“Dissemination of information for training” workshop

18-20 February 2008

Brussels

EN 1998
Eurocode 8: Design of structures for earthquake resistance

Organised by
European Commission: DG Enterprise and Industry, Joint Research Centre

with the support of
CEN/TC250, CEN Management Centre and Member States
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<th>Time</th>
<th>Session</th>
<th>Speaker</th>
<th>Institution/University</th>
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<tr>
<td>9:00-9:15</td>
<td>Opening. General introduction to EN 1998</td>
<td>E. Carvalho</td>
<td>Gapres SA</td>
</tr>
<tr>
<td>9:15-10:00</td>
<td>Sections 1 to 3: General; requirements; ground conditions and seismic action</td>
<td>E. Carvalho</td>
<td>Gapres SA</td>
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<tr>
<td>10:00-10:45</td>
<td>Sections 4 and 5: Design of buildings and concrete buildings</td>
<td>M. Fardis</td>
<td>University of Patras</td>
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<td>10:45-11:00</td>
<td>Coffee</td>
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<tr>
<td>11:00-11:45</td>
<td>Sections 6 and 7: Steel and composite steel-concrete buildings</td>
<td>A. Plumier</td>
<td>University of Liege</td>
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<td>11:45-12:15</td>
<td>Sections 8 and 9: Timber and masonry buildings</td>
<td>E. Carvalho</td>
<td>Gapres SA</td>
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<td>12:15-12:30</td>
<td>Section 10: Base isolation</td>
<td>E. Carvalho</td>
<td>Gapres SA</td>
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<tr>
<td>12:30-13:00</td>
<td>Discussion</td>
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<td>13:00-14:30</td>
<td>Lunch</td>
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<tr>
<td>14:30-15:15</td>
<td>Part 2: Bridges</td>
<td>B. Kolias</td>
<td>Denco S.A.</td>
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<tr>
<td>16:30-17:15</td>
<td>Part 5: Foundations, retaining structures and geotechnical aspects</td>
<td>E. Faccioli</td>
<td>Politecnico di Milano</td>
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<td>16:00-16:15</td>
<td>Coffee</td>
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<tr>
<td>15:15-16:00</td>
<td>Part 3: Assessment and retrofitting of buildings</td>
<td>P. Pinto</td>
<td>Università di Roma La Sapienza</td>
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<tr>
<td>17:15-17:30</td>
<td>Part 4: Silos, tanks and pipelines Part 6: Towers, masts and chimneys</td>
<td>E. Carvalho</td>
<td>Gapres SA</td>
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<tr>
<td>17.30-18:00</td>
<td>Discussion</td>
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All workshop material will be available at [http://eurocodes.jrc.ec.europa.eu](http://eurocodes.jrc.ec.europa.eu)
GENERAL INTRODUCTION TO EN 1998

E. Carvalho
Gapres SA
**Eurocode 8**

**General rules and seismic actions**

E C Carvalho, Chairman TC250/SC8

**Eurocode 8 - Design of structures for earthquake resistance**

- EN1998-1: General rules, seismic actions and rules for buildings
- EN1998-2: Bridges
- EN1998-3: Assessment and retrofitting of buildings
- EN1998-4: Silos, tanks and pipelines
- EN1998-5: Foundations, retaining structures and geotechnical aspects
- EN1998-6: Towers, masts and chimneys

All parts published by CEN (2004-2006)

**EN1998-1: General rules, seismic actions and rules for buildings**

- General
- Performance requirements and compliance criteria
- Ground conditions and seismic action
- Design of buildings
- Specific rules for:
  - Concrete buildings
  - Steel buildings
  - Composite Steel-Concrete buildings
  - Timber buildings
  - Masonry buildings
- Base isolation

**Objectives**

In the event of earthquakes:

- Human lives are protected
- Damage is limited
- Structures important for civil protection remain operational

Special structures – Nuclear Power Plants, Offshore structures, Large Dams – outside the scope of EN 1998

**Fundamental requirements**

No-collapse requirement:

- Withstand the design seismic action without local or global collapse
- Retain structural integrity and residual load bearing capacity after the event

For ordinary structures this requirement should be met for a reference seismic action with 10% probability of exceedance in 50 years (recommended value) i.e. with 475 years Return Period
Fundamental requirements

**Damage limitation requirement:**

Withstand a more frequent seismic action without damage

Avoid limitations of use with high costs

For ordinary structures this requirement should be met for a seismic action with 10% probability of exceedance in 10 years (recommended value) i.e. with 95 years Return Period

**Reliability differentiation**

Target reliability of requirement depending on consequences of failure

Classify the structures into importance classes

Assign a higher or lower return period to the design seismic action

In operational terms multiply the reference seismic action by the importance factor $\gamma_I$

### Importance classes for buildings

<table>
<thead>
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<th>Buildings</th>
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<tbody>
<tr>
<td>I</td>
<td>Buildings of minor importance for public safety, e.g. agricultural buildings, etc.</td>
</tr>
<tr>
<td>II</td>
<td>Ordinary buildings, not belonging in the other categories</td>
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<tr>
<td>III</td>
<td>Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, critical institutions, etc.</td>
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<td>IV</td>
<td>Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.</td>
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Importance factors for buildings (recommended values):

$\gamma_I = 0.8; 1.0; 1.2$ and $1.4$

Importance factors for buildings

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### Compliance criteria (design verifications):

**Ultimate limit state**

Resistance and Energy dissipation capacity

Ductility classes and Behaviour factor values

Overturning and sliding stability check

Resistance of foundation elements and soil

Second order effects

Non detrimental effect of non structural elements

Simplified checks for low seismicity cases ($a_g < 0.08 \text{ g}$)

No application of EN 1998 for very low seismicity cases ($a_g < 0.04 \text{ g}$)

### Specific measures

In zones of high seismicity formal Quality Plan for Design, Construction and Use is recommended

### Compliance criteria (design verifications):

**Damage limitation state**

Deformation limits (Maximum interstorey drift due to the “frequent” earthquake):

- 0.5% for brittle non structural elements attached to the structure
- 0.75% for ductile non structural elements attached to the structure
- 1.0% for non structural elements not interfering with the structure

Sufficient stiffness of the structure for the operability of vital services and equipment

DLS may control the design in many cases

### Compliance criteria (design verifications):

**Specific measures**

Simple and regular forms (plan and elevation)

Control the hierarchy of resistances and sequence of failure modes (capacity design)

Avoid brittle failures

Control the behaviour of critical regions (detailing)

Use adequate structural model (soil deformability and non strutural elements if appropriate)
Ground conditions

Five ground types:
A - Rock
B - Very dense sand or gravel or very stiff clay
C - Dense sand or gravel or stiff clay
D - Loose to medium cohesionless soil or soft to firm cohesive soil
E - Surface alluvium layer C or D, 5 to 20 m thick, over a much stiffer material

2 special ground types S₁ and S₂ requiring special studies
Ground conditions defined by shear wave velocities in the top 30 m and also by indicative values for N_SPT and c_u.

Table 3.1: Ground types

<table>
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<tr>
<th>Ground type</th>
<th>Description of stratigraphic profile</th>
<th>Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.</td>
<td>vₛ,30 (m/s)</td>
</tr>
<tr>
<td>B</td>
<td>Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.</td>
<td>vₛ,30 (m/s)</td>
</tr>
<tr>
<td>C</td>
<td>Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.</td>
<td>180 - 360</td>
</tr>
<tr>
<td>D</td>
<td>Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.</td>
<td>&lt; 180</td>
</tr>
<tr>
<td>E</td>
<td>A soil profile consisting of a surface alluvium layer with vₛ values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with vₛ &gt; 800 m/s.</td>
<td></td>
</tr>
<tr>
<td>S₁</td>
<td>Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI &gt; 40) and high water content</td>
<td>&lt; 100 (indicative)</td>
</tr>
<tr>
<td>S₂</td>
<td>Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A - E or S₁</td>
<td></td>
</tr>
</tbody>
</table>

Seismic zonation

Competence of National Authorities
Described by a_R (reference peak ground acceleration on type A ground)
Corresponds to the reference return period T_NCR
Modified by the Importance Factor γ_I to become the design ground acceleration (on type A ground) a_d = a_Rk · γ_I
Objective for the future updating of EN1998-1:
European zonation map with spectral values for different hazard levels (e.g. 100, 500 and 2,500 years)

Basic representation of the seismic action

Elastic response spectrum
Common shape for the ULS and DLS verifications
2 orthogonal independent horizontal components
Vertical spectrum shape different from the horizontal spectrum (common for all ground types)
Possible use of more than one spectral shape (to model different seismo-genetic mechanisms)
Account of topographical effects (EN 1998-5) and spatial variation of motion (EN1998-2) required in some special cases
Definition of the horizontal elastic response spectrum (four branches)

- \(0 \leq T \leq T_B\)  
  \[S_e(T) = a_g \cdot S \cdot (1 + (T/T_B) \cdot (\eta \cdot 2.5 - 1))\]

- \(T_B \leq T \leq T_C\)  
  \[S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5\]

- \(T_C \leq T \leq T_D\)  
  \[S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot (T_C / T)\]

- \(T_D \leq T \leq 4\)  
  \[S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot (T_C \cdot T_D / T^2)\]

Additional information for \(T > 4\) s in Informative Annex.

Normalised elastic response spectrum (standard shape)

Control variables
- \(S, T_B, T_C, T_D\) (NDPs)
- \(\eta \geq 0.55\) damping correction for \(\xi \neq 5\)

Fixed variables
- Constant acceleration, velocity & displacement spectral branches
- Acceleration spectral amplification: 2.5

Different spectral shape for vertical spectrum (spectral amplification: 3.0)

Elastic response spectrum

Two types of (recommended) spectral shapes

- **Type 1** - High and moderate seismicity regions \((M_s > 5.5)\)
- **Type 2** - Low seismicity regions \((M_s \leq 5.5); near field earthquakes

Optional account of deep geology effects (NDP) for the definition of the seismic action.

Recommended elastic response spectra

- **Type 1** - \(M_s > 5.5\)
- **Type 2** - \(M_s \leq 5.5\)
**Design spectrum for elastic response analysis**

(derived from the elastic spectrum)

\[
S_d(T) = \begin{cases} 
    a_g \cdot S \cdot (2/3 + T / T_b \cdot (2.5/q - 2/3)) & 0 \leq T \leq T_b \\
    a_g \cdot S \cdot 2.5/q & T_b \leq T \leq T_C \\
    a_g \cdot S \cdot (T_C / T) & T_C \leq T \leq T_D \\
    \beta \cdot a_g & T_D \leq T \leq 4 \text{s}
\end{cases}
\]

Specific rules for vertical action:

\[
a_v = 0.9 \cdot a_g \text{ or } a_v = 0.45 \cdot a_g, \text{ if } q \leq 1.5
\]

**Alternative representations of the seismic action**

**Time history representation** (essentially for NL analysis purposes)

- **Three simultaneously acting accelerograms**
  - **Artificial accelerograms**
    - Match the elastic response spectrum for 5% damping
    - Duration compatible with Magnitude (\( T_s \geq 10 \text{s} \))
    - Minimum number of accelerograms: 3
  - **Recorded or simulated accelerograms**
    - Scaled to \( a_g \cdot S \)
    - Match the elastic response spectrum for 5% damping

\( a \) — acceleration
\( S \) — spectral response
\( q \) — behaviour factor
\( \beta \) — lower bound factor (NDP recommended value: 0.2)
SECTIONS 4 AND 5: DESIGN OF BUILDINGS AND CONCRETE BUILDINGS

M. Fardis
University of Patras
Design of buildings for earthquake resistance, according to Eurocode 8-Part 1
(Buildings and concrete buildings)

Michael N. Fardis
University of Patras, Greece

EUROCODES
Background and Applications

STRUCTURE OF EN 1998-1:2004
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

Fundamental features of good structural layout

• Clear structural system.
• Simplicity & uniformity in geometry of structural system.
• Symmetry & regularity in plan.
• Significant torsional stiffness about vertical axis.
• Geometry, mass & lateral stiffness: regular in elevation.
• Redundancy of structural system.
• Effective horizontal connection of vertical elements at all floor levels.

Clear structural system

• System of:
  – plane frames continuous in plan, from one side of the plan to the opposite, w/o offsets or interruption in plan, or indirect supports of beams, and/or
  – (essentially) rectangular shear walls, arranged in two orthogonal horizontal directions.

Symmetry - regularity in plan

• Lateral stiffness & mass ~symmetric w.r.t to two orthogonal horizontal axes (full symmetry — response to translational horizontal components of seismic action will not include any torsion w.r.t the vertical axis).
• Lack of symmetry in plan often measured via “static eccentricity”, e, between:
  – centre of mass of storey (centroid of overlying masses, CM) and
  – centre of stiffness (CS, important during the elastic response)
• One of Eurocode 8 criteria for regularity in plan:
  – “torsional radius” r_x (r_y) = “ratio of:
    – torsional stiffness of storey w.r.to CS, to
    – storey lateral stiffness in y (x) direction, orthogonal to x (y)
• CS, CR & r_x, r_y unique & independent of lateral loading only in single-storey buildings.
• Another Eurocode 8 criterion for regularity in plan: compact outline in plan, enveloped by convex polygonal line. Re-entrant corners in plan don’t leave area up to convex polygonal envelope >5% of area inside outline.

Symmetry - regularity in plan (cont’d)

Torsional response → difference in seismic displacements between opposite sides in plan; larger local deformation demands on side experiencing the larger displacement (“flexible side”).

Collapse of building due to its torsional response about a stiff shaft at the corner (Athens, 1999 earthquake).
High torsional stiffness w.r.t to vertical axis

- Purely torsional natural mode w.r.t to vertical axis w/ \( T > T \) of lowest (purely translational natural mode \( T \))
  - Accidental torsional vibrations w.r.t to vertical axis by transfer of vibration energy from the response in the lowest translational mode to the torsional one → significant & unpredictable horizontal displacements at the perimeter.

- Avoided through Eurocode 8 criterion for regularity in plan:
  - "Torsional radii" \( r_x \) (better \( r_{mx} \)) & \( r_y \) (better \( r_{my} \))
  - Radius of gyration of floor mass in plan \( l_s = \sqrt{\frac{J_{pl}}{M_{total, floors}}} \)
  - For rectangular floor area:
    \[ r_x \geq \frac{l_s}{12}; \quad r_y \geq \frac{l_s}{12} \]

Means of providing torsional stiffness about a vertical axis: Shear walls or strong frames at the perimeter

Arrangements of shear walls in plan:
(a) preferable;
(b) drawbacks due to restraint of floors & difficulties of foundation at the corners;
(c) sensitive to failure of individual walls

Geometry, mass, stiffness: regular in elevation


Eurocode 8 criteria for regularity in elevation in buildings w/ setbacks

Geometry, mass & lateral stiffness: regular in elevation (cont’d)

Redundancy of structural system

- Provide large number of lateral-load resisting elements & alternative paths for earthquake resistance.
- Avoid systems w/ few large walls per horizontal direction, especially in buildings long in plan:
  - In-plane bending of long floor diaphragms in building with two strong walls at the 2 ends \( \rightarrow \) Intermediate columns overloaded, compared to results of design w/ rigid diaphragm.

Eurocode 8: Bonus to system redundancy:
- \( q_o \) proportional to \( u_{i0} \):

Continuity of floor diaphragms

- Need smooth/continuous path of forces, from the masses where they are generated due to inertia, to the foundation.
- Cast-in-situ reinforced concrete is the ideal structural material for earthquake resistant construction, compared to prefabricated elements joined together at the site: the joints between such elements are points of discontinuity.
- Floor diaphragms should have sufficient strength to transfer the inertia forces to the lateral-load-resisting system & be adequately connected to it.
- Large openings in floor slabs, due to internal patios, wide shafts or stairways, etc. may disrupt continuity of force path, especially if such openings are next to large shear walls near or at the perimeter.
- Vertical elements of lateral-force resisting system should be connected together, via combination of floor diaphragms & beams:
  - at all horizontal levels where significant masses are concentrated, and
  - at foundation level.
Continuity of floor diaphragms (cont’d)

EC8 DESIGN CONCEPTS FOR SAFETY UNDER DESIGN SEISMIC ACTION

1. Design for energy dissipation (typically normal through ductility): q > 1.5
   - Global ductility:
     - Structure forced to remain straight in elevation through shear walls, bracing system or strong columns (3M_y > 1.5M_y in frames)
   - Local ductility:
     - Plastic hinges detailed for ductility capacity derived from q-factor
     - Brittle failures prevented by overdesign/capacity design
   - Capacity design of foundations & foundation elements:
     - On the basis of overstrength of ductile elements of superstructure

2. Design w/o energy dissipation & ductility: q ≤ 1.5 for overstrength design only according to EC2 - EC7 (Ductility Class "Low" - DCL)
   - Only:
     - for Low Seismicity (NDP; recommended: PGA on rock < 0.08g)
     - for superstructure of base-isolated buildings

Force-based design for energy-dissipation & ductility, to meet no-collapse requirement under Design Seismic action:

- Structure allowed to develop significant inelastic deformations under design seismic action, provided that integrity of members & of the whole is not endangered
- Basis of force-based design for ductility:
  - Inelastic response spectrum of SDOF system having elastic-perfectly plastic F-D curve, in monotonous loading
  - For given period, 𝑇, of elastic SDOF system, inelastic spectrum relates:
    - Ratio q = 𝐹_𝑦/𝐹_e of peak force, 𝐹_𝑦, that would develop if the SDOF system was linear-elastic, to its yield force, 𝐹_e ("behaviour factor")
    - Maximum displacement demand of the inelastic SDOF system, δ_{max}, expressed as ratio to the yield displacement, δ_y: displacement ductility factor, 𝜇_y = δ_{max}/δ_y

Control of inelastic seismic response: Soft-storey mechanism avoided

- Soft storey collapse mechanism to be avoided via proper structural layout

Control of inelastic seismic response via capacity design

- For Dissipative Structures (except masonry):
  - Two Ductility Classes (DC):
    - DC H (High)
    - DC M (Medium)
  - Differences in:
    - q-values (usually q > 4 for DCH, 1.5 < q < 4 for DCM)
    - Local ductility requirements (ductility of materials or section, member detailing, capacity design against brittle failure modes)

- Not all locations or parts in a structure are capable of ductile behaviour & 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 will take place only in those members, regions and mechanisms capable of ductile behaviour & energy dissipation. The rest stay in the elastic range
- The 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_y, at least equal to the design value of the action effect due to the seismic design situation, 𝐹_s, from the analysis:
  - 𝑅_y ≤ 𝐹_s
- Normally linear analysis is used for the design seismic action (by dividing the elastic response spectrum by the behaviour factor, 𝑞)
EC8-PART 1: FOR ALL MATERIALS:

- "Secondary seismic elements":
  - Their contribution to resistance & stiffness for seismic actions neglected in design (& in linear analysis model, too);
  - Required to remain elastic under deformations due to design seismic action.
  - Designer free to assign elements to the class of "secondary seismic elements", provided that:
    - Their total contribution to lateral stiffness ≤ 15%;
    - Regularity classification does not change.

LINEAR ANALYSIS FOR DESIGN SEISMIC ACTION

- Reference approach: Force-based design with linear analysis:
  - Linear modal response spectrum analysis, with design response spectrum (elastic spectrum reduced by behaviour-factor q):
    - Applies always (except for seismic isolation with very nonlinear devices)
    - If:
      - building regular in elevation &
      - higher modes unimportant (fundamental $T < 4T_c$ & $<2T_c$; $T_c$ at end of constant spectral acceleration plateau)
      - (linear) Lateral force procedure emulating response-spectrum method:
        - $T$ from mechanics (Rayleigh quotient);
        - Reduction of forces by 15% if >2 storeys & $T < 2T_c$
  - Member verification at the Ultimate Limit State (ULS) for "Life-Safety" EQ in terms of forces (resistances)

REGULARITY OF BUILDINGS IN ELEVATION

- Qualitative criteria, can be checked w/o calculations:
  - Structural systems (walls, frames, bracing systems):
    - continuous to the top (of corresponding part).
  - Storey $K$ & $m$: constant or gradually decreasing to the top.
  - Individual floor setbacks on each side: < 10% of underlying storey.
  - Unsymmetric setbacks: < 30% of base in total.
  - Single setback at lower 15% of building: < 50% of base.
  - In frames (incl. infilled): smooth distribution of storey overstrength

REGULARITY OF BUILDINGS IN PLAN

- Criteria can be checked before any analysis:
  - $K$ & $m$ - symmetric w.r.t two orthogonal axes.
  - Rigid floors.
  - Plan configuration compact, w/ aspect ratio ≤ 4;
    - any recess from convex polygonal envelope: < 5% of floor area.
  - In both horizontal directions:
    - $f$ (toroidal radius of struct. system) > $l_1$ (radius of gyration of floor plan):
      - Translational fundamental $T(s) > torsional$.
    - $e_o$ (eccentricity between floor C.S. & C.M.) ≤ 0.3 $r$:
      - Conservative bound to satisfactory performance (element ductility demands – same as in torsionally balanced structure).
  - Alternative for buildings ≤ 10m tall:
    - In both horizontal directions: $r^2 ≥ l_1^2 + e_o^2$

FOR CONCRETE & MASONRY BUILDINGS

- Yield-point stiffness in analysis (50% of uncracked section EI):
  - Reduction in design seismic forces vis-a-vis use of full section EI
  - Increase of displacements for drift-control & P-Δ effects (goesizes of frame members).
Implementation of EC8 seismic design philosophy

- Damage limitation (storey drift ratio < 0.5-1%) under the damage limitation earthquake (~50% of "design seismic action"), using 50% of uncracked gross section stiffness.
- Member verification for the Ultimate Limit State (ULS) in bending under the "design seismic action", with elastic spectrum reduced by the behaviour factor $q$.
- In frames or frame-equivalent dual systems: Fulfilment of strong column/weak beam capacity design rule, with overstrength factor of 1.3 on beam strengths.
- Capacity design of members and joints in shear.
- Detailing of plastic hinge regions, on the basis of the value of the curvature ductility factor that corresponds to the $q$-factor value.

ULS Verification of dissipative zones

- The regions of members entrusted for hysteretic energy dissipation - called in Eurocode 8 “dissipative zones” - are designed & 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 design situation, $E_d$, from the analysis:

$$E_d \leq R_d$$

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

Seismic design of the foundation

- Objective: The ground and the foundation system should not reach its ULS before the superstructure, i.e. remain elastic while inelasticity develops in the superstructure.
- Means:
  - The ground and the foundation system are designed for their ULS under seismic action effects from the analysis derived for $q=1.5$, i.e. lower than the $q$-value used for the design of the superstructure, or
  - The ground and the foundation system are designed for their ULS under seismic action effects from the analysis multiplied by $\gamma$Rd/Edi, where $R_d$ force capacity in the dissipative zone or element controlling the seismic action effect of interest $E_d$ the seismic action effect there from the elastic analysis and $\gamma$ = 1.2.
- For individual spread footings of walls or columns of moment-resisting frames, $R_{Ed}$ is the minimum value of $F_{Ed}$ in the two orthogonal principal directions at the lowest cross-section of the vertical element where a plastic hinge can form in the seismic design situation;
- For individual spread footings of columns of concentric braced frames, $R_{Ed}$ is the minimum value of the $F_{Ed}$, among all diagonals which are in tension in the particular seismic design situation, for eccentric braced frames, $R_{Ed}$ is the minimum value of $V_{Ed}$, among all seismic links of the frame;
- For common foundations of more than one elements, $\gamma$Rd/Edi = 1.4.

Strong column/weak beam capacity design rule in frames or frame-equivalent dual systems (frames resist >50% of seismic base shear) above two storeys (except at top storey joints):

$$\sum M_{Ed} \geq \gamma M_{Ed}$$

- Overstrength factor $\gamma = 1.3$ on beam strengths $V_{Ed} = 1.3$

Beam & column flexural capacities at a joint in Capacity Design rule

EC8-PART 1: DAMAGE LIMITATION CHECK

- Seismic action for "damage limitation": NDP.
- Recommended for ordinary buildings: 10%/10yrs (95yr EQ);
- ~50% of "design seismic action" (475yr EQ).
- Interstorey drift ratio calculated for “damage limitation” action via “equal displacement rule” (elastic response):
  - <0.5% for brittle nonstructural elements attached to structure;
  - <0.75% for ductile nonstructural elements attached to structure;
  - < 1% for nonstructural elements not present or not interfering with structural response (damage limitation for structure).
- Concrete & masonry:
  - Elastic stiffness ~ 50% of uncracked gross-section stiffness.
- In concrete, steel or composite frames: damage limitation check governs member sizes.

STRUCTURE OF EN1998-1:2004

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-Conglomerate Composite Buildings
8 Specific Rules for Timber Buildings
9 Specific Rules for Masonry Buildings
10 Base Isolation
Seismic Design Philosophy for RC buildings

• **Ductility Classes (DC)**
  - Design based on energy dissipation and ductility:
    - **DC L (Low)**: Medium $q = 3$ x system overstrength factor ($\leq 1.3$).
    - **DC M (Medium)**: High $q = 4-4.5$ x system overstrength factor ($\leq 1.3$).
  - The aim of the design is to control the inelastic seismic response:
    - Structural layout & relative sizing of members ensures beam-sway mechanism.
    - Plastic hinge regions (beam ends, base of columns) are detailed to sustain inelastic deformation demands related to behaviour factor $q$.
      - $\alpha_{u} = q$ if $T > T_{c}$
      - $\alpha_{u} = 1 + (q - 1)T / T_{c}$ if $T \leq T_{c}$

Material limitations for "primary seismic elements"

<table>
<thead>
<tr>
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<td>Steel overstrength</td>
<td>No limit</td>
<td>No limit</td>
<td>$f_{u,k} \leq 1.25f_{y,k}$</td>
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</tbody>
</table>

NDP-partial factors for materials in ULS:

**Recommended:**

• Use same values as for persistent & transient design situations (i.e. concrete: $\gamma_{c} = 1.5$, steel: $\gamma_{s} = 1.15$);

Capacity design of members, against pre-emptive shear failure

$\alpha_u / \alpha_s$ in behaviour factor of buildings designed for ductility: due to system redundancy & overstrength

Normally:

- $\alpha_u \& \alpha_s$ from base shear - top displacement curve from pushover analysis.
- $\alpha_u$: seismic action at development of global mechanism.
- $\alpha_s$: seismic action at 1st flexural yielding anywhere.

- $\alpha_u / \alpha_s \geq 1.5$ (default values given between 1 to 1.3 for buildings regular in plan):
  - 1.0 for wall systems w/ just 2 uncoupled walls per horiz. direction;
  - 1.1 for one-storey frame or frame-equivalent dual systems, and wall systems w/ > 2 uncoupled walls per direction;
  - 1.2 for one-bay multi-storey frame or frame-equivalent dual systems, wall-equivalent dual systems & coupled wall systems;
  - 1.3 for multi-storey multi-bay frame or frame-equivalent dual systems.

- for buildings irregular in plan: default value = average of default values of buildings regular in plan and 1.0

Basic value, $q_{o}$, of behaviour factor - regular in elevation RC buildings

<table>
<thead>
<tr>
<th>Lateral-load resisting structural system</th>
<th>DC M</th>
<th>DC H</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inverted pendulum system*</td>
<td>1.5</td>
<td>2</td>
</tr>
<tr>
<td>Torsionally flexible structural system**</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Uncoupled wall system (&gt; 65% of seismic base shear resisted by walls, more than half by uncoupled walls) not belonging in one of the categories above</td>
<td>3</td>
<td>4.0 $\omega_{w}$</td>
</tr>
<tr>
<td>Any structural system other than those above</td>
<td>$3\omega_{w}$</td>
<td>$4.5\omega_{w}$</td>
</tr>
</tbody>
</table>

* : at least 50% of total mass in upper-third of the height, or with energy dissipation at base of a single element (except one-story frame or frame- equivalent dual systems at the top); walls in two horizontal directions in plan & with max. value of normalized axial load $\nu_{w}$ of all columns connected at the top by means of floor slabs.

** : 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: behaviour factor $q = 0.8q_{o}$

Wall or wall-equivalent dual systems: $q$ multiplied (further) by $\left(1 + a_{w}/3\right)$, where $a_{w}$: prevailing wall aspect ratio = $H/L_{w}$. 

Concrete grade No limit

Ductility Class DC L (Low) DC M (Medium) DC H (High)

Steel class per EN 1992-1-1, Table C1

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Material limitations for "primary seismic elements"
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I. Beams

- For steel B (µ = 5.75, f_y = 1.08-1.15) increase µ_d demand by 50%.
- Members w/ axial load & symmetric reinforcement, ωω ≤ 0.018f_y D:
  - Confining reinforcement (for walls: in boundary elements) with (effective) mechanical volumetric ratio:
    \[ \frac{\omega_\text{vol}}{\text{D}} = \frac{30\mu_f}{\gamma_{\text{con}}\mu_f + \gamma_\text{crown} D_\text{crown}} \approx 0.035 \]
  - ω_\text{vol}: volume of confining concrete; γ_\text{con}, γ_\text{crown}: confinement factors
  - ω_\text{vol}: mechanical ratio of longitudinal reinforcement in confinement elements:
    \[ \omega_\text{vol} = \frac{\psi_f}{\gamma_f} \mu_f \]
- Members w/ unsymmetric reinforcement (beams):
  - Max. mechanical ratio of tension steel:
    \[ \omega \leq \omega^* = 0.018f_y D \]

II. Columns

- Capacity design shear in column which is weaker than the beams:
  \[ V_{\text{CD}} = \frac{M_{\text{fact}} + M_{\text{fact}}}{B_D} \]
- Capacity design shear in (weak or strong) columns:
  \[ V_{\text{CD}} = \frac{M_{\text{fact}} + M_{\text{fact}}}{B_D} \]

III. Walls

- Over-design in shear, by multiplying shear forces from the analysis for the design seismic action, V_\text{fact}, by factor:\n  \[ \epsilon = \frac{V_{\text{fact}}}{V_{\text{fact}}} \]

- DC H squat walls (h/bw ≤ 2): Over-design for flexural overstrength of base w.r.t. analysis:
  \[ M_{\text{fact}} = \text{design moment at base section (from analysis),} \]
  \[ V_{\text{fact}} = \text{design flexural resistance at base section,} \]
  \[ \text{Y.m} = 1.2 \]

- DC H slender walls (h/bw > 2): Over-design for flexural overstrength of base w.r.t. analysis & for increased inelastic shear:
  \[ S_{\text{fact}}(\omega) \]
  where, \( \omega \) is the period of elastic response spectrum.

- Members w/o axial load & w/ unsymmetric reinforcement (beams):
  - Confining reinforcement (for walls: in boundary elements) with (effective) mechanical volumetric ratio:
    \[ \frac{\omega_\text{vol}}{\text{D}} = \frac{30\mu_f}{\gamma_{\text{con}}\mu_f + \gamma_\text{crown} D_\text{crown}} \approx 0.035 \]
  - ω_\text{vol}: volume of confining concrete; γ_\text{con}, γ_\text{crown}: confinement factors
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- Members w/ unsymmetric reinforcement (beams):
  - Max. mechanical ratio of tension steel:
    \[ \omega \leq \omega^* = 0.018f_y D \]
TYPES OF DISSIPATIVE WALLS

- Ductile wall:
  - Fixed at base, to prevent rotation there w.r.t. to rest of structural system.
  - Designed & detailed to dissipate energy only in flexural plastic hinge just above the base.
- Large lightly-reinforced wall (only for DC M):
  - Wall with horizontal dimension $L > 4m$, expected to develop limited cracking or inelastic behaviour, but to transform seismic energy to potential energy (uplift of masses) & energy dissipated in the soil by rigid-body rocking, etc.
  - Due to its dimensions, or lack-of-fixity at base wall cannot be designed for energy dissipation in plastic hinge at the base.

DESIGN & DETAILING OF DUCTILE WALLS

- Inelastic action limited to plastic hinge at base, so that cantilever relation between $q$ & $\mu$ can apply:
  - Wall provided with flexural overstrength above plastic hinge region (linear moment envelope with shift rule).
  - Design in shear for $V$ from analysis, times:
    \[ V_{Rd} = \frac{V_{Ed}}{V_{Ed}} \times \left( 1 + \frac{1}{2} \frac{V_{Ed}}{V_{Rd}} \right) \]
  - $M_{Rd}$ in plastic hinge region:
    \[ M_{Rd} = \frac{1}{2} \frac{V_{Ed}}{V_{Rd}} \times \left( 1 + \frac{1}{2} \frac{V_{Ed}}{V_{Rd}} \right) \]
  - $M_{Ed}$ in plastic hinge region:
    \[ M_{Ed} = \frac{1}{2} \frac{V_{Ed}}{V_{Ed}} \times \left( 1 + \frac{1}{2} \frac{V_{Ed}}{V_{Rd}} \right) \]
  - $S_T$ upper limit $T$ of constant spectral acc. region.
- In plastic hinge zone: boundary elements w/ confining reinforcement of effective mechanical volumetric ratio:
  \[ \rho_{\text{conf}} = 0.035 \]

LARGE LIGHTLY REINFORCED WALLS

- Wall system classified as one of large lightly reinforced walls if, in horizontal direction of interest:
  - At least 2 walls with $L > 4m$, supporting together > 20% of gravity load above (sufficient no. of walls / floor area & significant uplift of masses); if just one wall, $q=2$
  - Fundamental period $T_1 < 0.5s$ for fixity at base against rotation (: wall aspect ratio low)
- Systems of large lightly reinforced walls:
  - Only DC M ($q=3$). 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) storey 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.
(9) NDP (Nationally Determined Parameter) according to EC2. The Table gives the value recommended in EC2.

(1) $\gamma_p$ is the value of the curvature ductility factor that corresponds to the basic value, $\gamma_q$, of the behaviour factor used in the design.

(2) The minimum area of bottom steel, $A_{sw, min}$, is in addition to any compression steel that may be needed for the verification of the inflection point for the ULS in bending under the absolutely maximum negative (moment) moment for the “seismic design situation”, $M_{Ed}$.

(3) $h_b$ is the column depth in the direction of the bar, $\nu = \frac{N_{Ed}}{f_{yvd} d-w_0}$, for the minimum value of the axial load in the “seismic design situation”, with compression taken as positive.

(4) At a member end where the moment capacities around the joint satisfy $M_{Ed} \geq M_{Edw}$, replaced in the calculation of the design shear force, $V_{Ed}$ by $M_{Edw} \geq M_{Edw}$. 

(5) $z$ is the internal lever arm, taken equal to 0.6d or to the distance between the tension and the compression reinforcement, $d_{bw}$.

(b) Vmax is the absolutely largest of the two values, and is taken positive in the calculation of $V_{Ed}$ for the sign of $V_{Ed}$ is determined according to whether it is the same as that of $V_{Ed}$ or not.

Footnotes - Table on detailing & dimensioning primary seismic columns (previous page)

Footnotes - Table on detailing & dimensioning primary seismic columns (current page)

Footnotes - Table on detailing & dimensioning ductile walls (cont’d from previous page)

Footnotes - Table on detailing & dimensioning ductile walls (previous pages)
SECTIONS 6 AND 7: STEEL AND COMPOSITE
STEEL-CONCRETE BUILDINGS

A. Plumier
University of Liege
# EUROCODES

## Eurocode 8

### Sections 6 and 7.

**Steel and Composite Steel Concrete Buildings.**

**Prof. André PLUMIER**

### Eurocode 8 Sections 6 and 7.

1. Context and background of Sections on Steel and Composite Steel – Concrete.
2. Design Rules for Steel Structures
3. Design Rules for Composite Steel Concrete Structures
4. Dissemination

## 1. Context and Background

**Eurocode 8 rules on steel & composite structures**

1986. ECCS Design Recommendations
ECCS: European Convention for Constructional Steelwork
Aribert, Ballio, Mazzolani, Plumier, Sedlacek
1994. Eurocode 8 = ENV
Steel structures = ECCS Recommendations
Composite steel concrete: poor information
1994: Northridge earthquake 1995: Kobe earthquake
Many cracked steel connections

=>1994 – 2004: EU research projects
STEELQUAKE RECONS 3D Ispra test ICONS
Large testing installation funding: ECOEST ECOLEADER
+ US and Japan research => Improved design rules.

## Steel. EU Background. STEELQUAKE Project

Moment Resisting Connections of Steel Frames in Seismic Area
U.NAPOLI, U.SOFIA, NTUA, ATHENS, IST.LISBON, TIMISOARA, U.JULIJAHA, INCERC.TIMISOARA, ULIEGE.

Material behaviour. Ductility of members and connections.
Failure modes. Ductility demand. β-factors
Tests on subassemblages - full and partial strength connections.
Strain rate effects.

## Steel. US Background.

Following Northridge 1994: a strong push 200 million $ 10 years

**Output**

Material
Now equivalent to EN 1000

Design of connections for some time, 4 prescribed types or demonstration by tests
Now: open, requirements on plastic rotation capacity

Execution of connections => details
Cope holes
Reduced Beam sections etc
Reduced beam sections RBS or “dogbones”
- invented in Europe (1989)
- improved in US
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Comments

Steel connections damaged by hundreds as in 1994, Northridge earthquake: unlikely with Eurocodes 3 and 8 and European practice

EUROPEAN RESEARCH ACTIVITY ON SEISMIC BEHAVIOUR OF COMPOSITE MOMENT RESISTING FRAMES

1. Definition of problems & Data bank of references 1996-97
3. Definition of research needed 1997

Analysis of structures:
1) neglecting concrete How to disconnect and how far?
   Unsafe capacity design?

c) considering concrete. Problems:
   ► evaluate effective width of slab in the elastic & plastic field?
   ► define conditions of ductility of sections?
   ► behaviour factors?
   ► layout of shear connectors?
   ► contribution of transverse beam to M transfer?
   ► partial strength connections?

Ductility in beam ends.
Where to yield:
- Steel members: M+ OK M- buckling
- Concrete: Little, because degradation.
- Connectors: No, low cycle fatigue.
- Rebars: M- OK
- Connections Component method needed

SLAB DESIGN IN CONNECTION ZONE OF MOMENT FRAMES

Development of design approach for:
▶ dimensions of T section (steel profile + slab)
▶ specific “seismic” re-bars
▶ shear connectors on beams
▶ effective width and cyclic behaviour

Limited scope:
▶ rigid connections
▶ ductility by yielding of steel profile

Problems studied:
▶ density of slab reinforcements
▶ Contribution of transverse beam
▶ Disconnection of concrete
▶ Steel deck waves directions

Bi-directional cyclic response study of 3-D composite frame
= European JRC ISPRA test

A variety of design situations

In the concrete slab
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EUROCODES
Background and Applications

Composite. EU Background

Yielding, buckling and fracture of bottom flange

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EUROCODES
Background and Applications

Composite. EU Background

Shaking table & cyclic static tests on connections at NTUA.
Partial & full shear connection connection
Tests on shear connectors, Aribert-Lachal
Various loading histories
⇒design resistance of connectors
Compared Experimental Assessment of Steel and Composite Frames.
El Nashai & M. Tajji

Composites. EU Background

Cyclic test on Composite moment frame ULg – CEA

⇒ Redistribution of moments in beams
⇒ Density of shear connectors (2 density)
⇒ Slab design: reinforcements section and lay out
⇒ Effective width for I and M p
⇒ Low cycle fatigue of composite sections

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EUROCODES
Background and Applications

Composite. EU Background


DG XII - European Commission.
JRC Ispra, U.ROMA, U.PAVIA, U.PATRAS, LNEC, POLI.MILANO, GEO, INSA.LYON, ENS.CACHAN, U.LIEGE, TH.DARMSTADT, Imperial College, UP.MADRID.

+ invited contribution using mobility funds:
Trinity College Dublin
INSA de Rennes
University of Trento
University Federico II of Napoli
Politecnico di Milano.

ICONS Topic 4 Report = Background document to Eurocode 8
on composite steel concrete structures

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EUROCODES
Background and Applications

Composite. EU Background

6.1 General

Design Concepts

Ductility classes

Classical constructional steel
Charpy toughness: absorbed energy min 27J (at t°usage)
Distribution yield stresses and toughness such that:

Design of non dissipative structures
- requirements on steel material + bolts 8.8-10.9
- preferably in low seismicity regions
- K bracings may not be used

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EUROCODES
Background and Applications

2. Eurocode 8. Section 6: Steel Buildings

Required steel characteristics

- Classical constructional steel
- Change toughness: absorbed energy min 27J (at t°usage)
- Distribution yield stresses and toughness such that:
dissipative zones at intended places yielding at those places before the other zones leave the elastic range

Design                 Reality

Weak Beam Strong Column
Not respected!

Beam
f_{yb}=355 N/mm²

Correspondance between reality & hypothesis is required
### 6.2 Material

Conditions on \( f_y \) of dissipative zones

to achieve \( \gamma_{fmax} = \gamma_{fmax} \) to have a correct reference in capacity design

3 possibilities

a) Compute considering that in dissipative zones: \( f_{y,max} < 1.1 f_y \)

Ex: \( 5235, \gamma_y = 1.25 \) \( \Rightarrow f_{y,max} = 332 \text{ N/mm}^2 \)

An upper yield strength is specified for dissipative zones

b) Do design, based on a single nominal yield strength \( f_y \)

- use nominal \( f_y \) for dissipative zones

- use higher nominal \( f_y \) for non dissipative zones and connections

Ex: \( 5235 \) non dissipative zones

\( f_y = 355 \text{ N/mm}^2 \)

c) \( f_{y,max} \) of dissipative zones is measured

is the value used in design \( \Rightarrow \gamma_y = 1 \)

### 6.5.2 General Criteria

- Dissipative zones: adequate ductility and resistance

- Yielding, buckling, hysteretic behaviour do not affect stability.

- Elements in Compression or Bending

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<thead>
<tr>
<th>Ductility Class</th>
<th>Behaviour factor ( q )</th>
<th>Cross Sectional</th>
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<tbody>
<tr>
<td>DCH</td>
<td>( 2 \leq q &lt; 4 )</td>
<td>class 1</td>
</tr>
<tr>
<td>DCM</td>
<td>( 1.5 \leq q \leq 2 )</td>
<td>class 2</td>
</tr>
</tbody>
</table>

- Semi-rigid - partial strength connections:

- OK if:

  - adequate rotation capacity (corresponding deformations)
  - members framing into connections are stable
  - effect of connection deformations on drift analysis

- Non-dissipative parts and the elements connecting them to dissipative parts have overstrength (development of cyclic yielding of dissipative parts)

### 6.5 Connections in dissipative zones

- Flange weld or bolted non dissipative connections

- \( R_b \), resistance of the connection according to Eurocode 3

- \( R_u \), plastic resistance of the connected dissipative member

In EN1, \( R_b \) computed with "appropriate estimation \( f_y \) of the actual value of the yield strength" - "appropriate" was a problem

- The adequacy of design should be supported by experimental evidence...to conform with requirements defined...for each structural type and ductility class.

<table>
<thead>
<tr>
<th>Example: moment resisting frame plastic rotation capacity ( \delta_b )</th>
<th>( \delta_b = 8 \times 0.5 \text{ rad} )</th>
</tr>
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<tbody>
<tr>
<td>ductility class DCH</td>
<td>( \delta_b \geq 35 \text{ rad} )</td>
</tr>
<tr>
<td>DCM with ( \gamma = 2 )</td>
<td>( \delta_b = 25 \text{ rad} )</td>
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</tbody>
</table>

### 6.6 Moment frames. Design Criteria

Target global mechanism:

- plastic hinges in beams, not in columns

- (re)welding, at base, at top level, in 1 story buildings if in columns: \( N_d / N_{Ed} < 0.3 \)

General criterion:

- Beams

\[
\frac{M}{M_{pl,Rd}} \leq 0.5, \quad \frac{N}{N_{pl,Ed}} \leq 0.35
\]

- \( M_{Ed} = V_{Ed} / 1.5, N_{Ed} = 0.5 \)

- \( V_{Ed} \), capacity design to beam plastic moment \( M_{pl,Ed} \)

- \( V_{Ed} = V_{Ed,pl} + V_{Ed,cr} \)

- Columns

\[
M_{Ed} = (M_{pl,Ed} + M_{Ed,cr} / 1.5)
\]

- \( N_{Ed} = N_{Ed,pl} + 1.5 \cdot P_{Ed,cr} \)

- \( M_{pl,Ed} \), minimum section overstrength \( M_{pl,Ed} = M_{Ed,pl} / 1.5 \) of all beams dissipative zones.

- \( M_{Ed,cr} \), design bending moment in beam (principal moment)

-MpEd,cr, plastic moment
Rules for connections in dissipative zones

**Background and Applications**

- Homogeneous dissipative behaviour: section overstrength of diagonal
- Diagonals considered in the analysis under seismic action

**Beams and columns**

- Resist gravity loads
- Dissipative elements: diagonals in tension

**Diagonals**

- Standard model: only tension diagonals participate in structural resistance allowed to consider compression diagonal, if model OK
- Non-linear analysis

**Frames with diagonal bracings**

- Standard model: only tension diagonals participate in structural resistance allowed to consider compression diagonal, if model OK

**Frames with concentric bracings**

- Diagonals in tension & compression
- Compression and tension diagonals participate in structural resistance to seismic action

**Diagonals**

- N_{Ed}/N_{EdG3} = min \( \lambda \leq 2 \)
- \( \lambda = \frac{N_{Ed}}{N_{EdG3}} \)

**Connections in dissipative zones**

Connection design detail & Ductility classes: National Annexes

**Shear resistance of framed web panels**

- \( V_{wp,Ed} \)
- \( V_{wb,Rd} \)

**Frames with eccentric bracings**

Long links dissipate energy by yielding in bending intermediate links... shear

**Elements called “seismic links”**

- Designed to dissipate energy

**Categories:**

- Short links: dissipate energy by yielding in shear
- Long links: dissipate energy by yielding in bending
Stiffeners in links:
- Short links (shear on complete length)
- Long links (plastic hinges at both ends)

Members not containing seismic links:
- Short links
- Long links

Capacity design to the links:
- Checks: like for concentric bracings

6.9 Inverted pendulum structure
\[ \theta \leq 1.5 \]

6.10 Structures with concrete cores or concrete walls
Concrete structure is primary structure
- Dual structures
  - Moment-resisting frames and braced frames acting in the same direction:
    - Designed using a single q-factor.
  - Horizontal forces: distributed between frames according to their elastic stiffness.
- Mixed structures:
  - Moment-resisting frame with infills structurally disconnected from frame on lateral and top sides: design as steel structures.
  - Infills in contact: frame-infill interaction to take into account.

7.1 General

Design Options
- Steel only => Disconnection (defined)
- Composite => Rules EC4 + EC8

Design Concepts

<table>
<thead>
<tr>
<th>Type</th>
<th>q</th>
<th>Ductility class</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.5</td>
<td>DCL</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>DCM</td>
</tr>
<tr>
<td>3</td>
<td>5</td>
<td>DCH</td>
</tr>
</tbody>
</table>

Non dissipative structures: Eurocode 3 & 4
Requirements on steel material + bolts 8.8 - 10.9
Only in low seismicity regions
K bracings may not be used.

7.2 Materials

Steel: like for seismic design of steel structures
- \( f_y \) max (not more than 35% higher than the steel grade e.g. 235 for S 235)
- Toughness
Concrete: C20/25 < \( f_c \) < C40/50 => C30/35
Rebars: 2 classes (ductile-non ductile)
- \( f_u / f_y \) A%

7.3 Structural types

- Moment-resisting frames,
- Concentric braced frames,
- Eccentrically braced frames,
- Structural systems, R.C. walls behaviour

<table>
<thead>
<tr>
<th>Type</th>
<th>Behaviour factors q</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>DCH</td>
</tr>
<tr>
<td>2</td>
<td>DCM</td>
</tr>
<tr>
<td>3</td>
<td>DCH</td>
</tr>
</tbody>
</table>

Steel Concrete Structures.
Materials & Structural Types

- Composite MRF's
- Wall systems: Table

Analysis & Design. General

7.4. Structural Analysis
Scope: dynamic elastic
- \( E_s / E_c = 7 \)
- 2 stiffness of sections: effective concrete (\( M^+ \))
- Only rebar (\( M_r \))

7.5.2 General Criteria for Dissipative Structural Behaviour
Like steel 6.5.2

7.5.3 Plastic resistance of dissipative zones
Two plastic resistances considered:
- a lower bound in checks of sections of dissipative elements
- an upper bound for capacity design of elements & connections adjacent to the dissipative zone

\[ P_{ext} \] computed considering concrete and ductile steel components

\[ P_{int} \] computed considering all components in the section including non ductile ones (e.g. welded mesh).
**Composite connections in dissipative zones**

**Design objective:** integrity of concrete, yielding in steel
- Dissipative connections allowed
- Rebars in joint region: models satisfying equilibrium
- Yielding of rebars allowed
- In fully encased framed web panels: beam/column connections
- Panel zone resistance = concrete & steel shear panel resistance
- Yielding of rebars allowed
- In fully encased framed web panels of beam/column connections

\[ \text{Panel zone resistance} = \sum \text{concrete & steel shear panel resistance} \]

**Aspect ratio**

\[ \frac{h}{b_p} \]

**Composite connections in dissipative zones**

- Vertical rebars to take beam shear force
- If composite column, distribute beam shear between steel and concrete

**Members & classes of Ductility**

7.6 Roles for members, General

Local ductility of members in compression and/or bending

Steel and unencased steel parts of composite sections: EC3-EC4

**Limits for partially encased**

<table>
<thead>
<tr>
<th>Ductility Class</th>
<th>Structures</th>
<th>DCB</th>
<th>DCM</th>
<th>DCL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Partially Encased</td>
<td>DCH</td>
<td>4</td>
<td>1.5</td>
<td>1</td>
</tr>
<tr>
<td>with bars welded to flanges</td>
<td>DCM</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Filled Rectangular</td>
<td>DCL</td>
<td>2</td>
<td>4</td>
<td>6</td>
</tr>
</tbody>
</table>

**Steel beams with slab**

Design objective: - maintain integrity of slab
- yielding in steel section and/or rebars

Stiffness in plastic hinges
- P.N.A. = Plastic Neutral Axis
- \( \varepsilon_{cu} = \) concrete crushing strain
- \( \varepsilon_{apl} = \) plastic strain of steel

\[ \frac{x}{d} \leq \frac{\varepsilon_{cu}}{\varepsilon_{apl}} \]

**Steel beams with slab**

- Partial shear connection in dissipative zones of beams OK if
  - \( \phi \) in M>0 region, connection degree > 0.8
  - total resistance of connectors in M<0 region > plastic resistance of rebars.

- Reduction of shear resistance by a rib shape efficiency factor \( k_r \), if steel sheeting with ribs transverse to beams

\[ k_r = \begin{cases} 1 & \text{if } \phi \leq 0.8 \\ \frac{1}{2} & \text{if } \phi > 0.8 \end{cases} \]

Full shear connection required with non ductile connectors

**Columns**

- Columns generally not dissipative => EC 4 design
- Columns may be dissipative:
  - at ground level in moment frames
  - top & bottom of fully encased columns at any storey

Bond and friction shear resistance not reliable in cyclic conditions

- For shear resistance: strong restrictions (research needed)

- Partially encased: assumes concrete section resistance
- Fully encased: steel section resistance
- Partially encased: either steel or concrete considered resistance

Steel beams with slab

Definition of longitudinal & transverse elements + details in Moment Resisting Frame Structure

Slab « seismic » rebars

Moment Resisting Frames
Dissipative zones in beam with slab: vicinity of columns
"Seismic rebars" needed
Section and layout to achieve ductility => Annex C

Effective width b_eff

Effective width, b_eff

b_eff = 2 b_e

b_eff ≠ b_e for I elastic analysis

M_pl plastic resistance

Trans.beam b_e for M_Rd

b_e for I -Interior Present
M - (: 0,1 L_0,05 L

-Exterior Fixed to column M - (: 0,1 L_0,05 L

-Exterior Not active. M - (: 0,025 L

M+ : b_c/2 or h_c/2 0,025 L

Concrete disconnection rule
Beam plastic resistance: only steel if slab totally disconnected from steel frame in a diameter 2b_eff zone around a column

Moment Resisting Frames. Analysis.

In beams, two different stiffnesses:
\( E_1 \) part of spans submitted to \( M > 0 \) (slab uncracked)
\( E_2 \) part of spans submitted to \( M < 0 \) (slab cracked)

Or an equivalent inertia \( I_2 \):

\[ I_2 = 0.6 I_1 + 0.4 I_2 \]

Columns:

\[ (E_1 I_1 + 0.5 E_2 I_2) \]

\( E_s \) and \( E_{cm} \): modulus of elasticity for steel and concrete

\( I_a \), \( I_c \) and \( I_s \): moment of inertia of steel section, concrete and rebars

Composite frames may not be used as dissipative beams.

Concrete disconnection rule
Beam plastic resistance: only steel if slab totally disconnected from steel frame in a diameter 2b_eff zone around a column

Annex C

Seismic design of the slab reinforcements of composite T beams with slab in moment frames

General:
- Two conditions to ensure ductility in bending
- Avoid early buckling of steel section (classes of sections + \( x/d \))
- Avoid early crushing of slab concrete (\( x/d \) limit + rebars required)

EC4: negative moment & no transverse steel beam

Exterior Column Case

Façade beam to check in bending shear torsion

Rebars:
- Hairpin (EC4)
- Bars anchored in façade beam
3 Force Transfer Mechanisms of Slab Compression

Mechanism 1
Direct compression on column

\[ F_{RD1} = \frac{b}{c} \frac{d}{f} \]

Mechanism 2
Compression on column sides by concrete struts

\[ F_{RD2} = 0.7 \frac{h}{c} \frac{d}{f} \]

Mechanism 3
Connectors on facade steel beam

\[ F_{RD3} = n \times P_{RD} \]

where
- \( F_{RD1} \): Force due to direct compression on column
- \( F_{RD2} \): Force due to compression on column sides by concrete struts
- \( F_{RD3} \): Force due to connectors on facade steel beam
- \( P_{RD} \): Design resistance of one connector
- \( n \): Number of connectors in effective width

Maximum compression force \( F_{RD3} \) Transmitted if:

\[ F_{RD1} + F_{RD2} + F_{RD3} > b \frac{d}{f} \]

=> choose \( n \) to achieve adequate \( F_{RD3} \).

7.8 Composite concentrically braced frames
Concepts
- Yielding of diagonals in tension
- Tension only design & no composite braces

7.9 Composite eccentrically braced frames
- Dissipative action occurs through yielding in shear of links
- All other members remain elastic
- Links may be short or intermediate with a maximum length \( e \)
- Links are made of steel sections, possibly composite with slabs, not encased
- In a composite brace under tension, only the steel section is considered in the resistance of the brace
- Failure of connections is prevented

7.10 Systems made of reinforced concrete shear walls
- Composite with structural steel elements
Type 1 and 2 designed to behave as shear walls and dissipate energy in vertical steel sections and rebars

Type 1: Steel or composite frame with concrete infills
Type 2: Concrete walls reinforced by vertical steel sections

Shear carried by the reinforced concrete wall
Gravity and overturning moment carried by the wall acting compositely with the vertical boundary members
4. Dissemination

SEMINARS on Eurocode 8. Some possibilities.

In France, organised by:
- AFPS - Association Française de Génie Parasismique
  info on website: www.afps-seisme.org
- « Le Moniteur »
  info on website: www.groupelemoniteur.fr

In Norway:
- Tekna - The Norwegian Society of Chartered Scientific and Academic Professionals
  info on website: www.tekna.no

In Belgium:
- University of Liege, Department ARGENCO
  info on website: www.argenco.ulg.ac.be

Thank you for your attention!

4. Dissemination

BOOKS on Seismic Design closely related to Eurocode 8.

  Thomas Telford Publisher, 2005
  Explanations on Eurocode 8

  info on website: www.afps-seisme.org

- « Earthquake Resistant Steel Structures » 2008
  ArcelorMittal Technical Brochure free in French & English

- « Constructions en zone sismique » 2007
  Textbook for students University of Liege
  info on website: www.argenco.ulg.ac.be (in french, free Download)

4. Dissemination

Embedment length l required for embedding in walls of Type 3 designed to dissipate energy in the coupling beams:

\[ l = 1.5 \times \text{steel beam depth} \]

Rules on connections apply: face bearing plates, vertical rebars sections, etc.

7.11 Composite plate shear walls

- Designed to yield through shear of the steel plate
- Stiffened by encasement and attachment to reinforced concrete to prevent buckling of steel.

Thank you for your attention!
SECTIONS 8 AND 9: TIMBER AND MASONRY BUILDINGS

E. Carvalho
Gapres SA
**Eurocode 8**  
Timber and Masonry structures  
E C Carvalho, Chairman TC250/SC8

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**EN 1998-1**  
Section 8  
Specific rules for timber structures

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**Timber** is generally considered to be a good structural material for construction in seismic areas due to its:
- Light weight
- Reasonable strength in tension and in compression

both being relevant properties for the seismic performance of structures

---

<table>
<thead>
<tr>
<th>Structural material</th>
<th>Unit mass / (kg/m³)</th>
<th>Strength / Range of values (MPa)</th>
<th>Ratio f/ρ (10³ MPa / kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood</td>
<td>550</td>
<td>20 – 30</td>
<td>35 - 55</td>
</tr>
<tr>
<td>Structural steel</td>
<td>7800</td>
<td>275 - 365</td>
<td>35 - 45</td>
</tr>
<tr>
<td>Concrete</td>
<td>2400</td>
<td>19 - 30</td>
<td>10 - 30</td>
</tr>
<tr>
<td>Reinforced concrete</td>
<td>2400</td>
<td>2 – 3,5</td>
<td>0,8 - 1,5</td>
</tr>
<tr>
<td>Masonry</td>
<td>2100</td>
<td>4 - 8</td>
<td>1,9 - 3,8</td>
</tr>
</tbody>
</table>

---

The good performance of wood is reflected by its range of values $f/\rho$ similar to structural steel but
- Timber elements do not present large deformational ductility
- Response of timber elements up to failure is approximately linear elastic
- Collapse is sudden, mostly associated with defects inherent to the natural origin of timber

---

For timber structures EN 1998-1 distinguishes between dissipative and low-dissipative structural behaviour (as for concrete, steel and composite) However, in view of the limited ability of timber to behave nonlinearly:
- Energy dissipation should be mostly in connections
- Timber elements should respond linearly

Some (little) NL behaviour in compression perpendicular to grain may be expected.  
Tension perpendicular to grain markedly brittle.
Distinction between dissipative and low-dissipative structures depends mostly on the nature of the connections:

- Basic distinction in EN1998-1 between:
  - Semi-rigid joints
  - Rigid joints

Dissipation of energy in connections:

Two main sources:
1. Cyclic yielding of metallic (normally steel) dowel type fasteners of the connections (nails, staples, screws, dowels or bolts)
   - Stable mechanism with large hysteretic cycles
2. Crushing of the wood fibres bearing against the dowel
   - Thin hysteretic loops with significant degradation (due to the cavity being formed in front of the dowel)

Response of the connections depends mostly on the interaction between the two mechanisms:

Achieve good dissipative behaviour with proper balance between:
- Wood crushing
- Dowel yielding

More significant parameter: Slenderness of the dowel type element

Ductility Classes

Dissipative structures
- Ductility Class Medium (DCM)
- Ductility Class High (DCH)

Low-dissipative structures
- Ductility Class Low (DCL)

Ductility classification depends mostly on:
- Structural type/redundancy
- Nature of structural connections

Determine properties of dissipative zones by testing (prEN 12512) or Use deemed to satisfy rules (in EN 1998-1)

Materials and properties of dissipative zones:
- General requirements as in EN 1995-1-1 (and EN 1993-1 for steel elements)
- Additional requirements for DCM and DCH
  - Glued joints may not be considered dissipative
  - Density of particle board panels $\geq 650$ kg/m$^3$
  - Thickness of particle board and fibre board panels $\geq 13$ mm
  - Thickness of plywood sheathing $\geq 9$ mm

Nailed shear panel systems superior to conventional bracing
Avoid pull out of nails under transverse loading (avoid smooth nails or apply provisions against withdrawal)
**Behaviour factors DCM and DCH (q = 1.5 for DCL)**

<table>
<thead>
<tr>
<th>Structural types</th>
<th>DCM</th>
<th>DCH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall panels with glued diaphragms connected with nails and bolts</td>
<td>Glued panels q = 2.0</td>
<td>Nailed panels q = 3.0</td>
</tr>
<tr>
<td>Wall panels with nailed diaphragms connected with nails and bolts</td>
<td></td>
<td>Nailed panels q = 5.0 (q = 4.0)</td>
</tr>
<tr>
<td>Trusses</td>
<td>Doweled and bolted joints q = 2.0</td>
<td>Nailed joints q = 3.0</td>
</tr>
<tr>
<td>Mixed structures with timber framing and non-load bearing infills</td>
<td>q = 2.0</td>
<td></td>
</tr>
<tr>
<td>Hyperstatic portal frame with doweled and bolted joints</td>
<td>μ ≥ 4</td>
<td>q ≥ 2.5</td>
</tr>
<tr>
<td></td>
<td>q ≥ 6</td>
<td>q = 4.0</td>
</tr>
</tbody>
</table>

( ) for lower slenderness dowels

**Underlying requirements for the allowed q factors for timber structures:**

- **Buildings regular in elevation**
  - (if non-regular, reduce by 20% the values indicated for q)

  Dissipative zones able to sustain, without strength degradation larger than 20%, 3 fully reversed cycles at ductility demand of:
  - μ = 4 for DCM
  - μ = 6 for DCH

  Rotational ductility in portal frames or distortional displacement ductility in shear panels

**Deemed to satisfy rules for dissipative zones**

- Fasteners in doweled, bolted and nailed timber-to-timber and steel-to-timber joints:
  - Slenderness: ε/d ≥ 10 (or 8)
  - Diameter: d ≤ 12 mm

- In shear walls and diaphragms:
  - Wood-based material
  - Slenderness of nails: ε/d ≥ 4 (or 3)
  - Nail diameter: d ≤ 3.1 mm

  More demanding than EN 1995 (allows dowels and bolts up to 10 mm)

  Allowance for lower slenderness (values) with reduction of the q factor

**Detailing for DCM and DCH**

- **Connections**
  - Connections in compression members prevented from separating
  - Tight fitted bolts and dowels in pre-drilled holes.
  - Maximum diameter of bolts and dowels: 16 mm (larger diameters allowed with toothed ring connectors for confinement)
  - Dowels, smooth nails and staples with provisions against withdrawal
  - Provisions against splitting (metal or plywood plates)

**Safety verifications**

- Resistance models in accordance with EN 1995-1-1 with k_{mod} values for instantaneous loading

  - **DCM**
    - Partial factors γ_{M} as for the fundamental load combination
  - **DCM and DCH**
    - Partial factors γ_{M} = 1.0 as for the accidental load combination (important difference from concrete, steel and timber)
    - Provide sufficient overstrength to elements connected to dissipative zones
    - Increase partial factors by 1.3 in carpenter joints (to avoid brittle failures)
EN 1998-1

Section 9

Specific rules for masonry structures

Masonry is generally considered to present specific problems for construction in seismic areas due to its:

- Weight
- Poor strength in tension
- Brittle response in tension and compression

_all being relevant properties for the seismic performance of structures_

The specific problems of the seismic performance of masonry is reflect by the low of values \( f/\rho \) both in compression and tension

However

Masonry structures may present:

- Densely distributed walls
- Good robustness (if all elements are appropriately tied together)
- Dissipation of energy in a distributed fashion by widespread cracking (which has to be controlled either by tying or by distributed reinforcement)

Hence the seismic behaviour of masonry structures may be very much influenced by design

For masonry structures EN 1998-1 distinguishes between:

- Unreinforced masonry construction
- Confined masonry construction
- Reinforced masonry construction

Unreinforced masonry akin to the concept of Low-Dissipative structures

Use of unreinforced masonry (EN1996) is recommended only for Low seismicity cases (recommended NDP)

Unreinforced masonry (EN1998-1) may not be used if \( a_s \geq 0.20 \) g (recommended NDP that should depend on the requirements for materials properties)

Materials and bonding patterns

- General requirements as in EN 1996-1-1
- Additional requirements (all recommended NDPs)
  - Minimum strength of units
    - Normal to bed face: \( f_{ub} \geq 5 \) N/mm²
    - Parallel to bed face: \( f_{ub} \geq 2 \) N/mm²
  - Minimum strength of mortar
    - Unreinforced and confined: \( f_{um} \geq 5 \) N/mm²
    - Reinforced: \( f_{um} \geq 10 \) N/mm²
  - Masonry bond for perpend joints
    - Fully grouted joints with mortar
    - Ungrouted joints
  - Ungrouted with mechanical interlocking

Large number of NDPs reflect and is intended to accommodate the large variety of materials and construction practices for masonry across Europe

<table>
<thead>
<tr>
<th>Structural material</th>
<th>Unit mass ( \rho ) (kg/m³)</th>
<th>Strength ( f ) Range of values (MPa)</th>
<th>Ratio ( f/\rho ) ( (10^{-3} ) MPa / kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood (Compression)</td>
<td>550</td>
<td>20 – 30</td>
<td>35 – 55</td>
</tr>
<tr>
<td>Structural steel (Compression and tension)</td>
<td>7800</td>
<td>275 - 355</td>
<td>35 - 45</td>
</tr>
<tr>
<td>Concrete (Compression)</td>
<td>2400</td>
<td>25 - 80</td>
<td>10 - 30</td>
</tr>
<tr>
<td>Tension</td>
<td>2400</td>
<td>2 – 3.5</td>
<td>0.8 - 1.5</td>
</tr>
<tr>
<td>Reinforced concrete (Bending)</td>
<td>2500</td>
<td>10 - 25</td>
<td>4 - 10</td>
</tr>
<tr>
<td>Masonry (Compression)</td>
<td>2100</td>
<td>4 – 8</td>
<td>1.9 - 3.8</td>
</tr>
<tr>
<td>Tension</td>
<td>2100</td>
<td>0.3 – 0.5</td>
<td>0.1 - 0.2</td>
</tr>
</tbody>
</table>
Upper limits of behaviour factors (recommended NDPs)

<table>
<thead>
<tr>
<th>Type of construction</th>
<th>$q$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced masonry in accordance with EN 1996 alone</td>
<td>1.5</td>
</tr>
<tr>
<td>Unreinforced masonry in accordance with EN 1998</td>
<td>1.5 – 2.5</td>
</tr>
<tr>
<td>Confined masonry</td>
<td>2.0 – 3.0</td>
</tr>
<tr>
<td>Reinforced masonry</td>
<td>2.5 – 3.0</td>
</tr>
</tbody>
</table>

Structural analysis

- Uncracked or cracked (recommended) stiffness
- Cracked stiffness approx. 50%
- If appropriate (existence of coupling beams/spandrels) a frame analysis may be used
- Redistribution of base shear among walls

Construction rules and geometric conditions

- **General**
  - Connections between floors and walls
  - Floor continuity and effective diaphragm effect
    - **Shear walls**
      - Minimum effective thickness $t_{ef,min}$
      - Maximum height to thickness ratio $(h_{ef}/t_{ef})_{max}$
      - Minimum length to height ratio $(l/h)_{min}$

Geometric requirements for shear walls

<table>
<thead>
<tr>
<th>Masonry type</th>
<th>$t_{eff,min}$ (mm)</th>
<th>$(h_{ef}/t_{ef})_{max}$</th>
<th>$(l/h)_{min}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced, with natural stone units</td>
<td>350</td>
<td>9</td>
<td>0.5</td>
</tr>
<tr>
<td>Unreinforced, with any other type of units</td>
<td>240</td>
<td>12</td>
<td>0.4</td>
</tr>
<tr>
<td>Unreinforced, with any other type of units, in cases of low seismicity</td>
<td>170</td>
<td>15</td>
<td>0.35</td>
</tr>
<tr>
<td>Confined masonry</td>
<td>240</td>
<td>15</td>
<td>0.3</td>
</tr>
<tr>
<td>Reinforced masonry</td>
<td>240</td>
<td>15</td>
<td>No restriction</td>
</tr>
</tbody>
</table>

Additional requirements for unreinforced masonry

- Horizontal concrete beams or steel ties at floor levels in all walls
- Concrete beams reinforcement with at least 2 cm²

Additional requirements for confined masonry

- Horizontal and vertical confining elements bonded together and cast against the masonry
- Confining elements larger than 150 mm (interconnect the two masonry leaves in case of double-leaf masonry)
- Longitudinal reinforcement of confining elements with at least 3 cm² or 1% of cross sectional area
- Stirrups $d \geq 5$ mm spaced ≤ 150 mm
- Reinforcing steel Class B or C (EN 1992-1-1)
- Lap splices longer than 60 diameters
Additional requirements for confined masonry (cont.)

- **Vertical** confining elements:
  - At both edges of walls
  - At both sides of openings larger than 1.5 m²
  - Within wall spaced, at most, 5 m
  - At wall intersections more than 1.5 from other confining element

- **Horizontal** confining elements:
  - At every floor level
  - With vertical spacing not larger than 4 m

Additional requirements for reinforced masonry

- **Horizontal** reinforcement in bed joints spaced not more than 600 mm
- **Horizontal** reinforcement not less than 0.05% of cross sectional area of wall
- **Vertical** reinforcement (in pockets or holes in the units) not less than 0.08% of cross sectional area of wall
- **Reinforcing steel** Class B or C (EN 1992-1-1)
- **Lap splices** longer than 60 diameters

Additional requirements for reinforced masonry (cont.)

- **Vertical** reinforcement not less than 200 mm² and provided with 5 mm stirrups at 150 mm spacing:
  - At both free edges of all walls
  - At every wall intersection
  - Within wall spaced, at most, 5 m

Safety verifications

- **Resistance models** in accordance with EN 1996-1-1
- Specific partial factor \( \gamma_m \) for masonry and \( \gamma_s \) for steel to be defined as NDPs

  Recommended values:
  - For masonry: \( \gamma_m = \frac{2}{3} \) of the value from EN 1996-1-1, but not less than 1.5
  - For steel: \( \gamma_s = 1.0 \)

“Simple masonry buildings”

- Concept applicable **only** for Importance Classes I and II
- Explicit safety verification not mandatory
- Safety verification **implicit** with the fulfilment of some geometrical conditions

“Simple masonry buildings” conditions:

- Maximum number of storeys and minimum relative area of walls depending on \( a_s \) and type of masonry
- **Regularity** in plan
- **Compact** in plan shape
- **Shear walls** in both orthogonal directions and approximately symmetrical
- **Two parallel walls** in both orthogonal directions, placed close to the edges of the building
- **75% of weight** supported by shear walls
- Variation of mass and wall area between adjacent storeys limited to 20% (recommended NDP)
SECTION 10: BASE ISOLATION

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Eurocode 8
Base Isolation (for buildings)

E C Carvalho, Chairman TC250/SC8

EN1998-1: General rules, seismic actions and rules for buildings
Section 10 - Base isolation
Deals with seismically isolated structures (specific rules for buildings)

Aim:
Reduce the seismic response of the lateral-force resisting system by:
- Increasing the Fundamental Period
- Modifying the shape of the fundamental mode
- Increasing the damping
- Combining various effects

Distributed energy dissipation systems not covered by Section 10
Specific rules for Bridges in EN 1998-2

Base isolation strategies

- Period elongation
  - Reduction of spectral accelerations
  - Increase of displacements (mostly occurring in the isolation system)
- Force limitation
  - Reduction of the forces transmitted to the structure
  - Displacements controlled by damping (provided by the isolation system)
  - Design of the structure in terms of the capacity design concept

In both cases with increased energy dissipation capacity

Main Definitions

- Isolation system – Collection of components used for providing seismic isolation, which are arranged over the isolation interface.
- Isolation interface – Surface which separates the substructure and the superstructure and where the isolation system is located. Normally at the base of buildings, tanks and silos or between piers and deck in bridges.
- Isolator units – Elements constituting the isolation system.
- Full isolation – The structure is fully isolated if, in the design seismic situation, it remains within the elastic range.

Types of Isolator units

- Laminated elastomeric bearings (quasi-elastic response with equivalent damping from 5% - LDRBs to 20% - HDRBs)
- Elastoplastic devices (hysteretic response of metals)
- Viscous or friction dampers
- Pendulums (low friction sliders – stainless steel/PTFE)

Functions/capabilities

- Vertical-load carrying capacity
- Energy dissipation capacity
- Recentering capability
- Lateral restraint

Design Objectives as for non base-isolated structures:

In the event of earthquakes:
- Human lives are protected
- Damage (structural, non-structural and to contents) is limited
- Structures important for civil protection remain operational

Fundamental requirements as for non base-isolated structures:

- No-collapse requirement
- Damage limitation requirement

But an increased reliability is required for the isolating devices.
Magnification factor $\gamma_X$ on the seismic displacements of each unit (NDP recommended value $\gamma_X = 1.2$)
Compliance criteria

For the DLS, lifelines crossing joints remain elastic.

For the ULS, gas lines and other hazardous lifelines crossing joints accommodate the relative displacements including the magnification factor $\gamma$.

Interstorey drift limited as for non base isolated buildings.

At ULS, substructure and superstructure remain elastic.

General Design Provisions

Arrangement of devices allowing for inspection, maintenance and replacement.

Protection of devices against fire, chemical or biological attack.

Distribution of devices to minimize torsion effects.

Sufficient stiffness of structure above and below the isolation interface to avoid differential movement.

Sufficient space around the devices to allow free movement with no hammering.

Seismic action

Two horizontal and vertical components acting simultaneously.

Elastic spectrum (and alternative representations) as for non-base isolated buildings.

Site specific spectra required for Class IV buildings if distance from potentially active fault with a $M_s \geq 6.5$ is less than 15 km.

(When applicable) behaviour factor $q = 1.0$ except for the superstructure where $q = 1.5$ may be used.

Properties of isolation system

For analysis purposes use the most unfavourable values of mechanical properties (account for rate of loading, effect of vertical load, temperature and aging).

Maximum stiffness and Minimum damping for the evaluation of accelerations and forces.

Minimum stiffness, damping and friction for the evaluation of displacements.

In Class I and II buildings mean values may be used provided that extreme values are within 15% of the mean.

Structural Analysis

Equivalent linear analysis

Simplified linear analysis

Modal simplified linear analysis

Modal linear analysis

Time history analysis

Equivalent linear analysis

Use equivalent stiffness and damping at displacement $d_{dc}$ (evaluated in an iterative procedure):

Conditions (for “equivalence”):

• Effective (secant) stiffness of the Isolation System at total design displacement is not less than 50% of the effective stiffness at $0.2d_{dc}$.

• Effective damping of the Isolation System does not exceed 30%.

• The force-displacement characteristics of the Isolation System do not vary more than 10% due to the rate of loading and the vertical load variation (in the range of design values).

• The increase of force in the Isolation System for displacements between $0.5d_{dc}$ and $d_{dc}$ is not less than 2.5% of the total gravity load above the system (to provide recentering capability).
**Simplified (static) linear analysis**

The structure is assumed to behave like a SDOF system in both horizontal directions (superstructure acting as a rigid block).

**Conditions (for “simplification”):**

- Maximum eccentricity between stiffness of the Isolation System and centre of mass of the structure does not exceed 7.5% of plan.
- Distance from potentially active faults with \( \text{MS} \geq 6.5 \) greater than 15 km.
- Maximum plan dimension not greater than 50 m.
- Rigid substructure (to minimise differential displacements).
- All devices above elements of substructure that support vertical load.
- Effective period in the range: \( 3T_f \leq T_{eff} \leq 3 \) s \( (T_f \) is the period for the same structure with a fixed base). 

**Modal simplified linear analysis**

Applicable when the conditions for the simplified (static) analysis are met except the one for maximum eccentricity.

The structure is assumed to behave like a 3DOF system (superstructure acting as a rigid block with its motion described by 2 horizontal displacements and the rotation about the vertical axis).

**Modal linear analysis**

Applicable when the conditions for the simplified analysis are not met.

A linear model of the complete structural system including both the stiffness of the superstructure (according to the modelling rules applicable to “conventional” structures) and the “equivalent” stiffness and damping properties of the isolator units should be used for a complete modal analysis.

**Time history analysis**

Can always be used (any type of structure and isolation system).

- Mandatory if it is not possible to model the Isolation System with an equivalent linear system.
- The superstructure may be modelled elastically (for full isolation).
- The constitutive model of the devices shall represent its actual behaviour in the range of deformations and velocities associated with the seismic design situation (use the most unfavourable values).
- Seismic action represented as defined for time history analysis of “conventional” structures (alternative representations of the seismic action).
PART 2: BRIDGES

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**EUROCODE EN1998-2**  
SEISMIC DESIGN OF BRIDGES  
*Basil Kolias*

### Background and Applications

**Basic Requirements**

- **Non-Collapse**
  - Retain structural strength + residual resistance for emergency traffic.
  - Limit damage to areas of energy dissipation.

- **Damage Minimization**
  - Under probable seismic effects.

### Analysis Methods

#### Equivalent Linear Analysis:
- Elastic force analysis (response spectrum) forces from unlimited elastic response divided by global behaviour factor = q.

\[
\text{design spectrum} = \frac{\text{elastic spectrum}}{q}
\]

#### Stiffness of Ductile Elements:
- Secant stiffness at the theoretical yield

#### Non-linear Dynamic Time-History Analysis:
- In combination with response spectrum analysis without relaxation of demands.
- For irregular bridges.
- For bridges with seismic isolation.

#### Non-linear Static Analysis (Push-Over):
- For irregular bridges.

### Seismic behaviour of bridges

**Ductility Classes**

- **Limited Ductile Behaviour:**  
  \[ q \leq 1.50 \]

- **Ductile Behaviour:**  
  \[ 1.50 < q \leq 3.50 \]
Compliance Criteria for Elastic Analysis

- **Limited Ductile Behaviour:**
  - Section verification with seismic design effects $A_{Ed}$
  - Verification of non-ductile failure modes (shear and soil) with elastic effects $gA_{Ed}$ and reduction of resistance by $\gamma_{Ed} = 1.25$

- **Ductile Behaviour:**
  - Flexural resistance of plastic hinge regions with design seismic effects $A_{Ed}$
  - All other regions and non-ductile failure modes (shear of elements & joints and soil) with capacity design effects $A_C$
  - Local ductility ensured by special detailing rules (mainly confinement).

Compliance Criteria for Elastic Analysis

- **Control of Displacements:**
  - Assessment of seismic displacement $d_e$
    
    $d_e = \eta \mu d_{Ed}$
    
    $d_{Ed} = \text{result of elastic analysis}$
    
    $\eta = \text{damping correction factor}$
    
    $\mu = \text{displacement ductility as follows:}$
    
    when $T \geq T_0 = 1.25 T_C$:
    
    $\mu = q$
    
    when $T < T_0$:
    
    $\mu = (q-1)T_0 / T + 1 \leq 5q - 4$

- **Provision of adequate clearances for the total seismic design displacement:**
  
  $d_{Ed} = d_e + d_o + 0.5d_T$
  
  $d_o$ due to permanent and quasi-permanent actions.
  
  $d_T$ due to thermal actions.
  
  For roadway joints: 40% $d_e$ and 50% $d_T$

Compliance Criteria for Non-linear Analysis

- **Chord rotation:**
  
  $\theta = \theta_y + \theta_p$

- **Ductile Members:**
  
  Deformation verification
  
  Plastic chord rotations of plastic hinges:
  
  $\theta_{p,E} \leq \theta_{p,d}$, $\theta_{p,d} = \theta_{p,u} / Y_{RP}$, $Y_{RP} = 1.40$
  
  $\theta_{p,u}$ = probable (mean) capacity from tests or derived from ultimate curvatures
Non-ductile members:
- **Force verification** as in elastic analysis for regions outside plastic hinges and non-ductile failure modes, with capacity design effects replaced by:
  \[ Y_{R,\text{Bet}} A_{Ed} \]
  with \( Y_{R,\text{Bet}} = 1.25 \)
- Design resistances:
  \[ R_d = R_k / \gamma_M \]

Seismic Action
- **Two types of elastic response spectra:**
  - Type 1 and 2.
  - 5 types of soil: A, B, C, D, E.
- **4 period ranges:**
  - short, constant acceleration, velocity and displacement.
- **Design spectrum = elastic spectrum / \( q \).**
- **3 importance classes:**
  - \( \gamma_I = 1.3, 1.0, 0.85 \).

Seismic Action: Spatial Variability
- **Spatial variability model should account for:**
  - Propagation of seismic waves
  - Loss of correlation due to reflections/refractions
  - Modification of frequency content due to different mechanical properties of foundation soil

Displacement sets defined from:
- \( d_g = 0.025 a_g S T_c T_D \): max particle displ. corresponding to the ground type (EC8-1)
- \( L_g \) is the distance beyond which seismic motion is completely uncorrelated

<table>
<thead>
<tr>
<th>Ground Type</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>( L_g (m) )</td>
<td>600</td>
<td>500</td>
<td>400</td>
<td>300</td>
<td>500</td>
</tr>
</tbody>
</table>
**Seismic Action: Spatial Variability**

**Displacement set A**

- uniform expansion/contraction
  - displacement of support $i$ relative to support 0
  
  \[ d_i = \varepsilon_i L_s \leq d_s \sqrt{2} \]
  \[ \varepsilon_i = \frac{d_s \sqrt{2}}{L_s} \]

**Displacement set B**

- with opposite directions at adjacent piers

\[ \Delta d_i = \pm \beta \varepsilon_i L_{aw,i} \]
\[ d_i = \pm \Delta d_i / 2 \]
\[ \beta_i = \begin{cases} 0.5 & \text{same ground type} \\ 1.0 & \text{different ground type} \end{cases} \]

**Regular / Irregular Bridges**

- Criterion based on local required force reduction factors $r_i$ of the ductile members $i$:
  
  \[ r_i = q \frac{M_{Ed,i}}{M_{Rd,i}} = q \times \text{Seismic moment} / \text{Section resistance} \]

- A bridge is considered regular when the "irregularity" index:
  
  \[ \rho_i = \frac{\max(r_i)}{\min(r_i)} \leq \rho_0 = 2 \]

- Piers contributing less than 20% of the average force are not considered

**Capacity Design Effects**

- Correspond to the section forces under permanent loads and a seismic action creating the assumed pattern of plastic hinges, where the flexural overstrength:
  
  \[ M_o = \gamma_o M_{Rd} \]

  has developed with:

  \[ \gamma_o = 1.35 \]

- Simplifications satisfying the equilibrium conditions are allowed.

**Detailing Rules**

**Confinement reinforcement**

- Increasing with:
  
  - Normalised axial force: $\eta_k = \frac{N_{Ed}}{(A_f f_{Ed})}$
  - Axial reinforcement ratio $\rho$ (for $\rho > 0.01$).

- Not required for hollow sections with:
  
  - $\eta_k \leq 0.20$ and restrained reinforcement.

- Rectangular hoops and cross ties or Circular hoops or spirals or overlapping spirals
**Detailing Rules**

### Restraining of axial reinforcement against buckling
- Max support spacing:
  \[ s_L \leq \delta \phi_L \leq 5 \leq \frac{2.5 (f_y / f_p) + 2.25}{6} \]
- Minimum amount of transverse ties:
  \[ A_t/s_T = \sum A_s f_{ys} / 1.6 f_{yt} \text{ (mm}^2/m) \]

### Hollow piers
- In the region of the plastic hinges
- Limitation of wall slenderness ratio:
  \[ b/t \text{ or } D/t \leq 8 \]

### Pile foundations
- Rules for the location and required confinement of probable plastic hinges

**Detailing Rules**

### Bearing and seismic links.
- Holding down devices.
- Shock transmission units (STU).
- Min. overlap lengths at movable supports.
- Abutments and retaining walls.
- Culverts with large overburden.

\[ \gamma_s = \frac{v_g}{v_s} \]

**Bridges with Seismic Isolation**

### The isolating system arranged over the isolation interface reduces the seismic response by:
- Either lengthening of the fundamental period.
- Or increasing of the damping.
- Or (preferably) by combination of both effects.

**Design properties of the isolating system**
- Nominal design properties (NDP) assessed by prototype tests, confirming the range accepted by the Designer.
- Design is required for:
  - Upper Bound design properties (UBDP).
  - Lower Bound design properties (LBDP).
- Bounds of Design Properties result either from tests or from modification factors.

**Analysis methods**
- Fundamental or multi mode spectrum analysis (subject to specific conditions).
- Non-linear time-history analysis.
**Bridges with Seismic Isolation**

### Substructure

- Design for limited ductile behaviour: \( q \leq 1.50 \)

### Compliance criteria

- **Isolating system**
  - Displacements increased by factor: \( \gamma_{IS} = 1.50 \)
  - Sufficient lateral rigidity under service conditions is required.

### Lateral restoring capability (revision)

- **Governing parameter:** \( d_{cd}/d_r \)
  - \( d_{cd} = \) design displacement
  - \( d_r = F_0/K_p = \) maximum residual displ.

- **Condition (1):** insignificant residual displ.: \( \frac{d_r}{d_f} < \delta \)
  \( \delta = 0.5 \)

- **Condition (2):** adequate capacity for accumulated residual displ.: \( d_{cd} \geq d_u + \gamma_d d_{cd} \delta \)
  \( \rho_\delta = 1 + 1.35 \left( \frac{d_r}{d_f} \right)^4 \left( 1 + 80 \frac{d_r}{d_f} \right)^{-1} \)
  \( \gamma_{ls} = 1.20 \)

### Seismic Deformation Capacity of Piers

- **Ultimate Displacement**

- **Chord rotation** \( \theta_u = \theta_f + \theta_p \)

- Plastic chord rotation \( \theta_p \) derived:
  - Directly from appropriate tests
  - From the curvature, by integration
Deformation Capacity of Piers

- Ultimate curvature: $\phi_u = \frac{\varepsilon_{su} - \varepsilon_{cu}}{d}$
  - Reinforcement: $\varepsilon_{su} = 0.075$ (EN1992-1-1)
  - Unconfined concrete: $\varepsilon_{cu} = -0.0035$ (EN1992-1-1)
  - Confined concrete:
    $$\varepsilon_{cu,c} = -0.004 - \frac{1.4\rho f_{ym} \varepsilon_{su}}{f_{cm,c}}$$

- Mean material properties
  - Reinforcement
    - $f_{ym}/f_{yk} = 1.15$, $f_{sm}/f_{sk} = 1.20$, $\varepsilon_{su} = \varepsilon_{uk}$
  - Concrete
    - $f_{cm} = f_{ck} + 8$ (MPa), $E_{cm} = 22(f_{cm}/10)^{0.3}$
  - Stress-strain diagram of concrete
    - Unconfined concrete: $\varepsilon_{cf} = -0.007f_{cm}^{0.3}$

Confined concrete - Mander model

Chord rotation: $\theta_u = \theta_y + \theta_{p,u}$

$$\theta_{p,u} = (\Phi_{u} - \Phi_{y})L_p\left(1 - \frac{L_p}{2L}\right)$$

Calibration with test results

- Database:
  - 64 tests on R/C pier elements.
    - 31 circular, 25 rectangular, 8 box sections
  - Curvature analysis for each test specimen
  - Non-linear regression for the coefficients of:
    $$L_p = 0.10L + 0.015f_{yk}d_s$$
Non-linear Static Analysis

- Based on the equal displacements rule
- Analysis directions:
  - x: Longitudinal
  - y: Transverse

Load distribution

- Load increment at point $i$ at step $j$
  - $\Delta F_{ij} = \Delta \alpha_i G_i \zeta_i$
  - distribution constant along the deck: $\zeta_i = 1$
  - distribution proportional to first mode shape

Horizontal load increased until the displacement at the reference point reaches the design seismic displacement of elastic response spectrum analysis ($q = 1$), for $Ex + 0.3Ey$ and $Ey + 0.3Ex$

Reference point is the centre of mass of the deformed deck

Thank you !!!
PART 5: FOUNDATIONS, RETAINING STRUCTURES AND GEOTECHNICAL ASPECTS

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Object and salient characteristics of EN1998-5:2003

The norm:
- "rules for the siting and foundation soil of structures for earthquake resistance", and
- "covers the design of different foundation systems, ... of earth retaining structures ... under seismic actions"

From Part 1: "... It shall be verified that both the foundation elements and the foundation soil are able to resist the action effects resulting from the response of the superstructure without substantial permanent deformations."

Salient characteristics and innovative aspects
- Complementarity with Eurocode 7 (EN 1997-1), which does not cover earthquake resistant design.
- Introduction and use of dynamic soil properties ($\tau_{\text{cyc}}$, shear wave velocity $v_s$, and damping) in addition to standard static properties ($\tan \phi'$, $c_u$, $q_u$).
- Different approaches to safety and strength verifications, depending on seismicity level and type of soil.
- In situ investigations, or gathering of reliable equivalent data, to determine the elastic design response spectrum.
- Recognition of seismically-induced permanent ground deformations as a design criterion.

Ground properties
- Static undrained parameter values ($c_u$, $\tan \phi'$) can generally be used for strength verifications, with recommended values of material partial factors ($\gamma_m$).
- For cohesionless soil the strength parameter is the cyclic undrained shear strength $c_{u,cyc}$ which should take the possible pore pressure build-up into account.
- The soil shear modulus:
  $$G = \frac{W}{g} v_s^2$$
  and the soil damping ratio are introduced, for use in SSI calculations, as well as their dependence on the seismic shear strain in the soil (through the design ground acceleration).
- For the evaluation of the liquefaction potential, the cyclic soil resistance against liquefaction (based on field performance in past earthquakes) is used, which depends on SPT blowcount or cone penetration resistance.

Design seismic action: dependence on ground type
The reference ground motion model at a point on the surface is the acceleration elastic response spectrum:

$$S_e(T) = a_g f(T, S, T_B, T_C, T_D)$$

The dependence is established through:
- The constant soil factor $S$, which does not modify the spectrum shape but only amplifies it, and takes a single value for each ground type.
- The "control periods" $T_B, T_C, T_D$ which modify the shape by enlarging the spectral plateau.

Remark: it would be desirable to make $S$ a smoothly varying function of $V_{30}$, e.g.:

$$S = (V_{30}/V_s)^{b_s}$$

with $b_s$ and $V_s$ constants.
Identification of ground types

\[ \sum_{i=1}^{N} V_{i} = 30 \]

Weighted \( V \)

Particular cases:
- Types S1, S2
- Special studies

Seismic action: dependence on ground type

In seismic geotechnical verifications, such as:
- Slope stability
- Liquefaction hazard occurrence
- Stability of retaining works

The seismic action is directly determined by the design ground acceleration \( a_{gS} \), multiplied by a reductive coefficient, as follows:
- \( 0.5 \frac{a_{gS}}{g} \) seismic coefficient for pseudo-static slope stability verifications
- 0.65 \( a_{gS} \) effective acceleration for checking the liquefaction potential in a saturated sand deposit
- \( \left[ \frac{(a_{g}/g)_{S}}{r} \right] \) seismic coefficient for computing the dynamic thrust in the pseudo-static verification of a retaining work.

Requirements for siting and foundation soils

Amplitude response (averaged over frequency) along slopes of different geometry

Example of simplified 3D representation of a settlement on a crest: Baiardo

Requirements of the construction site: Proximity to seismically active surface faults

1. Buildings of importance classes II, III, IV defined in EN 1998-1:2004, 4.2.5, shall not be erected in the immediate vicinity of tectonic faults recognised as being seismically active in official documents issued by competent national authorities.
2. An absence of movement in the Late Quaternary may be used to identify non-active faults for most structures that are not critical for public safety.
3. Special geological investigations shall be carried out for urban planning purposes and for important structures to be erected near potentially active faults in areas of high seismicity, in order to determine the ensuing hazard in terms of ground rupture and the severity of ground shaking.

Izmit, Turkey, M 7.3 earthquake of August 1999
Fault rupture (double stranded) near Golcuk
**Requirements for siting and foundation soils**

**POTENTIALLY LIQUEFIABLE SOILS**

Detailed guidelines and charts (Annex B) are provided for evaluating the liquefaction susceptibility of saturated cohesionless foundation soils through the well known empirical method based on **NSPT** or **CPT** resistance.

- The guidelines are not unduly conservative, because evaluation of liquefaction susceptibility can be omitted if:
  - The sandy soil layer or lens lies at more than 15 m depth from ground surface.
  - The design ground acceleration is less than 0.15 g and, at the same time, **NSPT** is sufficiently high, or the content of plastic fines in the soil is sufficiently high.

A minimum safety factor of 1.25 (in terms of shear stresses) is recommended.

---

**Foundation system**

**BASIC RULE**

Only one foundation type to be used for the same structure, unless this consists of dynamically independent units. E.g., use of piles and shallow foundations in the same structure must be avoided, except for bridges and pipelines.

**DESIGN ACTION EFFECTS**

Action effects transmitted to the foundations are evaluated according to capacity design considerations for dissipative structures (high ductility), while allowable seismic action combination applies for non-dissipative structures (essentially responding in the elastic range).

**DIRECT FOUNDATIONS (Footings)**

Stability of footings is to be checked against sliding failure, i.e.,

\[ V_{sd} \leq N_{sd} \tan \delta + E_{pd} \]

where:
- \( V_{sd} \) is the shear force resistance.
- \( N_{sd} \) is the friction force resistance.
- \( \delta \) is the inclination of the structural load.
- \( E_{pd} \) is the bearing capacity resistance.

and against bearing capacity failure (Annex F, see following two slides). Factors to be taken into account include: Inclination and eccentricity of structural load, inertia forces in the soil, pore pressure effects, non-linear soil behaviour.
Bounding surface of external loads
A single expression has been obtained for both cohesive and granular soils; it is introduced in Annex F of Part 5.

Foundation system

Piles and piers
Should be designed to resist both:
(a) Inertia forces from the superstructure, and
(b) Kinematic forces due to the earthquake-induced soil deformations.

The latter apply if all of the following conditions occur:

b1. Class D, S1, or S2 soil profile with consecutive layers of sharply contrasting stiffness
b2. Design ground acceleration > 0.10 g, and
b3. The supported structure is of importance category III or IV.

Although piles will generally be designed to remain elastic, they may under certain conditions develop plastic hinges at their head.

Kinematic forces on piles
Kinematic forces induced by earthquake on piles in "stable" soils

Design earthquake

\[ s(z) \text{: seismically induced horizontal displacements in the soil} \]

Kinematic forces induced by earthquake on piles in "unstable" loose soils

Stable soil
Unstable loose sandy soil
Stable soil
Horizontal displacements induced in soil by eqk ("flow failure")

Effects of kinematic forces on piles

General requirements and considerations
- Permanent displacements/tilting may be acceptable, provided functional or aesthetic requirements are not violated
- Structural choice is based on static loads, but seismic action may lead to different solution
- Build-up of significant PWP in backfill or supported soil is to be absolutely avoided
- Methods of analysis should in principle account for:
  - Inertial and interaction effects between structure and soil (even non-linear)
  - Hydrodynamic effects in the presence of water (in the soil, and on outer face of wall)
  - Compatibility of deformations of soil, wall, and free tendons

NB: After introducing requirements for general methods of analysis, the code only provides prescriptions for pseudo-static verifications.

Earth retaining structures

Earth retaining structures
Experimental observations (flexible retaining walls)

1) Effects on retaining walls in historical earthquakes
   - Liquifiable soils (harbour facilities) ___ Collapse (0.6 – 4m displacements)
     Loma Prieta (California, 1989) – $M_0=7.1$
     Kobe (Giappone, 1995) – $M_0=8.0$
     Bhu (India, 2001) – $M_0=7.6$
   - Non-liquifiable soils ___ Good behaviour (<10cm displacements)
     Northridge (California, 1994) – $M_0=6.8$
     Kobe (Giappone, 1995) – $M_0=6.9$
     Taiwan (1999) – $M_0=7.0$

2) Dynamic experimental tests
   - Shaking table
   - Dynamic centrifuge tests

Earth retaining structures

Resistance and stability verifications

- Foundation wall: the following need to be verified:
  - Stability of slope
  - Stability w.r.t. to failure by sliding and loss of bearing capacity, for shallow foundations.
  - Anchorage:
    - Design actions: permanent gravity loads, horizontal thrust $E_d$, seismic action.
    - Ground categories B, C, D: governed by $V_s,30$
      Soil profile properties
      Excavation depth $h$
      Unit weight $\gamma$

- Anchorages: they shall assure equilibrium and have a sufficient capacity to adapt to the seismic deformations of the ground. The distance $L_a$ between the anchor and the wall shall exceed the distance $L_d$, required for non-seismic tests:
  $$ L_a = L_d + (1 + 1.5 \gamma a_h) $$

- Backfill material: must be immune from liquefaction.

- Structural stability under the combination of the seismic action with other possible loads, equilibrium must be achieved without exceeding the strength of any structural element.

Earth retaining structures

Simplified (pseudo-static) analysis: the seismic action can be reduced by a kind of ductility factor $\gamma$:

$$ k_i = a_c \gamma \frac{S_i}{E_d} $$

$S_i$ = $0.5E_o$ (Spectrum Type 1) $\gamma = 0.33 \gamma$ (Spectrum Type 2)

Values of reduction factor $\gamma$ and residual displacement

<table>
<thead>
<tr>
<th>Type of retaining structure</th>
<th>$\gamma$</th>
<th>Acceptable residual displacement, d (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free gravity walls that can accept a displacement</td>
<td>2</td>
<td>2000</td>
</tr>
<tr>
<td>as above, but “behind”</td>
<td>1.5</td>
<td>1000</td>
</tr>
<tr>
<td>Reinforced concrete walls, anchored or braced walls, reinforced concrete walls located on vertical piles, restrained treatment at sun and bridge deck</td>
<td>1</td>
<td>900</td>
</tr>
</tbody>
</table>

For loose, saturated granular soils: $\gamma = 1$ and PS against liquefaction not less than 2.

The provisions for pseudo-static analysis follow a standard approach (Mononobe and Okabe), given in Annex E.

Examples of numerical analyses: simple flexible wall


Soil profile properties

Ground categories B, C, D: governed by $V_s,30$

Coarse grained, dry, $v_s=250\text{m/s}$, $\phi=34^\circ$, $c=0$, $\psi=0^\circ$, $\beta=0^\circ$, $\delta=0^\circ$

RC flexible wall

Unit weight $\gamma=25\text{kN/m}^3$, $\gamma_s=3.3$, $E_d=28\text{GPa}$

Excavation depth $h=3.5\text{m}$, Embedment “d”

Pre-dimensioning [EC 8]

1) Design strength: $\tan \phi = 0.25$
2) Seismic coefficients
   $$ \phi' = \phi \frac{1}{1 + \frac{1.5}{2} \gamma a_h} $$
3) Thrust
   $$ P_{Ed} = \frac{\gamma}{1 + 1.5 \gamma a_h} $$
4) Rotational equilibrium w.r.t. to base
5) 20% increase in embedment: d

Examples of numerical analyses: simple flexible wall


Material models

Soil: coarse grained, homogeneous, $V_s>150\text{m/s}$, elastic-plastic non associated constitutive model, Mohr-Coulomb rupture criterion($\phi=32^\circ$).

Flexible wall: 13m height, 5m excavation, linear elastic behaviour, $E=28\text{GPa}$, $\nu=0.3$.

Base excitation

2 groups of 7 accelerograms on ground type A

Zone I

$\zeta_{d,\text{a}}=0.35g$ $\zeta_{d,\text{b}}=0.25g$

Average response spectra

matching EC8 elastic spectrum
Results of EC8 pseudo-static vs. 2D dynamic (FEM) analyses
(from the M. Eng. Thesis of O. Zanoli, Politecnico di Milano)
Bending moment profiles in retaining wall

Eurocode 8: pseudo-static analyses with assigned \( k_h \) values for \( r = 1, 2 \);
Dynamic analyses: average \( M_{\text{max}} \) values over groups of input accelerograms

Ground type B
(Both pseudo-static and dynamic FEM analyses performed with commercial software FLAC)

Examples of numerical analyses
Complexity of wave propagation phenomena in soil – flexible wall systems
(other example with \( a_{\text{max}} = 0.15g \))
Nonlinear dynamic analyses performed with FEM code Gefdyn

Suggested reference:
Eurocode 8: Design of structures for earthquake resistance.
General rules, seismic actions, design rules for buildings, foundations and retaining structures
Michael N. Fardis, Eduardo Carvalho, Amr Elnashai, Ezio Faccioli, Paolo Pinto and Andre Plumier.
Published by Thomas Telford, UK, 2006
PART 3: ASSESSMENT AND RETROFITTING OF BUILDINGS

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Eurocode 8 Part 3
Assessment and retrofit of buildings

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University of Rome La Sapienza
Italy

Performance requirements

<table>
<thead>
<tr>
<th>Hazard (return period of the design spectrum)</th>
<th>Required performance</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_{DR}=2275$ years (2% in 50 years)</td>
<td>Near Collapse (NC) (heavily damaged, very low residual strength &amp; stiffness, large permanent drift but still standing)</td>
</tr>
<tr>
<td>$T_{DR}=475$ years (10% in 50 years)</td>
<td>Significant damage (SD) (significantly damaged, some residual strength &amp; stiffness, non-structural comp. damaged, uneconomic to repair)</td>
</tr>
<tr>
<td>$T_{DR}=225$ years (20% in 50 years)</td>
<td>Limited damage (LD) (only lightly damaged, damage to non-structural components economically repairable)</td>
</tr>
</tbody>
</table>

Compliance criteria

EC8 Part 3, 2.2.1(1): "Compliance with the requirements in 2.1 is achieved by adoption of the seismic action, method of analysis, verification and detailing procedures contained in this Part of EN1998".

Remarks:
- The criteria are not consistent with the definitions of the LS's. The NC-LS, for ex., is described as a state of severe damage extending over the entire structural system, and such as to bring it close to collapse.
- If the verifications would have to be satisfied for all individual primary elements, very few existing buildings would be exempted from some form of intervention.

Compliance criteria

A more consistent framework:
- The analyst should identify a number of structural situations that are realistically conducive to the LS under consideration.
- Such situations depend on the building topology and involve in general both single components and specific groups of components.

The ensemble of critical situations is conveniently arranged in the classical form of a fault tree. In the fault tree representation the state of the system is described as a serial arrangement of sub-systems, some of which are made of a number of components working in parallel.
Compliance criteria

A more consistent framework:
with reference to a fault tree representation as in the example, the state of the system is determined by the value of a scalar quantity defined as:

\[ Y = \max_{i=1,N} \min_{j=1,N_j} R_{ij} \]

where:
- \( R_{ij} \) = ratio between demand and capacity at the \( j \)-th component of the \( i \)-th subsystem
- \( N_j \) = number of components in subsystem \( i \)

\( Y=1 \) implies attainment of the LS under consideration

Deterministic approach to epistemic uncertainty

The confidence factor

Treatment of uncertainty

Knowledge levels (KL) and Confidence factor (CF)

An additional partial material factor, as the CF, is appropriate to cover the (generally) larger uncertainties on existing structures.

Uncertainties due to lack of knowledge such as, for example, the ignorance on whether a structural detail (or even a structural member) is present at all, cannot be accounted for by means of a CF.

Structural reliability theory offers standard tools for dealing with this different kind of uncertainty (epistemic uncertainty).

The confidence factor: limits of the approach

Steps of the procedure

a) Establish a set of alternative possible models of the structure
b) Based on experience and available evidence, assign a weight to each of the models: \( \sum w_i = 1 \)
  - Weights represent the degree of belief the analyst has on each of the models
c) For each model perform a seismic risk analysis (probability of exceeding the considered LS): \( P_{LS,i} \)
d) Calculate the final unconditional risk as the weighted sum of the above conditional risk:

\[ P_{LS} = \sum w_i P_{LS,i} \]

Probabilistic approach to epistemic uncertainty

Steps of the procedure

a) and b) same as for probabilistic procedure
c) For each model calculate the value of the state variable \( Y_i \) according to the rules of EC8 Part 3
d) Calculate the best estimate of \( Y \) as the weighted sum of individual \( Y_i \) and check:

\[ \sum w_i Y_i \leq 1 \]

Comment

The above procedure is just one order of rigour higher than customary sensitivity analysis, where the subjective judgment enters in the final selection of just one model and acceptance of the results it gives, while in the above procedure subjectivity enters in the assignment of the weights.
Methods of analysis: linear

Linear analysis with unreduced elastic seismic action (1/2)
- Lateral force and modal response spectrum
- Usable subject to a substantial uniformity, over all ductile primary elements, of the ratio between elastically calculated demand and corresponding capacity, i.e.
  \[
  \max(D_i/C_i) \min(D_i/C_i) < 2.5 \text{ (suggested, but no } > 3)\
  \]
- Limited practical experience indicates that when the above condition is satisfied the results from elastic multi-modal analysis compare well with those from non linear

This proves that the above condition represents a true physical quantitative definition of regularly of a structure from a seismic point of view, a definition that should supersede the semi-quantitative and rather arbitrary definitions given in EC8 Part 1

Methods of analysis: linear

Linear analysis with unreduced elastic seismic action (2/2)
- The lateral force method is less accurate and not computationally advantageous: it might well be dropped
- Modal response spectrum is accurate when the conditions for applicability are satisfied but the percentage of buildings complying with them is anticipated being not very large
- Application of linear methods to masonry structures is problematic due:
  - The condition related to D/C ratios is not of clear application, especially in case of a FE modeling of the structure
  - There are additional stiff conditions to be fulfilled: vertical continuity of all walls, tight floors, maximum stiffness ratio between walls at each floor level than 2.5, etc.

The above remarks point towards a generalised recourse to non linear methods.

Methods of analysis: linear

Multi-modal pushover: a convenient proposal (Chopra and Goel, 2002)
- Use several spatial lateral load patterns, corresponding to all significant modes: \( F_i = M_{D_1} \)
- Perform a pushover analysis and evaluate the desired response quantities \( R \), for each modal pattern and for each of the two horizontal components of the seismic action \( E_x \) and \( E_y \) and for the two signs \( (R_{E_x} - R_{E_y}) \)
- Combine the results from the above analyses according to the SRSS rule
  \[
  R = R_0 + \sum (R_{E_x} - R_{E_y})^2 + (R_{E_x} - R_{E_y})^2
  \]

Methods of analysis: non-linear static 1/3

The reference version of the pushover method in EC8 Part 3 is the same as in Part 1
- This version provides satisfactory results when:
  - The structure is essentially symmetric and torsionally rigid
  - The case of unsymmetrical (but still single-mode dominated) buildings is treated in EC8 Part 1 by means of an hybrid procedure whereby:
  - The loading pattern is still planar (uniform or modal)
  - The displacements of the different sides of the building obtained from the pushover analyses are amplified by a factor based on the results of spatial modal analysis
- In EC8 Part 3 a note is added in 4.4.5 saying that when \( T_1 \approx 4T_c \) or \( T_1 > 2s \) the effects of higher modes should be taken into account (not a ‘P’, hence not obligatory)

Methods of analysis: non-linear static 2/3

Problem with modal combination of member forces (absolute value)
- Unrealistically high normal forces and bending moments
- Shear forces not in equilibrium with bending moments
- Shear verification of columns: influence of the value of \( N \) both in the demand \( V(N) \) and in the capacity \( V_s(N) \)
  - Approximate solution: evaluate the D/C ratio mode by mode \( V_s(N) / V_s(N) \) (same sign of \( N \) on both D and C) and then check:
    \[
    \sum (V_s(N) / V_s(N)) < 1
    \]
    (damage variable analogy)
Ductile members (beam-columns & walls in flexure): the demand quantity is the chord-rotation at the ends, as obtained from the analysis, either linear or non-linear. Brittle mechanisms (shear): the demand quantity is the force acting on the mechanism.

Linear analysis: the ductile transmitting mechanisms can be:
- Yielded: the force is given by the analysis
- Non-linear analysis: forces as obtained from the analysis

Verifications (if LM accepted)
- In terms of strength:
  - Use mean values of properties divided by the CF

Non-linear Model
- Linear Model (LM)
  - Forces as obtained from the analysis

Mechanically-based models capable of accounting for all internal deterioration mechanisms that develop in inadequately detailed RC members are not available. Resort has been made to a large database collecting tests made in the past in order to derive empirical expressions.

\[
\alpha_{LM} = 0.03 \left( \frac{\max(0, \epsilon_{mc} - \epsilon_{cr})}{\max(0, \epsilon_{mc} - \epsilon_{cr})} \right)^{0.5} \frac{1}{\gamma_{cr}} \frac{1}{\gamma_{cr}^2} 25 \leq \gamma_{cr} \leq 1.25
\]

where \( \gamma \) = normalised axial force
\( \alpha_{LM} \) = mech. reinforcement ratio of compression and tension reinf.
\( l_e \) = shear span
\( \beta \) = net height of the section
\( \alpha \) = confinement effectiveness factor
\( \rho_{tr}, \rho_{dk} \) = transverse and diagonal reinforcement ratio

- Standard situation due to the simultaneous application of the orthogonal components of the seismic action
- No guidance in EC8 Part 3 (lack of adequate knowledge of the behaviour at ultimate)
- Limited experimental evidence (Fardis, 2006) supports the assumption of an elliptical interaction domain for biaxial deformation at ultimate
- Proposal:
  - For each mode evaluate the bidirectional demand/capacity ratio (BDCR)
    \[
    BDCR = \left( \frac{\epsilon_{mc}}{\epsilon_{cr}} \right) \left( \frac{l_e}{\beta} \right) \left( \frac{\rho_{tr} + \rho_{dk}}{\gamma_{cr}} \right)
    \]
    - Check that \( \sum (BDCR) < 1 \)

The well-known three-terms additive format for the shear strength has been retained. The expressions for the three contributions have been derived using the same database as for the flexural capacity, augmented by test results of specimen falling in shear after initial flexural yielding:

\[
V_s = 0.8 \left( \frac{A_s}{A_{eff}} \right) \min \left( N \cdot 0.55 \cdot A_f, \left( 1 - 0.55 \min \left( \frac{5}{2} \rho_{sl}, 5 \right) \right) \right)
\]

where \( A_{eff} \) = neutral axis depth
\( N = \) compressive axial force \( > 0 \) if tensile
\( A_{eff} = \) cross-section area
\( \rho_{sl} = \) plastic part of ductility demand
\( \rho_{sl} = \) total longitudinal reinforc.
The section covers traditional strengthening techniques, such as concrete and steel jacketing, as well as the use of FRP plating and wrapping, for which results from recent research are incorporated.

Guidance in the use of externally bonded FRP is given for the purposes of:

- increasing shear strength (contribution additive to existing strength)
- increasing ductility of critical regions (amount of confinement pressure to be applied, as function of the ratio between target and available curvature ductility)
- preventing lap-splice failure (amount of confinement pressure to be applied, as function of the bar diameters and of the action already provided by existing closed stirrups)