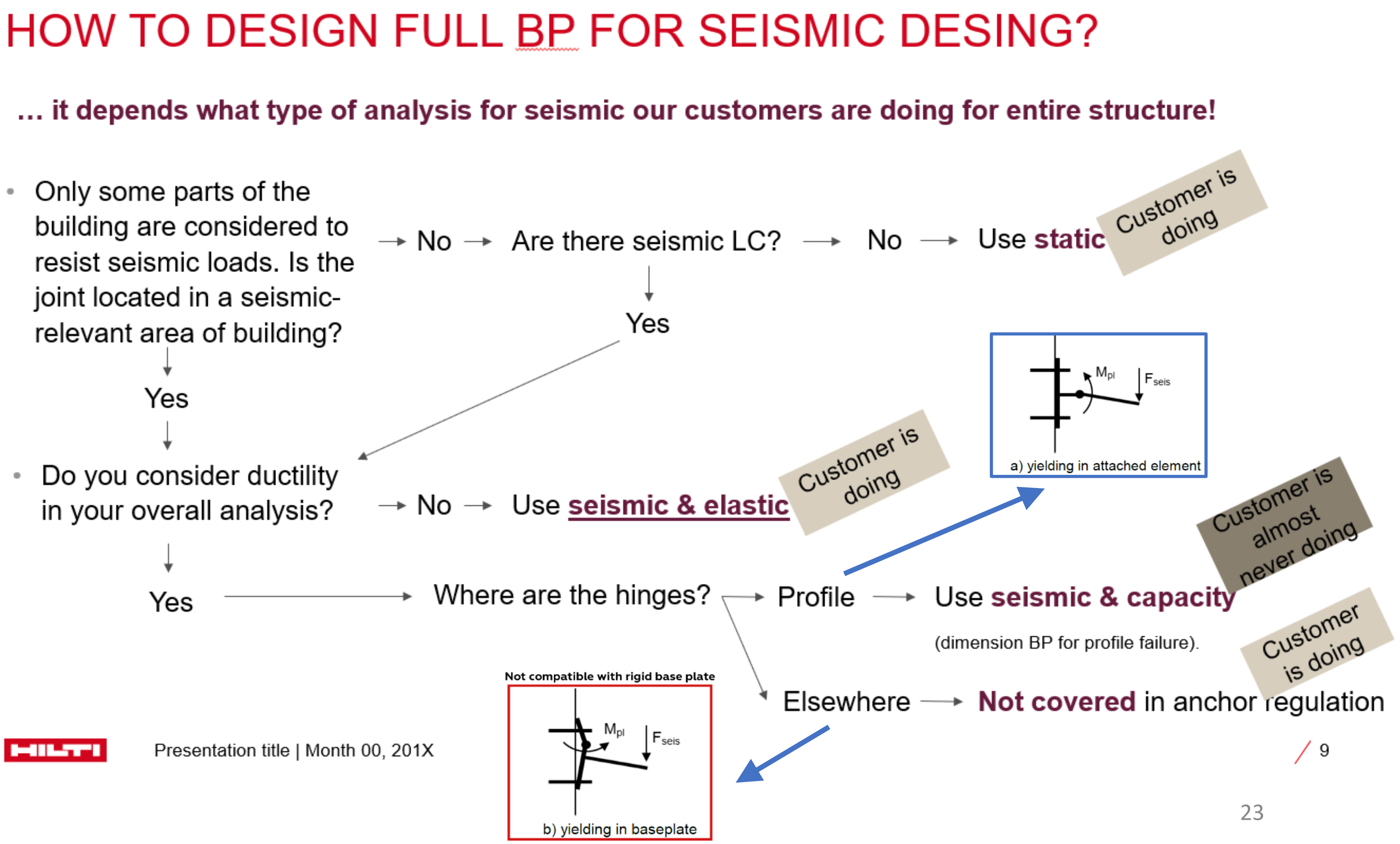
A part of the Eurocode 3 is dedicated to the design of joints in steel structures (EN 1993-1-1-8).
The specific design methods given for determining the design moment resistance of a joint are based on an assumed distribution of internal forces. A joint may be represented by a **rotational spring connecting the centre lines of the connected members at the point of intersection.**
Joints are composed by a series of **basic component** and the design moment-rotation characteristic of a joint depend on the properties of its basic components. The basic components are listed in Table 6.1 of EN 1933-1-8.

Table 6.1: Basic joint components

Component			Reference to application rules		
			Design Resistance	Stiffness coefficient	Rotation capacity
1	Column web panel in shear		6.2.6.1	6.3.2	6.4.2 and 6.4.3
2	Column web In transverse compression		6.2.6.2	6.3.2	6.4.2 and 6.4.3
3	Column web in transverse tension		6.2.6.3	6.3.2	6.4.2 and 6.4.3
4	Column flange in bending		6.2.6.4	6.3.2	6.4.2 and 6.4.3
5	End-plate in bending		6.2.6.5	6.3.2	6.4.2
6	Flange cleat in bending		6.2.6.6	6.3.2	6.4.2

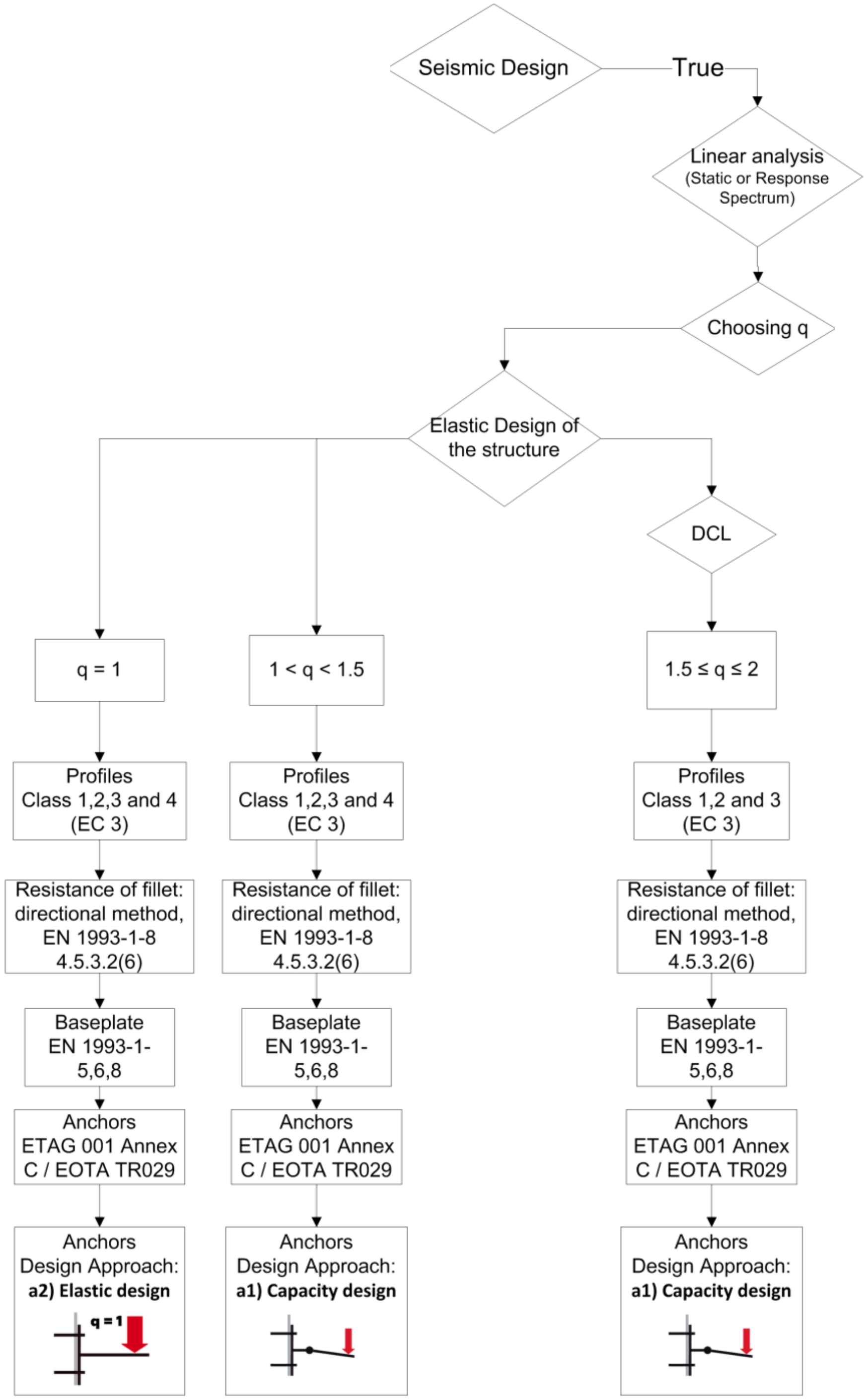
Current level of knowledge and further developments or needs

The main outcomes of the investigation, whose pivotal results are summarised in this report, align very well with the state-of-the art and possible (design) situation depicted by the following figures, as received by Hilti.



Design procedure

This flowchart, which is identical to the one see before, now finds its way into this document and the information contained therein could be merged with the detailed explanations given in the above Sections, on a point-by-point basis.

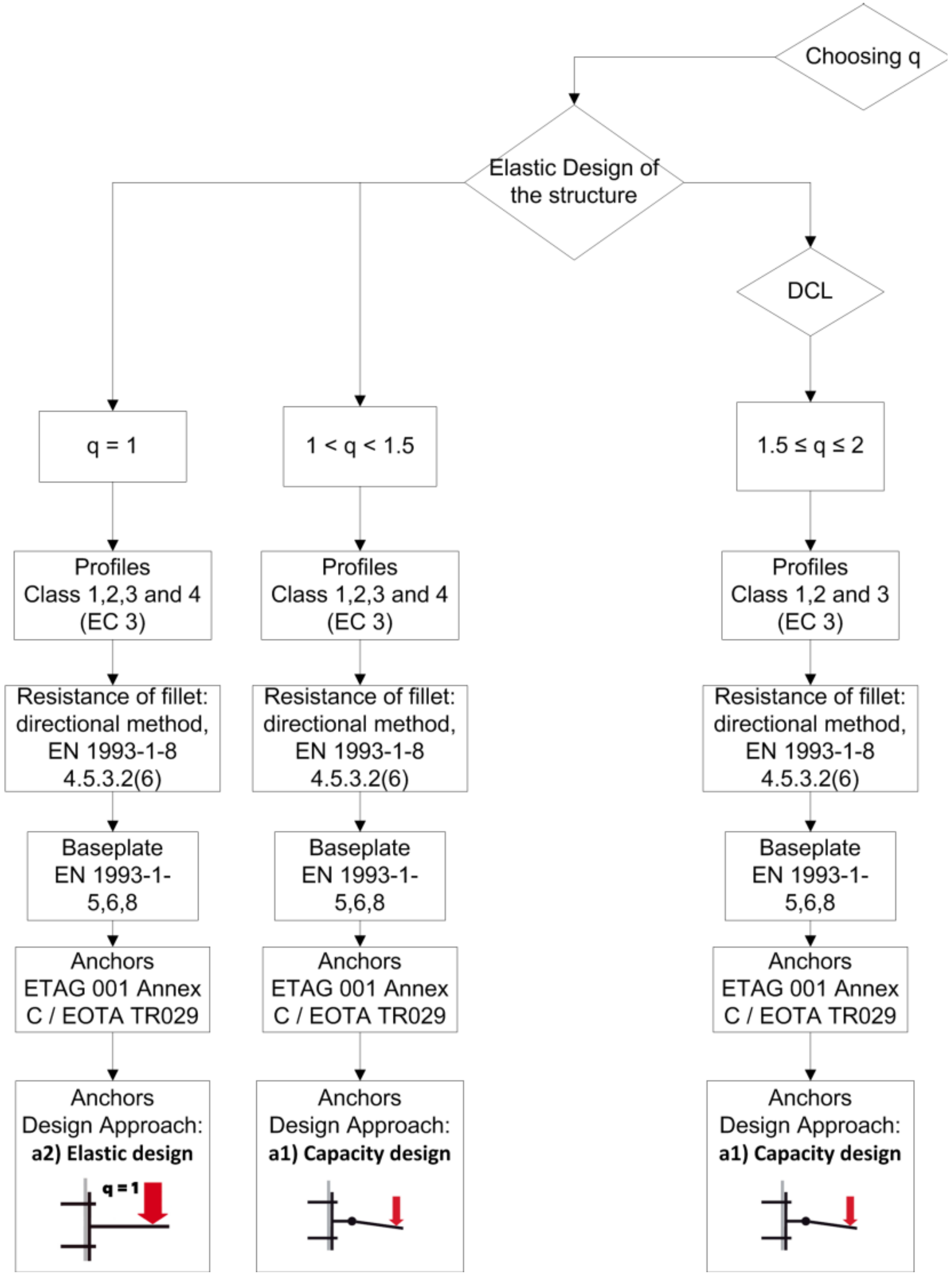


Design procedure for linear analysis

This focus is shows the design procedure for base connection of steel structures, under the hypothesis of perform linear analysis.

Two main design approach are possible for the structures, depending on the selected behavior factor q:

- **Elastic design;**
- **Capacity design**.



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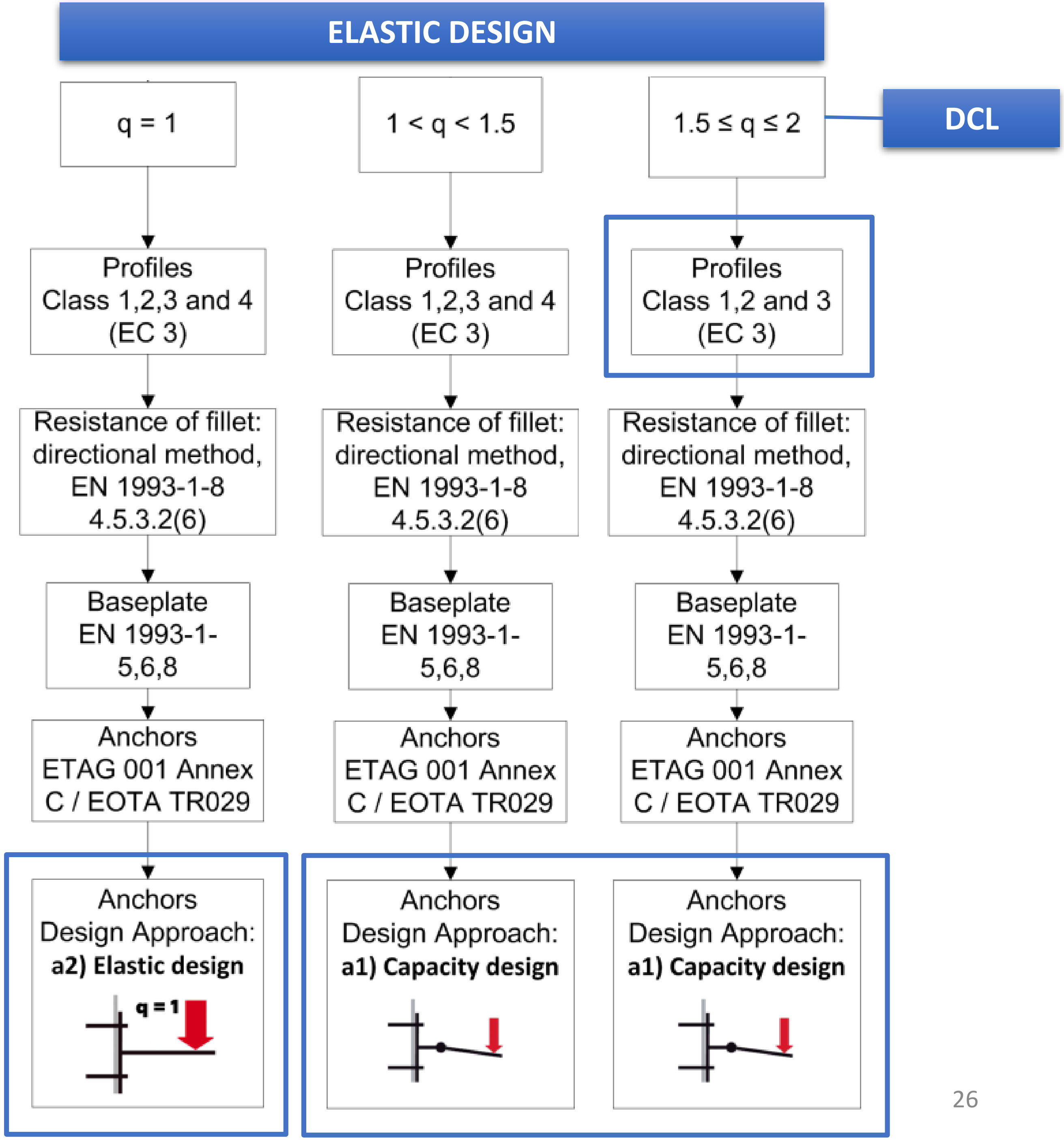
Design procedure for Elastic and Low dissipative DCL behaviour

This focus on the flowchart represents the design procedure to be adopted in order to design the base connection of a steel structure assuming ‘*pure elastic*’ behaviour (q=1) or ‘*substantially elastic*’ (1 < q < 5) or low dissipative behaviour (1.5 ≤ q ≤ 2).



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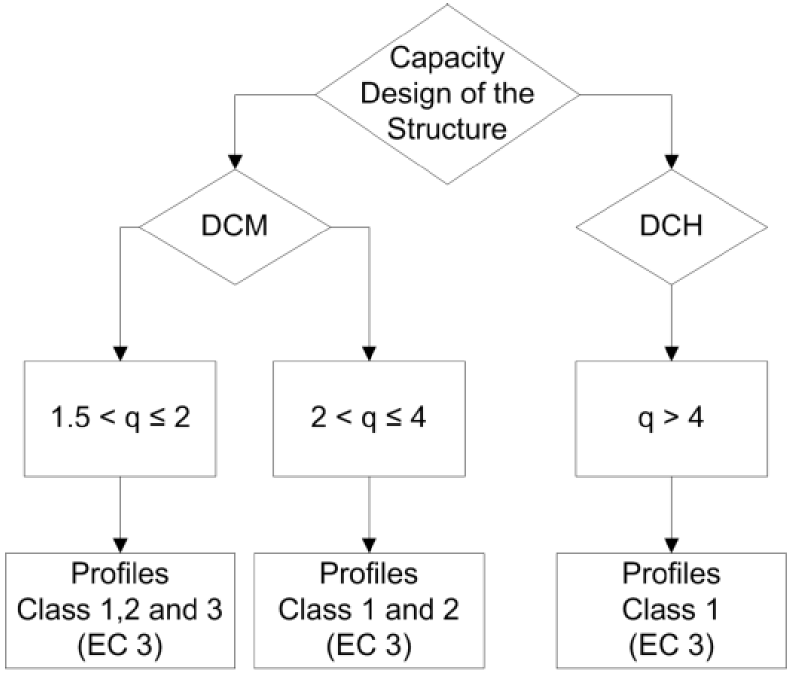
Design procedure for Medium DCM and High DCH dissipative behaviour

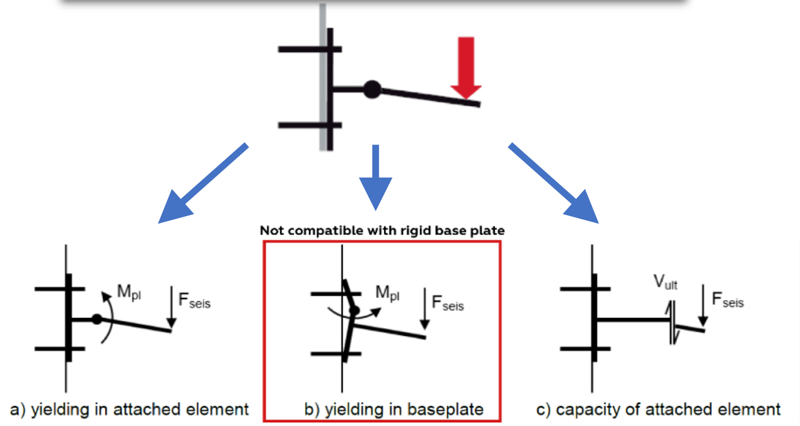
Overstreght criteria is required

Very high action effects are expected on the base connection It is possible to develop this type of base connection also for capacity design?

Applicability for all structural behaviour DCL, DCM and DCH

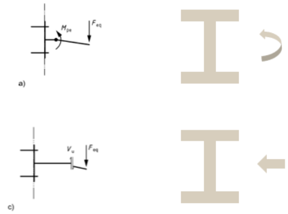


Anchor design approach a2)



CAPACITY DESIGN AND ISSUES:

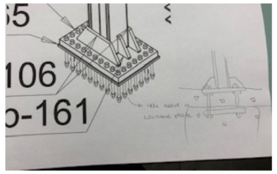
For capacity design, yielding on the attachment is necessary. Following the approach on the anchor guideline, means that a failure in the profile is needed, before the rest of the joint fails.



$M_{pl,Rd}$ * over strength factor

$F_{pl,Rd}$ * over strength factor

One of our customers did a capacity joint and ended up with:



Forces which create failure in the attachment (**profile**) are very high!

If these forces need to be equilibrated by anchors → geometrically not possible.

Furthermore, regulations require that capacity design for joints needs to have experimental tests to validate results.