Distinguished Webinar Series in Earthquake Engineering and Seismology
Organized by the Canadian Association of Earthquake Engineering

Design of concrete buildings for earthquake resistance in the evolution from the first to the second generation of European Standards for structural & geotechnical design

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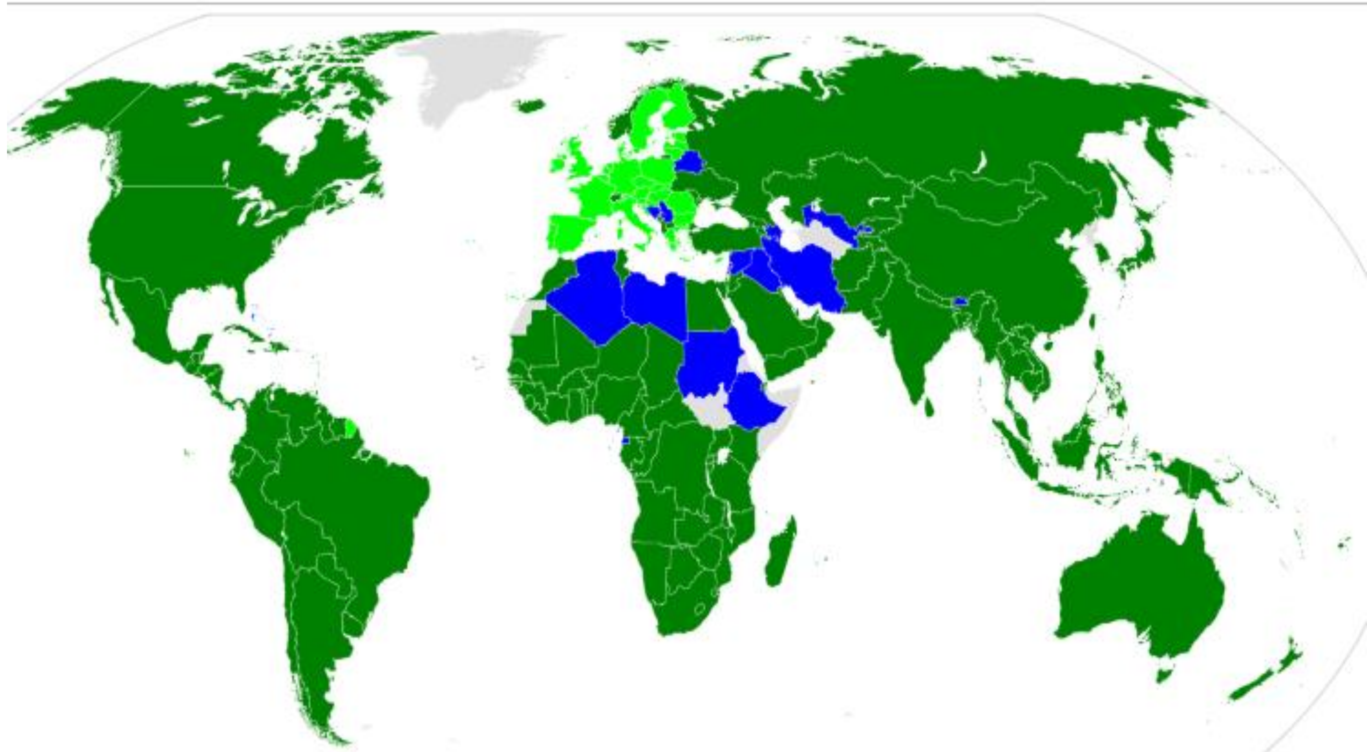
Introductory Note:
The role of Technical Standards in a global economy and in the European Economic Area

Technical Standards as barriers to international trade



World Trade Organisation (WTO):

- It binds its 164 Members to ensure that technical standards or regulations do not create unnecessary obstacles to trade.
- It has created the basis for International Standards (IS), ensuring their supremacy to national ones.

WTO Agreement on Technical Barriers to Trade (“Standards Code”)

- “Members shall ensure that technical regulations or standards are not prepared, adopted or applied with a view to or with the effect of creating unnecessary obstacles to international trade”.
- “Where technical regulations or standards are required and relevant international standards exist or their completion is imminent, Members shall use them .. as a basis for their technical regulations”.
- “A Member preparing, adopting or applying a technical regulation which may have a significant effect on trade of other Members shall .. explain the justification for that technical regulation .. ”.
- “Members shall play a full part, within the limits of their resources, in the preparation by appropriate international standardizing bodies of International Standards for products for which they either have adopted, or expect to adopt, technical regulations or standards.. ”.
- “..unnecessary duplication should be avoided between the work under this Agreement and that of governments in other technical bodies..”.

CEN
European Committee for Standardisation

34 CEN Members:

27 EU countries

- Austria
- Belgium
- Bulgaria
- Croatia
- Cyprus
- Czech Republic
- Denmark
- Estonia
- Finland
- France
- Germany
- Greece
- Hungary
- Ireland
- Italy
- Latvia
- Lithuania
- Luxembourg
- Malta
- Netherlands
- Poland
- Portugal
- Romania
- Slovakia
- Slovenia
- Spain
- Sweden

3 EFTA countries

- Switzerland
- Norway
- Iceland

- Serbia
- North Macedonia
- Turkey
- United Kingdom

Agreement between ISO and CEN ("Vienna Agreement")

CEN achieved a derogation from the "Standards Code" and the freedom to develop European Technical Standards

"Agreement on technical co-operation between ISO and CEN" (the "Vienna Agreement" 1991)

The “Vienna Agreement” 1991

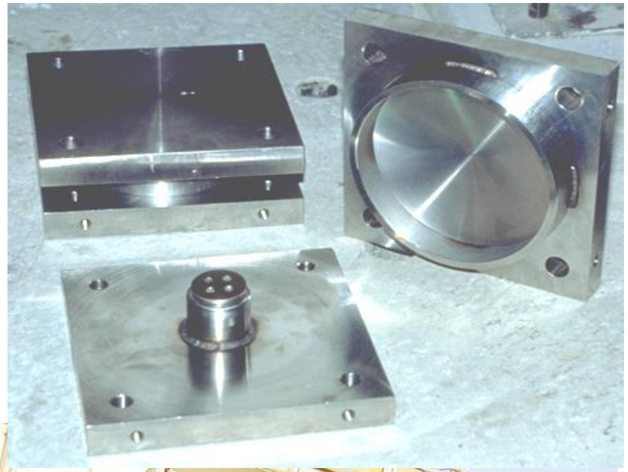
- .. recognises the primacy of International Standards.
- .. recognises that particular needs of the Single European Market might require .. standards for which a need has not been recognised at the international level
- The prioritization of ISO work is such that in some instances CEN needs to undertake work which is urgent in the European context, but less so in the international one.
- ISO recognises and respects that CEN operates within and must respect a political environment set both in the EEA and through a co-operation of the European Standards Organizations;
- ISO and CEN are committed to values such as transparency, openness, coherence, impartiality and relevancy. CEN supports coherence via withdrawal of conflicting national standards upon publication of a European standard;
- Standards development is done in either ISO or CEN, but both bodies ensure that the processes of consensus confirmation and approval are synchronized to achieve the objective of simultaneous publication;
- The transfer of work from CEN to ISO is the preferred route, but is not automatic;
- When expected results are not attained, the party which is not satisfied can decide to proceed separately;
- CEN committs to respond to comments from non-CEN ISO members.

The Eurocodes as European Standards

What are the Eurocodes? What is their role?

- The Eurocodes are a set of 58 European Standards (EN) for the structural, geotechnical and seismic design of buildings and civil engineering works, as well as of structural components thereof.
- They serve over 500000 engineers in EU or EFTA Member States and other CEN countries (SRB, MK, TR, UK).
- They underpin a market with an annual worth of ~65 billion Euro of professional services.
- They promote free-of-technical-obstacles access to a construction sector which produces ~10% of the GNP in the Single European Market of over 500 million people (the largest in the world, in terms of purchasing power).

The Eurocodes belong to the set of European Standards for construction



<table>
<tr><td colspan="2"><u>Structural design</u> standards (CEN): The <u>Eurocodes</u></td></tr>
<tr><td><u>Material</u> standards (steel, concrete, etc) – CEN;
<u>Product</u> standards (e.g., structural bearings, prefabricated structural members) - CEN</td><td>ETAs: European Technical Approvals (eg, Fibre-Reinforced Polymers, prestressing systems, etc.) - EOTA</td></tr>
<tr><td colspan="2"><u>Execution</u> standards (construction of concrete structures, steel structures) - CEN</td></tr>
<tr><td colspan="2"><u>Test</u> standards - CEN</td></tr>
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Commission Recommendation: Implementation/Use of Eurocodes

European Commission: "Commission Recommendation on the implementation and use of Eurocodes for construction works & structural construction products" Brussels (2003)

- Member States (MSs) should adopt the Eurocodes as a suitable tool for designing construction works, checking the mechanical resistance of components or checking the stability of structures.
- The Eurocodes are to be used by contracting authorities in technical specifications relating to the coordination of procedures for the award of public service contracts ... Technical specifications are to be defined by the contracting authorities by reference to national standards implementing European standards.
- MSs should take all necessary measures to ensure that structural construction products calculated in accordance with the Eurocodes may be used, and should therefore refer to the Eurocodes in their national regulations on design.
- MSs should inform the Commission of all national measures in accordance with the Recommendation.

Objectives of the Eurocodes

- Means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC & European Regulation 305/2011/EU, particularly with Requirements N°1 – Mechanical resistance and stability – and N°2 – Safety in case of fire;
- Basis for specifying contracts for construction works and related engineering services;
- Framework for drawing up harmonized technical specifications for construction products (ENs and ETAs);
- Means to improve the functioning of the Single Market for products & engineering services, by removing obstacles arising from different nationally codified practices for structural design

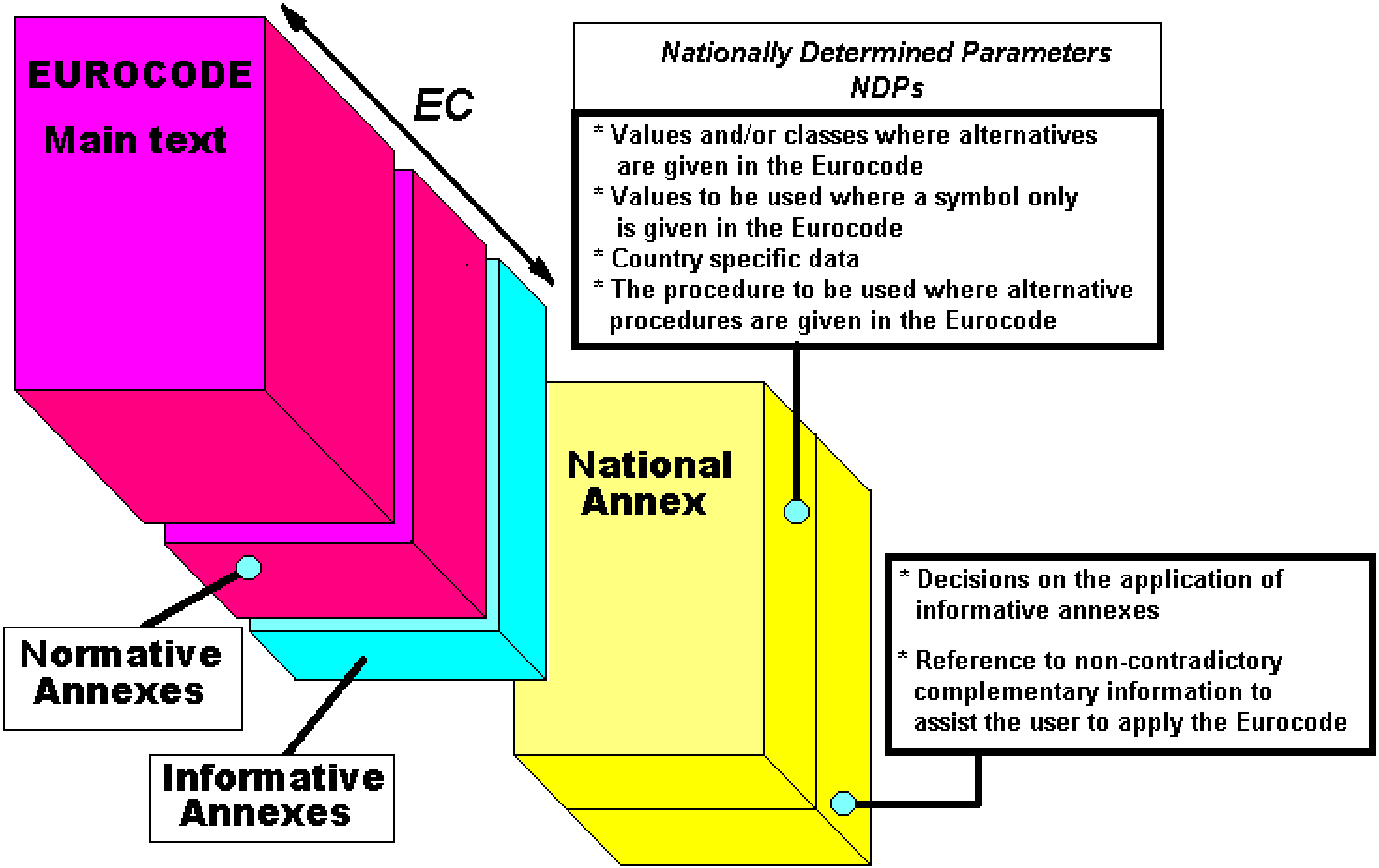
Important features of the Eurocode system

- Comprehensive & integrated set of codes covering:
    - all structural materials;
    - practically all types of construction works;
  - in a consistent, harmonized and user-friendly manner (similar document structure, symbols, terminology, verification criteria, analysis methods, etc.),
  - with hierarchy and cross-referencing among different Eurocodes and Eurocode parts
  - without overlapping or duplication.
- Eurocode system is ideal for application in a large number of countries with different traditions, materials, environmental conditions, etc., as it has built-in flexibility to accommodate such differences.

Flexibility in the Eurocode system

- The Eurocodes do not allow design with rules other than their own.
- National choice may be exercised through the **National Annex**, only where the Eurocode itself explicitly allows:
  1. Choosing a value for a parameter, for which a symbol or a range of values is given in the Eurocode;
  2. Choosing among alternative classes fully described in the Eurocode;
  3. Adopting an Informative Annex or referring to an alternative national document, complementing and not contradicting the Eurocode.
- Items of national choice: **Nationally Determined Parameters (NDPs)**
- National choice through NDPs:
  - On issues controlling safety (national competence) & where there are geographic or climatic differences (eg, seismic hazard)
- For cases 1 & 2, the Eurocode itself recommends (in a Note) a choice. The European Commission urges Member States to adopt the recommended choice, to minimize diversity within the Single Market.
- If a National Annex does not include the national choice for a NDP, the designer and/or the owner may choose, for the particular project.

National implementation of the Eurocodes



History of the Eurocodes

The Eurocodes in the European Economic Community

25/3/1957 Rome Treaty

Public Procurement Directive 71/305/CEE

1976 Steering Committee of Eurocodes

10/3/1979 European Monetary System

1980 First Eurocodes (ECs)

12/7/1986 Unique Act New Approach

1989 Construction Products Directive 89/106/CEE
Essential requirements
1) Mechanical resistance & stability
2) Safety in case of fire
3) Hygiene, health, environment
4) Safety in use
5) Protection against noise
6) Energy economy, heat retention
Interpretative documents 1994

1990 Transfer of ECs to CEN TC250

Directive 92/50/CEE Public procurement contracts

1991-96 pre-Standards ENVs

Directive 93/37/CEE Public works contracts

1/11/93

The Eurocodes in the European Union

1/11/93

1997
Directive 97/52/CE
Public procurement services

1998
«Conversion» of ENVs to ENs starts

2003 Commission's Recommendation to Member States 03/C4639 /CEE "Implementation & use of Eurocodes"
1) Adopt ECs
2) Use ECs in Specs for public sector & energy, water, transport, telecom works
3) Member States competent on safety & economy: Nationally Determined Parameters (NDPs)
4) Compare, harmonize NDPs

2004
Directive 04/18/CE
Public procurement

2007
All Eurocodes published as ENs

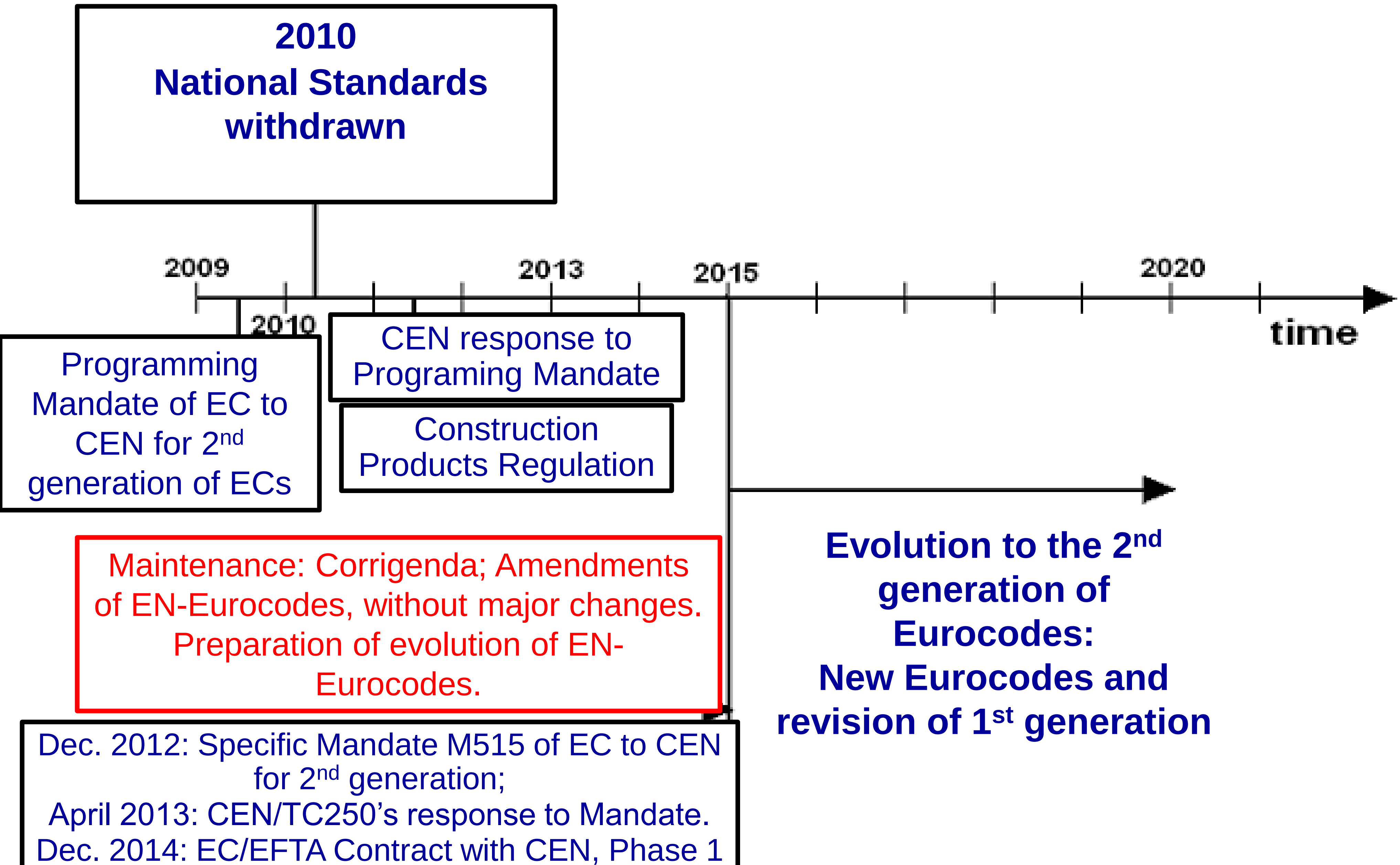
2011 Construction Products Regulation 305/2011/EU
Basic requirements
1) Mechanical resistance & stability
2) Safety in case of fire
3) Hygiene, health, environment
4) Safety in use
5) Protection against noise
6) Energy economy, heat retention
7) Sustainable use of resources

Dec. 2012
Specific Mandate M515
EC invites CEN to develop work programme for 2nd generation of EN-Eurocodes

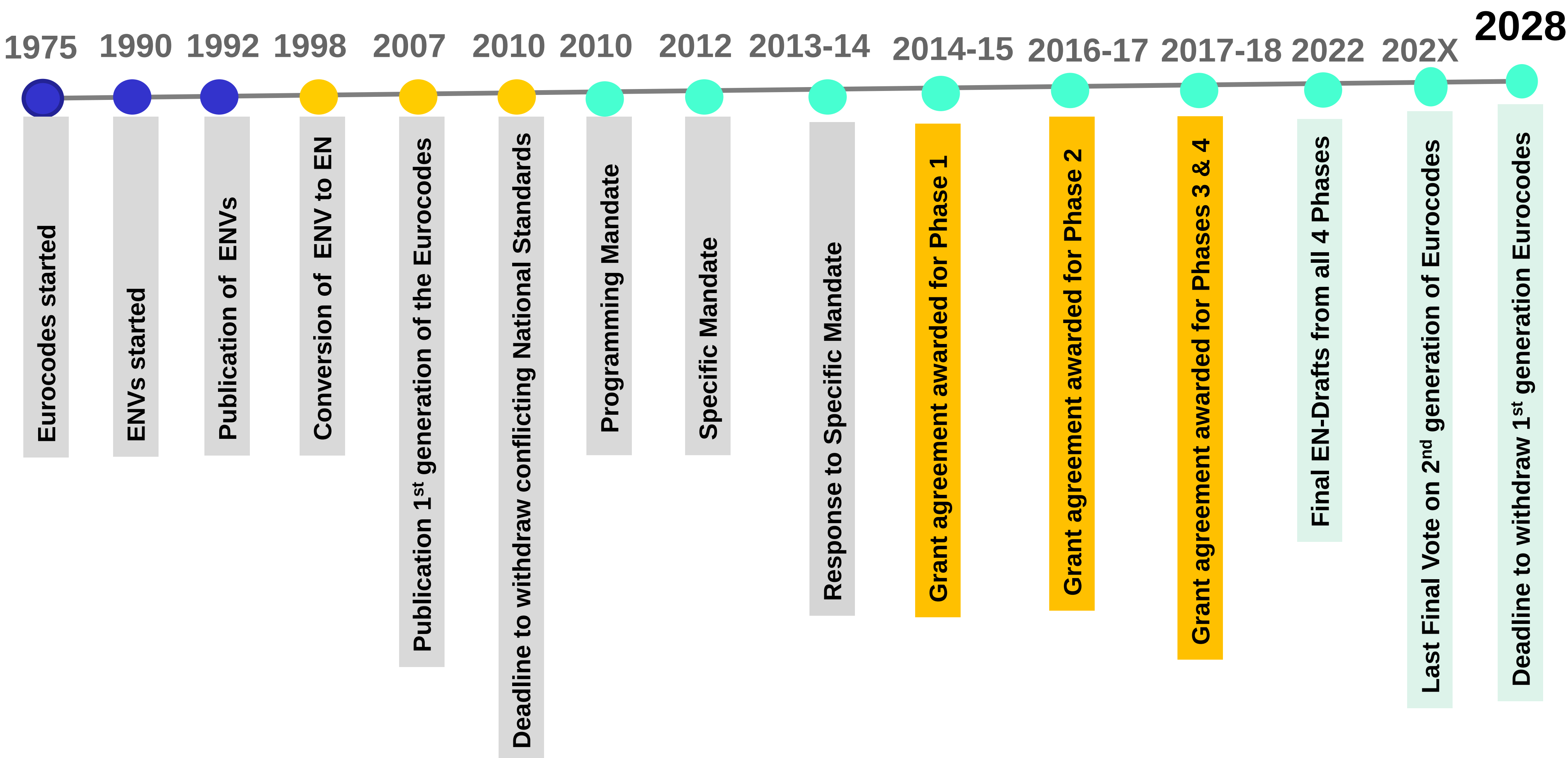
2014
Directive 14/24/EU
Public procurement

Dec. 2014
Grant Agreement EC to CEN: Phase 1 of second generation of EN-Eurocodes

From the 1st to the 2nd generation



Timeline of the various generations of Eurocodes



General objectives of the evolution to the second generation

- Reduce number of Nationally Determined Parameters.
- Enhance “Ease of use” by:
    - Improving clarity;
    - Simplifying routes through the Eurocodes;
    - Limiting, where possible, alternative application rules;
    - Avoiding/removing rules of little practical use in design;
    - etc.
- Fill voids in scope.
- Consolidate; produce short, succinct texts.
- Ensure stability for the users:

The present and the future

The first generation of EN-Eurocodes

EN 1990 Eurocode: Basis of structural design

EN 1991 Eurocode 1: Actions on structures

EN 1992 Eurocode 2: Design of concrete structures

EN 1993 Eurocode 3: Design of steel structures

EN 1994 Eurocode 4: Design of composite (steel-concrete) structures

EN 1995 Eurocode 5: Design of timber structures

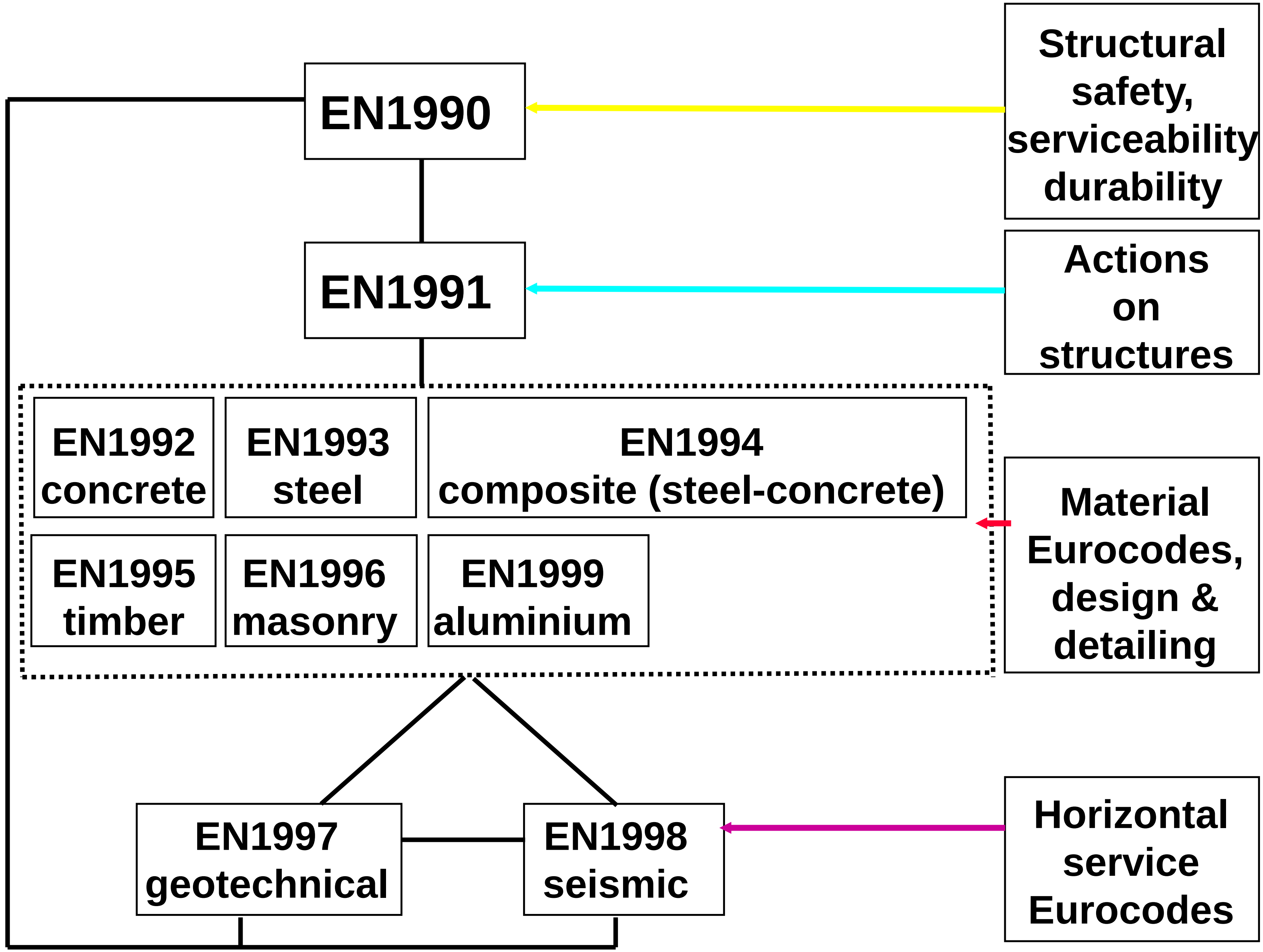
EN 1996 Eurocode 6: Design of masonry structures

EN 1997 Eurocode 7: Geotechnical design

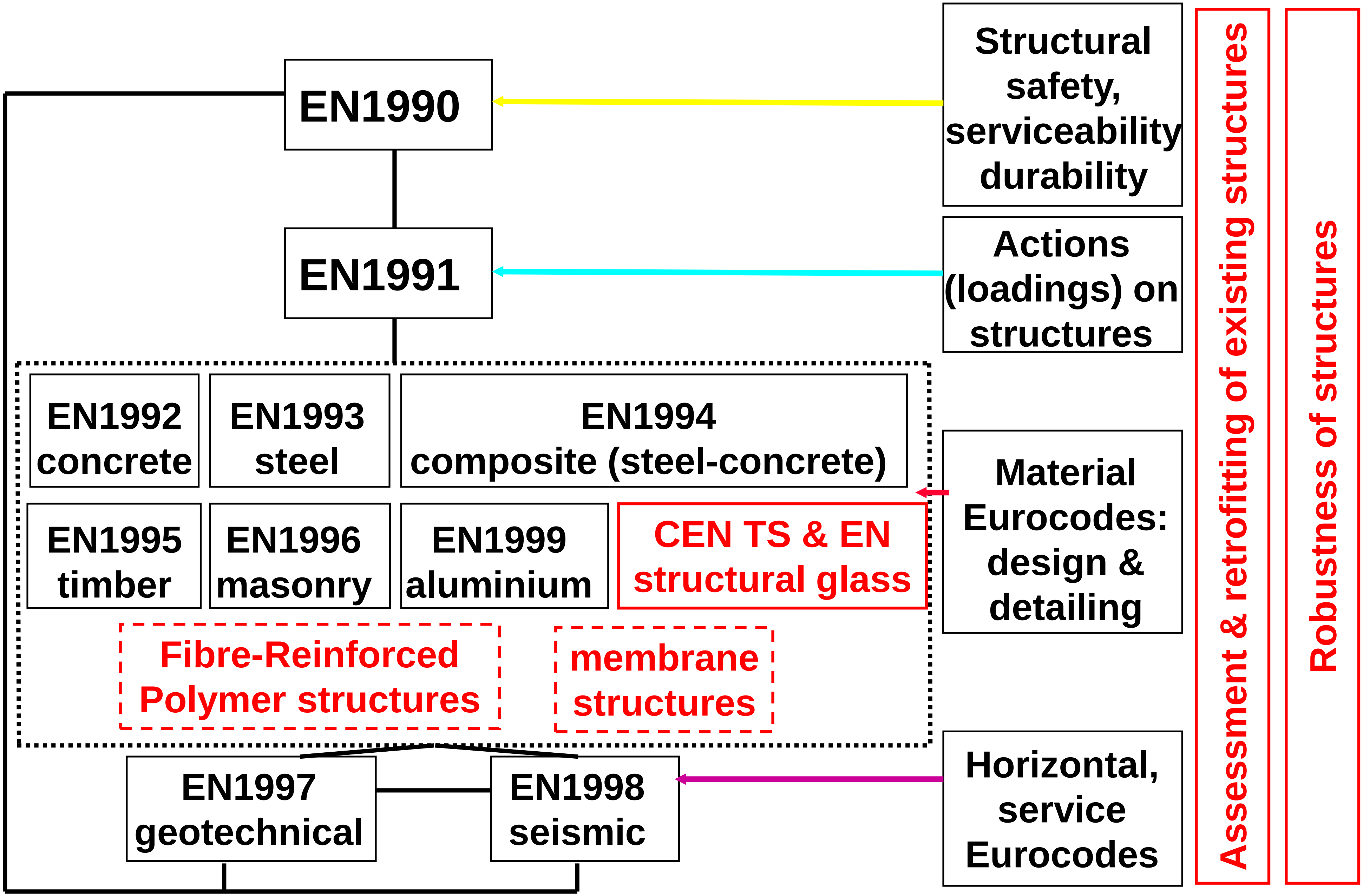
EN 1998 Eurocode 8: Design of structures for earthquake resistance

EN 1999 Eurocode 9: Design of aluminium structures

Interrelation and Hierarchy of Eurocodes



New elements in the 2nd generation of Eurocodes



Eurocode 8 “Design of structures for earthquake resistance”

Eurocode 8 Parts

1st generation		2nd generation	
EN1998-1	General rules, seismic actions, rules for buildings	**EN1998-1-1**	General rules, seismic action
EN1998-2	Bridges	**EN1998-1-2**	Rules for new buildings
EN1998-3	Assessment and retrofitting of buildings	**EN1998-2**	Bridges
EN1998-4	Silos, tanks, pipelines	**EN1998-3**	Assessment and retrofitting of buildings and bridges
EN1998-5	Foundations, retaining structures, geotechnical aspects	**EN1998-4**	Silos, tanks and pipelines, towers, masts and chimneys
EN1998-6	Towers, masts, chimneys	**EN1998-5**	Geotechnical aspects, foundations, retaining and underground structures

EN 1998-1:2004 General rules, seismic actions, rules for buildings

1. **General**
2. **Performance Requirements and Compliance Criteria**
3. **Ground Conditions and Seismic Action**
4. **Design of Buildings**
5. **Specific Rules for Concrete Buildings**
6. **Specific Rules for Steel Buildings**
7. **Specific Rules for Steel-Concrete Composite Buildings**
8. **Specific Rules for Timber Buildings**
9. **Specific Rules for Masonry Buildings**
10. **Base Isolation**

Annex A (Informative): Elastic Displacement Response Spectrum

Annex B (Informative): Determination of Target Displacement for Nonlinear Static (Pushover) Analysis

Annex C (Normative): Design of the Slab of Steel-Concrete Composite Beams at Beam-Column Joints in Moment Resisting Frames

EN 1998-1-1:202X General rules, seismic action

EN 1998-1-2:202X Rules for new buildings

4. Basis of design
5. Modelling and structural analysis
6. Verification of structural elements to limit states
7. Ancillary elements
8. Base isolated buildings
9. Buildings with energy dissipation systems
10. Specific rules for concrete buildings
11. Specific rules for steel buildings
12. Specific rules for composite steel–concrete buildings
13. Specific rules for timber buildings
14. Specific rules for masonry buildings
15. Specific rules for aluminium buildings

Annex A (Informative) Characteristics of earthquake resistant buildings and in plan regularity
Annex B (Informative) Natural eccentricity and torsional radius
Annex C (Normative) Floor accelerations for ancillary elements
Annex D (Normative) Buildings with energy dissipation systems
Annex E (Normative) Seismic design of connections for steel buildings
Annex F (Normative) Steel light weight structures
Annex G (Normative) Design of composite connections in dissipative composite steel-concrete moment resisting frames
Annex H (Informative) Seismic design of exposed and embedded column base connections
Annex I (Normative) Design of the slab of steel-concrete composite beams at beam-column joints in moment resisting frames
Annex J (Informative) Drift limits for eccentrically loaded unreinforced masonry piers
Annex K (Informative) Simplified evaluation of drift demands on infilled frames
Annex L (Normative) Load-deformation relationships of dissipative timber components for non-linear analyses

EN 1998-1-2:202X Rules for new buildings

4. Basis of design
4.1. Building classification
4.2. Seismic actions
4.3. Compliance criteria
4.4. Characteristics of earthquake resistant buildings
5. Modelling and structural analysis
5.1. Modelling
5.2. Minimum design eccentricity in buildings
5.3. Methods of analysis
6. Verification of structural elements to limit states
6.2. Verification of Significant Damage (SD) limit state
6.3. Verification to other limit states
10. Specific rules for concrete buildings
10.2. Basis of design and design criteria
10.3. Materials requirements
10.4. Structural types, behavior factors, limits of seismic action and limits of drift
10.5. Beams
10.6. Columns
10.7. Beam-column joints
10.8. Ductile walls
10.9. Large walls
10.10. Flat slabs
10.11. Provisions for anchorages and laps
10.12. Provisions for concrete diaphragms
10.13. Prestressed concrete
10.14. Precast concrete structures
10.15. Design and detailing of foundations

Performance Objectives

In both EN1998-1:2004 and EN1998-1-1:202X

Performance Objectives

1. Life safety
2. Limitation of damage
3. Facilities important for civil protection remain operational (reliability differentiation).

Performance-based Seismic Engineering

- Present-day seismic design codes for new buildings <u>hide</u> the *ends*, i.e., the seismic performance target, and emphasize the (prescriptive) *means*, e.g.:

  the behavior factor q, for reduction of the elastic spectrum in linear analysis,

  the member detailing rules, etc.

- Performance-based seismic design is <u>transparent</u>: it targets specific and measurable performance for a set of seismic hazard levels, e.g. for ordinary buildings:

Performance Level	**Hazard Level**	
Fully Operational	**Frequent earthquake**	**(25-72 yrs)**
Limited Damage (for Immediate Occupancy)	**Occasional earthquake**	**(72-225 yrs)**
Life Safety	**Rare earthquake**	**(475 yrs)**
Collapse Prevention	**Very rare earthquake**	**(800-2500yrs)**

  - Pros: Better property protection; flexibility in conceptual design
  - Cons: Exra work in design.

Limit State design in Europe:

Early Performance-based design for all sorts of loadings

- The 1970 CEB "International Recommendations for the design and construction of concrete structures" introduced "Performance Levels" for any type of loading (not just for earthquake) under the name "Limit States":
- Limit State (LS) = state of unfitness to (intended) purpose:
  - "Ultimate" LSs (ULS) concern safety of people or structure;
  - "Serviceability" LSs (SLS) concerns operation of the facility and damage to property;
  - Intermediate "Damageability" LSs.
- Limit State concept: the backbone of all Eurocodes (as mandated by Eurocode 1990 "Basis of structural design"), including Eurocode 8.

In EN1990:2002- Eurocode: Basis of structural design:

- <u>Ultimate limit states</u> concern:
  - the safety of people
  - the safety of the structure
- <u>Serviceability limit states</u> concern:
  - the functioning of the structure
  - the comfort of people
  - the appearance of the structure

<table>
<tr><th>Limit State
Design Situation</th><th>ULS</th><th>• loss of equilibrium of the structure or any part of it, considered as a rigid body;
• failure by excessive deformation, transformation of the structure or any part of it into a mechanism, rupture, loss of stability of the structure or any part of it, including supports and foundations;
• failure caused by fatigue or other time-dependent effects.</th><th colspan="2">SLS</th></tr>
<tr><td>Persistent</td><td colspan="2">✓</td><td colspan="2">✓</td></tr>
<tr><td>Transient</td><td colspan="2">✓</td><td colspan="2">✓</td></tr>
<tr><td>Accidental</td><td colspan="2">✓</td><td colspan="2"></td></tr>
<tr><td>Seismic</td><td colspan="2">✓</td><td>✓</td><td></td></tr>
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Limit States in EN 1998-1-1:202X

Limit State	Facility operation	Structural condition
Operational **(OP) SLS**	Continued use; any damage may be repaired later	Only slight damage to structure and infills/partitions
Damage Limitation **(DL) SLS**	Safe, but normal use temporarily interrupted;	Light structural damage (localised bar yielding, concrete cracking or spalling). Insignificant permanent drift. Structure retains full resistance with minor decrease in stiffness; infills/partitions have distributed cracks
Significant Damage **(SD) ULS**	No threat to life during event; emergenc / temporary use only; repair feasible, but maybe uneconomic	Significant structural damage or moderate permanent drifts; sufficient capacity for gravity loads; infills or partitions damaged but not collapsed
Near Collapse **(NC) ULS**	Unsafe for emergency use; life safety during earthquake almost ensured, not guaranteed (falling debris)	Heavy structural damage, or large permanent drifts, or infills/partitions collapsed; strength (barely) sufficient for gravity loads

EN1998-1:2004: Reliability differentiation depending on consequences

- Classify building in Important Class, depending on consequences of failing to meet the performance requirements.
- Assign a higher or lower mean return period to the design seismic action of each class.
- Implementation: multiply the seismic action which applies to structures of Ordinary Importance by the importance factor γ_I

Combination of the seismic action with other actions

- «Seismic design situation»: $\sum_{j\geq 1} G_{k,j}"+"A_{Ed}"+"\sum_{i\geq 1}\psi_{2,i}Q_{k,i}$

$\sum_{j\geq 1} G_{k,j}$: Permanent actions (characteristic or nominal values)

$\sum_{i\geq 1}\psi_{2,i}Q_{k,i}$: Quasi-permanent values of variable actions $\psi_{2,i} =\sim 0.3$

$A_{Ed} = \gamma_{\mathrm{I}} A_{Ek}$: (Design) seismic action

A_{Ek}: Characteristic seismic action, γ_{I}: Importance factor of structure

EN 1998-1:2004 Recommended Importance classes & factors for buildings

	Building type (NDP)	γ_I (NDP)
I	Minor importance for public safety (warehouses, agricultural buildings)	0.8
II	Ordinary (residential or office buildings, small buildings)	1
III	Major consequences of collapse: Grandstands, large buildings, schools, assembly halls, cultural facilities	1.2
IV	Of vital importance for civil protection: hospitals, fire stations, power plants	1.4

EN1998-1-1/-1-2:202X: Recommended Consequence Classes (CC), performance factors $\gamma_{LS,CC}$ and return periods $T_{LS,CC}$ for the seismic action four Limit States -coefficient δ for the Seismic Action Class in buildings

	Building type (NDP)	$\gamma_{LS,CC}$ (NDP)			$T_{LS,CC}$ [years] (NDP)			δ
		DL (& OP)	SD	NC	DL (& OP)	SD	NC	
CC1	see I above	0.4	0.8	1.2	50	250	800	0.6
CC2	see II above	0.5	1	1.5	60	475	1600	1
CC3a	see III above	0.5	1.2	1.8	60	800	2500	1.25
CC3b	see IV above	0.6	1.5	2.2	100	1600	5000	1.6

- The performance factor $\gamma_{LS,CC}$ multiplies the 475yr seismic action of CC2, like the importance factor
- Coefficient δ multiplies the 475yr constant spectral acceleration of the elastic spectrum at the surface of the ground to give the Seismic Action Class index, which is used to characterise the seismicity of the site.

Seismic Action class of a structure in EN1998-1-1:202X

Seismic Action Class index = Coefficient δ of structure (depends on CC)
Times the
value of 475yr of elastic spectrum at ground surface in constant spectral acceleration plateau

Seismic action class	Seismic action index S_d (m/s^2)	Design to Eurocode 8
Very low	< 1.25	Not necessary
Low	1.25 – 3.0	With simplified rules (NDP)
Moderate	3.0 – 6.25	(possibly simpler, elastic design)
High	> 6.25	

“Reference Seismic Action” and “Reference Return Period” in EN1998-1:2004 and EN1998-1-1:202X

- The seismic action for which new structures of **Importance Class II** or **Consequence Class CC2** (ordinary) are designed for **Significant Damage** is called “Reference seismic action”.
- Its mean return period is termed “Reference return period” and is a NDP with recommended value 475 years (10% probability of exceedance of Reference seismic action” in 50 years).

The seismic action in EN1998-1:2004 and EN1998-1-1:202X (the future)

Ground conditions

- Eurocode 8 requires appropriate investigations (in situ and/or in the lab) to identify the ground conditions, in order to:
  - classify the soil profile for the selection of the elastic response spectrum appropriate to the site;
  - identify possible soil behavior detrimental to the seismic response of the superstructure.

EN1998-1:2004: Standard Ground types

		$v_{s,30}$ (m/s)	N_{SPT}	c_u (kPa)
A	Rock with ≤5m weaker surface material	>800	_	_
B	Very dense sand, gravel or very stiff clay, ≥ several tens of m	360-800	>50	>250
C	Dense or medium-dense sand, gravel or stiff clay, several tens to many hundreds m	180-360	15-50	70-250
D	Loose-to-medium cohesionless soil or soft-to-firm cohesive soil	<180	<15	<70
E	5 to 20m surface alluvium layer with v_s of type C or D, underlain by v_s>800m/s material			
S_1	≥10m thick soft clay/silt with PI > 40 and high water content	<100	_	10-20
S_2	Liquefiable soils, sensitive clays, or any other soil not of type A – E or S_1			

EN1998-1-1:202X: Standard Ground types

	Ground class	stiff	medium stiffness	soft
Depth class	$v_{s,H}$ (m/sec)	400-800	250-400	150-250
	H_{800} (m)			
very shallow	$H_{800} \leq 5$m	A	A	E
shallow	$5\text{m} < H_{800} \leq 30$m	B	E	E
intermediate	$30\text{m} < H_{800} \leq 100$m	B	C	D
deep	$H_{800} > 100$m	B	F	F

EN 1998-1:2004: The «design seismic action»

- The SD Limit State should be met under the «design seismic action».
- The «design seismic action» of structures of ordinary importance (Class II) is called «Reference seismic action». Its mean return period is called «Reference Return Period».
- The Reference Return Period of the Reference Seismic action is a NDP, with recommended value of 475 years.
- The Reference Seismic action is described (in the national zonation maps) in terms of a single parameter:

  the Reference Peak Ground Acceleration (PGA) on Rock, $a_{gR.}$
- The design ground acceleration on rock (Ground A), a_g, is defined as the product of the reference PGA times the importance factor:

$$a_g = \gamma_I a_{gR}$$

- In addition to the Reference Peak Ground Acceleration on Rock, the Reference Seismic action is defined in terms of the Elastic Response Spectrum for 5% damping.

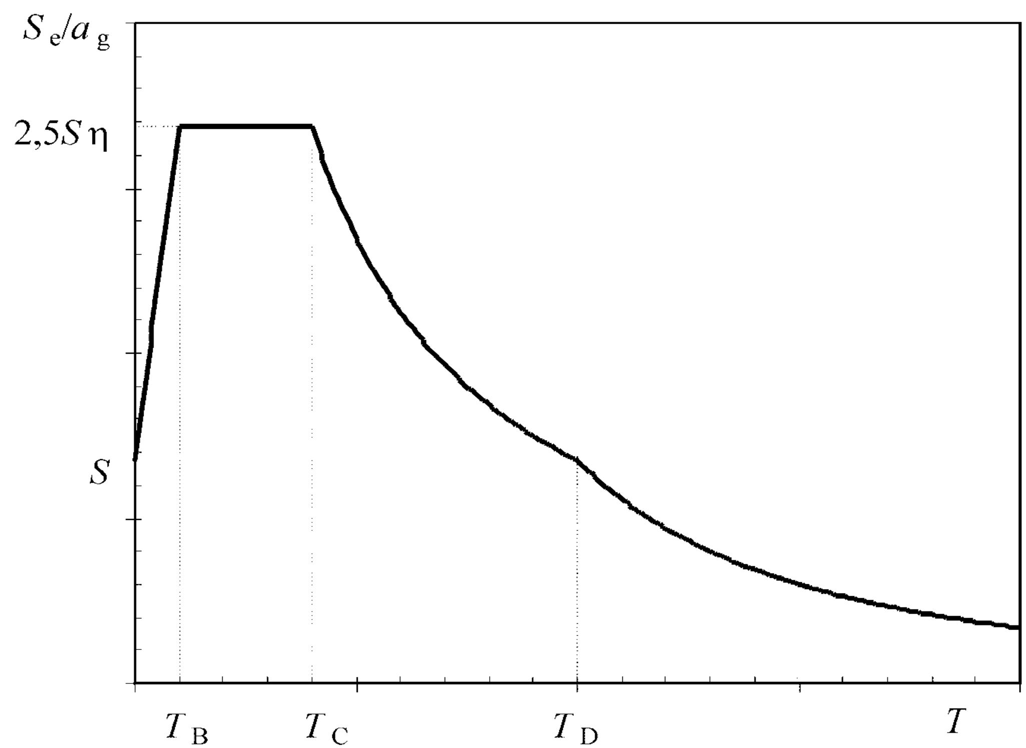
Representation of the seismic action by the Elastic Response Spectrum for 5% damping.

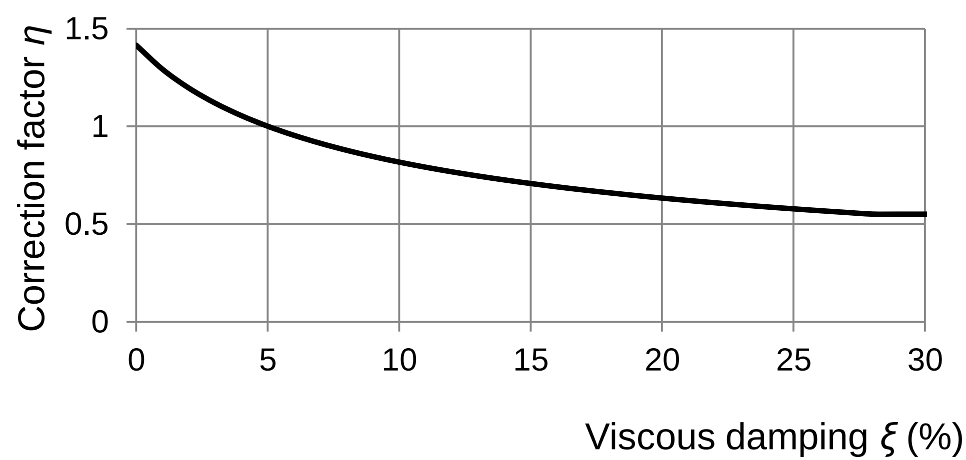
- Seismic action defined in terms of <u>Elastic Response Spectrum for 5% viscous damping.</u>
- Same spectrum applies to the two orthogonal independent horizontal components of the seismic action; a different one for the vertical.
- The shapes of the spectra depend on the ground type.
- The spectra of the "reference seismic action" on Ground of Type A (rock):
  - In EN1998-1:2004 are defined in terms of:
    - the Peak Ground Acceleration" (PGA) on rock, $\mathbf{a_{g,R}}$.
  - In EN1998-1-1:202X in terms of the Spectral Acceleration on rock:
    - at the constant-acceleration plateau of the spectrum, $\mathbf{S_{\alpha,ref}}$, and
    - at 1 sec period, $\mathbf{S_{\beta,ref}}$.
- Zonation maps in the National Annex to EN1998-1:2004 give $\mathbf{a_{g,R}}$;
- Those of EN1998-1-1:202X will give $\mathbf{S_{\alpha,ref}}$ and $S_{\beta,ref}$, or just $S_{\alpha,ref}$ with $S_{\beta,ref}$ taken equal to 20%, 30% or 40% of $S_{\alpha,ref}$, for $S_{\alpha,ref} \leq 2.5 m/s^2$, $2.5 m/s^2 < S_{\alpha,ref} \leq 5 m/s^2$ or $5 m/s^2 < S_{\alpha,ref}$, respectively ("low", "moderate" and "high" seismicity, according to EN1998-1-1:202X).

EN1998-1:2004: Standard elastic response spectral shape

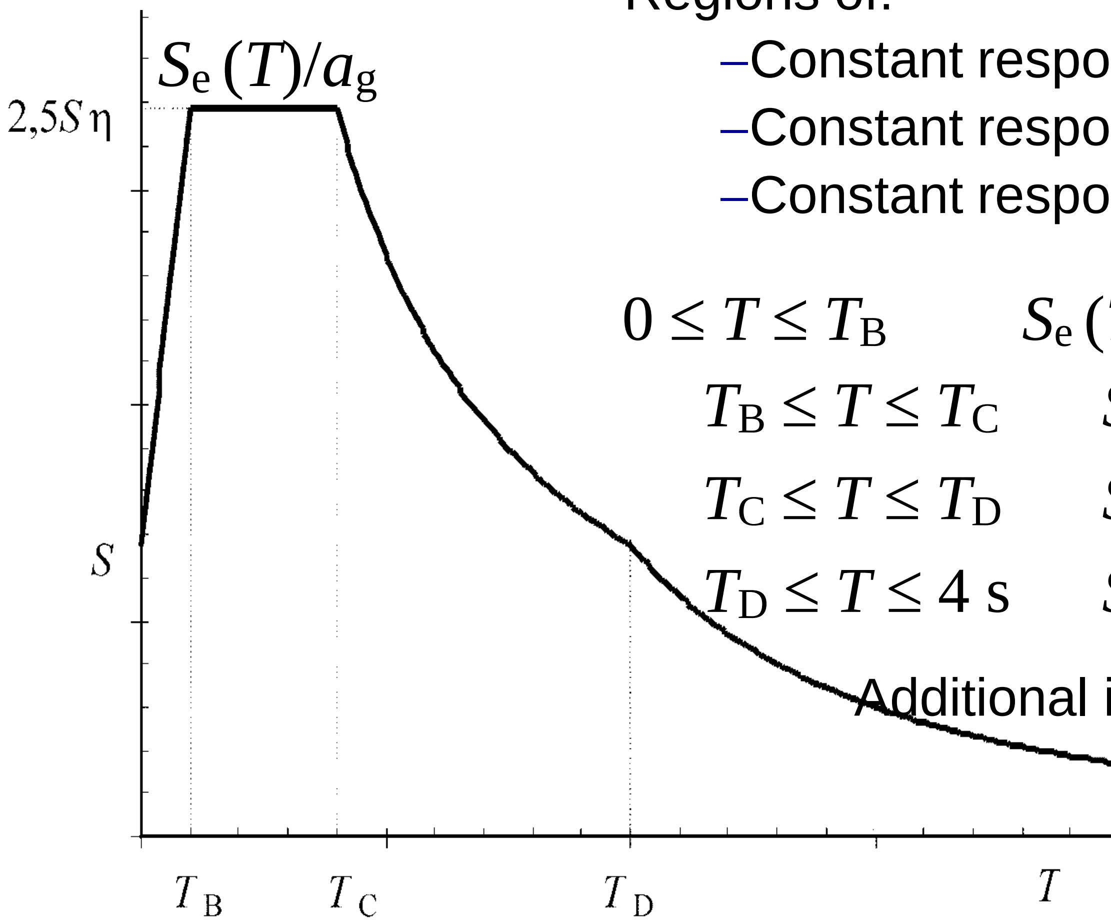
- Uniform amplification of spectrum by soil factor S (including PGA at soil surface, to Sa_g).
- Corner periods T_B, T_C, T_D, and S:
  - NDPs, with the same value for all Limit States and Return Periods of the seismic action.
- Constant spectral acceleration =
  - 2.5-times PGA at soil surface for horizontal spectrum,
  - 3-times for vertical.
- Damping correction factor:

$$\eta = \sqrt{10/(5+\xi)} \geq 0{,}55$$





EN1998-1:2004: Horizontal elastic response spectrum $S_e(T)$



Regions of:

- Constant response spectral pseudo-acceleration
- Constant response spectral pseudo-velocity
- Constant response spectral displacement

$0 \le T \le T_B \qquad S_e(T) = a_g \,.\, S \,.\, (1+T/T_B \,.\, (\eta \,.\, 2{,}5 \text{ -}1))$

$T_B \le T \le T_C \qquad S_e(T) = a_g \,.\, S \,.\, \eta \,.\, 2{,}5$

$T_C \le T \le T_D \qquad S_e(T) = a_g \,.\, S \,.\, \eta \,.\, 2{,}5\,(T_C/T)$

$T_D \le T \le 4\text{ s} \qquad S_e(T) = a_g \,.\, S \,.\, \eta \,.\, 2{,}5\,(T_C \,.\, T_D/T^2)$

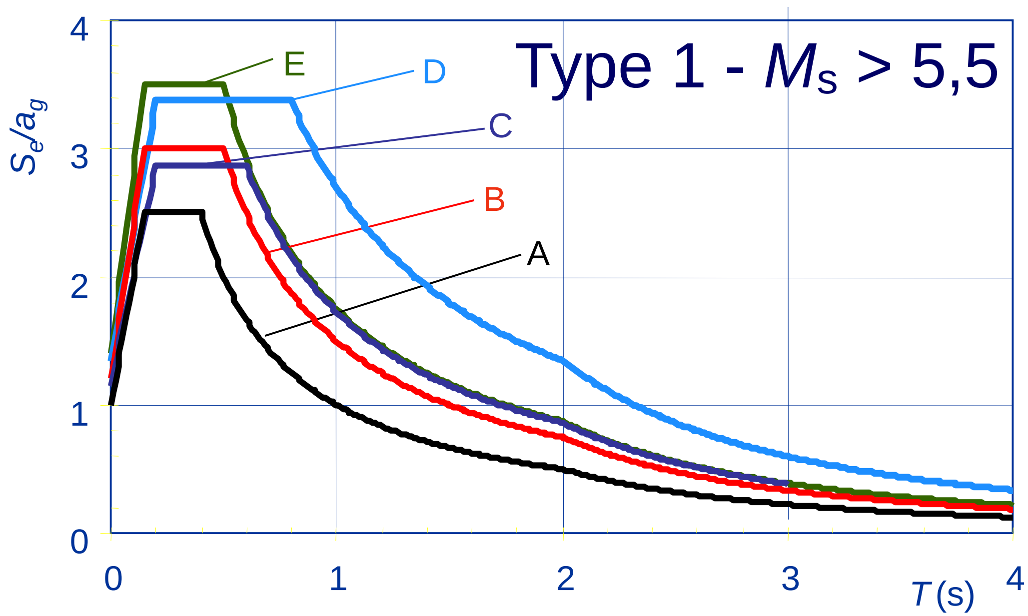
Additional information for $T > 4$ s in Annex

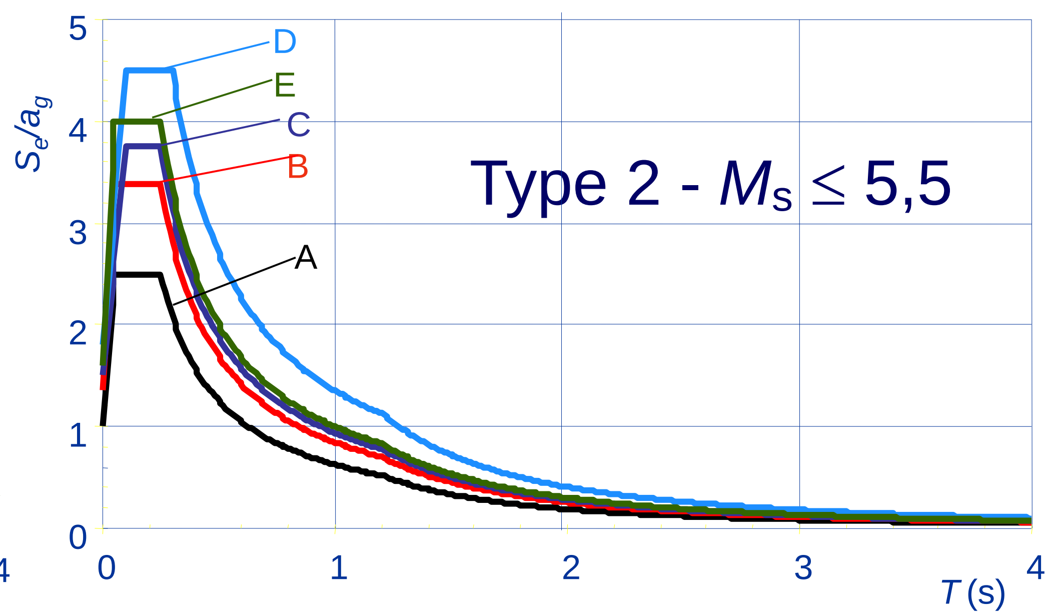
S	soil factor (NDP)
a_g	design ground acceleration on type A ground: $a_g = \gamma_I a_{gR}$
T_B, T_C, T_D	corner periods in the spectrum (NDPs)
η	damping correction factor ($\eta = 1$ for 5% damping)

EN1998-1:2004 Two recommended elastic spectral shapes

- Depending on the most significant contributions to the hazard at a site:
  - Type 1 - High and moderate seismicity regions ($M_S > 5{,}5$)
  - Type 2 - Low seismicity regions ($M_S \leq 5{,}5$); near field earthquakes

	Type 1				Type 2			
Ground Type	S	T_B (s)	T_C (s)	T_D (s)	S	T_B (s)	T_C (s)	T_D (s)
A	**1,0**	**0,15**	**0,4**	**2,0**	**1,0**	**0,05**	**0,25**	**1,2**
B	**1,2**	**0,15**	**0,5**	**2,0**	**1,35**	**0,05**	**0,25**	**1,2**
C	**1,15**	**0,2**	**0,6**	**2,0**	**1,5**	**0,1**	**0,25**	**1,2**
D	**1,35**	**0,2**	**0,8**	**2,0**	**1,8**	**0,1**	**0,3**	**1,2**
E	**1,4**	**0,15**	**0,5**	**2,0**	**1,6**	**0,05**	**0,25**	**1,2**





EN1998-1-1:202X: Horizontal elastic response spectrum $S_e(T)$

$$0 \le T \le T_A:\ S_e(T) = \frac{S_\alpha}{F_A}$$

$$\eta = \sqrt{\left(10 + \frac{T_C(\xi - 5)}{T_C + 30\,T}\right) / (5 + \xi)}$$

$$T_A \le T \le T_B:\ S_e(T) = \frac{\eta(T - T_A) + \dfrac{T_B - T}{F_A}}{T_B - T_A} S_\alpha$$

$$T_B \le T \le T_C:\ S_e(T) = \eta S_\alpha$$

$$S_\alpha = F_T F_\alpha S_{\alpha,RP}$$

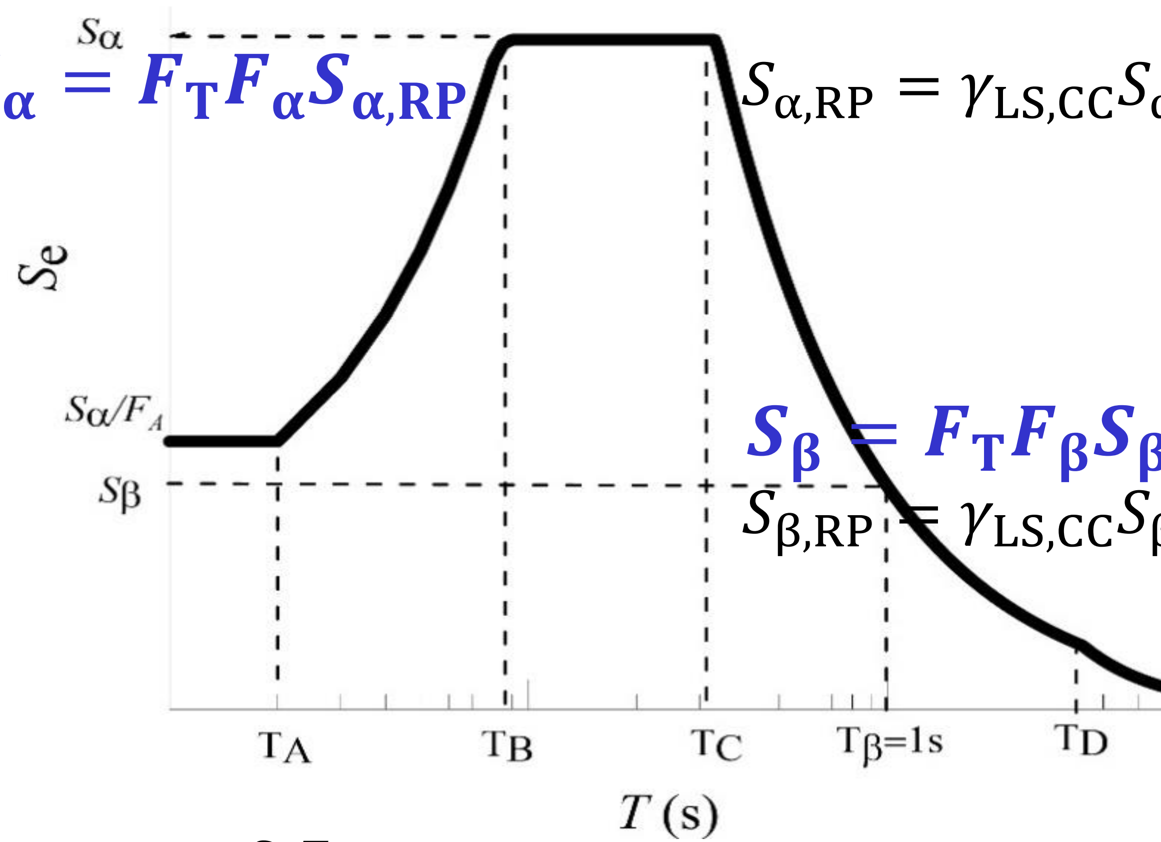
$$S_{\alpha,RP} = \gamma_{LS,CC} S_{\alpha,ref}$$

$$T_C \le T \le T_D:\ S_e(T) = \eta \frac{S_\beta T_\beta}{T}$$

$$S_\beta = F_T F_\beta S_{\beta,RP}$$

$$S_{\beta,RP} = \gamma_{LS,CC} S_{\beta,ref}$$

$$T \ge T_D:\ S_e(T) = \eta T_D \frac{S_\beta T_\beta}{T^2}$$



F_A=2.5

$$T_A = 0.02\text{s},\ 0.05\text{s} \le T_B = \frac{T_C}{4} \le 0.1\text{s},\ T_C = \frac{S_\beta T_\beta}{S_\alpha},\ T_D = \max[2; 1 + S_{\beta,RP}(\text{m/s}^2)]\ \text{s}$$

EN1998-1-1:202X Nonlinear amplification of spectral values

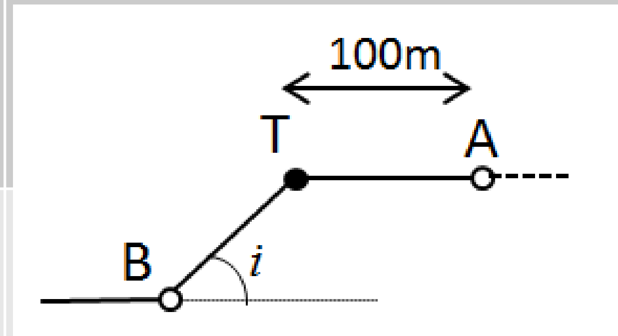
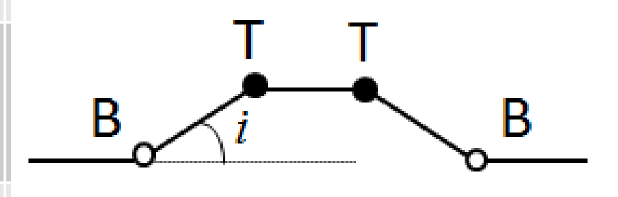
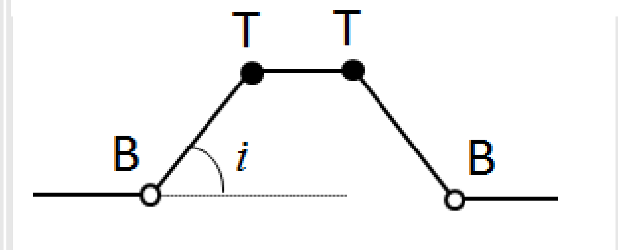
$$S_{\alpha} = F_{\mathrm{T}} F_{\alpha} S_{\alpha,\mathrm{RP}} \qquad S_{\beta} = F_{\mathrm{T}} F_{\beta} S_{\beta,\mathrm{RP}}$$

$$S_{\alpha,\mathrm{RP}} = \gamma_{\mathrm{LS,CC}} S_{\alpha,\mathrm{ref}} \qquad S_{\beta,\mathrm{RP}} = \gamma_{\mathrm{LS,CC}} S_{\beta,\mathrm{ref}}$$

<table>
<tr><th rowspan="2">Ground type</th><th colspan="2">F_{α}</th><th colspan="2">F_{β}</th></tr>
<tr><th>H_{800}, $v_{s,H}$ available</th><th>Default value</th><th>H_{800}, $v_{s,H}$ available</th><th>Default value</th></tr>
<tr><td>A</td><td>1,0</td><td>1,0</td><td>1,0</td><td>1,0</td></tr>
<tr><td>B</td><td rowspan="3">$\left(\frac{v_{s,H}}{800}\right)^{-0,4 r_{\alpha}}$</td><td>$1,3(1-0,01\, S_{\alpha,RP})$</td><td rowspan="3">$\left(\frac{v_{s,H}}{800}\right)^{-0,7\, r_{\beta}}$</td><td>$1,6(1-0,02\, S_{\beta,RP})$</td></tr>
<tr><td>C</td><td>$1,6(1-0,02\, S_{\alpha,RP})$</td><td>$2,3(1-0,03\, S_{\beta,RP})$</td></tr>
<tr><td>D</td><td>$1,8(1-0,04\, S_{\alpha,RP})$</td><td>$3,2(1-0,1\, S_{\beta,RP})$</td></tr>
<tr><td>E</td><td>$\left(\frac{v_{s,H}}{800}\right)^{-0,4 r_{\alpha}\frac{H}{30}\left(4-\frac{H}{10}\right)}$</td><td>$2,2(1-0,05\, S_{\alpha,RP})$</td><td>$\left(\frac{v_{s,H}}{800}\right)^{-0,7\, r_{\beta}\frac{H}{30}}$</td><td>$3,2(1-0,1\, S_{\beta,RP})$</td></tr>
<tr><td>F</td><td>$0,9\left(\frac{v_{s,H}}{800}\right)^{-0,4 r_{\alpha}}$</td><td>$1,7(1-0,04\, S_{\alpha,RP})$</td><td>$1,25\left(\frac{v_{s,H}}{800}\right)^{-0,7\, r_{\beta}}$</td><td>$4(1-0,1\, S_{\beta,RP})$</td></tr>
<tr><td></td><td colspan="4">$r_{\alpha} = 1 - 15\frac{S_{\alpha,RP}}{v_{s,H}}$, $r_{\beta} = 1 - 15\frac{S_{\beta,RP}}{v_{s,H}}$, $S_{\alpha,RP}$, $S_{\beta,RP}$ in m/s^2, $v_{s,H}$ in m/s</td></tr>
</table>

EN1998-1-1:202X (& Informative Annex of EN1998-5: 2004) Topographic amplification (top of hills or crest of a ridge)

$$S_{\alpha} = F_{T} F_{\alpha} S_{\alpha,RP} \qquad S_{\beta} = F_{T} F_{\beta} S_{\beta,RP}$$

Topography description	F_T	Simplified sketch
Flat ground surface, slope or isolated ridge with average slope angle i<15° or height <30m	1,0	
Slopes with average slope angle i > 15°	1,2	
Ridge with width at the top much smaller than at the base & average slope angle 15° < i <30°	1,2	
Ridge with width at the top much smaller than at the base and average slope angle i > 30°	1,4	

Values of F_T refer to top point T in the simplified sketches; linear decrease of F_T between points T and B (base) or A (100 m distance from T) where F_T = 1 applies.

EN1998-1:2004: Elastic spectra for special ground types S_1, S_2

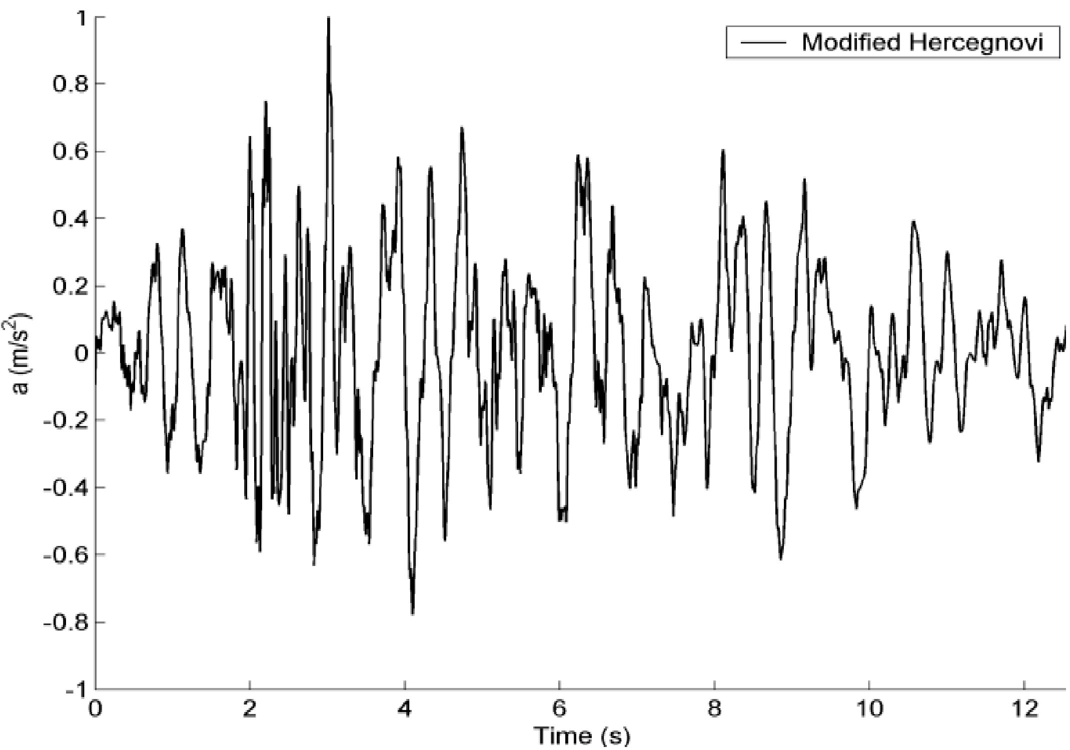
- Through a special site-specific study.
- S_1
  - ≥10m thick soft clay/silt with PI > 40 & high water content
  - Establish dependence of response spectrum on thickness and v_s value of soft clay/silt layer and on its stiffness contrast with the underlying materials (low internal damping and abnormally long range of linear behavior, likely to cause abnormal site amplification).
- S_2:
  - Liquefiable soils, sensitive clays, or any other soil not of type A – E or S_1
  - Consider possibility of soil failure.

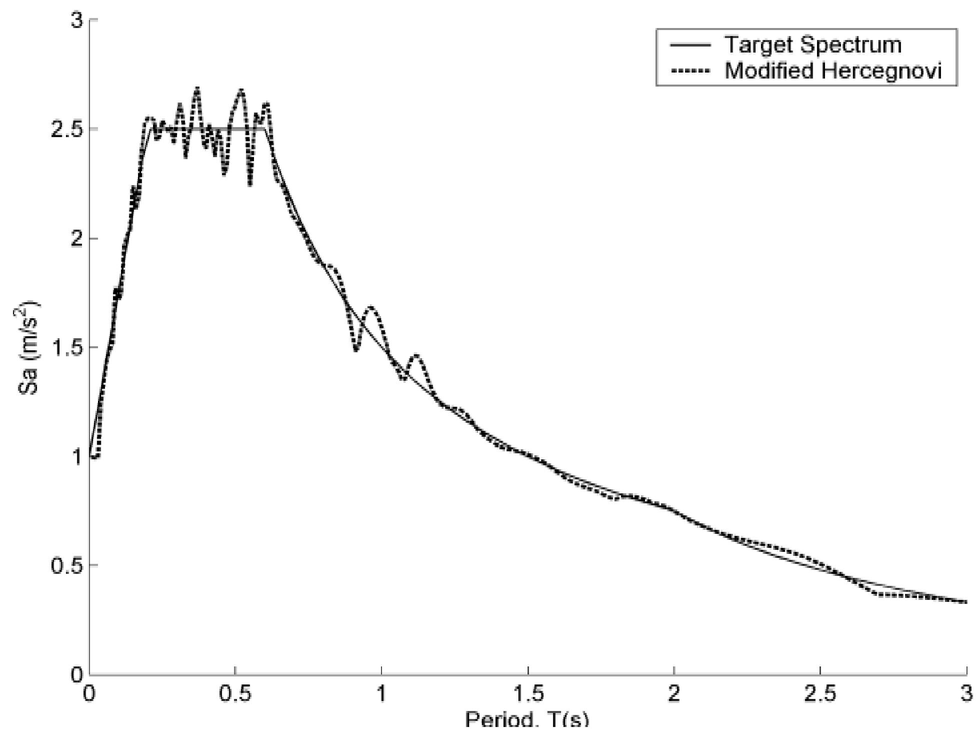
EN1998-1:2004: Ground motion acceleration records for response-history analysis

- Historic or simulated records preferred over artificial ones
  - **Simulated records:** from mathematical model of the source dominating the seismic hazard (rupture event, wave propagation via the bedrock to the site and via the subsoil to the ground surface).
  - **Historic records:** from seismic events with magnitude, fault distance & mechanism of rupture consistent with those dominating the hazard for the design seismic action. Travel path & subsoil conditions of recording station should resemble those of the site.
  - **Artificial ("synthetic") records:** mathematically derived from the target elastic spectrum (unrealistic if rich in all frequencies in the same way as the target spectrum; perfect matching of spectrum to be avoided).

EN1998-1:2004: Ground motion records for response-history analysis

- Component records scaled so that elastic spectra values ≥ 90% of code spectra (in the range of 1.5x to 20% of the fundamental period along the component).
- For pairs of horiz. components this is applied to SRSS of spectral values, taking 0.9√2 ~1.3.
- **≥ 7** independent seismic events (component or pair time-histories) needed if analysis results for peak response quantities are averaged;
- **≥ 3** if most adverse peak response from all the analyses is used.





Criteria and rules to satisfy the performance requirements

Performance-based design of new buildings in EN1998-1:2004 and EN1998-1-1:202X, EN1998-1-2:202X

- Two-(and-a-half) performance levels design:
  - ULS design of the structure (for ductility) for **Significant Damage;** Importance Class II or Consequence Class CC2 (ordinary) buildings, under the “Reference seismic action”.
  - SLS verification of infills/partitions for **Damage Limitation** under a frequent (~100 years) earthquake.
  - (implicit **Collapse Prevention** under a very strong/rare, but unspecified, earthquake thanks to Capacity Design.

EN1998-1:2004: Verification of Damage Limitation LS

- In buildings: Interstory drift ratio calculated for DL seismic action is:
  - < 0.5% for brittle nonstructural elements attached to structure;
  - < 0.75% for ductile nonstructural elements attached to structure;
  - < 1% for the structure (nonstructural elements not interfering w/ structural response.
- Recommended seismic action for Damage Limitation: 95 year return period: ~50% of 475 year seismic action.
- In frame buildings the damage limitation verification controls member sizes.

EN1998-1-2:202X: Property protection at various LSs

- In RC buildings: Interstory drift ratio for SD seismic action should be:

< 2% (equivalent to the 1% limit for non-interacting infills under a DL seismic action taken as 50% of SD one in EN1998-1:2004).

- If infills interfere with structural response, interstory drift ratios should respect the limits in the Table, to meet the performance requirements of the corresponding LS

Masonry Type	at IO [%]	at DL [%]	at SD [%]
Ductile masonry infills	0,38	0,75	2,00
Unreinforced masonry with clay units in Groups 1, 2 or 3 with thickness ≥ 200 mm and f_k ≥ 3MPa	0,23	0,45	1,40
Unreinforced masonry with units of Group 4	0,13	0,25	0,90

* In infills with openings, limits taken 30% lower; in confined or reinforced ones, 20% higher.

- Infills interfering with the structure:
  - may be neglected in the model of the analysis, if the imbalance in the product of their length parallel to an axis through the Centre of Stiffness of the story times the squared distance from that axis is less than 50% between the two sides of the axis;
  - for imbalance from 50% to 200% of the side with less infills, the internal forces and deformations from the analysis of the bare frame are increased by 30%;
  - for larger imbalance, infills should be included in the model as equivalent struts with width 25% of the diagonal, and verified at the SD LS in horizontal shear.

EN 1998-1:2004 Compliance to SD (and NC) LSs

Two options:

1. **Design for energy dissipation & ductility: q >1.5**
   - **Global ductility:**
     - Structure forced to remain straight in elevation through shear walls, bracing system or strong columns ($\Sigma M_{Rc} > 1.3 \Sigma M_{Rb}$ in frames):
   - **Local ductility:**
     - Plastic hinges detailed for ductility capacity derived from q-factor;
     - Brittle failures prevented by overdesign/capacity design
   - **Foundation (capacity-) designed to stay elastic:**
     - On the basis of overstrength of ductile elements of superstructure.

   (Or: Foundation elements - incl. piles - designed & detailed for ductility)
2. **Design for strength, w/o energy dissipation & ductility: q $\leq$ 1.5 for overstrength; design as for non-seismic loadings (Ductility Class "Low"– DCL) Only:**
   - for Low Seismicity (NDP; recommended: for PGA on rock $\leq$0.08g)
   - for superstructure of base-isolated buildings.

EN1998-1:2004:
Force-based design for energy dissipation & ductility

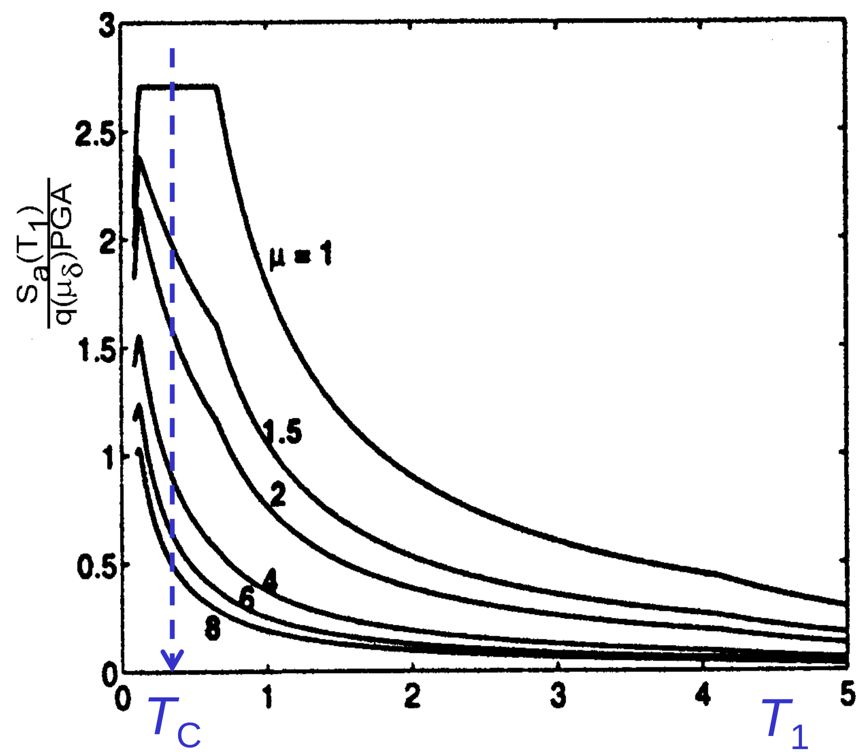
- Structure allowed to develop significant inelastic deformations under design seismic action, provided that the integrity of members & of the whole are not endangered.
- Basis of force-based design for ductility:
  - inelastic response spectrum of SDoF system having elastic-perfectly plastic F-δ curve, in monotonic loading.
- For given period, T, of elastic SDoF system, inelastic spectrum relates:
  - ratio $q = F_{el}/F_y$ of peak force, F_{el}, that would develop if the SDoF system was linear-elastic, to its yield force, F_y, (behavior factor)

  to

  - maximum displacement demand of the inelastic SDOF system, δ_{max}, expressed as ratio to the yield displacement, δ_y : global displacement ductility factor, $\mu_\delta = \delta_{max}/\delta_y$

Elastic design w/ force reduction and ductility

- In design for ductility: 5%-damped elastic spectrum reduced by (prescriptive) behavior factor *q*, (depends on the type, layout, regularity & redundancy of structural system):
- Global ductility:
  - One-to-one correspondence between *q* & global displacement ductility factor, μ_δ
  - Inelastic spectra of SDOF system (Vidic, Fajfar, Fischinger 1994):
    - If $T_1 \geq T_C$: $\boldsymbol{\mu_\delta = q}$
    - If $T_1 < T_C$: $\boldsymbol{\mu_\delta = 1+(q-1)T_C/T_1}$



Buildings of any material in EN1998-1:2004 or EN1998-1-2:202X

- Three Ductility Classes (DC): (except in masonry buildings):
  - DCH (High), DCM (Medium), DCL (Low) in EN1998-1:2004;
  - DC3, DC2, DC1 in EN1998-1-2:202X.
  - Differences in:
    - behavior factor q:
      - usually $q > 4$ in DCH/DC3;
      - $1.5 < q < 4$ in DCM/DC2
      - $q=1.5$ (usually) in DCL/DC1.
    - Local ductility requirements
      - ductility of materials or section,
      - member detailing,
      - capacity design against brittle failure modes.
- Heightwise irregular buildings: q-factor reduced by 20%.

Buildings of any material in EN1998-1-2:202X

- The behavior factor q for the reduction of the elastic spectrum in force-based design is split in three factors:
  - $q = q_s q_R q_D$
  - q_s= 1.5 due to (member and material) overstrength;
  - $q_R \geq 1$ reflects redundancy of structural system;
  - $q_D \geq 1$ reflects ductility/ capacity to deform inelastically and dissipate energy in cyclic loading.

Control of inelastic seismic response via capacity design

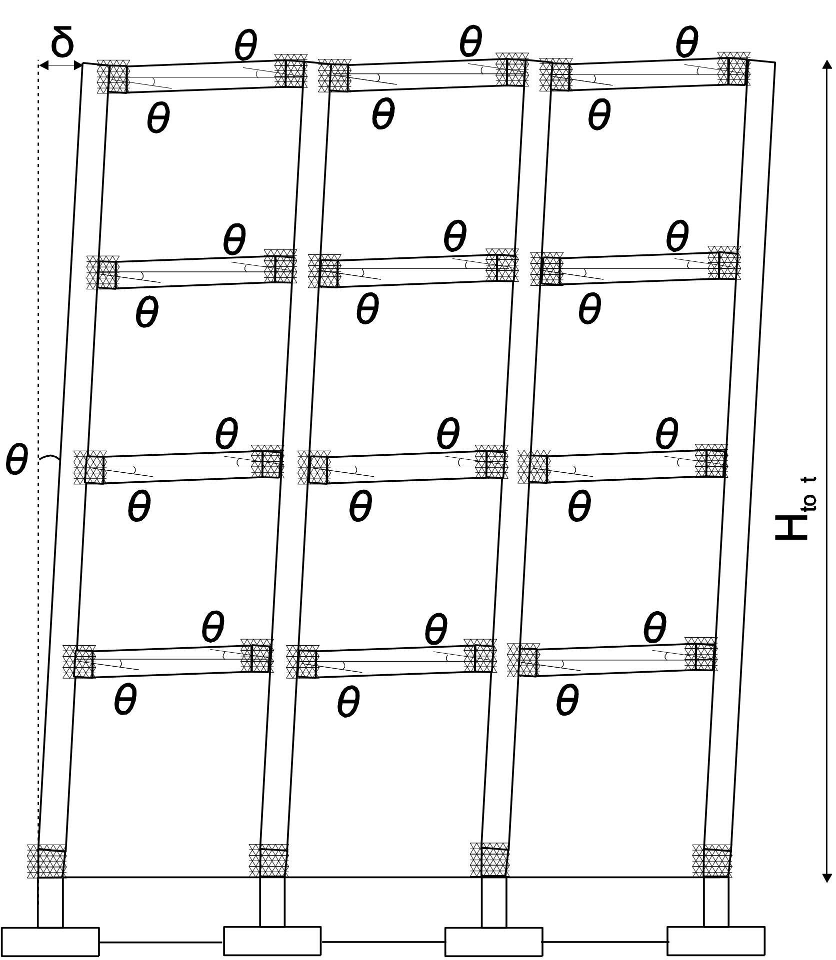
- Not every location or member of a structure is capable of ductile behavior & energy dissipation.
- **"Capacity design"** provides the necessary hierarchy of strengths between adjacent structural members or regions & between different mechanisms of load transfer within the same member, to ensure that inelastic deformations take place only where ductile behavior & energy dissipation is possible; the rest of the structure stays in the elastic range.
- Regions of members entrusted for hysteretic energy dissipation are called in Eurocode 8 **"dissipative zones"**; they are designed and detailed to provide the required ductility & energy-dissipation capacity.
- Before their design & detailing for the required ductility & energy-dissipation capacity, "dissipative zones" are dimensioned to provide a design value of ULS force resistance, R_d, at least equal to the design value of the action effect due to the seismic action, E_d, from the analysis:

$$\boldsymbol{R_d > E_d}$$

- Normally linear analysis is used for the design seismic action (by dividing the elastic response spectrum by the behavior factor, q)

From global (μ_δ) to local (chord rotation) ductility factor μ_θ

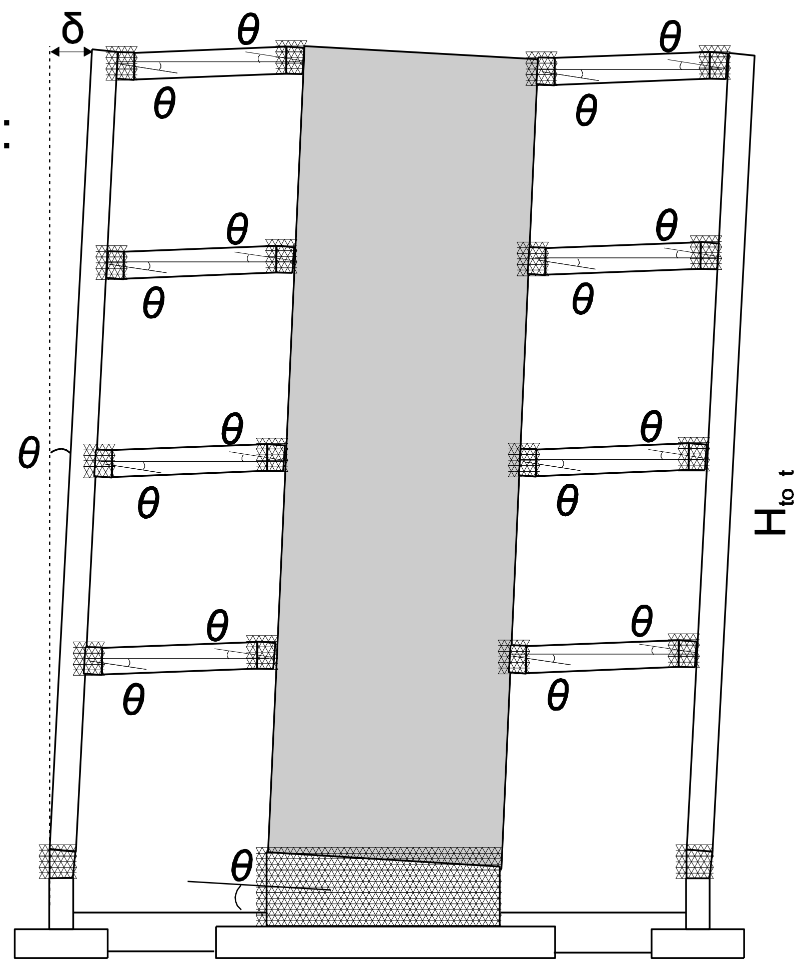
Ductility demands uniformly spread throughout the plastic mechanism thanks to a stiff and strong vertical spine of a building:



At the base of vertical members and at beam ends:

$$\boldsymbol{\theta} = \delta / \boldsymbol{H_{tot}}$$

$$\boldsymbol{\mu_\theta = \mu_\delta}$$



Strong-column/weak-beam capacity design: $\sum M_{Rc} \geq 1.3 \sum M_{Rb}$

walls taking ≥ 50% of seismic base shear

EN1998-1:2004: Design of the foundation by capacity design

- **Objective:** The soil & the foundation system may not reach their ULS before the superstructure: stay elastic when superstructure is inelastic.
- **Means (in EN1998-1:2004):**

1. The soil and the foundation system are designed for the ULS under seismic action effects from an analysis with q=1.5 (< q used for the superstructure); **or**
2. The soil and the foundation system are designed for the ULS under seismic action effects from the analysis multiplied by:

$$\gamma_{Rd}(R_{di}/E_{di}) \leq q, \ (\gamma_{Rd}=1.2, \text{ or } = 1 \text{ if } q \leq 3)$$

   - R_{di}: force capacity in dissipative zone controlling seismic action effect of interest
   - E_{di}: seismic action effect in that dissipative zone from the linear analysis
   - For individual spread footings of walls or columns:

     R_{di}/E_{di}= minimum value of M_{Rd}/M_{Ed} in the two orthogonal principal directions at the lowest cross-section of the vertical element where a plastic hinge may form;
   - For common foundations of more than one elements, $\gamma_{Rd}(R_{di}/E_{di})$ =1.4;

   or

3. Soil designed for seismic action effects as in 1 or 2, but foundation system designed for the seismic action effects from the analysis (with the value of q of the superstructure) and capacity design, and detailed for ductility, like the superstructure.

Performance-based assessment/retrofitting of existing buildings in EN1998-3: 2005 (Part 3 of EC8)

- **Up to three-tier seismic assessment/retrofitting:**

  Limit States (Performance Levels):

  - **Damage Limitation**
  - **Significant Damage**
  - **Near Collapse.**

- Flexibility for country/owner/designer to choose how many and which Limit States to meet and under what Hazard Level.
- A note mentions as objectives for ordinary new buildings:
  - Damage Limitation: Occasional earthquake (225 yrs???)
  - Significant Damage: Rare earthquake (475 yrs)
  - Near Collapse: Very rare earthquake (2500 yrs)

EN1998-1-1 & -1-2:202X: Displacement-based design with nonlinear analysis & direct verification of deformations

- Nonlinear analysis, static (pushover) or dynamic (t-history)
- Member verification at the SD or NC Limit State in terms of:
  - <u>deformations</u> in <u>ductile</u> members/mechanisms (flexure);
  - <u>forces</u> (resistances) in <u>brittle</u> members/mechanisms (for RC: shear).
- Deformation capacities:

  →Given in EN 1998-1-1:202X - for concrete, following the approach in Annex A of EN1998-3:2005 (Assessment & retrofitting), but with more complete and widely applicable models.
- Force-based approach, with elastic spectrum reduced by the behavior factor, q, retained, but restricted to SD Limit State – and to DL or OP ones with q = 1.

Analysis for the seismic action

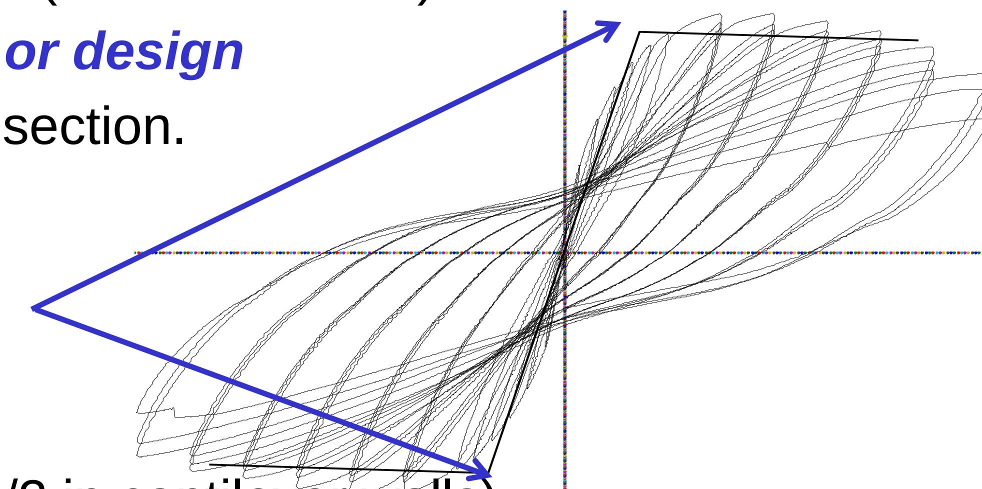
Elastic stiffness for the analysis

- To simulate SDOF system with bilinear force-displacement relation (: basis of inelastic spectrum relating global displacement ductility factor and force-reduction/behavior factor):
  - ➢ Use secant-to-yield-point stiffness in the analysis
  - ➢ In concrete & masonry buildings:
  - Unless more accurately determined (eg, the value given in EN1998-3:2005 or EN1998-1-1:202X), use 50% of uncracked gross section stiffness as secant-to-yield-point stiffness:
- Compared to use of full uncracked section stiffness:
  - ➢ Design seismic forces reduced
  - ➢ Displacements for drift-control & P-Δ effects increased (govern size of frame members).

Elastic stiffness: Controls dominant period(s) of nonlinear response

- ***For forced-based design:***
  – EI=50% of uncracked section stiffness overestimates by ~100% secant-to-yield-point stiffness;
- overestimates force demands (safe-sided);
- underestimates displacement demands (unconservative).
- ***For displacement-based evaluation or design***
  – EI= Secant stiffness at yielding of end section.

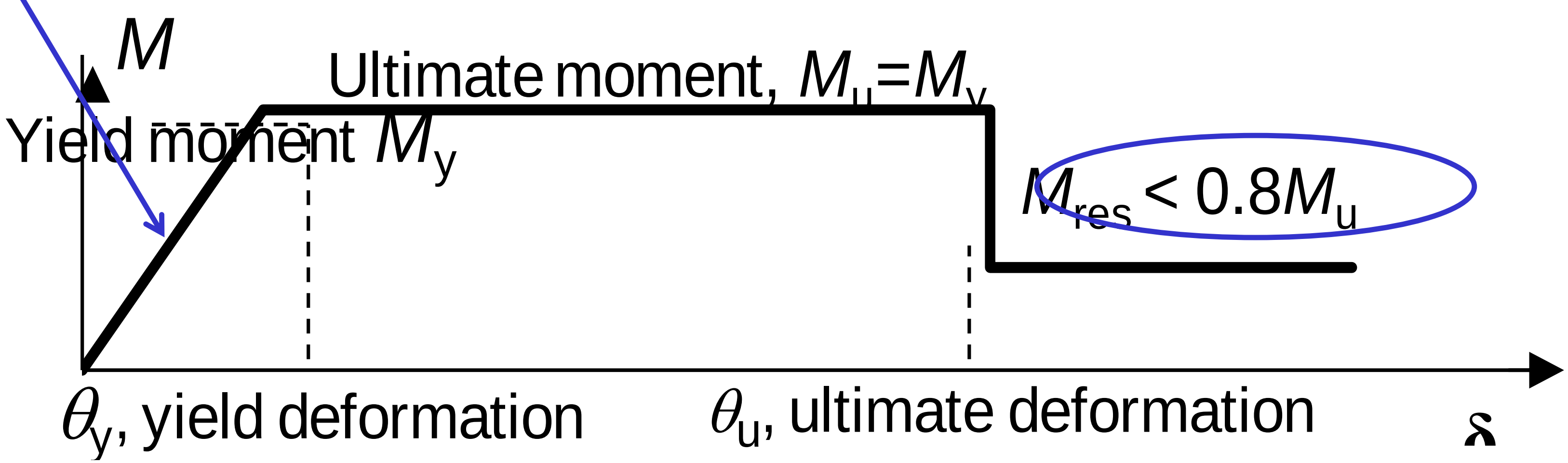
$$EI = M_y L_s / 3\theta_y$$



– Effective stiffness of shear span L_s
– $L_s = M/V$ (~$L_{cl}/2$ in beams/columns, ~$H_w/2$ in cantilever walls),
– M_y, θ_y: moment & chord rotation at yielding;
– Average EI of two member ends in positive or negative bending.

EN1998-1-1:202X: Idealized envelope of cyclic moment-deformation behavior in displacement-based approach

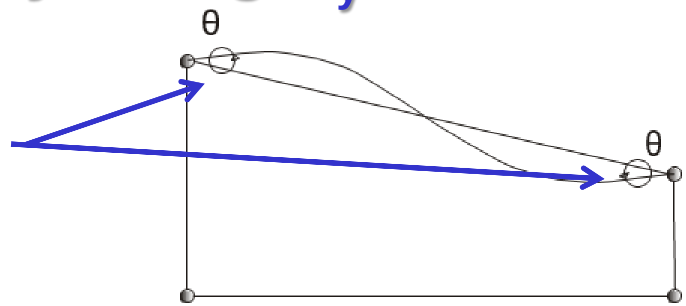
Effective elastic stiffness:
secant-to-yield-point:
$EI = M_y L_s / 3\theta_y$



member deformation: chord-rotation

EN1998-1-1:202X: Member chord rotation at yielding, θ_y

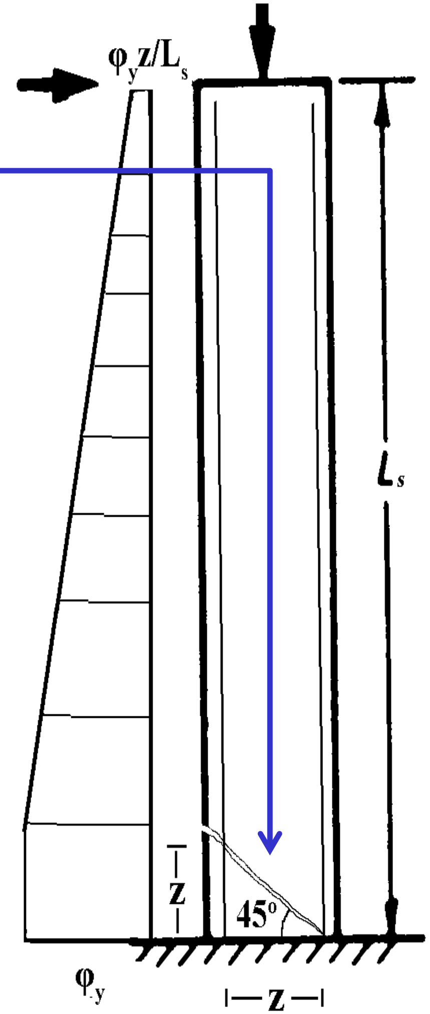
(Chord rotation at the end of a member, θ: angle between normal to end section and chord connecting member ends at the displaced position).



θ_y = sum of:

1. a flexural component:
   - $\varphi_y(L_s+z)/3$ if 45º-cracking of member precedes flexural yielding of its end section (shear force at flexural yielding, M_y/L_s> shear strength w/o shear reinforcement);
   - $\varphi_y L_s/3$ if it doesn't
2. a shear deformation,
   - beams/rect. columns: $0.0019(1+h/1.6L_s)$
   - walls/box sections: $0.0011(1+h/3L_s)$
   - circular columns : $0.0025\max[0; 1-L_s/8D]$
3. fixed-end-rotation due to slippage of tension bars from their anchorage outside the member length;

   at yielding of the end section: $\theta_{slip,y} = \varphi_y d_{bL} f_y/8\sqrt{f_c}$ (MPa)



"Secondary seismic elements"

- Their contribution to resistance/stiffness for seismic actions considered unreliable: Neglected in analysis model for the seismic action.
- Elements outside EN1998-1's scope or violating its rules (eg, very eccentric beam/column connections) taken as "secondary seismic".
- Designer free to consider elements as "secondary seismic", provided that:
  - Regularity classification of building does not change.
  - Their total contribution to lateral stiffness is:
    - $\leq$15% of that of "primary seismic elements"; or
    - (in EN1998-1-2:202X only): $\leq$30% of that of "primary elements" but the latter are verified under the most unfavorable results of two analyses: one "with", the other "without" the "secondary elements".
- "Secondary elements" should be verified under the deformations imposed by the SD seismic action to:
  - (in EN1998-1:2004) remain elastic.
  - (in EN1998-1-2:202X) maintain their gravity-load-bearing role; columns which yield should be detailed like DC2 columns.

Two analyses for the verification: with or without secondary elements

Linear analysis for Force-based design in EN1998-1:2004 and EN998-1-1 & 1-2:202X

- **Reference design/analysis approach:**
- Linear modal response spectrum analysis, with <u>design response spectrum</u> (elastic spectrum reduced by <u>behavior-factor q</u>):
  - Applies always (except to seismic isolation for very nonlinear devices)
- If:
  - building heightwise regular & $T<4T_C$ and <2s (EN1998-1:2004),
  - <30m tall & $T<4T_C$ and <1.5s (EN1998-1-2:202X)

    (T_C: T at end of constant spectral acceleration plateau):

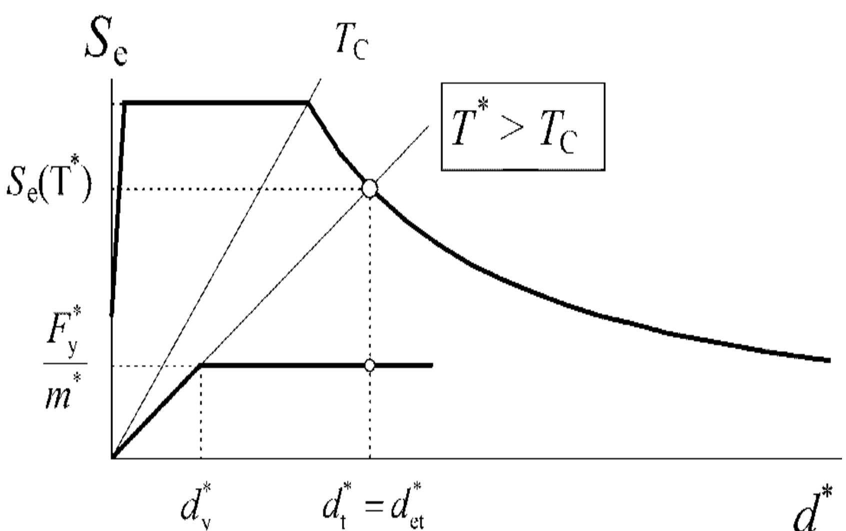
  Lateral force procedure emulating response-spectrum method
  - Fundamental T (mechanics, eg, Rayleigh quotient) gives base shear from a single entry of the design response spectrum;
  - Base shear reduced by 15% if >2 stories & $T<2T_C$ (& <1.2s in EN1998-1-2:202X)
- Members verified at ULS for SD seismic action in terms of forces.

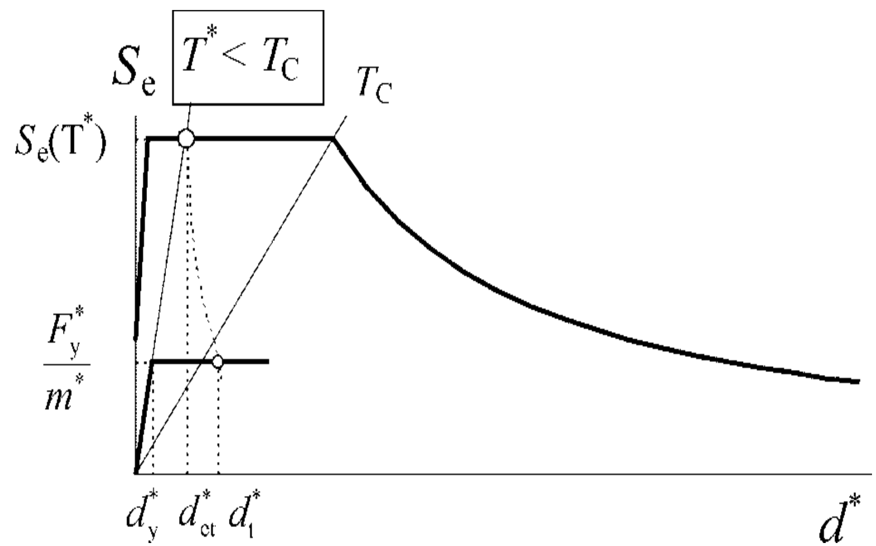
Linear analysis - Details

- **Reference method: modal response spectrum analysis, with spectrum reduced by behavior factor**
  - Number of modes to be taken into account:
    - Everyone with modal mass ≥5% of total in direction of application of seismic action;
    - Sufficient to collectively account for ≥ 90% of total mass in each direction of application of the seismic action.
  - Combination of modal responses:
    - CQC (Complete Quadratic Combination);
    - SRSS (Square-Root-of-Sum-of-Squares) if ratio of successive modal periods > 0.9 & < 1/0.9.
- **Lateral force procedure ("equivalent static"):**
  - Static lateral forces on story or nodal masses proportional to the mass times its distance from the base (inverted triangular heightwise distribution).

Nonlinear Static (Pushover) analysis w/ verification of deformations

- Lateral forces proportional to shape of mode w/ largest mass in direction of analysis - heighwise linear if lateral force approach applies.
- N2 method.
- Stiffness taken as the secant to first member yielding over structure.
- Target displacement from 5%-damped elastic spectrum:
  - equal displacement if $T>T_C$ $\mu=1+(q-1)T_c/T$, if $T<T_C$ (T_C: transition period)





- In EN1998-1-2:202X, the results of pushover analysis are corrected so that the base shear is at least 60% of that from linear analysis with the elastic spectrum reduced by the behavior factor q.

Accidental eccentricity

- There is no “dynamic” amplification of “natural” eccentricity.
- **EN1998-1:2004:** Accidental displacement of all masses in direction normal to the horizontal seismic action component, by:
  - $e_i = \pm 0.05 L_i$ ($\pm 0.1 L_i$ if there are irregular-in-plan masonry infills), L_i: plan dimension normal to the horizontal seismic action component
- Taken into account by means of:
  - For lateral force or modal response spectrum procedure, by linear static analysis under torques (w.r.to vertical axis) on story or nodal masses equal to the story or nodal forces of the lateral force procedure, times $e_i = 0.05 L_i$ (same sign at all stories or nodes) and superposition of the action effects to those due to the horizontal seismic action components w/o the accidental eccentricity.
  - For nonlinear response-history analysis, by shifting all story masses by the accidental eccentricity, analyzing four models instead of one and taking for every response measure the most adverse outcome from the four analyses.
- **EN1998-1-2:202X:** accidental eccentricity neglected, if it is less than the natural eccentricity of Centre of Mass w.r.to Centre of Stiffness.

Combination of peak effects of individual seismic action components

- For linear analysis:
  - Rigorous approach : SRSS-combination of seismic action effects EX, EY, EZ of individual components X, Y, Z: $\mathbf{E=\pm\sqrt{(EX^2+EY^2+EZ^2)}}$
  - Approximation: $\mathbf{E=\pm\max(|EX|+0.3|EY|+0.3|EZ|;}$
    $$\mathbf{|EY|+0.3|EX|+0.3|EZ|;}$$
    $$\mathbf{|EZ|+0.3|EX|+0.3|EY|).}$$
  - The approximation is followed **in EN1998-1:2004** for nonlinear static (Pushover) analysis, with component Z neglected and internal forces from above combinations not exceeding the member force resistances. Problems with this approach if the modal load patterns have components in both horizontal directions; addressed in the extension of the Pushover approach adopted **in EN1998-1-2:202X**, to cover 3D, torsional and higher mode effects.
- Time-history nonlinear analysis:

  Seismic action components X, Y, Z applied simultaneously.

Extension of Pushover analysis in EN1998-1-2:202X to cover 3D, torsional & higher-mode effects

- Pushover analysis under modal lateral loads in a single horizontal direction.
- Accompanied by a linear analysis under the corresponding horizontal seismic action component – preferably modal, even when the conditions for applying the lateral force procedure are met.
- The linear analysis should account for the accidental eccentricity – if larger than the natural – and the concurrent orthogonal component – be it with the 0.3:1 approximation.
- Displacements at any point in the structure from the pushover analysis should be scaled-up (never down) so that they are the same proportion of those at the control node as in the linear analysis (the latter including the effects of the orthogonal component and torsion – natural or accidental).
- For higher mode effects, local deformations from pushover analysis – including chord rotations – are further scaled-up (not down) by a ratio of ratios: of the ratio of interstory drifts at the Mass Centre of the story of interest from linear and pushover analyses, to that of lateral displacements at the control node from these analyses.
- Simplifications of the above corrections are provided.

Regularity of buildings in elevation

- Effects of regularity in elevation:
  - In EN1998-1:2004: Lateral force analysis not applicable;
  - Behavior factor q reduced by 20%
- Criteria: Qualitative, can be checked without calculations:
  - Structural systems (walls, frames): *continuous to the roof*.
  - story *K & m: constant or gradually decreasing to the top (by <20% per story in EN1998-1-2:202X)*.
  - story strength/demand from analysis*: heightwise smooth variation (<30% story-to-story difference in EN1998-1-2:202X)*
  - In EN1998-1:2004: Floor setbacks:
    - on each side: < *10% of underlying story.*
    - Asymmetric: < *30% of base in total.*
    - Single setback at lower 15% of building: < *50% of base.*

Calculation of displacements for linear analysis

- If linear analysis with the design response spectrum gives d_e as the displacement at a point of the structural system,
  - the “real” displacement of that point, d_s, is computed as:

    $$d_s = q_{\mathrm{displ}}\, d_\mathrm{e}$$

  - q_{displ} equal to q if fundamental period $> T_C$;
  - $q_{displ} = 1+(q-1)T_C/T < 3q$ if fundamental period $< T_C$.
- Displacements are not to be taken greater than those which would had been calculated from linear analysis with $q = 1$.

2^{nd}-Order (P-Δ) effects

- 2^{nd}-order effects computed at the story level (index: i) via their ratio to the 1^{st}-order effects of seismic action (in terms of story moments):

$$\boldsymbol{\theta_i = N_{tot,i} \Delta\delta_i / V_i H_i}$$

  - $N_{tot,i}$: total vertical load at and above story i in seismic design situation;
  - V_i = story shear in story i due to SD seismic action;
  - H_i = height of story i.
  - $\Delta\delta_i$ = interstory drift at story i due to SD seismic action:
  - if linear analysis with the design spectrum is used:
    - $\Delta\delta_i$ is that from the analysis times $q_d = 1+(q-1)\max(1;T_C/T) < 3q$.
    - In EN1998-1-2:202X, V_i is the shear in story i from the design spectrum due to the SD seismic action, times the components of the behavior factor due to overstrength and system redundancy, $q_s q_R$
- In the usual case where $\boldsymbol{\theta_i \leq 0.1}$ at all stories 2^{nd}-order effects ignored;
- If $\boldsymbol{\theta_i > 0.1}$ at any story, 2^{nd}-order effects taken into account by dividing all 1^{st}-order effects by $(1-\theta_i)$;
- $\boldsymbol{\theta_i > 0.2}$ at any story: Geometrically nonlinear analysis.

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Column capacity design in ductile frames of any material

- Strong column/weak beam capacity design to avoid soft-story mechanisms:

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

- Required by EN1998-1:2004 in primary columns of DCH or DCM frames (in RC buildings also in frame-equivalent dual systems) of over two stories.
- In EN1998-1-2:202X:
  - The requirement is limited to DC3 RC frames and frame-equivalent systems of two or more stories – except in 25% of the columns per frame and at the ground story of two-story buildings with axial load ratio less than 0.3.
  - In DC2 RC frames and frame-equivalent dual systems:
    - If designed with the Displacement-based approach, soft-stories considered to be prevented by meeting the chord-rotation verifications at column ends.
    - In DC2 RC frames or frame-equivalent dual systems designed with Force-based approach, soft-stories considered to be prevented if moment resistances at the ends of the n columns where plastic hinges may form (index i) meet the following at every story ($\theta_{u,min}^{pl}$: minimum column plastic rotation capacity in story, $d_{e,top}$: displacement of the top from linear analysis)

$$q_{\mathrm{S}} q_{\mathrm{R}} V_{\mathrm{tot},storey} (q - q_{\mathrm{S}}) d_{\mathrm{e,top}} \leq 2 \sum_{i=1}^{i=n} M_{\mathrm{Rd,column\ i}}\, \theta_{\mathrm{u,min}}^{\mathrm{pl}}$$

Design and detailing of RC buildings

Material limitations for "primary seismic elements"

EN 1998-1:2004	DC L or M	DC H
Concrete strength class, MPa	≥ 16	
10%-fractile yield strength of steel, f_{yk}, MPa	400 to 600	
10%-fractile hardening ratio of steel, $(f_t/f_y)_{k,0.10}$	≥1.08	≥1.15 <1.35
10%-fractile strain at maximum stress, $\varepsilon_{su,k,0.10}$	≥5.0%	≥7.5%
95%-fractile actual yield strength, $f_{yk,0.95}/f_{yk}$	-	≤1.25

EN 1998-1-2:202X	DC 1	DC 2 or 3
Concrete strength class, MPa	≥ 16	≥ 20
10%-fractile yield strength of steel, f_{yk}, MPa	400 to 700	
10%-fractile hardening ratio of steel, $(f_t/f_y)_{k,0.10}$	≥1.08	
10%-fractile strain at maximum stress, $\varepsilon_{su,k,0.10}$	≥5.0%	

Frame, wall, or dual systems in RC buildings

- **Definitions:**
  - Frame system: Frames take >65% of seismic base shear, V_{base}
  - Wall system: Walls take > 65% of V_{base}.
  - Dual system: Walls and frames take between 35 % & 65% of V_{base} each.
  - Frame-equivalent dual system: Frames take between 50 % & 65% of V_{base}.
  - Wall-equivalent dual system: Walls take between 50 % & 65% of V_{base}.

- **Eurocode 2 definition of wall: Wall ≠ column in that its cross-section is elongated (l_w/b_w>4)**

Restrictions in the use of DCs and structural systems depending on seismicity

- **EN 1998-1:2004**
- In cases other than of low seismicity (475yr PGA at the surface of the ground >1 m/sec^2, i.e., 475yr constant spectral acceleration of the elastic spectrum at the surface >2.5m/sec^2) DCL not recommended.

- **EN 1998-1-2:202X**
- If the Seismic Action Class index (δ times the 475yr constant spectral acceleration of the elastic spectrum at the surface of the ground) is:
  - >2.5 m/sec^2:
    - frame or dual structures should be designed for DC 2 or 3;
  - >5 m/sec^2:
    - frame structures should be designed for DC 3,
    - wall structures for DC 2.

Deformation limits in EN1998-1-1:202X for verifications and detailing in the displacement-based approach

EN1998-1-1:202X: Cyclic plastic rotation capacity, for rectangular compression zone & continuous ribbed bars

$$\theta_u = \theta_y + \theta_u^{pl} = \theta_y +$$

$$0.0206\left(1-0.22a_{nc}\right)\left(1-0.41a_{w,r}\right)\left(1-0.31a_{nr}\right)\left(0.2^{\nu}\right)\left[\frac{\max\left(0.01;\omega_2\right)}{\max\left(0.01;\omega_1\right)}\right]^{0.25} f_c^{\,0.1}\left[\min\left(9;\frac{L_s}{h}\right)\right]^{0.35} 24^{\left(\frac{\alpha\rho_s f_{yw}}{f_c}\right)} 1.225^{100\rho_d}$$

- $a_{w,r}$ = 1 for rect. walls, $a_{w,r}$=0 in all other cases;
- a_{nr} =1 in T-, H-, U-, box sections, a_{nr}=0 for rectangular sections;
- a_{nc} = 1 for poorly detailed members, a_{nc} = 0 for well-detailed ones;
- $\nu=N/bhf_c$; *b:* width of compression zone, *N:* axial force, >0 for compression;
- $\omega_1=(\rho_1 f_{y1}+\rho_v f_{yv})/f_c$ mech. steel ratio in entire tension zone (flange & web);
- $\omega_2=\rho_2 f_{y2}/f_c$ mechanical reinforcement ratio for the compression zone;
- $L_s/h=M/Vh$: shear-span-to-depth ratio at the section of maximum moment;
- $\rho_s=A_{sh}/b_w s_h$: ratio of transverse steel parallel to the plane of bending;
- α: confinement effectiveness factor:

$$\alpha = \left(1-\frac{s_h}{2b_o}\right)\left(1-\frac{s_h}{2h_o}\right)\left(1-\frac{\sum b_i^{\,2}/6}{b_o h_o}\right)$$

s_h: centreline spacing of stirrups,
b_o, h_o: confined core dimensions to centreline of hoop;
b_i: centerline spacing on section perimeter of longitudinal bars (index: *i*) engaged by a stirrup corner or cross-tie.

Cyclic plastic chord rotation capacity for any RC X-section

- **Plastic part of ultimate chord rotation** at member end from ultimate & yield curvature of end section, φ_u, φ_y, L_s=M/V: shear span at member end, L_{pl}: "plastic hinge length".

$$\theta_u^{pl} = (\varphi_u - \varphi_y) L_{pl}\left(1 - \frac{L_{pl}}{2L_s}\right) + \Delta\theta_{slip,u-y}$$

  $\Delta\theta_{slip,u\text{-},y}$: post-yield part of fixed-end-rotation due to slippage of longitudinal bars from anchorage zone outside the member length.

- **$\boldsymbol{\varphi_u}$, $\boldsymbol{\varphi_y}$: from plane-section analysis.**

– For φ_y: linear σ-ε relations till yielding of tension or compression chord.

– For φ_u: parabola-rectangle σ-ε diagram for concrete in compression, bilinear with linear strain-hardening for the reinforcing steel.

Calculation of φ_u should take into account all possible failure modes:

a) rupture of tension reinforcement in the full, unspalled section;

b) exceedance of concrete ultimate strain ε_{cu2} at the extreme compression fibers of unspalled section;

c) rupture of tension bars in the confined core after spalling of the cover;

d) exceedance of the ultimate strain $\varepsilon_{cu2,c}$ of the confined core after spalling.

Failure mode (b) governs over (c) or (d), if the moment resistance of the confined core > 80% of that of the full unspalled and unconfined section.

EN1998-1-1:202X: Ultimate strains in a RC member in cyclic flexure

- **Before spalling:**
  - Steel: $\varepsilon_{su}=0.4\varepsilon_{u,k}$, Concrete: $$0.0035 \le \varepsilon_{cu} = \left(18.5/h(mm)\right)^2 \le 0.01$$
- **After spalling:**
  - Steel: $\varepsilon_{su}=(4/15)\varepsilon_{u,k}$ $$\left(1+3d_b/s_h\right)\left(1-0.75\exp(-0.4N_{bars,compression}\right)$$
  - Concrete: $f_{cc} = f_c(1+K)$, $\varepsilon_{co,c} = \varepsilon_{co}(1+5K)$, $K = \min\left(4\frac{a\rho_w f_{yw}}{f_c}; 3{,}5\left(\frac{a\rho_w f_{yw}}{f_c}\right)^{\frac{3}{4}}\right)$
    - for rect. compression zone: $$\varepsilon_{cu,c} = \varepsilon_{cu} + 0.04\sqrt{a\rho_w f_{yw}/f_c}$$
    - for circular sections: $$\varepsilon_{cu,c} = \varepsilon_{cu} + 0.07\sqrt{a\rho_w f_{yw}/f_c}$$
    - for triangular compression zone: $$\varepsilon_{cu,c} = \varepsilon_{cu} + 0.12\sqrt{a\rho_w f_{yw}/f_c}$$

▪ ρ_w: ratio of transverse reinforcement in direction of bending (or minimum in two transverse directions for biaxial bending); f_{yw}: its yield stress,

▪ α: confinement effectiveness factor:
  – rectangular sections: $$\alpha = \left(1-\frac{s_h}{2b_o}\right)\left(1-\frac{s_h}{2h_o}\right)\left(1-\frac{\sum b_i^{\,2}/6}{b_o h_o}\right)$$
  – circular sections & circular hoops: $$\alpha = \left(1-\frac{s_h}{2D_o}\right)^2$$
  – circular sections & spiral reinforcement: $$\alpha = \left(1-\frac{s_h}{2D_o}\right)$$

s_h: centerline spacing of stirrups,
D_o: confined core diameter to centreline of hoop.

EN1998-1-1:202X: Cyclic plastic chord rotation capacity (*cont'd*)

- Between yielding of end section and ultimate curvature there in cyclic loading, yielding of tension bars penetrates into their anchorage zone: the fixed-end-rotation of the end section increases due to slip of tension bars from their anchorage by:

$$\Delta\theta_{slip,u-y} = 4.25 d_b (\varphi_u + \varphi_y)$$

- If φ_u, φ_y, $\Delta\theta_{slip,u-y}$ are determined as above:

  Plastic hinge length L_{pl} in cyclic loading

  – beams, rect. columns or walls, T-, H-, U-, box-sections:

$$L_{pl} = 0.3h\left[1 + 0.4\min\left(9; \frac{L_s}{h}\right)\right]\left(1 - \frac{1}{3}\sqrt{\min\left(2.5; \max\left(0.05; \frac{b_w}{h}\right)\right)}\right)\left(1 - 0.45\min(0.7; \nu)\right)$$

  – circular columns:

$$L_{pl} = 0.7D\left[1 + \frac{1}{7}\min\left(9; \frac{L_s}{D}\right)\right]\left(1 - 0.7\min(0.7; \nu)\right)$$

- **In members with poor detailing, L_{pl} increases by 30%**

EN 1998-1-1 & -1-2:202X – Verification of flexural deformations at SD and NC Limit States

$$\theta_{\mathrm{R,NC}} = \frac{1}{\gamma_{\mathrm{Rd},\theta}}\left(\theta_{\mathrm{y}} + \theta_{\mathrm{u}}^{\mathrm{pl}}\right)$$

$$\theta_{\mathrm{R,SD}} = \frac{1}{\gamma_{\mathrm{Rd},\theta}}\left(\theta_{\mathrm{y}} + 0.5\theta_{\mathrm{u}}^{\mathrm{pl}}\right)$$

$\gamma_{\mathrm{Rd},\theta}$=1.6 (provisional value)

Design and detailing in the force-based approach

EN 1998-1:2004

Basic value q_o of behavior factor- regular in elevation RC buildings

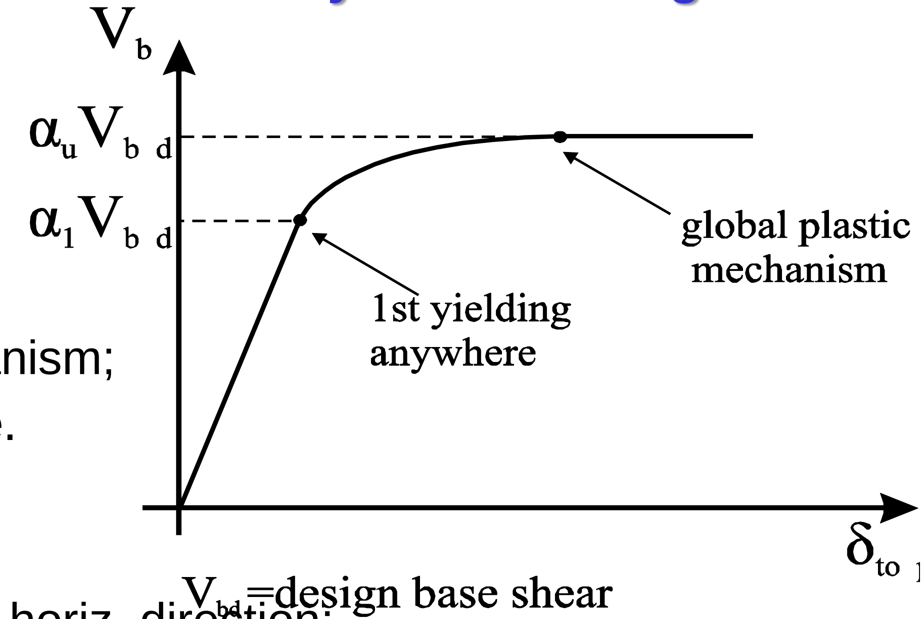
Lateral-load resisting structural system	DC L	DC M	DC H
Inverted pendulum system*	1.5	1.5	2
Torsionally flexible structural system**	1.5	2	3
Uncoupled wall system (>65% of base shear taken by walls; >half by uncoupled walls) not belonging in one of the categories above	1.5	3	$4\alpha_u/\alpha_1$
Any structural system other than those above	1.5	$3\alpha_u/\alpha_1$	$4.5\alpha_u/\alpha_1$

***Inverted pendulum system**: $\geq$ 50% of total mass in upper-third of the height, or all energy dissipation takes place at the base of a single element (except one-story frames with all columns connected at the top via beams in both horizontal directions in plan and with max. value of normalized axial load in seismic design situation $v_d \leq 0.3$).

** **Torsionally flexible structural system**: at any floor: radius of gyration of floor mass > torsional radius in one or both main horizontal directions (sensitive to torsional response about vertical axis).

- Buildings irregular in elevation: behavior factor $\mathbf{q = 0.8q_o}$;

α_u/α_1 in q-factor of buildings for system redundancy & overstrength



Normally:
α_u & α_1 from base shear-top displacement curve of a pushover analysis.

- α_u: seismic action at development of global mechanism;
- α_1 : seismic action at 1st flexural yielding anywhere.
- $\alpha_u / \alpha_1 \leq 1.5$;
- default values for buildings regular in plan:

= 1.0 for wall systems w/ just 2 uncoupled walls per horiz. direction;

= 1.1 for: one-story frame or frame-equivalent dual systems, or wall systems w/ > 2 uncoupled walls per direction;

= 1.2 for: (one-bay multi-story frame or frame-equivalent dual systems), wall-equivalent dual systems or coupled wall systems;

= 1.3 for:multi-story multi-bay frame or frame-equivalent dual systems.

- for buildings irregular in plan:

default value = average of default value of buildings regular in plan and 1.0

EN 1998-1-2:202X

Basic value q of behavior factor- <u>regular in elevation</u> RC buildings

		q_R	q_D		q	
			DC2	DC3	DC2	DC3
Frame or frame-equivalent dual structures	multi-story, multi-bay frames or frame-equivalent dual structures	1,3	1,3	2,0	2,5	3,9
	multi-story, one-bay frames	1,2			2,3	3,6
	one-story frames	1,1			2,1	3,3
Wall- or wall-equivalent dual structures	wall-equivalent dual structures	1,2	1,3	2,0	2,3	3,6
	coupled walls structures	1,2	1,4		2,5	3,6
	uncoupled walls structures	1,0	1,3		2,0	3,0
	large walls structures	--	--		3,0	
Flat slab structures*		1,1	1,2	--	2,0	--
Inverted pendulum system		1.0	1.5	1.5	1.5	1.5

* provisional; too conservative

- Buildings <u>irregular in elevation or torsionally flexible systems</u>:
  behavior factor $\underline{\boldsymbol{q = 0.8q_o}}$;

EN 1998-1:2004

Ductility of plastic hinges by detailing them for a target curvature ductility factor μ_φ derived from the q-factor

- $\mu_\varphi = 2q_o - 1$ if $T_1 \geq T_c$
- $\mu_\varphi = 1 + 2(q_o - 1)T_c/T_1$ if $T_1 < T_c$
  - T_1: fundamental period of building,
  - T_c: T at upper limit of constant spectral acceleration region,
  - q_o: *q-factor* unreduced for irregularity in elevation (multiplied with M_{Ed}/M_{Rd} at a wall base).
- For steel class B (ε_u: 5-7.5%, f_t/f_y: 1.08-1.15) increase μ_φ-demand by 50%

EN 1998-1:2004 Means for achieving μ_φ in plastic hinges

- **Base of columns, ductile walls** (symmetric reinforcement $\omega=\omega'$):
  - Confining reinforcement (for walls: in boundary elements) with (effective) mechanical volumetric ratio:

$$\alpha\omega_{wd} = 30\mu_\varphi(v_d+\omega_v)\varepsilon_{yd}b_c/b_o - 0.035$$

    $v_d=N_d/b_chf_{cd}$; $\varepsilon_{yd}=fy_d/E_s$;

    b_c: width of compression zone; b_o: width of confined core;

    ω_v: mechanical ratio of longitudinal web reinforcement $=\rho_v f_{yd,v}/f_{cd}$

  - DC H columns not meeting the strong-column/weak-beam rule ($\Sigma M_{Rc}<1.3\Sigma M_{Rb}$), should have full confining reinforcement at the end regions of all stories, not just at the (building) base;
  - DC H strong columns ($\Sigma M_{Rc}>1.3\Sigma M_{Rb}$) are also provided w/ confining reinforcement for μ_φ corresponding to 2/3 of q_o at the end regions of every story.
- **Beams:**
  - Max. mechanical ratio of tension steel:

$$\omega \leq \omega' + 0.0018/\mu_\varphi\,\varepsilon_{yd}$$

EN 1998-1-2:202X

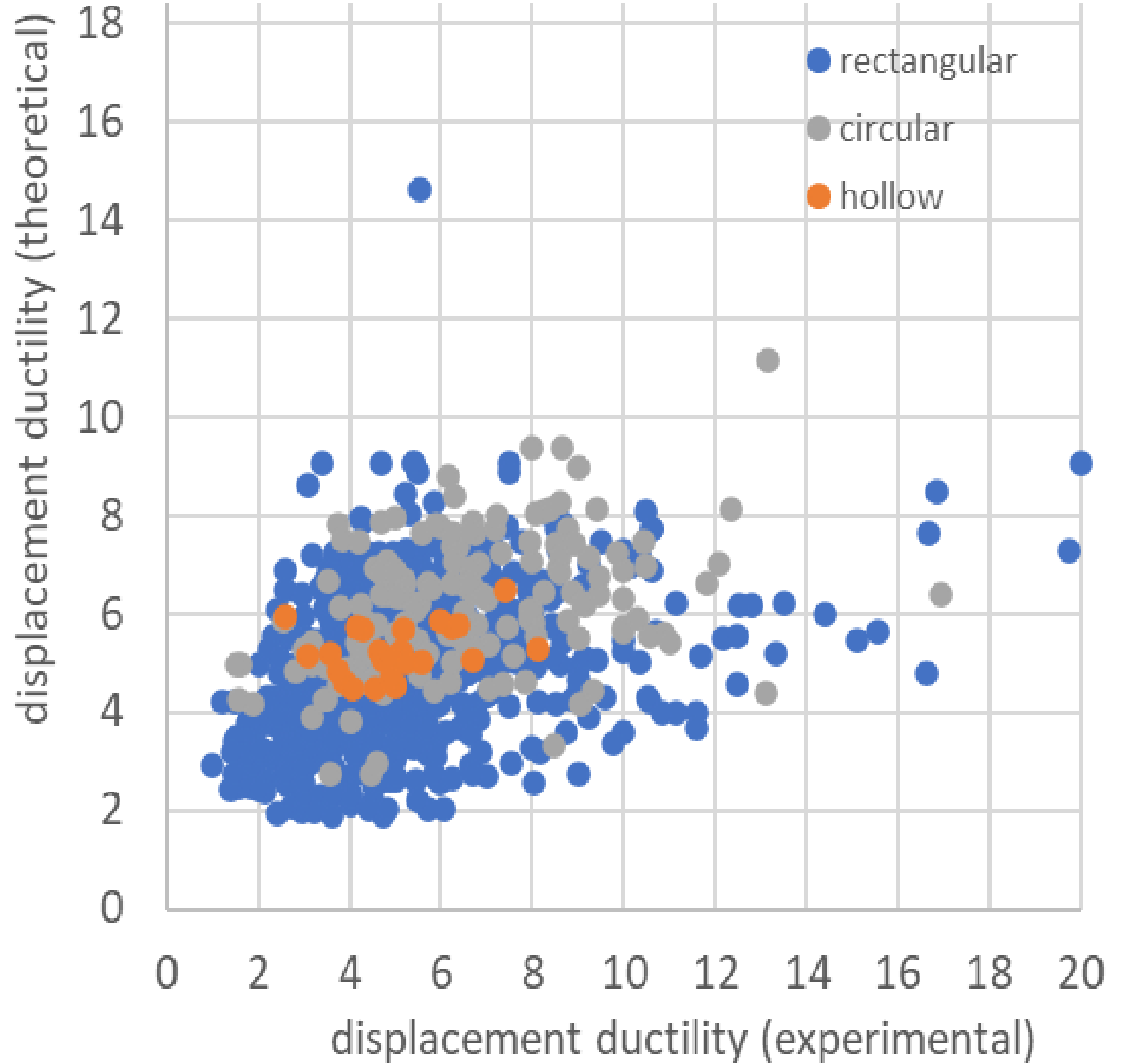
In force-based approach: Verification of plastic hinges for the chord-rotation ductility factor μ_θ derived from q-factor

Ratio of:

- ultimate chord rotation (plastic chord-rotation capacity plus chord-rotation at yielding) to
- chord-rotation at yielding

(both computed according to EN1998-1-1:202X)

should exceed product of the system-redundancy-dependent and ductility-dependent parts of behavior factor used in design.



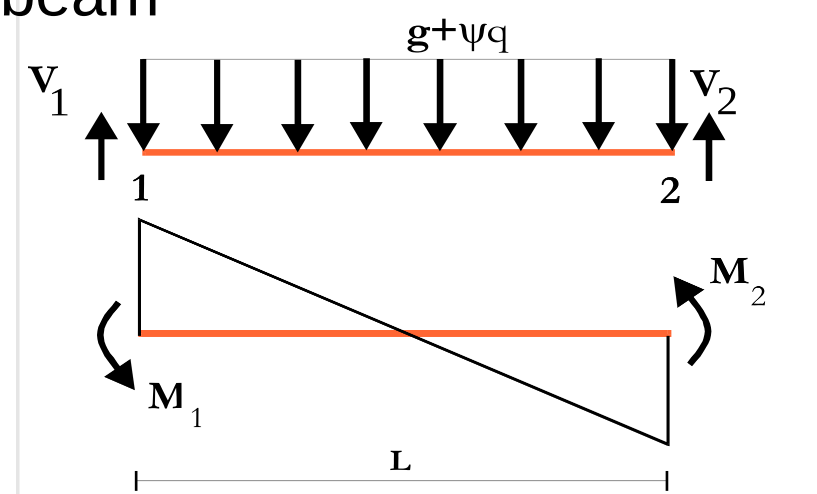
Capacity design of RC members, against pre-emptive shear failure

Beams

- Equilibrium of forces and moments on a beam

$$V_1 = V_{g+\psi q,1} + \frac{M_2 + M_1}{l_{cl}}$$

$$V_2 = V_{g+\psi q,2} - \frac{M_1 + M_2}{l_{cl}}$$

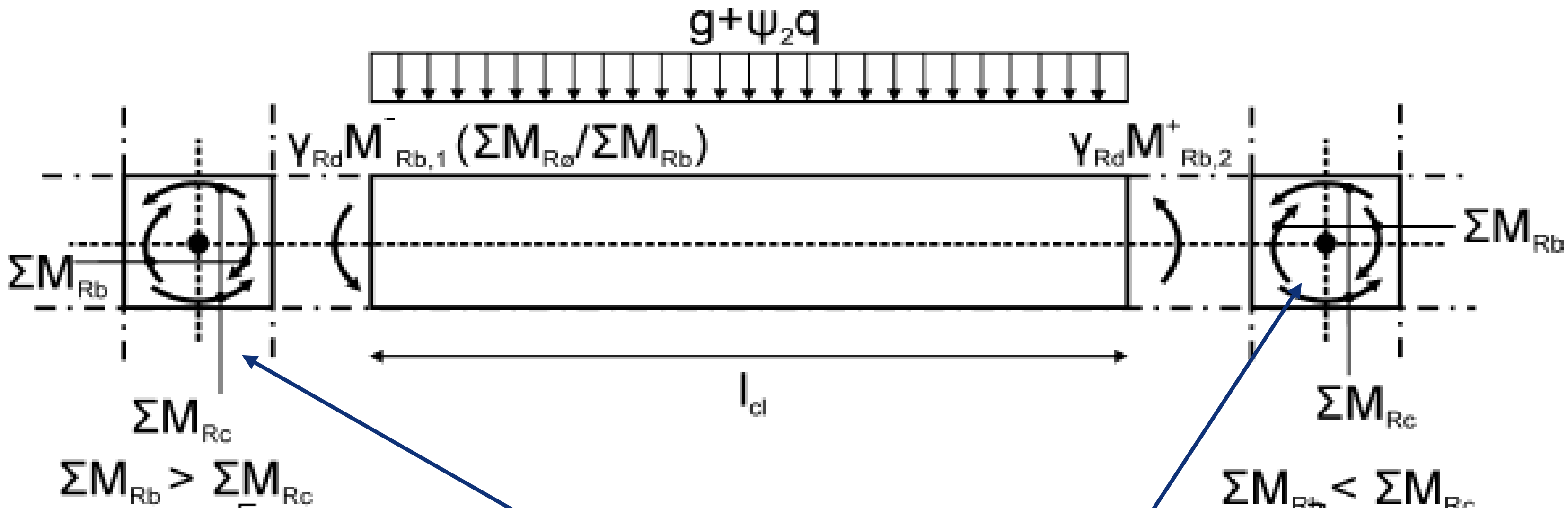


- Capacity-design shear in a beam weaker than the columns:

$$V_{CD,1} = V_{g+\psi q,1} + \gamma_{Rd} \frac{M^{-}_{Rd,b1} + M^{+}_{Rd,b2}}{l_{cl}}$$

$$V_{CD,2} = V_{g+\psi q,2} + \gamma_{Rd} \frac{M^{+}_{Rd,b1} + M^{-}_{Rd,b2}}{l_{cl}}$$

Capacity-design shear in beams (weak or strong)



$$\max V_{\mathrm{i,d}}(x)=\frac{\gamma_{\mathrm{Rd}}\left[M_{\mathrm{Rd,bi}}\;\min\left(1;\frac{\sum M_{\mathrm{Rd,c}}}{\sum M_{\mathrm{Rd,b}}}\right)_i+M_{\mathrm{Rd,bj}}^{+}\;\min\left(1;\frac{\sum M_{\mathrm{Rd,c}}}{\sum M_{\mathrm{Rd,b}}}\right)_j\right]}{l_{cl}}+V_{\mathrm{g}+\psi\mathrm{q,o}}(x)$$

- EN 1998-1:2004
  - in DC M $\gamma_{Rd}=1.0$,
  - in DC H $\gamma_{Rd}=1.2$
- EN 1998-1-2:202X
  - in DC 2 and 3: $\gamma_{Rd}=1.1$

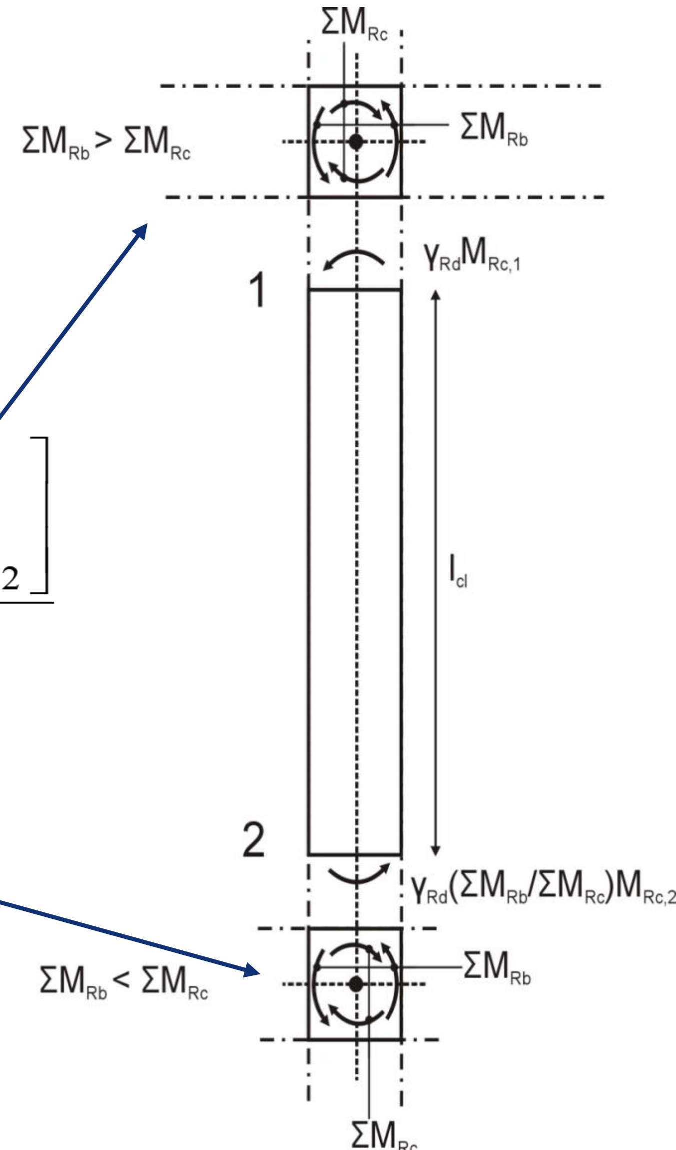
Columns

Capacity-design shear in column which is weaker than the beams:

$$V_{CD}^{+}=\gamma_{Rd}\frac{M_{Rd,c1}^{+}+M_{Rd,c2}^{-}}{h_{cl}} \qquad V_{CD}^{-}=\gamma_{Rd}\frac{M_{Rd,c1}^{-}+M_{Rd,c2}^{+}}{h_{cl}}$$

Capacity-design shear in (weak or strong) columns - Eurocode 8:

$$V_{CD,c}=\frac{\gamma_{Rd}\left[M_{Rd,c1}\min\left(1;\frac{\sum M_{Rd,b}}{\sum M_{Rd,c}}\right)_1+M_{Rd,c2}\min\left(1;\frac{\sum M_{Rd,b}}{\sum M_{Rd,c}}\right)_2\right]}{h_{cl}}$$



- EN 1998-1:2004
  - in DC M $\gamma_{Rd}=1.1$
  - in DC H $\gamma_{Rd}=1.3$
- EN 1998-1-2:202X
  - in DC 2 and 3: $\gamma_{Rd}=1.1$

Ductile Walls – EN1998-1:2004

Over-design in shear, by multiplying shear forces from linear analysis for design seismic action, V'_{Ed}, by factor ε, accounting for overstrength of plastic hinge at the base and higher modes after plastic hinging there:

DC M walls:

$$\varepsilon = \frac{V_{Ed}}{V'_{Ed}} = 1.5$$

DC H squat walls ($h_w/l_w \leq 2$):

$$\varepsilon = \frac{V_{Ed}}{V'_{Ed}} = \gamma_{Rd}\left(\frac{M_{Rdo}}{M_{Edo}}\right) \leq q$$

Over-design for flexural overstrength of the base w.r.to analysis

M_{Edo}: design moment at base section (from analysis),

M_{Rdo}: design flexural resistance at the base section,

γ_{Rd}=1.2

$$\varepsilon = \frac{V_{Ed}}{V'_{Ed}} = \sqrt{\left(\gamma_{Rd}\frac{M_{Rdo}}{M_{Edo}}\right)^2 + 0.1\left(q\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \leq q$$

DC H slender walls ($h_w/l_w > 2$):

Over-design for flexural overstrength of the base w.r.to analysis & for increased inelastic shears

$S_e(T)$: ordinate of elastic response spectrum

T_C: upper limit period of constant spectral acceleration region

T_1: period of mode with the largest participating mass in direction of V_{Ed}

Ductile Walls – EN1998-1-2:202X

Design shear forces: $V_{\mathrm{Edw}}(z) = \varepsilon(z)\, V'_{\mathrm{Edw,1}}(z) \leq q\, V'_{\mathrm{Edw}}(z)$

$V'_{\mathrm{Edw}}(z)$: from the combination of shears in all modes from the analysis;

$V'_{\mathrm{Edw,1}}(z)$: shear in mode with largest participating mass in direction of V_{Ed}

DC 2 walls: $\boldsymbol{\varepsilon(z) = q}$

DC 3 walls:

$$\boldsymbol{\varepsilon(z)} = \sqrt{\left(\gamma_{\mathrm{Rd}} \frac{M_{Rdo}}{M_{Edo}}\right)^2 + m(z)\left(q\,\frac{S_e(T_C)}{S_e(T_1)}\right)^2}$$

M_{Edo}: design moment at base section (from the analysis),

M_{Rdo}: design flexural resistance at base section,

γ_{Rd}=1.2

$m(z) =$ 0.1 in lower-third of wall height;

= 0.05 in middle-third;

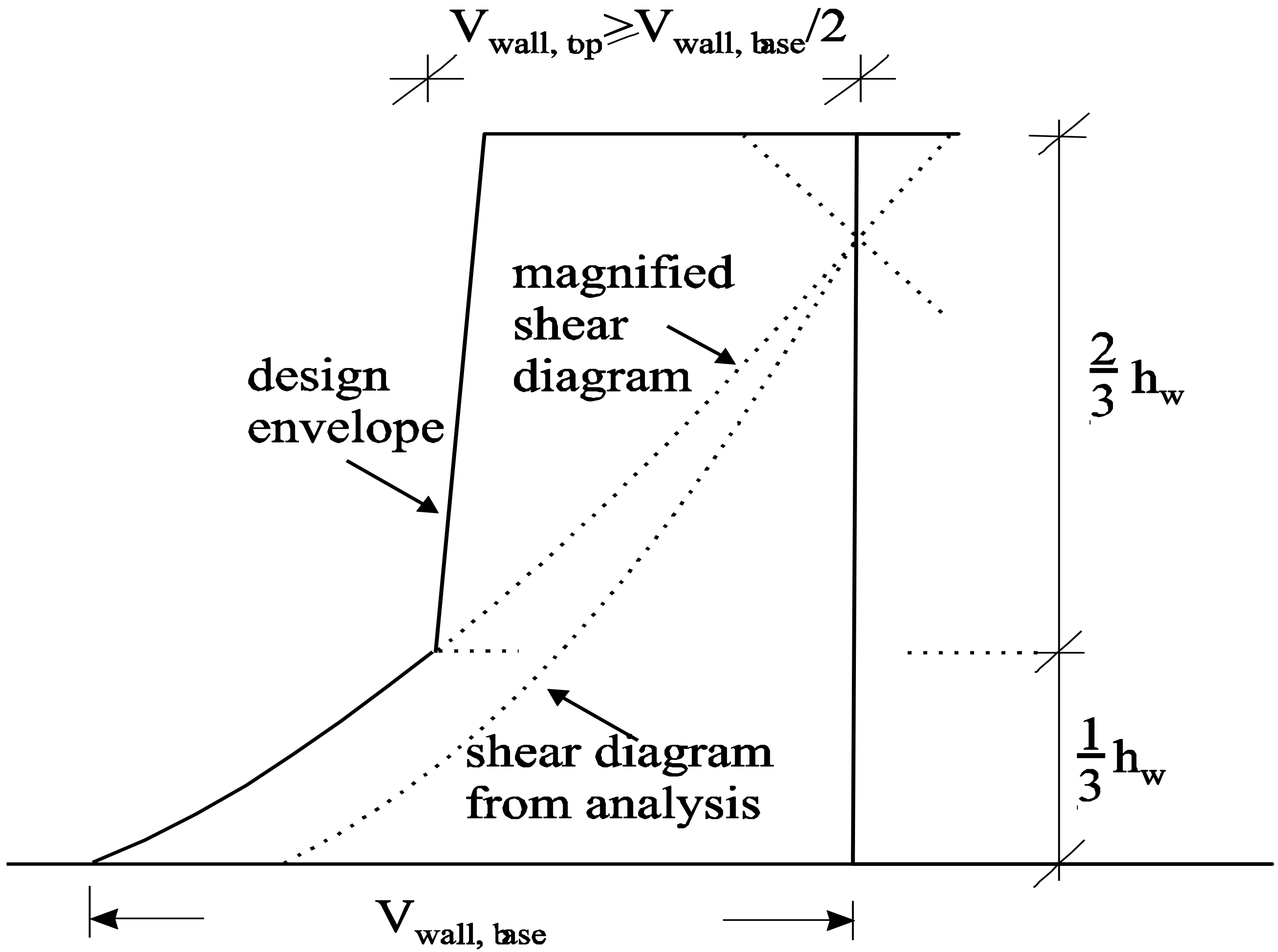
= 0.25 in upper-third.

$S_e(T)$: ordinate of elastic response spectrum

T_C: upper limit period of constant spectral acceleration region

T_1: period of mode with the largest participating mass in direction of V_{Ed}

Design shears in “dual” structural systems



To account for increase in the upper story shears due to higher mode inelastic response (after plastic hinging at the base)

Types of walls:

Ductile walls for moderate or high seismicity

Large, lightly reinforced walls for low or moderate seismicity

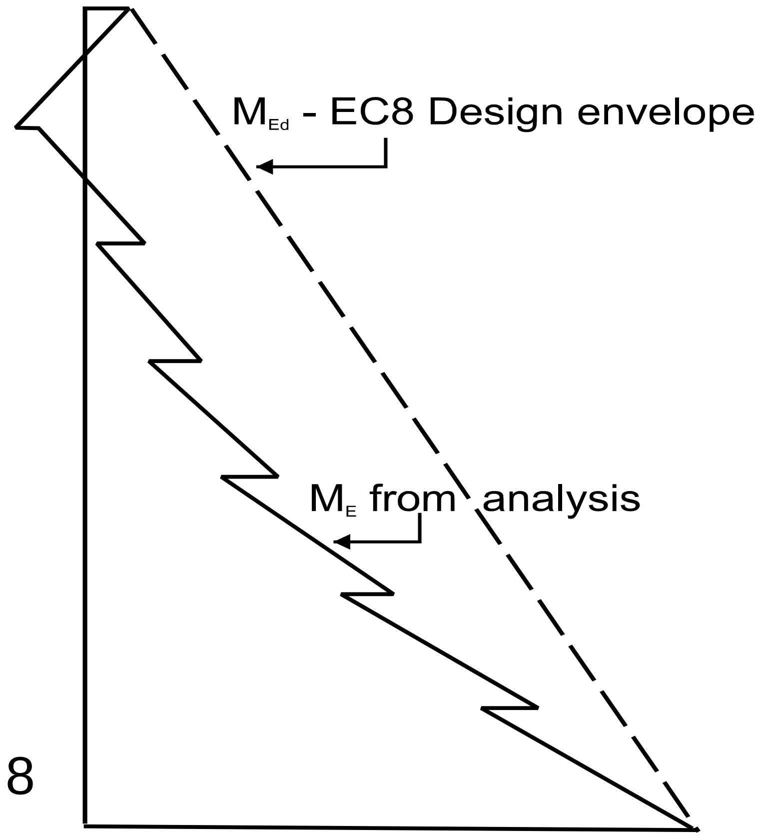
Two types of dissipative RC walls

- **Ductile walls:**
  - Fixed at the base, to prevent rotation there with respect to rest of structural system.
  - Designed & detailed to dissipate energy only in flexural plastic hinge just above the base.
- **Large lightly-reinforced walls- only in DCM or DCL, DC1 or DC2**
  - Walls with horizontal dimension $l_w \geq$ 4m, expected to develop limited cracking or inelastic behavior during design seismic action, but to transform seismic energy to potential energy (uplift of masses) & to energy radiated back into the soil by rigid-body rocking, etc.
  - Large X-sectional length, lack-of-fixity at the base or connection with transverse walls prevent plastic hinging at the base of such walls and hence energy dissipation in plastic hinges.

Ductile walls: Overdesign in bending

Strong column/weak beam capacity design is not required in wall or wall-equivalent dual systems (i.e. in those where walls resist >50% of seismic base shear)

But:
all ductile walls are designed in flexure,
to ensure that plastic hinge
develops only at the base:



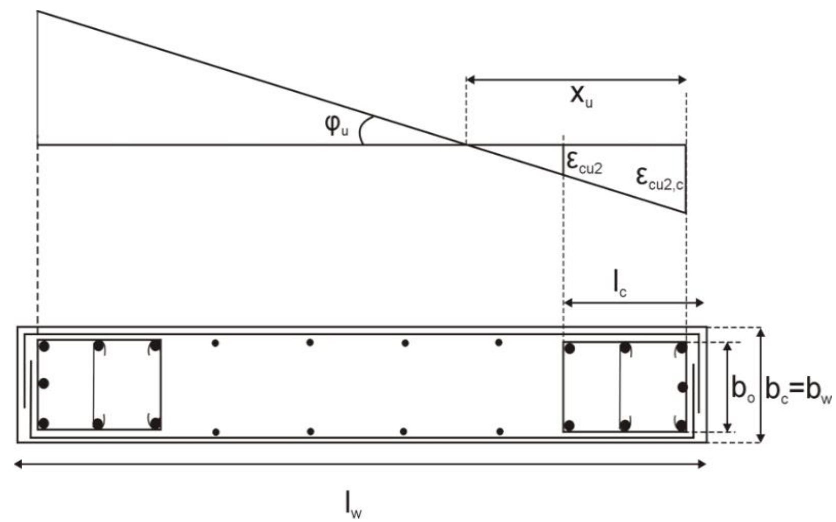
Typical moment diagram in a concrete wall
from the analysis & linear envelope for its
(over-)design in flexure according Eurocode 8

Ductile walls: Design in bending & shear - detailing

- Inelastic action limited to a plastic hinge at the base, so that the cantilever relation between q & μ_φ applies:
  - Wall is provided with flexural overstrength above plastic hinge region (linear moment envelope with shift rule);
  - Design in shear for V from analysis, times:

  1.5 for DC M

  $[(1.2\ M_{Rd}/M_{Ed})^2+0.1(qS_e(T_c)/S_e(T_1))^2]^{1/2} < q$ for DC H



- In plastic hinge zone: boundary elements w/ confining reinforcement having effective mechanical volumetric ratio:

$$\alpha\omega_{wd}=30\mu_\varphi(v_{d+}\omega_{\square})\varepsilon_{yd}b_c/b_o-0.035$$

over at least the part of the compression zone depth: $x_u=(v_d+\omega_v)l_w\varepsilon_{yd}b_c/b_o$

where the strain is between: $\varepsilon^*_{cu}=0.0035+0.1\alpha\omega_w$ & $\varepsilon_{cu}=0.0035$

Examples of large walls

Large lightly reinforced concrete walls

- Wall system classified as one of large lightly reinforced walls if, in horizontal direction of interest:
  - At least 2 walls with l_w>4 m, supporting together >20% of gravity load above (: sufficient no. of walls / floor area & significant uplift of masses); if one wall: q=2
  - Fund. period T_1<0.5s for fixity at the base against rotation (: low wall aspect ratio)
- Systems of large lightly reinforced walls:
  - q=3;
  - special (less demanding) dimensioning & detailing.
- Rationale: For large walls, minimum reinforcement of ductile walls implies:
  - very high cost;
  - flexural overstrength that cannot be transmitted to ground.

  On the other hand, large lightly reinforced walls:
  - preclude (collapse due to) story mechanism,
  - minimize nonstructural damage,
  - have shown satisfactory performance in strong EQs.
- If structural system does not qualify as one of large lightly reinforced walls, all its walls designed & detailed as ductile walls.

Design/detailing of large lightly reinforced walls -EC8

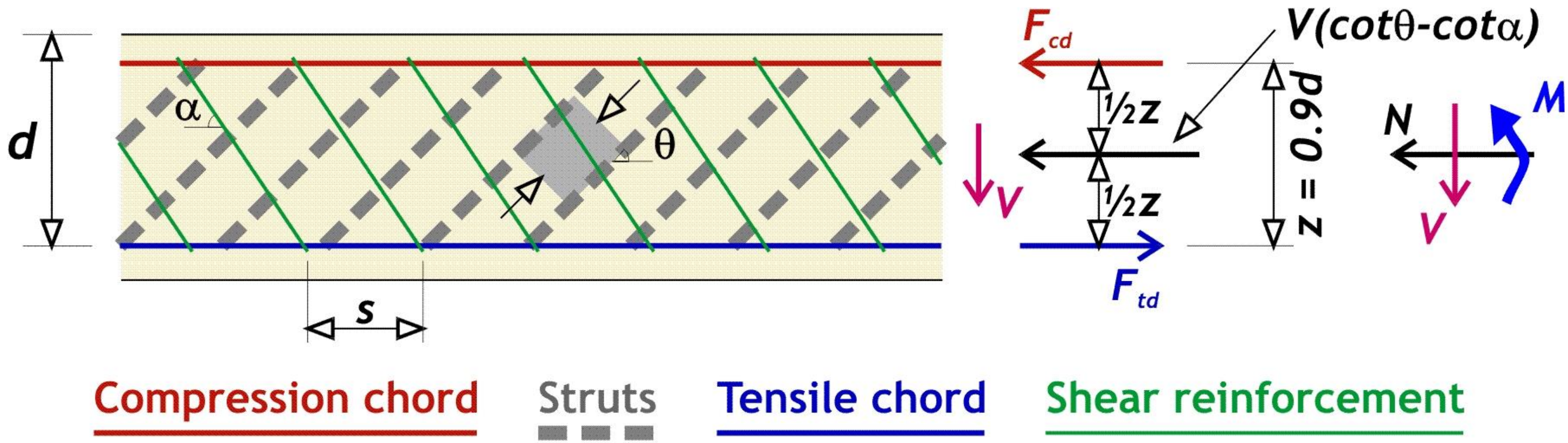
- Vertical steel tailored to demands due to *M* & *N* from analysis
  - Little excess (minimum) reinforcement, in order to minimize flexural overstrength.
- Shear verification for *V* from analysis times $(1+q)/2 \sim 2$:
  - If so-amplified shear demand is less than (design) shear resistance without shear reinforcement:

    No (minimum) horizontal reinforcement. Reason:
    - Inclined cracking prevented (horizontal cracking & yielding due to flexure mainly at construction joints);
    - If inclined cracking occurs, crack width limited by deformation-controlled nature of response (vs. force-controlled non-seismic actions covered in EC2), even without min horizontal steel.

New approach for cyclic shear resistance of prismatic members based on MCFT concepts and consistent with new approach for monotonic shear resistance in Eurocode 2

EN1992-1-1:2004 –
Shear resistance of members with shear reinforcement

Variable strut inclination model: $1<\cot\theta<2.5$, $22°<\theta<45°$



If $\mathbf{V_{Ed} > V_{Rdc,min}}$

$\mathbf{V_{Ed} \leq V_{Rd,s} = \rho_w b_w z f_{ywd}(\cot\theta+\cot\alpha)\sin\alpha}$,

$\mathbf{V_{Ed} \leq V_{Rd,max} = b_w z \nu f_{cd}/(\cot\theta+\tan\theta)}$, $\nu=0.6\,(1-f_{ck}/250)$

$\mathbf{V_{Rdc,min} = [C_{Rd,c} k (100\rho_l f_{ck})^{1/3} + k_1 \sigma_{cp}] b_w d}$, $k=1+\sqrt{(200/d(mm))}\leq 2$,

$\rho_l = A_{sl}/b_w d$: tension steel ratio, $\sigma_{cp}=N_{Ed}/A_c$ mean axial stress in section,

Recommended values: $C_{Rd,c}=0.18/\gamma_C$, $k_1=0.15$

EN1998-1:2004 – Members with shear reinforcement

		DCL	DCM	DCH
All members	$V_{Rd,c,min}$	$(C_{Rd,ck}(100\rho_l f_{ck})^{1/3}+k_1\sigma_{cp})b_w d$		
Frame beam	$V_{Rd,c}$	0		
	$V_{Rd,s}$	$\rho_w b_w z f_{ywd}\cot\theta$		$\rho_w b_w z f_{ywd}$
	$V_{Rd,max}$	$0.3(1- f_{ck}/250)\ b_w z f_{cd}\ \sin 2\theta$		$0.3(1- f_{ck}/250)\ b_w z f_{cd}$
Frame column	$V_{Rd,c}$	0		
	$V_{Rd,s}$	$\rho_w b_w z f_{ywd}\cot\theta$		
	$V_{Rd,max}$	$0.3(1- f_{ck}/250)\ b_w z f_{cd}\ \sin 2\theta$		
Wall other than below	$V_{Rd,c}$	0		
	$V_{Rd,s}$	$\rho_w b_w z f_{ywd}\cot\theta$		
	$V_{Rd,max}$	$0.3(1- f_{ck}/250)\ b_w z f_{cd}\ \sin 2\theta$		$0.12(1- f_{ck}/250)\ b_w z f_{cd}\ \sin 2\theta$
Wall w/ $L_s/h<2$	$V_{Rd,c}$	0		$V_{Rd,c,min}$
	$V_{Rd,s}$	$\rho_w b_w z f_{ywd}\cot\theta$		$\rho_w b_w z f_{ywd}$
	$V_{Rd,max}$	$0.3(1- f_{ck}/250)\ b_w z f_{cd}\ \sin 2\theta$		$0.12(1- f_{ck}/250)\ b_w z f_{cd}\ \sin 2\theta$

$$\textbf{DCL, DCM:}\ 1 \le \cot\theta = \sqrt{(2/3 - f_{ck}/225) f_{cd}/\rho_w f_{ywd} - 1} \le 2.5$$

$$\textbf{DCH walls:}\ 1 \le \cot\theta = \sqrt{0.4\,(2/3 - f_{ck}/225) f_{cd}/\rho_w f_{ywd} - 1} \le 2.5$$

$$\textbf{DCH walls } \mathbf{1.3<L_s/h<2:}\ 1 \le \cot\theta = \frac{1 \pm \sqrt{1-\alpha^2}}{\alpha} \le 2.5\,, \qquad \alpha = \frac{\rho_w f_{ywd}/1.2 + \tau_{Rdc,min}}{\left(0.12 - \frac{f_{ck}}{1250}\right) f_{cd}}$$

EN1992-1-1:202X –
Shear resistance of members with shear reinforcement

Strain-dependent variable strut inclination model: $1<\cot\theta<2.5$, $22°<\theta<45°$

$$V_{Ed} \leq V_{Rd,max} = b_w z \nu f_{cd}/(\cot\theta+\tan\theta),\ \boldsymbol{\nu} = \frac{\mathbf{1}}{\mathbf{1+110\epsilon_1}} \leq \mathbf{1.0}$$

$\epsilon_{\mathbf{1}} = \epsilon_x + (\epsilon_x + 0.001)\cot^2\theta$: tensile strain @ right angles to struts,

$\boldsymbol{\epsilon_x} = \frac{\epsilon_{x,t} + \epsilon_{x,c}}{2}$: (elastic) longitudinal strain at (cracked) section mid-depth

$$\boldsymbol{\epsilon_{x,t}} = \frac{1}{A_{s1}E_s}\left(\frac{M}{z} - \frac{N}{2} + \frac{V}{2}\cot\theta\right) \leq \frac{1}{A_{s1}E_s}\left(\frac{maxM}{z} - \frac{N}{2}\right)$$

$$\boldsymbol{\epsilon_{x,c}} = \frac{1}{A_{cc}E_c}\left(\frac{M}{z} + \frac{N}{2} - \frac{V}{2}\cot\theta\right) \leq \frac{1}{A_{cc}E_c}\left(\frac{maxM}{z} + \frac{N}{2}\right)$$

- Shear resistance to concentrated load at short distance L_s to support (angle of compr. stress field $\theta<\beta=\mathrm{atan}(h/L_s)$):

$$V_{Rd} = (\rho_w b_w z \sigma_{sw})\cot\beta + \left(k_\varepsilon \eta_{fc} f_{cd}\right)(b_w z)\ \sin^2\theta\,(\cot\theta - \cot\beta) \leq V_{Rd,max}$$

$$\sigma_{sw} = E_s[(\epsilon_x + 0.001)\cot^2\theta - 0.001] \leq f_{ywd}$$

EN1998-1-2:202X –
Shear resistance of members with shear reinforcement

$$\boldsymbol{\nu} = \frac{\mathbf{1}}{\mathbf{1.6(1+110\epsilon_1)}} \leq \mathbf{1.0}$$

Estimation of inelastic longitudinal strain at section mid-depth:

- From fictitious elastic moment from linear analysis with elastic spectrum and q=1, carried out to estimate inelastic displacements deformations:

$$\boldsymbol{\epsilon_{x,t}} = \frac{1}{A_{s1}E_s}\left(\frac{M_{\mathrm{el}}}{z} - \frac{N}{2} + \frac{V}{2}\cot\theta\right), \qquad \boldsymbol{\epsilon_{x,c}} = \frac{1}{A_{\boldsymbol{cc}}E_{\boldsymbol{c}}}\left(\frac{M_{\mathrm{el}}}{z} + \frac{N}{2} - \frac{V}{2}\cot\theta\right)$$

- From inelastic moment and chord-rotation ductility factor obtained from nonlinear analysis carried out to find inelastic internal forces/deformations:

$$\boldsymbol{\epsilon_{x,t}} = \frac{1}{A_{s1}E_s}\left(\mu_\theta\frac{M_{i\mathrm{nel}}}{z} - \frac{N}{2} + \frac{V}{2}\cot\theta\right) \quad \boldsymbol{\epsilon_{x,c}} = \frac{1}{A_{\boldsymbol{cc}}E_{\boldsymbol{c}}}\left(\mu_\theta\frac{M_{\mathrm{inel}}}{z} + \frac{N}{2} - \frac{V}{2}\cot\theta\right)$$

- From moment and behavior factor q from linear analysis with design spectrum (elastic divided by q) carried out to compute inelastic internal forces

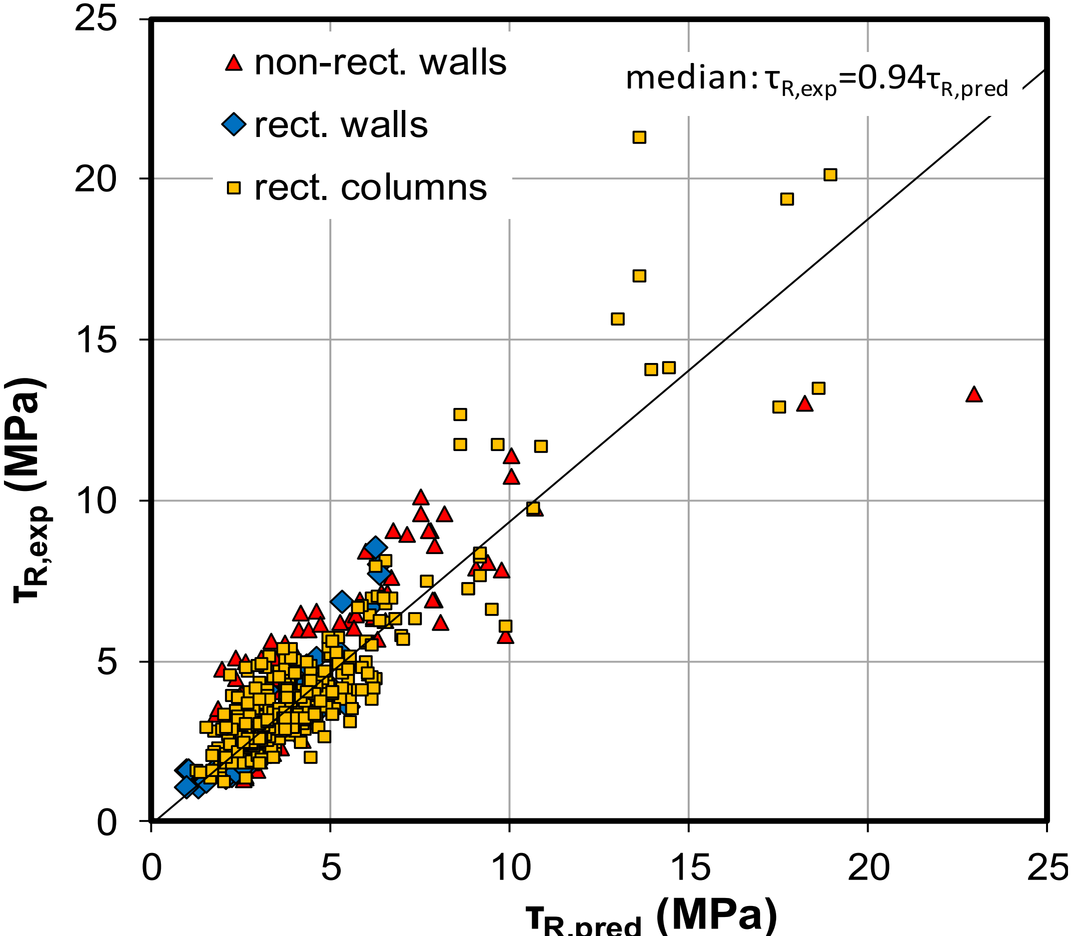
$$\boldsymbol{\epsilon_{x,t}} = \frac{1}{A_{s1}E_s}\left(q\frac{M_{\mathrm{el}}}{z} - \frac{N}{2} + \frac{V}{2}\cot\theta\right), \qquad \boldsymbol{\epsilon_{x,c}} = \frac{1}{A_{\boldsymbol{cc}}E_{\boldsymbol{c}}}\left(q\frac{M_{\mathrm{el}}}{z} + \frac{N}{2} - \frac{V}{2}\cot\theta\right)$$

EN1998-1-2:202X Cyclic shear resistance (*cont'ed*)

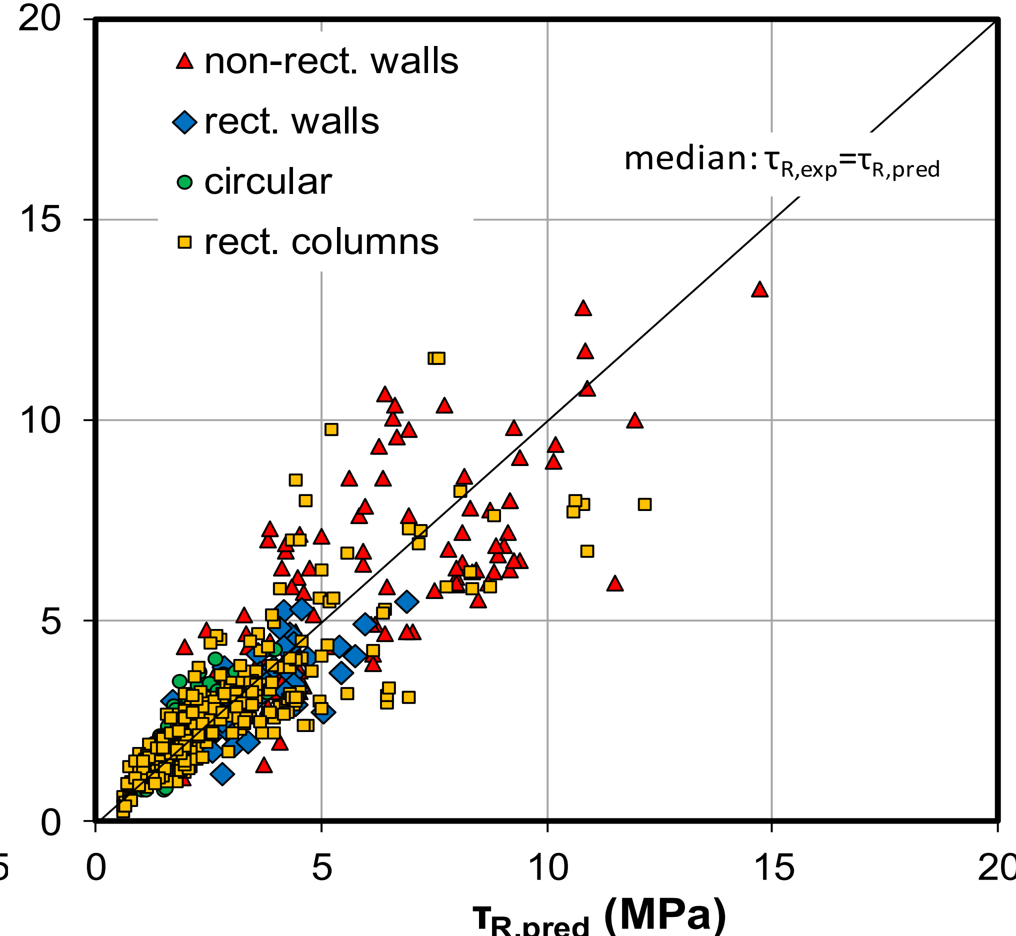
- Squat members (angle of compression field θ<β=atan(h/L_s))

$$V_{Rd} = (\rho_w b_w z \sigma_{sw}) \cot\beta + \left(k_\varepsilon \eta_{fc} f_{cd}\right)(b_w z)\ \sin^2\theta\,(\cot\theta - \cot\beta) \le V_{Rd,max}$$

$$\sigma_{sw} = E_s[(\epsilon_x + 0.001)\cot^2\theta - 0.001] \le f_{ywd}$$



521 cyclic tests with shear failure without yielding in flexure (cov 28%)



571 cyclic tests with shear failure after yielding in flexure (cov 29%)

New approach for cyclic shear resistance of beam-column joints based on MCFT concepts and consistent with new approach for prismatic members in Eurocode 2

Compression Field-based approach

$$V_{Rh,d} = max\left(V_{jh,cr}; V_{Rhd,min} + V_{Rhd,CF}\right)$$

- Interior joints $$V_{Rhd,min} = 1.2 f_{ctd} b_j \sqrt{h_c h_b}$$
- Exterior joints $$V_{Rhd,min} = 0.5 f_{ctd} b_j \sqrt{h_c h_b}$$
- If: (compression positive)

$$1 + \frac{N_h}{b_c h_b f_{ctd}} > \frac{N_v}{b_c h_c f_{cd}}, \quad 1 + \frac{N_v}{b_c h_c f_{ctd}} > \frac{N_h}{b_c h_b f_{cd}}$$

- If: $$V_{jh,cr} = \frac{f_{ctd} f_{cd}}{f_{ctd} + f_{cd}} \sqrt{\left(1 + \frac{N_h}{b_c h_b f_{ctd}} - \frac{N_v}{b_c h_c f_{cd}}\right)\left(1 + \frac{N_v}{b_c h_c f_{ctd}} - \frac{N_h}{b_c h_b f_{cd}}\right)} b_j h_c$$

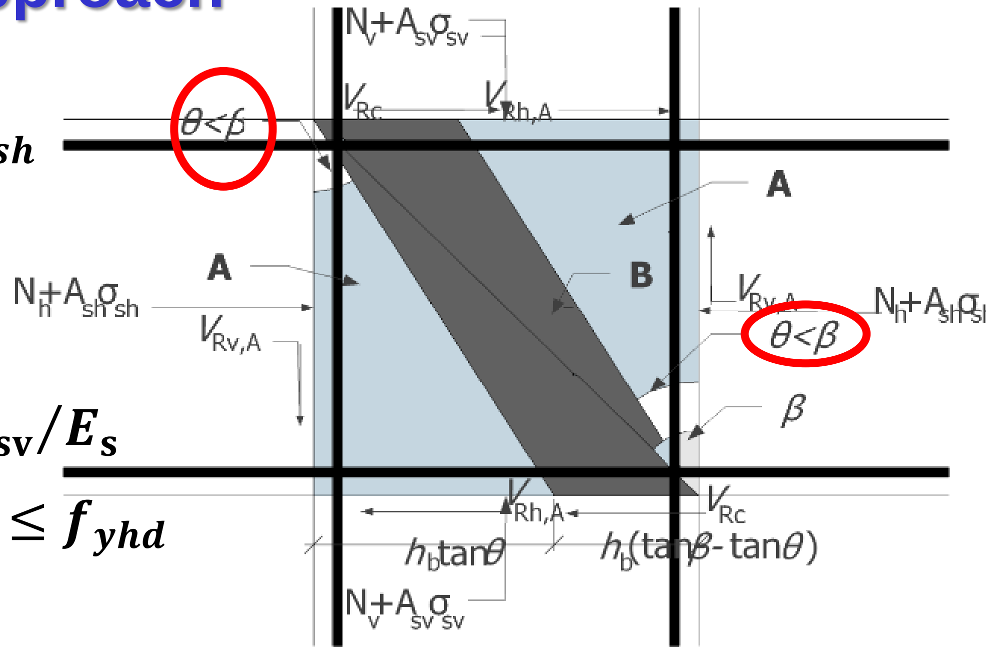
$$\frac{N_h}{b_c h_b f_{cd}} > 1 + \frac{N_v}{b_c h_c f_{ctd}} > 0, \quad 1 + \frac{N_h}{b_c h_b f_{ctd}} > 0$$

- If: $$V_{jh,cr} = f_{ctd} \sqrt{\left(1 + \frac{N_h}{b_c h_b f_{ctd}}\right)\left(1 + \frac{N_v}{b_c h_c f_{ctd}}\right)} b_j h_c$$

$0 > 1 + \frac{N_v}{b_c h_c f_{ctd}}$ or $1 + \frac{N_h}{b_c h_b f_{ctd}} < 0$ $\quad V_{jh,cr} = 0$

Compression Field approach

$$V_{Rhd,CF} = \min\left(\frac{\max(N_v + A_{sv}\sigma_{sv};0)}{\cot\theta}; N_h + A_{sh}\sigma_{sh} + (\cot\theta - \cot\beta)\sin^2\theta\, v_j f_{cd} h_c b_j\right)$$



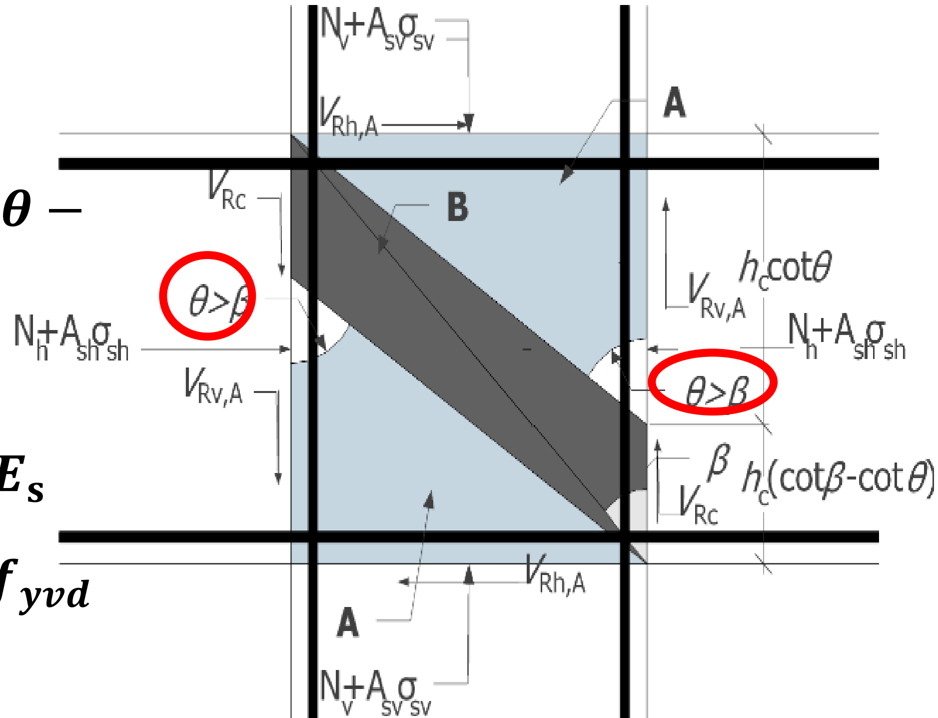
$$\varepsilon_1 = \varepsilon_v + (\varepsilon_v + 0.001)\cot^2\theta > 0 \qquad \varepsilon_v = \sigma_{sv}/E_s$$

$$\sigma_{sh} = E_s\left[(\varepsilon_v + 0.001)\cot^2\theta - 0.001\right] \le f_{yhd}$$

$$\sigma_{sv} = \left[V_{Rhd,CF}\cot\theta - N_v\right]/A_{sv} \le f_{yvd}$$

$$\boxed{v_j = \frac{1}{1.4(1 + 100\epsilon_1)}}$$

$$V_{Rhd,CF} = \min\left(\frac{\cot\theta}{\cot\beta}(N_h + A_{sh}\sigma_{sh}); \frac{\max(N_v+A_{sv}\sigma_{sv};0)}{\cot\beta} + (\tan\theta - \tan\beta)\cos^2\theta\, v_j f_{cd} h_c b_j\right)$$

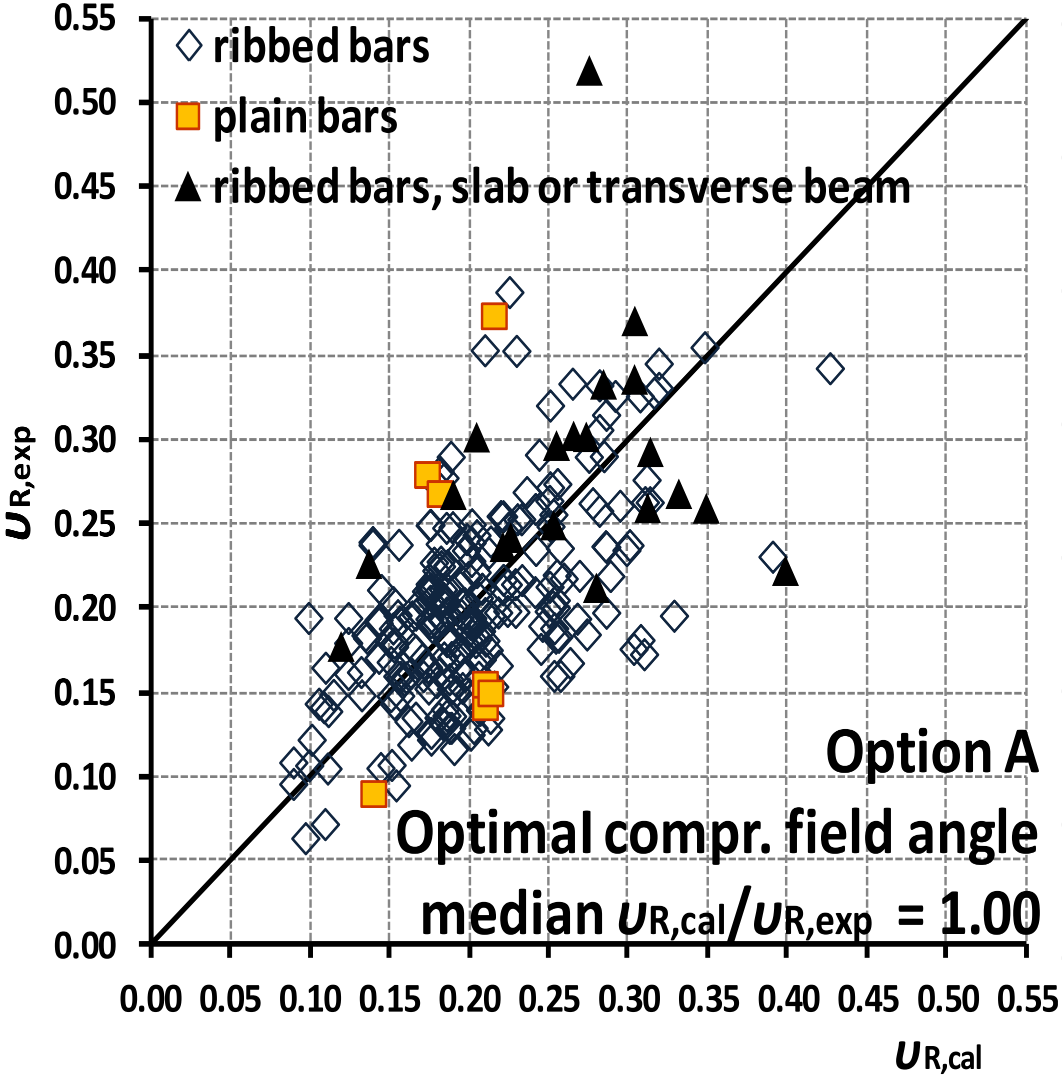


$$\varepsilon_1 = \varepsilon_h + (\varepsilon_h + 0.001)\tan^2\theta > 0 \qquad \varepsilon_h = \sigma_{sh}/E_s$$

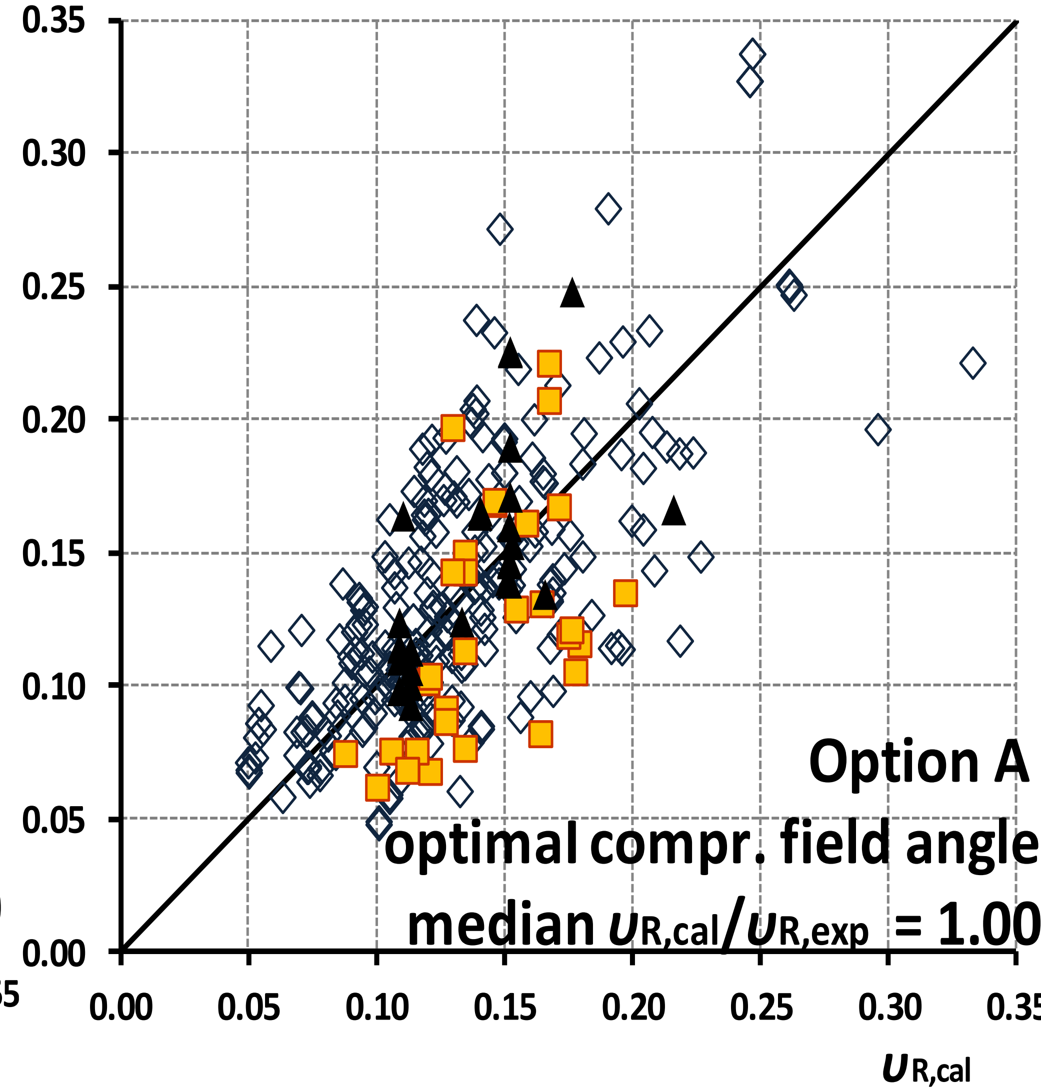
$$\sigma_{sv} = E_s\left[(\varepsilon_h + 0.001)\tan^2\theta - 0.001\right] \le f_{yvd}$$

$$\sigma_{sh} = \left[V_{Rhd,CF}\tan\theta - N_h\right]/A_{sh} \le f_{yhd}$$

263 cyclic tests of interior joints with shear failure (cov 26%)



300 cyclic tests of exterior joints with shear failure (cov 28%)



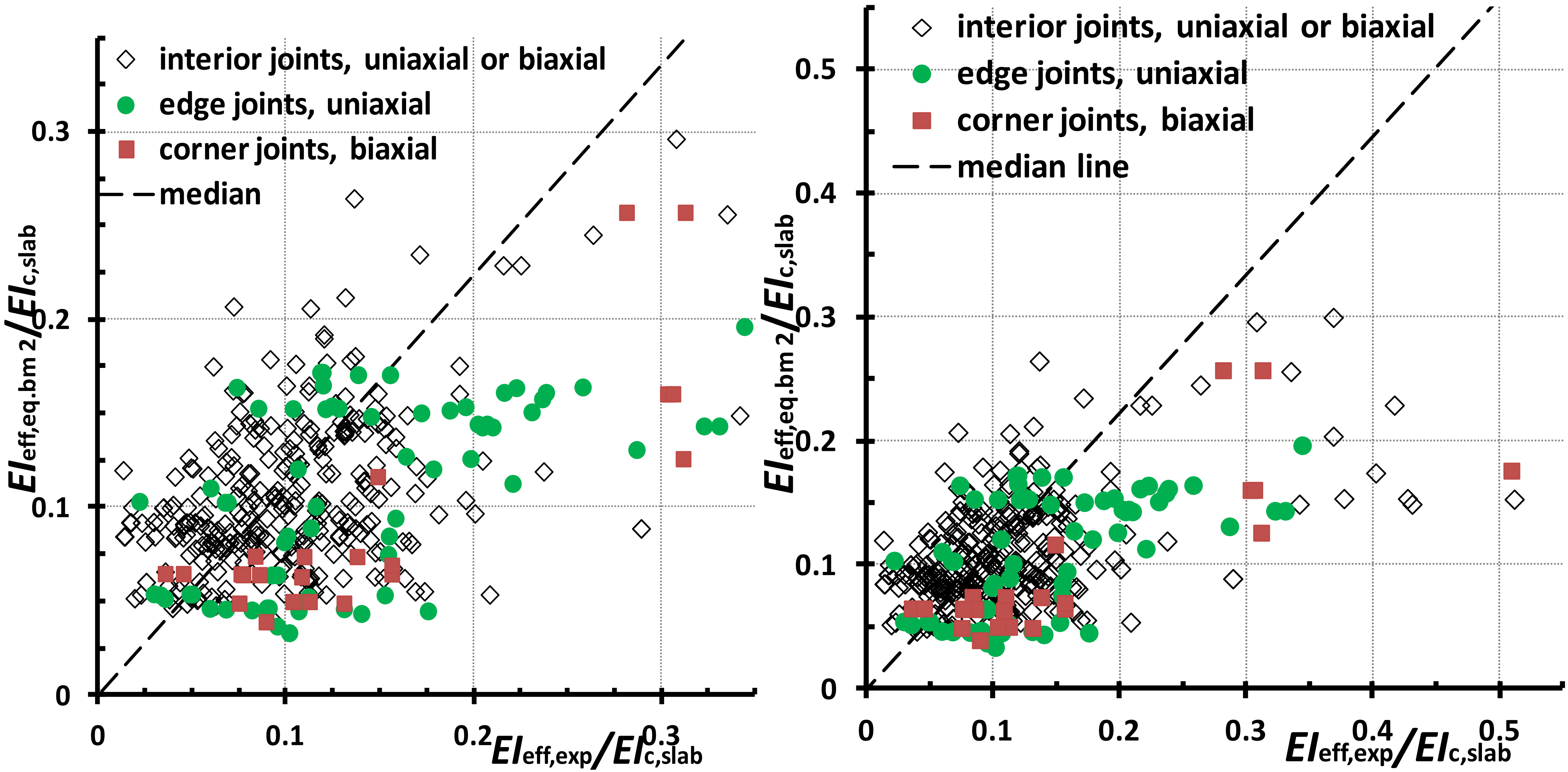
Proposal for design of flat slabs as “primary” seismic elements

- Modelling of slab stiffness in analysis.
- Capacity-design type of calculation of unbalanced moment transferred from a column to the slab.
- Resistance of slabs to cyclic punching shear.

“Equivalent beam”

Prismatic member connecting two adjacent columns in plane of bending with:

- theoretical span the axial distance between the columns’ centroids, L_{slab};
- cross-sectional depth equal to the slab thickness,
- top and bottom reinforcement ratios at the end sections those of the support strip between the connected columns,
- concrete strength, cover to reinforcement, etc, those of the slab,
- **half-width** on each side of the axis between the columns centroids, equal to the smaller of:
  - the distance to the slab’s edge,
  - one-half the axial distance to the nearest parallel “equivalent beam”, and
  - **$0.18L_{slab}$ or $0.2L_{slab,clear}$ at all types of joints**.



interior joints, uniaxial or biaxial
edge joints, uniaxial
corner joints, biaxial
median
$EI_{eff,eq.bm\,2}/EI_{c,slab}$
$EI_{eff,exp}/EI_{c,slab}$
interior joints, uniaxial or biaxial
edge joints, uniaxial
corner joints, biaxial
median line
$EI_{eff,eq.bm\,2}/EI_{c,slab}$
$EI_{eff,exp}/EI_{c,slab}$

Ratio of slab's experimental secant-to-yield-stiffness to theoretical of equivalent beam

$$\frac{EI_{exp}}{\frac{M^{-}_{y,eq.b}}{\theta^{-}_{y,eq.b}}\frac{L_{slab}}{6}}$$

Secant-to-yield-stiffness ratio: test-to-equiv. beam w/ half-width 20% of clear span

Joint's position within plane of bending	**interior joint**		**edge joint**		**All**
Position normal to plane of bending	interior joint	edge joint	interior joint	corner joint	
From cyclic tests					
# tested joints	175	19	48	17	259
mean ratio	1.07	1.49	1.56	1.86	1.24
median ratio	0.88		1.44		1.01
CoV of ratio	0.57	0.38	0.53	0.32	0.59
From monotonic tests					
# tested joints	112	5	18	7	142
mean ratio	1.05	1.2	1.13	1.11	1.07
median ratio	0.99		0.93		1.02
CoV of ratio	0.53		0.45	0.41	0.52
All tested joints with deflection measurement					
# tested joints	287	24	66	24	401
mean ratio	1.05	1.51	1.44	1.72	1.18
median ratio	0.93	1.42	1.33	1.72	1.01
CoV of ratio	0.57	0.35	0.54	0.36	0.56

Capacity-design unbalanced moment from column to slab

Slab-column joints should be designed for the vertical shear from the analysis in the seismic design situation and an unbalanced column moment in each orthogonal direction of the column equal to the smaller of:

1. The sum of design values of moment resistances of the supporting column at the interfaces with the slab above and below its joint with the column;
2. The slab's flexural resistance at a support section normal to the plane of bending, computed:
   - either as 0.75-times the sum of design values of sagging and hogging moment resistances of the full width of the slab tributary to the joint (neglecting at edge joints the sagging moment resistance);
     or
   - at interior columns, as 1.3-times the sum of design sagging and hogging moment resistances of the support sections of "equivalent beams" framing into opposite sides of the column in the plane of bending,
   - at edge columns, the design value of hogging moment resistance of the support section of the equivalent beam connected to the column.

mean, median, 90%-fractile, CoV of ratio of experimental unbalanced column moment to computed yield moment or moment of resistance of full slab width or of the equivalent beam

Joint position in plane of bending		interior joint							edge joint							All
Position normal to plane of bending		interior joint			edge joint		F or NF	All	interior joint			corner joint		F or NF	All	
Failure mode		P	FP	F or NF	P	F or NF			P	FP	F or NF	P	F or NF			
sample size (tested joints):		206	55	91	18	28	119	398	96	13	41	65	26	67	245	639
Unbalanced-to-yield moment of full slab width:	mean ratio	0.51	0.72	0.84	0.59	0.97	0.87	0.65	0.61	0.58	0.74	0.58	0.51	0.65	0.62	0.64
	median ratio	0.51	0.75	0.69			0.72	0.59	0.56		0.68	0.5		0.61	0.55	0.58
	CoV of ratio	0.44	0.25	0.74	0.41	0.79	0.76	0.67	0.48	0.37	0.54	0.71	0.3	0.53	0.55	0.63
	90%-fractile	0.8	0.96	1.58			1.65	1.18	0.99		1.24	1.07		1.08	1.04	1.13
As above, for equivalent beam	mean ratio	0.9	1.2	1.29	0.7	1.0	1.22	1.03	0.63	0.77	0.61	0.64	0.55	0.59	0.63	0.88
	median ratio	0.81	1.15	1.16			1.16	0.93	0.58		0.48	0.64		0.5	0.57	0.79
	CoV of ratio	0.47	0.32	0.45	0.18	0.67	0.5	0.49	0.47	0.5	0.64	0.53	0.45	0.58	0.51	0.52
	90%-fractile	1.45	1.71	2.03			2.0	1.67	1.01		1.09	1.06		1.0	1.04	1.49
Unbalanced-moment/moment resistance-of-full-slab-width	mean ratio	0.4	0.54	0.53	0.35	0.77	0.59	0.45	0.43	0.44	0.54	0.42	0.41	0.49	0.44	0.46
	median ratio	0.38	0.55	0.51			0.53	0.45	0.41		0.5	0.38		0.47	0.43	0.44
	CoV of ratio	0.46	0.34	0.38	0.29	01	0.51	0.51	0.48	0.35	0.6	0.76	0.38	0.56	0.58	0.54
	90%-fractile	0.63	0.78	0.8			0.97	0.78	0.69		0.94	0.78		0.83	0.76	0.77
As above, for equivalent beam	mean ratio	0.75	0.94	0.95	0.56	0.62	0.87	0.81	0.49	0.56	0.5	0.49	0.48	0.49	0.5	0.69
	median ratio	0.7	0.87	0.92			0.85	0.78	0.46		0.45	0.48		0.44	0.47	0.65
	CoV of ratio	0.49	0.34	0.36	0.27	0.48	0.41	0.45	0.48	0.37	0.5	0.56	0.45	0.48	0.49	0.52
	90%-fractile	1.23	1.37	1.4			1.34	1.28	0.79		0.82	0.84		0.79	0.81	1.14

P: Punching failure; F: Flexural failure; FP: Punching/flexural failure; NF: Non-failure in the test

Proposed cyclic punching shear resistance – modification to Annex I of EN1992-1-1:202X

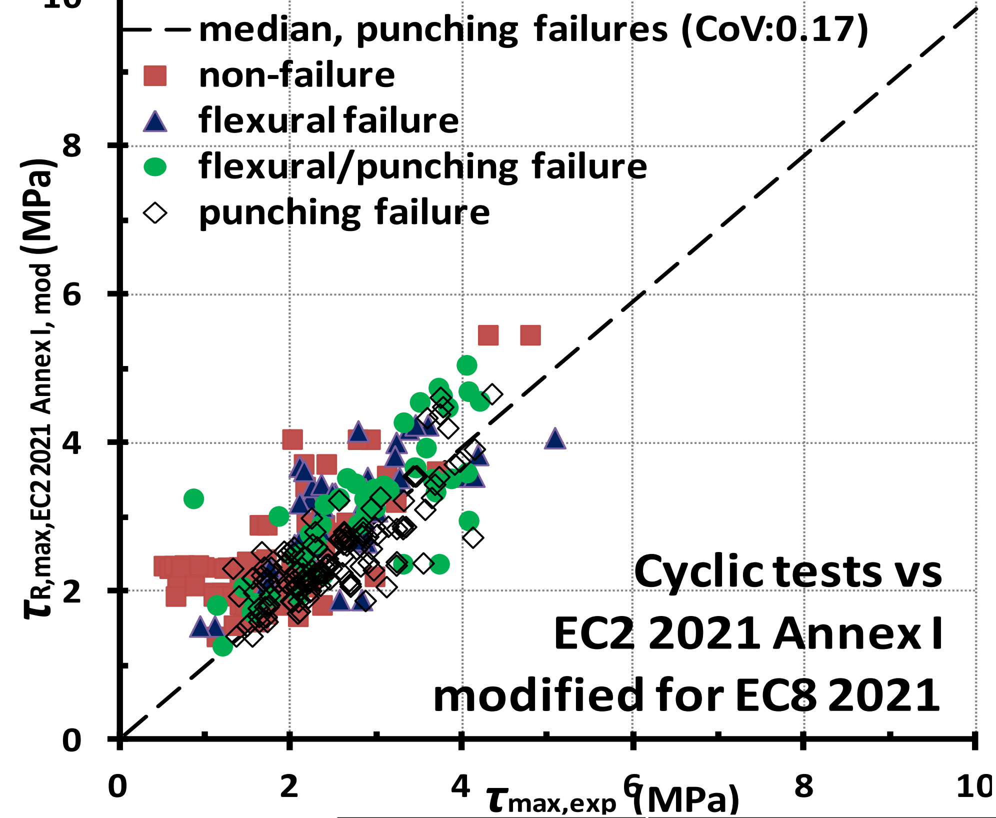
Shear resistance without shear reinforcement:

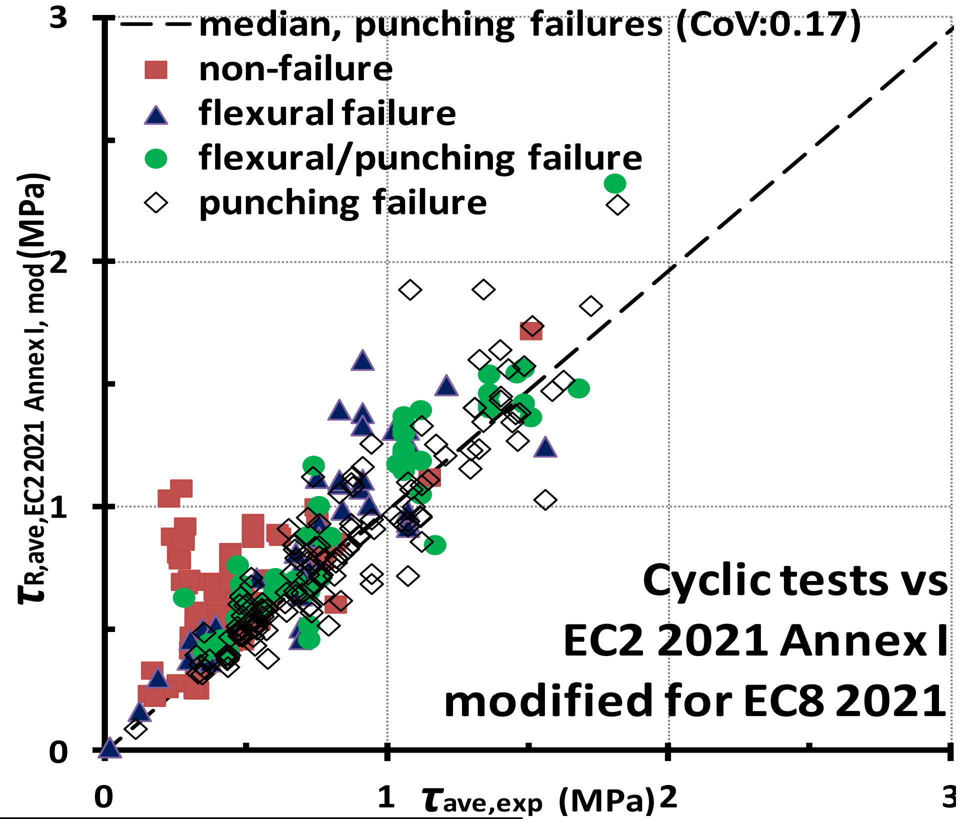
$$\tau_{\max} = \left(1 + 1{,}1\frac{\sqrt{e_{\mathrm{x}}^2 + e_{\mathrm{y}}^2}}{b_b}\right)\frac{\mathrm{V}}{b_{0,5}d_{\mathrm{v}}} < \tau_{\mathrm{Rd,c}}(MPa) = 0.4(100\rho_1)^{1/3}\sqrt{f_{\mathrm{ck}}(MPa)}$$

Shear resistance with shear reinforcement:

$$\eta_{\mathrm{sys}}^{1/2}\tau_{\mathrm{R,c}} \geq \tau_{\mathrm{Rd}} = \tau_{\mathrm{Rd,c}} + 0.8\rho_{\mathrm{w}}f_{\mathrm{yw}d} \geq \rho_{\mathrm{w}}f_{\mathrm{ywd}}$$

$$\eta_{sys} = 1.15\frac{d_{sys}}{d} + 0.63\left(\frac{b_{1,red}}{d}\right)^{1/4} - 0.85\frac{s_o}{d_{sys}}$$





Joint's position – loading	reinforced in shear	# tested joints				
		P	FP	F	NF	Total
Interior-uniaxial	No	68	15	11	16	110
	Yes	18	27	33	34	113
Edge-uniaxial	No	22	3	1	19	45
	Yes	9	6	-	6	21
Interior or edge-biaxial	No	12	-	-	4	16
	Yes	2	4	-	-	6
Corner-biaxial	No	5		-	8	13
	Yes	6	1	-	7	14
Totals		142	56	45	94	337

Summary of RC member detailing rules – EN1998-1:2004 vs EN1998-1-2:202X

EN1998-1:2004 - Beam longitudinal reinforcement

	DC H	DC M	DC L
"critical region" length at member end	$1.5h$	h	
$\rho_{min} = A_{s,min}/bd$ at the tension side	$0.5 f_{ctm}/f_{yk}$ (1)		$0.26 f_{ctm}/f_{yk}$ (1), 0.13% (2)
$\rho_{max} = A_{s,max}/bd$ in critical regions (2)	$\rho' + 0.0018 f_{cd}/(\mu_\phi \varepsilon_{yd} f_{yd})$ (3)		0.04
$A_{s,min}$, top and bottom bars	$2\Phi14$ (308mm^2)	-	
$A_{s,min}$, top bars in the span	$0.25 A_{s,top\text{-}supports}$	-	
$A_{s,min}$, bottom bars in critical regions	$0.5 A_{s,top}$ (4)		-
$A_{s,min}$, bottom bars at supports	$0.25 A_{s,bottom\text{-}span}$ (2)		
anchorage length for diameter d_{bL} (5)	$l_{bd} = a_{tr}[1-0.15(c_d/d_{bL}-1)](d_{bL}/4) f_{yd}/(2.25 f_{ctd} a_{poor})$ (6),(7),(8),(9)		

(1) f_{ctm} (MPa)$=0.3(f_{ck}$(MPa)$)^{2/3}$: mean tensile strength of concrete; f_{yk} (MPa): nominal yield stress of longitudinal bars

(2) NDP (Nationally Determined Parameter) per EC2; value recommended in EC2 is given here

(3) ρ': steel ratio at the opposite side of the section; μ_ϕ: curvature ductility factor corresponding to basic value of behavior factor, q_o, applicable to the design; $\varepsilon_{yd} = f_{yd}/E_s$.

(4) This $A_{s,min}$ is additional to the compression steel from the ULS verification of the end section in flexure under the extreme hogging moment from the analysis for the seismic design situation.

(5) Anchorage length in tension reduced by 30% if bar end extends by $\geq 5d_{bL}$ beyond a bend$\geq 90^o$.

(6) c_d: concrete cover of anchored bar, or one-half the clear spacing to nearest parallel anchored bar if it is smaller

(continued next slide)

EN1998-1:2004 - Beam longitudinal reinforcement *(cont'd)*

(continued from previous slide)

(7) $a_{tr} = 1\text{-}k(n_w A_{sw}\text{-}A_{s,t,min})/A_s \geq 0.7$, with A_{sw}: cross-sectional area of tie-leg within the cover of the anchored bar; n_w: number of such tie legs over the length l_{bd}; $k = 0.1$ if the bar is at a corner of a hoop or tie, $k = 0.05$ otherwise; $A_s = \pi d_{bL}{}^2/4$ and $A_{s,t,min}$ is specified in EC2 as equal to $0.25A_s$.

(8) $f_{ctd}=f_{ctk,0.05}/\gamma_c = 0.7f_{ctm}/\gamma_c = 0.21f_{ck}{}^{2/3}/\gamma_c$: design value of 5%-fractile tensile strength of concrete.

(9) $a_{poor} = 1.0$ if the bar is in the bottom 0.25 m of the beam depth, or (in beams deeper than 0.6 m) ≥ 0.3 m from the beam top; otherwise, $a_{poor} = 0.7$.

EN1998-1:2004 - Beam transverse reinforcement

	DC H	**DC M**	**DC L**
outside critical regions			
spacing, $s_h \leq$	$0.75d$		
$\rho_w = A_{sh}/b_w s_h \geq$	$(0.08\sqrt{f_{ck}(\text{MPa})})/f_{yk}(\text{MPa})$ (1)		
in critical regions			
diameter, $d_{bw} \geq$	6 mm		
spacing, $s_h \leq$	$6d_{bL}$ (2), $h/4$, $24d_{bw}$, 175mm	$8d_{bL}$ (2), $h/4$, $24d_{bw}$, 225mm	-

(1) NDP (Nationally Determined Parameter) per EC2; value recommended in EC2 is given here

(2) d_{bL}: minimum diameter of all top and bottom longitudinal bars within the critical region.

EN1998-1-2:202X - Beam longitudinal reinforcement

		DC 3	DC 2	DC 1
"critical region" length at member end		h		
$\rho_{min}=A_{s,min}/bd$ at the tension side		$0.5f_{ctm}/f_{yk}$ (1)		Such that moment resistance exceeds cracking moment
$\rho_{max}=A_{s,max}/bd$ in critical regions (2)	$f_{ck}\leq 25$ MPa	$\rho'+0.013-0.002f_{yk}/100$	$\rho'+0.015-0.002f_{yk}/100$	-
	$25 < f_{ck}$(MPa)<50	$\rho'+0.026-0.004f_{yk}/100$	$\rho'+0.028-0.004f_{yk}/100$	
	$f_{ck}\geq 50$ MPa	$\rho'+0.035-0.005f_{yk}/100$	$\rho'+0.037-0.005f_{yk}/100$	
$A_{s,min}$, bottom bars at supports		$0.25A_{s,bottom\text{-}span}$ (2)		

(1) f_{ctm} (MPa)=0.3(f_{ck}(MPa))$^{2/3}$ if $f_{ck}\leq$50 MPa, f_{ctm} (MPa)=1.1(f_{ck}(MPa))$^{1/3}$ if f_{ck}>50 MPa; f_{yk} (MPa): nominal yield stress of longitudinal steel.

ρ': steel ratio at the opposite side of the section; f_{yk} in MPa.

EN1998-1-2:202X - Beam transverse reinforcement

	DC 3	DC 2	DC 1
outside critical regions			
spacing, $s_h\leq$	$0.75d$		
$\rho_w=A_{sh}/b_w s_h\geq$	$(0.08\sqrt{f_{ck}(\text{MPa})})/f_{yk}(\text{MPa})$ (1)		
in critical regions			
spacing, $s_h\leq$	$8d_{bL}$ (2), $h/4$, $24d_{bw}$	$12d_{bL}$ (2), $h/4$, $30d_{bw}$	-

(1) Value may be reduced by 10 or 20%, when ductility class B or C steel is used, respectively.

(2) d_{bL}: minimum diameter of all top and bottom longitudinal bars within the critical region.

EN1998-1:2004 - Column longitudinal reinforcement

	DC H	DC M	DC L
$\rho_{min} = A_{s,min}/A_c$	1%		$0.1N_d/A_c f_{yd}$, 0.2% (1)
$\rho_{max} = A_{s,max}/A_c$	4%		4% (1)
diameter, d_{bL}	≥8mm		
number of bars per side	≥3		≥2
spacing along the perimeter of bars restrained by a tie corner or hook	≤150mm	≤200mm	-
distance along perimeter of unrestrained bar to nearest restrained one	≤150mm		
lap splice length (2)	$l_0 = 1.5[1-0.15(c_d/d_{bL}-1)]a_{tr}(d_{bL}/4)f_{yd}/(2.25f_{ctd})$ (3), (4), (5)		

(1) NDP (Nationally Determined Parameter) per EC2; value recommended in EC2 is given here

(2) Anchorage length in tension is reduced by 30% if the bar end extends by ≥ $5d_{bL}$ beyond a bend ≥ 90^o.

(3) c_d: minimum of: concrete cover of lapped bar and 50% of clear spacing to adjacent lap splice.

(4) $a_{tr} = 1-k(2n_w A_{sw}-A_{s,t,min})/A_s$, with k = 0.1 if the bar is at a corner of a hoop or tie, k = 0.05 otherwise; A_{sw}: cross-sectional area of a column tie; n_w: number of ties in the cover of the lapped bar over the outer third of the length l_0; $A_s = \pi d_{bL}^2/4$ and $A_{s,t,min}$ is specified in EC2 as equal to A_s.

(5) $f_{ctd} = f_{ctk,0.05}/\gamma_c = 0.7f_{ctm}/\gamma_c = 0.21f_{ck}^{2/3}/\gamma_c$: design value of 5%-fractile of tensile strength of concrete

EN1998-1:2004 - Column transverse reinforcement

	DC H	DC M	**DC L**
critical region length[1] ≥	1.5h_c, 1.5b_c, 0.6m, $H_{cl}/5$	h_c, b_c, 0.45m, $H_{cl}/6$	h_c, b_c,
	Outside the critical regions		
diameter, $d_{bw} \geq$	6mm, $d_{bL}/4$		
spacing, $s_w \leq$	20d_{bL}, h_c, b_c, 400mm		
at lap splices of bars with d_{bL}>14mm, $s_w \leq$	12d_{bL}, 0.6h_c, 0.6b_c, 240mm		
	In critical regions [2]		
diameter, $d_{bw} \geq$ [3]	6mm, $0.4\sqrt{(f_{yd}/f_{ywd})}d_{bL}$	6mm, $d_{bL}/4$	
spacing, $s_w \leq$ [3], [4]	6d_{bL}, $b_o/3$, 125mm	8d_{bL}, $b_o/2$, 175mm	as outside critical regions
mechanical ratio $\omega_{wd} \geq$[5]	0.08	-	
effective mechanical ratio $a\omega_{wd} \geq$ [4], [5], [6], [7]	$30\mu_\phi^* \nu_d \varepsilon_{yd} b_c/b_o - 0.035$	-	
	In the critical region at the base of the column (at the connection to the foundation)		
mechanical ratio $\omega_{wd} \geq$	0.12	0.08	-
effective mechanical ratio $a\omega_{wd} \geq$ [4], [5], [6], [8], [9]	$30\mu_\phi \nu_d \varepsilon_{yd} b_c/b_o - 0.035$		-

(1) h_c, b_c, H_{cl}: column sides and clear length.

(2) For DC M: If a value of $q \leq 2$ is used in design, transverse reinforcement in critical regions of columns with axial load ratio $\nu_d \leq 0.2$ may follow rules for DCL columns.

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EN1998-1:2004 - Column transverse reinforcement *(cont'd)*

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(3) For DC H: In the two lower stories of the building, the requirements on d_{bw}, s_w apply over a distance from the end section not less than 1.5 times the critical region length.

(4) Index c denotes full concrete section; index o the confined core to centreline of perimeter hoop; b_o is the smaller side of this core.

(5) ω_{wd}: volume ratio of confining hoops to confined core (to centerline of perimeter hoop) times f_{ywd}/f_{cd}.

(6) $a = (1\text{-}s/2b_o)(1\text{-}s/2h_o)(1\text{-}\{b_o/[(n_h\text{-}1)h_o]+h_o/[(n_b\text{-}1)b_o]\}/3)$: confinement effectiveness factor of rectangular hoops at spacing s, with n_b legs parallel to the side of the core with length b_o and n_h legs parallel to the side of length h_o.

(7) For DCH: at column ends protected from plastic hinging through the capacity design check at beam-column joints, μ_ϕ^* is the value of the curvature ductility factor that corresponds to 2/3 of the basic value, q_o, of the behavior factor applicable to the design; at the ends of columns where plastic hinging is not prevented, because of the exemptions from the application of the strong column-weak beam rule, μ_ϕ^* is taken equal to μ_ϕ defined in Note (8) (see also Note (9)); $\varepsilon_{yd}= f_{yd}/E_s$.

(8) μ_ϕ: curvature ductility factor corresponding to basic value, q_o, of behavior factor

(9) For DCH: The requirement applies also in the critical regions at the ends of columns where plastic hinging is not prevented, because of the exemptions from the application of the strong column-weak beam rule.

EN1998-1-2:202X - Column longitudinal reinforcement

	DC 3	DC 2	DC 1
$\rho_{min} = A_{s,min}/A_c$	1%		$0.1N_d/A_c f_{yd}$, 0.2% (1)
$\rho_{max} = A_{s,max}/A_c$	4%		4% (1)
diameter, d_{bL}	≥12 mm		
number of bars per side	≥3		≥2
spacing along the perimeter of bars restrained by a tie corner or hook	≤200mm	≤250mm	-
distance along perimeter of unrestrained bar to nearest restrained one	≤150mm		

(1) NDP (Nationally Determined Parameter) per EC2; the value recommended in EC2 is given.

EN1998-1-2:202X - Column transverse reinforcement

	DC 3	DC 2	DC 1
critical region length (1) ≥	h_c, b_c, 0.45m, $H_{cl}/6$		h_c, b_c,
Outside the critical regions			
diameter, $d_{bw} \geq$	$d_{bL}/4$		
spacing, $s_w \leq$	$15d_{bL}$, h_c, b_c, 300mm		
at lap splices of bars with d_{bL}>14mm, $s_w \leq$	$9d_{bL}$, $0.6h_c$, $0.6b_c$, 180mm		
In critical regions			
diameter, $d_{bw} \geq$	6mm, $d_{bL}/4$		
spacing, $s_w \leq$(2)	$8d_{bL}$, $b_o/2$, 175mm	$9d_{bL}$, $b_o/2$, 200mm	as at bar laps w/ d_{bL}>14mm
mechanical ratio $\omega_{wd} \geq$ (3)	0.08	0.05	-

(1) h_c, b_c, H_{cl}: column sides and clear length.

(2) Index o denotes confined core to centreline of perimeter hoop; b_o is the smaller side of core.

(3) ω_{wd}: volume ratio of confining hoops to confined core (to centreline of perimeter hoop) times f_{ywd}/f_{cd}

EN1998-1:2004 - Walls

	DC H	DC M	DC L
critical region height, h_{cr}	$\geq \max(l_w, H_w/6)$ (2) $\leq \min(2l_w, h_{storey})$ if wall ≤ 6 stories $\leq \min(2l_w, 2h_{storey})$ if wall > 6 stories		-
Boundary elements			
a) in critical height region:			
- length l_c from wall edge $\geq$	$0.15l_w$, $1.5b_w$, part of the section where $\varepsilon_c > 0.0035$		-
- thickness b_w over $l_c \geq$	0.2m; $h_{st}/15$ if $l_c \leq \max(2b_w, l_w/5)$, $h_{st}/10$ otherwise		-
- vertical reinforcement:			
ρ_{min} over $A_c = l_c b_w$	0.5%		0.2% (1)
ρ_{max} over A_c	4% (1)		
spacing along perimeter of bars restrained by tie corner or cross-tie hook	$\leq$150mm	$\leq$200mm	-
- confining hoops (index w)(3)			
diameter, $d_{bw} \geq$	6mm, $0.4\sqrt{(f_{yd}/f_{ywd})}d_{bL}$	6mm,	wherever $\rho_L >$ 2% in section: as over rest of the wall (see case b below)
spacing, $s_w \leq$ (4)	$6d_{bL}$, $b_o/3$, 125mm	$8d_{bL}$, $b_o/2$, 175mm	
$\omega_{wd} \geq$ (3)	0.12	0.08	
$a\omega_{wd} \geq$ (4), (5)	$30\mu_\phi(\nu_d+\omega_v)\varepsilon_{yd}b_w/b_o - 0.035$		
b) over the rest of the wall height:	Wherever in the section $\varepsilon_c > 0.2\%$: $\rho_{v,min} = 0.5\%$; elsewhere: 0.2% In parts of the section where $\rho_L > 2\%$: - distance of unrestrained bar in compression zone to nearest restrained bar $\leq$ 150mm; - hoops with $d_{bw} \geq \max(6\text{mm}, d_{bL}/4)$, spacing $s_w \leq \min(12d_{bL}, 0.6b_{wo}, 240\text{mm})$ (1) till distance $4b_w$ above or below floor slab /beam; $s_w \leq \min(20d_{bL}, b_{wo}, 400\text{mm})$ (1) beyond that distance		

EN1998-1:2004 - Walls *(cont'd)*

(1) NDP (Nationally Determined Parameter) per EC2; the value recommended in EC2 is given here

(2) l_w: long side of rectangular wall section or rectangular part thereof; H_w: total height of wall; h_{story}: story height.

(3) (In DC M only) The DCL rules apply to the confining reinforcement of boundary elements, if: under the maximum axial force in the wall from the analysis for the seismic design situation, the wall axial load ratio $\nu_d = N_{Ed}/A_c f_{cd}$ is ≤ 0.15; or, if $\nu_d \leq 0.2$ but the q-value used in the design is $\leq$ 85% of the q-value allowed when the DC M confining reinforcement is used in boundary elements.

(4) Notes (4), (5), (6) of Table for EN1998-1:2004 columns apply to the confined core of boundary elements.

(5) μ_ϕ: value of the curvature ductility factor corresponding to the product of the basic value q_o of the behavior factor times the ratio M_{Edo}/M_{Rdo} of the moment at the wall base from the analysis for the design seismic action to the design value of moment resistance at the wall base for the axial force from the same analysis; $\varepsilon_{yd} = f_{yd}/E_s$; ω_{vd}: mechanical ratio of vertical web reinforcement.

EN1998-1:2004 - Walls *(cont'd)*

	DC H	DC M	DC L
Web			
thickness, $b_{wo} \geq$	max(150mm, h_{storey}/20)		-
vertical bars (index: v):			
$\rho_v = A_{sv}/b_{wo}s_v \geq$	0.2%, but 0.5% wherever in the section ε_c>0.002		0.2% (1)
$\rho_v = A_{sv}/b_{wo}s_v \leq$	4%		
$d_{bv} \geq$	8mm	-	
$d_{bv} \leq$	b_{wo}/8	-	
spacing, $s_v \leq$	min(25d_{bv}, 250mm)	min(3b_{wo}, 400mm)	
horizontal bars (index: h):			
$\rho_{h,min}$	0.2%	max(0.1%, 0.25ρ_v) (1)	
$d_{bh} \geq$	8mm	-	
$d_{bh} \leq$	b_{wo}/8	-	
spacing, $s_h \leq$	min(25d_{bh}, 250mm)	400mm	
$\rho_{v,min}$ at construction joints (6)	max(0.25%; $\dfrac{1.3 f_{ctd} - N_{Ed}/A_c}{f_{yd} + 1.5\sqrt{f_{cd} f_{yd}}}$)	-	

(6) N_{Ed}: minimum axial load from the analysis for the seismic design situation (positive for compression); $f_{ctd}=f_{ctk,0.05}/\gamma_c = 0.7f_{ctm}/\gamma_c = 0.21f_{ck}^{2/3}/\gamma_c$: design value of 5%-fractile tensile strength of concrete.

EN1998-1-2:202X - Walls

	DC 3	DC 2	DC 1
critical region height, h_{cr}	$\geq l_w$, $H_w/6$, $\leq 2l_w$, $\leq h_{story}$ if wall ≤6 stories, $\leq 2h_{story}$ if >6 stories		-
Boundary elements			
a) in critical height region:			
- length l_c from wall edge ≥	$0.15l_w$, $1.5b_w$		-
- thickness b_w over l_c ≥	0.2m; $h_{st}/15$ if $l_c \leq 2b_w$, $l_w/5$; $h_{st}/10$ otherwise		-
- vertical reinforcement:			
diameter, d_{bL}	≥12 mm		
number of bars per side	≥3		-
ρ_{min} over $A_c = l_c b_w$	1%		0.2%, $0.5f_{ctm}/f_{yk}$ (1)
ρ_{max} over A_c	4% (1)		
spacing (along perimeter) of bars restrained by tie corner or X-tie	≤200mm	≤250mm	-
- confining hoops (index w): $\omega_{wd} \geq$	0.08	0.05	-
b) over the rest of the wall height:	As in the web (see below)		
Web			
thickness, $b_{wo} \geq$	150mm, $h_{story}/20$, $l_w/40$		-
vertical bars (index: v):			
$\rho_v = A_{sv}/b_{wo}s_v \geq$	0.25%; 0.5% wherever in the section $\varepsilon_c > 0.002$		0.2%, $0.5f_{ctm}/f_{yk}$ (1)
$\rho_v = A_{sv}/b_{wo}s_v \leq$	4%		
spacing, $s_v \leq$	250mm	300mm	$3b_{wo}$, 400mm(1)
horizontal bars (index: h):			
$\rho_{h,min}$	0.25%		$0.5f_{ctm}/f_{yk}$, $\rho_v/4$ (1)
spacing, $s_h \leq$	250mm	300mm	400mm(1)

Thank you !