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JAPAN SOCIETY OF CIVIL ENGINEERS

*"New developments in Technology and Standards for Reinforced Concrete in Europe and Japan"*

20th November 2009, ATHENS, GREECE

# EN 1998: EUROCODE 8
# DESIGN OF STRUCTURES
# FOR EARTHQUAKE RESISTANCE

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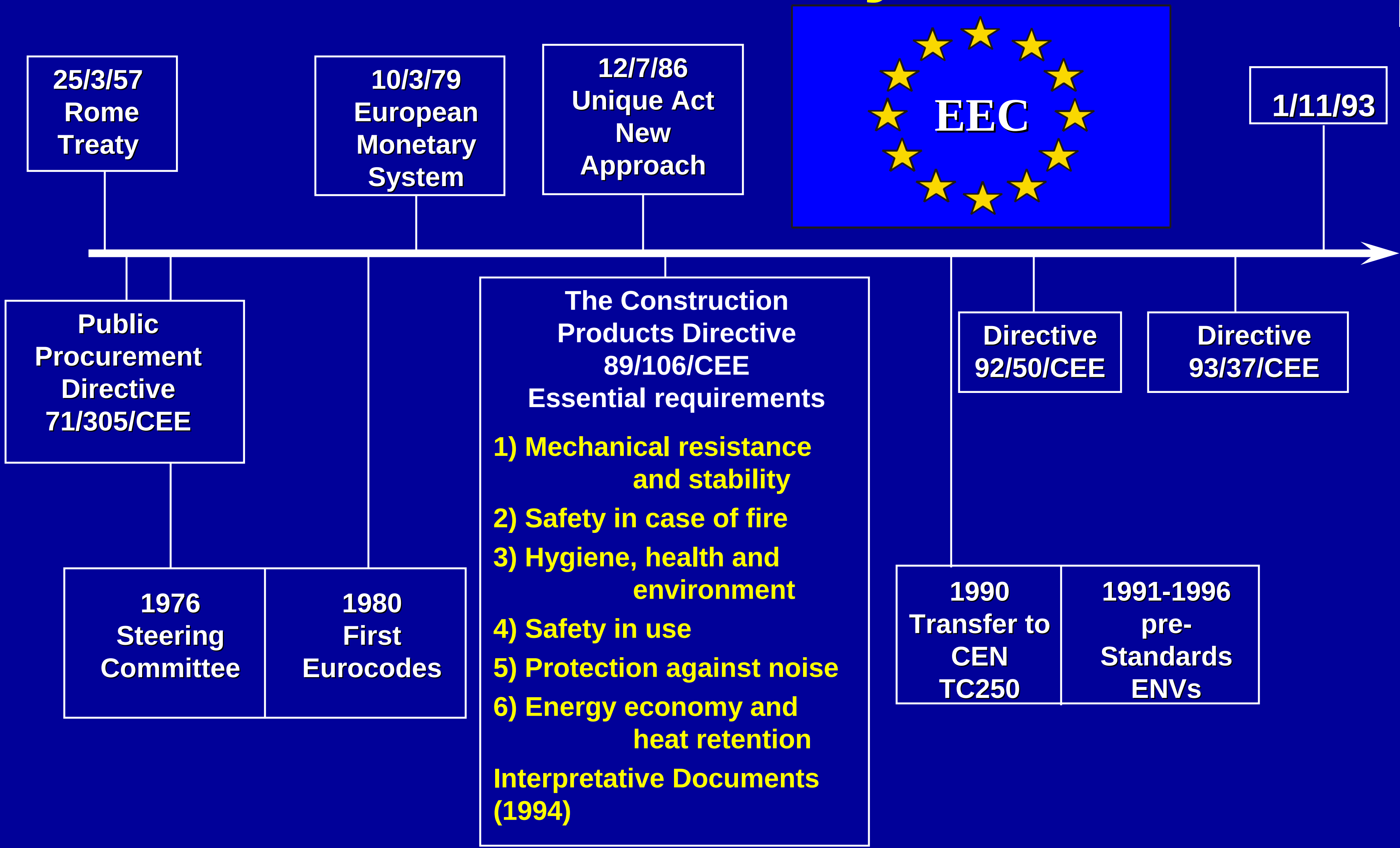
# Part I:
# The Eurocode context

# The ECs in the European Economic Community



# The ECs in the European Union

EU

1/11/93

Directive 97/52/CE

1998
«Conversion»
of ENVs to ENs
started

2002
First
Standards EN

The Commission's
Recommendation
to Member States

03/C4639 /CEE:
Implementation and use of
Eurocodes

1) Adopt ECs

2) Use ECs as basis of Specs in public sector and energy, water, transport & telecommunication sectors

3) Member States competent on safety and economy: NDPs

4) Compare, harmonise NDPs

2005
End of
«conversions» of
ENVs to ENs

# CONVERSION OF EUROCODES FROM ENV TO EN

- Subject: 56 ENs
- Period: 1998-2005
- Roles:
  - Financing, Implementation & Control: *European Commission, DG-Enterprise*
  - Institutional & Management: *CEN*
  - Administration & overall Technical Coordination: *CEN/TC250*
  - Technical responsibility for individual Eurocodes: *TC250/SCs*
  - 1st Draft: *Project Teams* of nationally-nominated experts, working with *SC*
  - Redrafting & Decisions: *National Standards Bodies (NSB)* via *SC* & *Formal Vote*
- Phases (for each EC part):
  - 1st Draft by Project Team on the basis of national comments for ENV; technical discussion, redrafting & decisions in SC: *2-3 yrs*
  - Examination of Draft by NSBs, redrafting, translation to French, German, Formal Vote *(weighted voting; qualified majority),* publication by CEN *~2 yr*
  - National versions of EN, including National Annex with national choices: *2 yrs*
  - Parallel use of existing national provisions & EN-packages: *3yrs from last EN*
  - Withdrawal of conflicting national standards: *2010-11*

# Objectives of Eurocodes

The Member States of the EU and EFTA recognise that Eurocodes serve as reference documents for the following purposes :

→ as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire;

→ as a basis for specifying contracts for construction works and related engineering services;

→ as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

# Objectives of Eurocodes (cont'd)

In addition, the Eurocodes are expected to:

- improve the functioning of the single market for products and engineering services by removing obstacles arising from different nationally codified practices for the assessment of structural reliability;
- improve the competitiveness of the European construction industry and the professionals and industries connected to it, in countries outside the European Union.

# IMPORTANT FEATURES OF EUROCODE-SYSTEM

- Comprehensive & integrated system covering:
  - all structural materials;
  - practically all types of construction works;
- in a consistent, harmonised & user-friendly manner (similar document structure, symbols, terminology, verification criteria, analysis methods, etc.),
- with hierarchy & cross-referencing among different ECs & EC-parts
- w/o overlapping & duplication.

- EC-system ideal for application in a large No. of countries w/ different traditions, materials, environmental conditions, etc., as it has built-in flexibility to accommodate such differences.

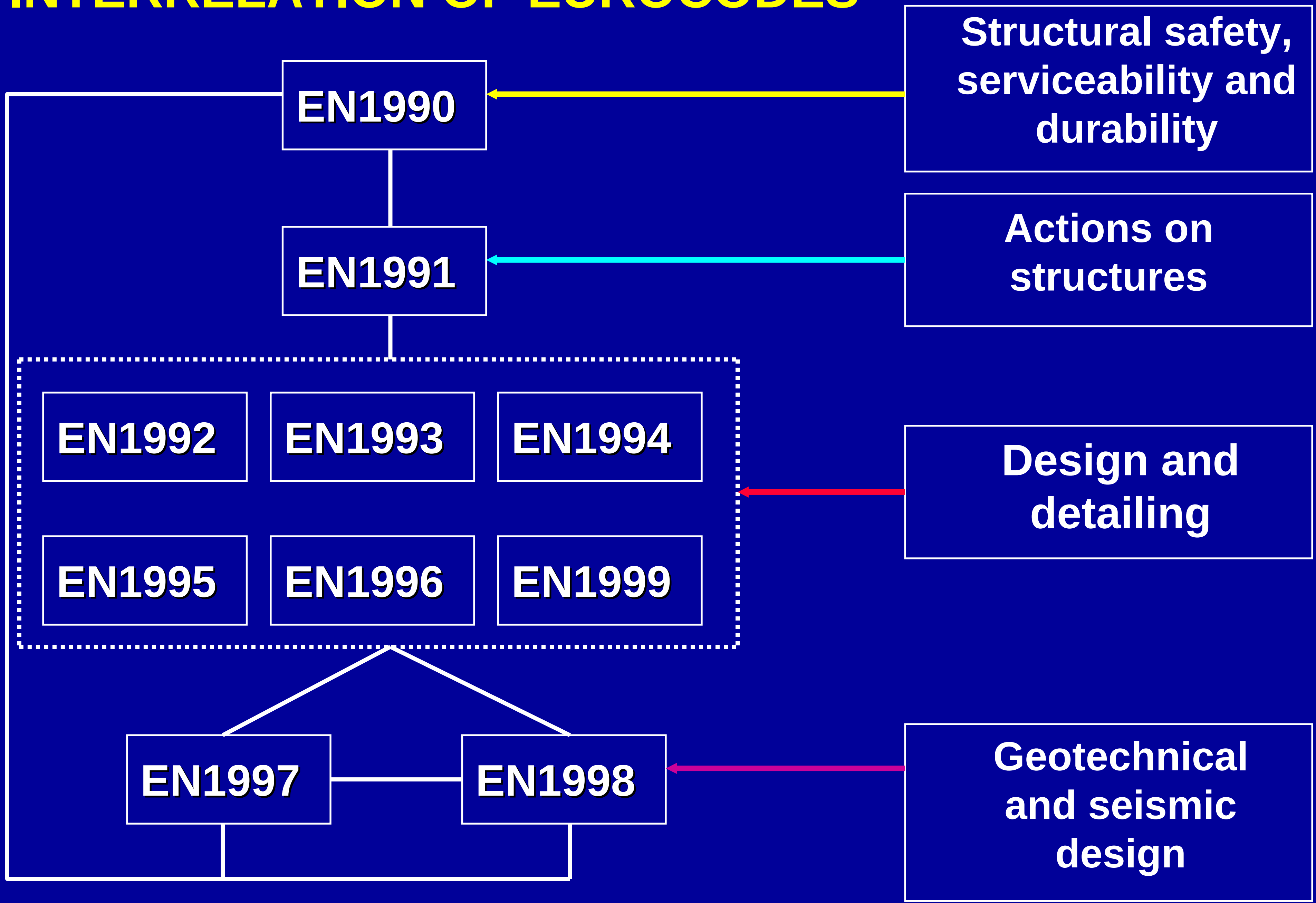
# European Standards (ENs)

| | |
|---|---|
| Design standards : The Eurocodes | |
| Material standards (steel, concrete, etc.) and Product standards (Structural bearings, Isolation devices, etc.) | ETAs: European Technical Approvals (FRPs, Prestressing systems, Isolation/dissipation devices, etc.) |
| Execution standards (e.g., standards for the execution of concrete or steel structures) | |
| Test standards | |

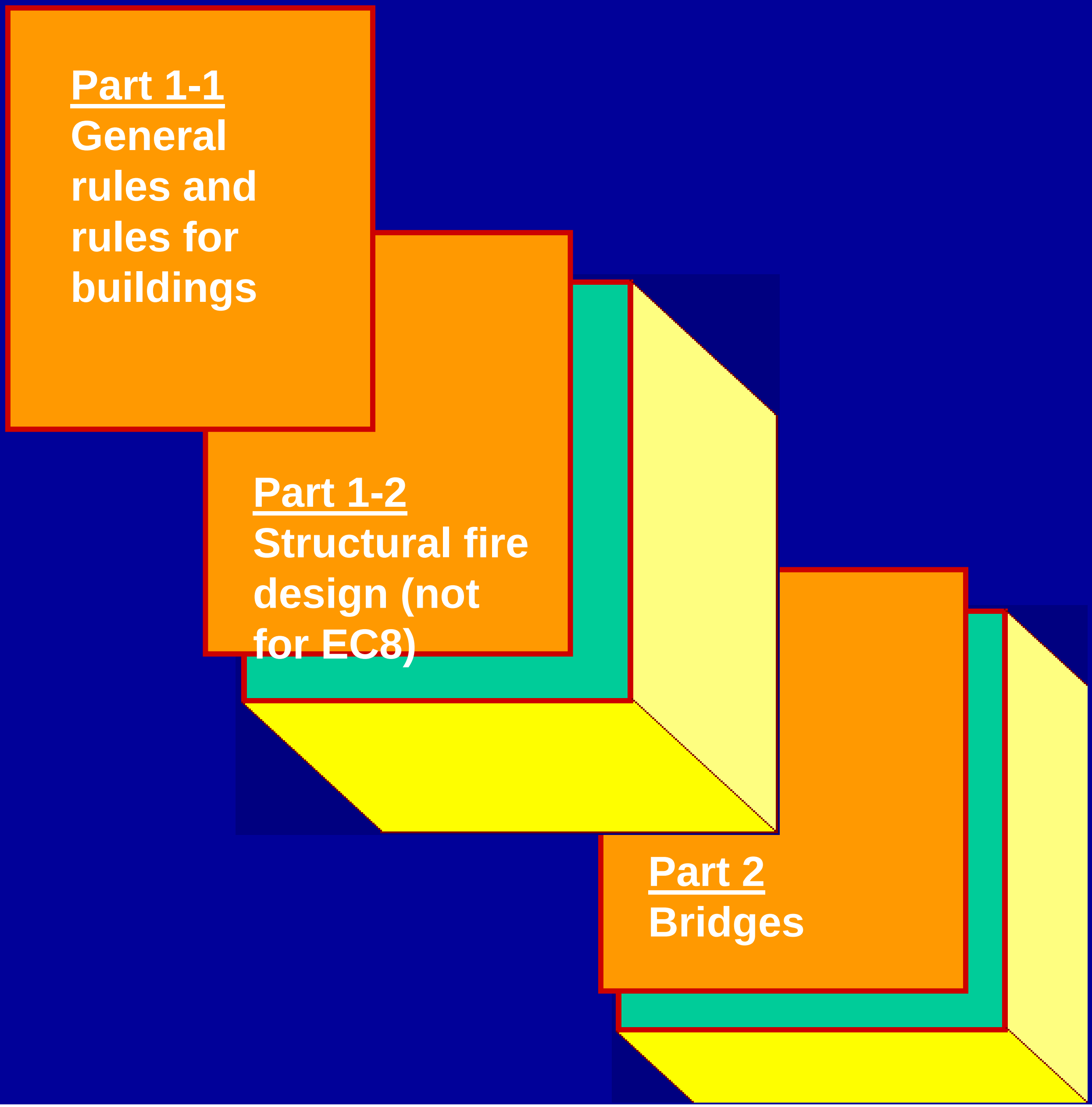
# THE EN-EUROCODES

| | | |
|---|---|---|
| EN 1990 | Eurocode | : Basis of structural design |
| EN 1991 | Eurocode 1 | : Actions on structures |
| EN 1992 | Eurocode 2 | : Design of concrete structures |
| EN 1993 | Eurocode 3 | : Design of steel structures |
| EN 1994 | Eurocode 4 | : Design of composite steel and concrete structures |
| EN 1995 | Eurocode 5 | : Design of timber structures |
| EN 1996 | Eurocode 6 | : Design of masonry structures |
| EN 1997 | Eurocode 7 | : Geotechnical design |
| EN 1998 | Eurocode 8 | : Design of structures for earthquake resistance |
| EN 1999 | Eurocode 9 | : Design of aluminium structures |

# INTERRELATION OF EUROCODES



# Organisation of Eurocodes 2, 3, 4, 5, (8)



# EN 1990 – Eurocode : Basis of structural design

**Foreword**

**Section 1 : General**

**Section 2 : Requirements**

**Section 3 : Principles of limit states**

**Section 4 : Basic variables**

**Section 5 : Structural analysis & design assisted by testing**

**Section 6 : Verification by the partial factor method**

**Annex A1(N):** Application for buildings

**Annex A2 (N):** Application for bridges

**Annex B (I):** Management of structural reliability for construction works

**Annex C (I):** Basis for partial factor design & reliability analysis

**Annex D (I):** Design assisted by testing

# EN 1990 – Eurocode : Basis of structural design

## (future) ANNEXES

A3 (N): Application for towers, masts & chimneys
A4 (N): Application for silos and tanks
A5 (N): Application for cranes and machinery

E1 (I?): Structural bearings
E2 (I?): Expansion joints
E3 (I?): Pedestrian parapets
E4 (I?): Vehicle parapets
E5 (I?): Ropes and cables

# Eurocode 1 – Actions on structures

- **<u>GENERAL ACTIONS</u>**
  - **EN 1991-1-1: Densities, self-weight, imposed loads on buildings**
  - **EN 1991-1-2: Actions on structures exposed to fire**
  - **EN 1991-1-3: Snow loads**
  - **EN 1991-1-4: Wind actions**
  - **EN 1991-1-5: Thermal actions**
  - **EN 1991-1-6: Actions during execution**
  - **EN 1991-1-7: Accidental actions**
- **EN 1991-2: Traffic loads on bridges**
- **EN 1991-3: Actions due to cranes and machinery**
- **EN 1991-4: Actions in silos and tanks**

# Eurocode 2 – Design of concrete structures

- EN1992-1-1: General rules and rules for buildings
- EN1992-1-2: Structural fire design
- EN1992-2: Reinforced and prestressed concrete bridges
- EN1992-3: Liquid retaining and containing structures

# Eurocode 3 – Design of steel structures

- **EN1993-1-1:** General rules and rules for buildings
- **EN1993-1-2:** Structural fire design
- **EN1993-1-3:** Cold-formed thin gauge members & sheeting
- **EN1993-1-4:** Stainless steels
- **EN1993-1-5:** Plated structural elements
  **EN1993-1-6:** Strength and stability of shell structures
- **EN1993-1-7:** Strength and stability of planar plated structures transversely loaded
- **EN1993-1-8:** Design of joints
- **EN1993-1-9:** Fatigue strength of steel structures
- **EN1993-1-10:** Selection of material for fracture toughness and through thickness properties
- **EN1993-1-11:** Use of high-strength tensile elements

# Eurocode 3 – Design of steel structures (cont'd)

- EN1993-2: Steel bridges
- EN1993-3-1: Towers and masts
- EN1993-3-2: Chimneys
- EN1993-4-1: Silos
- EN1993-4-2: Tanks
- EN1993-4-3: Pipelines
- EN1993-5: Piling
- EN1993-6: Crane supporting structures

# Eurocode 4 – Design of composite steel and concrete structures

- **EN1994-1-1:** **General rules and rules for buildings**
- **EN1994-1-2:** **Structural fire design**
- **EN1994-2:** **Composite bridges**

# Eurocode 5 – Design of timber structures

- EN1995-1-1: General rules and rules for buildings
- EN1995-1-2: Structural fire design
- EN1995-2: Timber bridges

# Eurocode 6 – Design of masonry structures

- **EN1996-1-1:** **Common rules for reinforced and unreinforced masonry structures**
- **EN1996-1-2:** **Structural fire design**
- **EN1996-2:** **Design, selection of materials and execution of masonry**

# Eurocode 7 – Geotechnical design

- EN1997-1: General rules
- EN1997-2: Ground investigation and testing

# Eurocode 8 – Design of structures for earthquake resistance

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

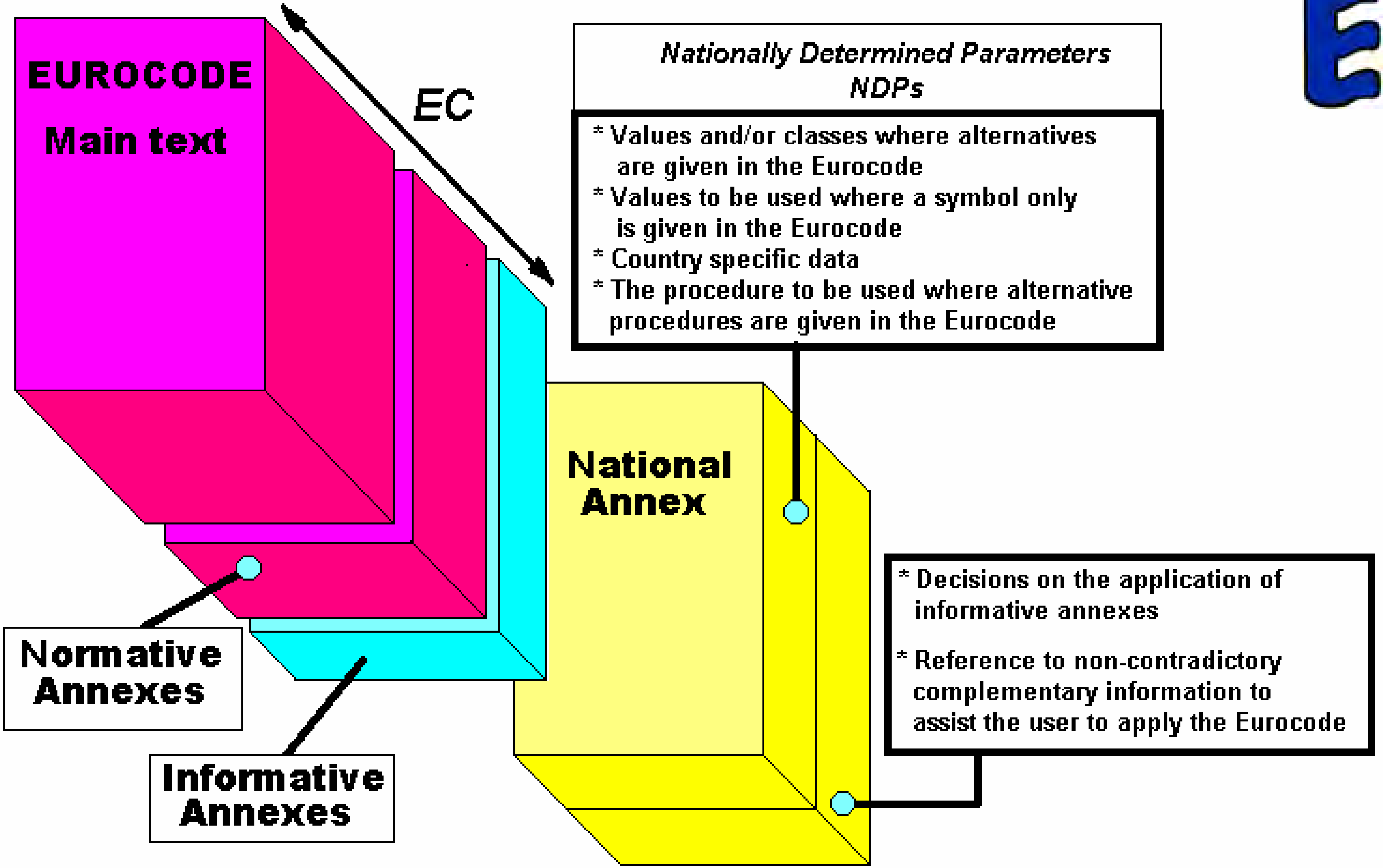
# Eurocode 9 – Design of aluminium structures

- EN1999-1-1: General rules – Structures
- EN1999-1-2: General rules - Structural fire design
- EN1999-1-3: Additional rules for structures susceptible to fatigue
- EN1999-1-4: Supplementary rules for trapezoidal sheeting
- EN1999-1-5: Supplementary rules for shell structures

# FLEXIBILITY WITHIN EUROCODE FRAMEWORK

- Eurocodes (ECs) or National Annexes cannot allow design with rules other than those in the ECs.
- National choice can be exercised through the National Annex, only where the Eurocode itself explicitly allows:
  1. Choosing a value for a parameter, for which a symbol or range of values is given in the Eurocode;
  2. Choosing among alternative classes or models detailed in the Eurocode;
  3. Adopting an Informative Annex or referring to alternative national document.
- Items of national choice in 1-2: Nationally Determined Parameters NDPs
- National choice through NDPs:
  - Wherever agreement on single choice cannot be reached;
  - On issues controlling safety, durability & economy (national competence) & where geographic or climatic differences exist (eg. Seismic Hazard)
- For cases 1 & 2, the Eurocode itself recommends (in a Note) a choice. The European Commission will urge countries to adopt recommendation(s), to minimize diversity within the EU.
- If a National Annex does not exercise national choice for a NDP, designer will make the choice, depending on conditions of the project.

# IMPLEMENTATION OF EUROCODES



# European Commission, Guidance Paper L: “Application and use of Eurocodes” CONSTRUCT 01/483 Rev.1, Brusells, 2001

- The determination of the levels of safety of buildings and civil engineering works and parts thereof, including aspects of durability and economy, is .. within the competence of the Member States.
- Possible difference in geographical or climatic conditions (e.g. wind or snow), or in ways of life, as well as different levels of protection that may prevail at national, regional or local level … will be taken into account … by providing choices in the EN Eurocodes for identified values, classes, or alternative methods, to be determined at the national level (named Nationally Determined Parameters, NDPs). Thus allowing the Member States to choose the level of safety, including aspects of durability and economy, applicable to works in their territory.
- When Member States lay down their NDPs, they should:
  - choose from the classes included in the EN Eurocodes, or
  - use the recommended value, or choose a value within the recommended range of values, for a symbol where the EN Eurocodes make a recommendation, or
  - when alternative methods are given, use the recommended method, where the EN Eurocodes make a recommendation,
  - take into account the need for coherence of the NDPs laid down for the different EN Eurocodes and the various Parts thereof.
- Member States are encouraged to co-operate to minimize the number of cases where recommendations for a value or method are not adopted for their nationally determined parameters.
- The NDPs laid down in a Member State should be made clearly known to the users of the EN Eurocodes and other parties concerned, including manufacturers.
- When EN Eurocodes are used for the design of construction works, or parts thereof, the NDPs of the Member State on whose territory the works are located shall be applied.
- Any reference to a EN Eurocode design should include the information on which set of NDPs was used, whether or not the NDPs .. used correspond to the recommendations given in the EN Eurocodes.

# European Commission, Guidance Paper L: “Application and use of Eurocodes” CONSTRUCT 01/483 Rev.1, Brusells, 2001

- National Provisions should avoid replacing any EN Eurocodes provisions, e.g. Application Rules, by national rules (codes, standards, regulatory provisions, etc.).
- When, however, National Provisions do provide that the designer may – even after the end or the coexistence period – deviate from or not apply the EN Eurocodes or certain provisions thereof (e.g. Application Rules), then the design will not be called “a design according to EN Eurocodes”.
- When Eurocodes Parts are published as European standards, they will become part of the application of the Public Procurement Directive (PPD).
- In all cases, technical specifications shall be formulated in public tender enquiries and public contracts by referring to EN Eurocodes, in combination with the NDPs applicable to the works concerned.
- However, the reference to EN Eurocodes is not necessarily the only possible reference allowed in a Public contract. The PPD foresees the possibility for the procuring entity to accept other proposals, if their equivalence to the EN Eurocodes can be demonstrated by the contractor.
- Consequently, the design of works proposed in response to a Public tender can be prepared according to:
  - EN Eurocodes (including NDPs) which give a presumption of conformity with all legal European requirements concerning mechanical resistance and stability, fire resistance and durability, in compliance with the technical specifications required in the contract for the works concerned;
  - Other provisions expressing the required technical specification in terms of performance. In this case, the technical specification should be detailed enough to allow tenderers to know the conditions on which the offer can be made and the owner to choose the preferred offer. This applies, in particular, to the use of national codes, as long as Member States maintain their use in parallel with EN Eurocodes (e.g. a Design Code provided by National Provisions), if also specified to be acceptable as an alternative to an EN Eurocode Part by the Public tender.

## European Commission: “Commission Recommendation on the implementation and use of Eurocodes for construction works & structural construction products”. Document No. C(2003)4639, Brussels (2003)

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

……….

- Member States should inform the Commission of all national measures in accordance with the Recommendation.

# European Commission: "Commission Recommendation on the implementation and use of Eurocodes for construction works & structural construction products". Document No. C(2003)4639, Brussels (2003)

……..

- For each Nationally Determined Parameter (NDP), the Eurocodes give a recommended value. However, Member States may choose a different specific value as the NDP, if they consider it necessary in order to ensure that building and civil engineering works are designed and executed in a way that does not endanger the safety of persons, domestic animals or property
- Member States should use the recommended values provided by the Eurocodes when NDPs have been identified in the Eurocodes. They should diverge from those recommended values only where geographical, geological or climatic conditions or specific levels of protection make the necessary. Member States should notify the Commission of the NDPs in force on their territory within two years of the date on which the Eurocodes became available.
- In order to achieve a higher level of harmonization, a comparison of the various NDPs implemented by the Member States should be undertaken and, where appropriate, they should be aligned.
- Member States should, acting in coordination under the direction of the Commission, compare the NDPs implemented by each Member State and assess their impact as regards the technical differences for works or parts of works. Member States should, at the request of the Commission, change their NDPs in order to reduce divergence from the recommended values provided by the Eurocodes.

……..

- Member States should inform the Commission of all national measures in accordance with the Recommendation.

# EN 1998-1:2004

# General rules, seismic actions, rules for buildings

| | | No. of NDPs |
|---|---|---|
| 1. | General | _ |
| 2. | Performance Requirements and Compliance Criteria | 2 |
| 3. | Ground Conditions and Seismic Action | 8 |
| 4. | Design of Buildings | 7 |
| 5. | Specific Rules for Concrete Buildings | 11 |
| 6. | Specific Rules for Steel Buildings | 6 |
| 7. | Specific Rules for Steel-Concrete Composite Buildings | 4 |
| 8. | Specific Rules for Timber Buildings | 1 |
| 9. | Specific Rules for Masonry Buildings | 15 |
| 10. | Base Isolation | 1 |
| Annex A (Informative): | Elastic Displacement Response Spectrum | 1 |
| Annex B (Informative): | Determination of the Target Displacement for Nonlinear Static (Pushover) Analysis | 1 |
| Annex C (Normative): | Design of the Slab of Steel-Concrete Composite Beams at Beam-Column Joints in Moment Resisting Frames | _ |
| | Total: | 57 |

# EN 1998-5:2004
# Foundations, retaining structures, geotechnical aspects

| | | No. of NDPs |
|---|---|---|
| 1. | General | – |
| 2. | Seismic Action | – |
| 3. | Ground Properties | 1 |
| 4. | Requirements for Siting and for Foundation Soils | 1 |
| 5. | Foundation System | 1 |
| 6. | Soil-Structure Interaction | – |
| 7. | Earth Retaining Structures | – |
| Annex A (Informative): | Topographic Amplification Factors | 1 |
| Annex B (Normative): | Empirical Charts for Simplified Liquefaction Analysis | – |
| Annex C (Informative): | Pile-Head Static Stiffnesses | 1 |
| Annex D (Informative): | Dynamic Soil-Structure Interaction (SSI). General Effects and Significance | 1 |
| Annex E (Normative): | Simplified Analysis for Retaining Structures | – |
| Annex F (Informative): | Seismic Bearing Capacity of Shallow Foundations | 1 |
| | | Total: 7 |

# EN 1998-3:2005
# Assessment and Retrofitting of buildings

| | | No. of NDPs |
|---|---|---|
| 1. | General | _ |
| 2. | Performance Requirements and Compliance Criteria | 3 |
| 3. | Information for Structural Assessment | 2 |
| 4. | Assessment | 2 |
| 5. | Decisions for Structural Intervention | _ |
| 6. | Design of Structural Intervention | _ |
| Annex A (Informative): | Concrete Structures | 1 |
| Annex B (Informative): | Steel or Composite Structures | 1 |
| Annex C (Informative): | Masonry Buildings | 1 |
| | Total: | 10 |

- Normative part: General rules
- All material-specific aspects: In Informative (nonbinding) Annexes

# EN 1998-2:2005: Bridges

| | | No. of NDPs |
| --- | --- | --- |
| 1. | Introduction | – |
| 2. | Performance Requirements and Compliance Criteria | 8 |
| 3. | Seismic Action | 4 |
| 4. | Analysis | 2 |
| 5. | Strength Verification | 3 |
| 6. | Detailing | 6 |
| 7. | Bridges with Seismic Isolation | 4 |
| Annex A (Informative): | Probabilities Related to the Reference Seismic Action. Guidance for the Selection of Design Seismic Action during the Construction Phase | 1 |
| Annex B (Informative): | Relationship between Displacement Ductility and Curvature Ductility Factors of Plastic Hinges in Concrete Piers | 1 |
| Annex C (Informative): | Estimation of the Effective Stiffness of Reinforced Concrete Ductile Members | 1 |
| Annex D (Informative): | Spatial Variability of Earthquake Ground Motion: Model and Methods of Analysis | 1 |
| Annex E (Informative): | Probable Material Properties and Plastic Hinge Deformation Capacities for Non-Linear Analyses | 1 |

*(Cont'd next page)*

# *(Cont'd)* EN 1998-2:2005: Bridges

| | | No. of NDPs |
|---|---|---|
| Annex E (Informative): | Added Mass of Entrained Water for Immersed Piers | 1 |
| Annex F (Normative): | Calculation of Capacity Design Effects | — |
| Annex G (Informative): | Static Nonlinear Analysis (Pushover) | 1 |
| Annex J (Normative): | Variation of Design Properties of Seismic Isolator Units | 2 |
| Annex JJ (Informative): | λ-Factors for Common Isolator Types | 1 |
| Annex K (Informative): | Tests for Validation of Design Properties of Seismic Isolator Units | 1 |
| | Total: | 38 |

# EN 1998-6:2005
# Towers, Masts and Chimneys

| | | No. of NDPs |
|---|---|---|
| 1. | General | – |
| 2. | Performance Requirements and Compliance Criteria | – |
| 3. | Seismic Action | 2 |
| 4. | Design of Earthquake Resistant Towers, Masts and Chimneys | 4 |
| 5. | Specific Rules for Reinforced Concrete Chimneys | – |
| 6. | Special Rules for Steel Chimneys | – |
| 7. | Special Rules for Steel Towers | – |
| 8. | Special Rules for Guyed Masts | – |
| Annex A (Informative): | Linear Dynamic Analysis accounting for Rotational Components of the Ground Motion | 1 |
| Annex B (Informative): | Modal Damping in Modal Response Spectrum Analysis | 1 |
| Annex C (Informative): | Soil-Structure Interaction | 1 |
| Annex D (Informative): | Number of Degrees of Freedom and of Modes of Vibration | 1 |
| Annex E (Informative): | Masonry Chimneys | 1 |
| Annex E (Informative): | Electrical Transmission Towers | 1 |
| | Total: | 12 |

# EN 1998-4:2006

# Silos, Tanks and Pipelines

| | | No. of NDPs |
|---|---|---|
| 1. | General | – |
| 2. | General Principles and Application Rules | 6 |
| 3. | Specific Principles and Application Rules for Silos | 1 |
| 4. | Specific Principles and Application Rules for Tanks | 2 |
| 5. | Specific Principles and Application Rules for Above-ground Pipelines | – |
| 6. | Specific Principles and Application Rules for Buried Pipelines | – |
| Annex A (Informative): | Seismic Analysis Procedures for Tanks | 1 |
| Annex B (Informative): | Buried Pipelines | 1 |
| | Total: | 11 |

# EC8 Parts - Key dates

| EC8 Part | Title | Approval by formal vote | Availability from CEN | National publication - National Annexes |
|---|---|---|---|---|
| 1: EN1998-1 | General rules, seismic actions, rules for buildings | Feb 04 | Dec. 04 | Dec. 06 |
| 2: EN1998-2 | Bridges | June 05 | Nov. 05 | Nov. 07 |
| 3: EN1998-3 | Assessment and retrofitting of buildings | Feb 05 | June 05 | June 07 |
| 4: EN1998-4 | Silos, tanks, pipelines | April 06 | July 06 | July 08 |
| 5: EN1998-5 | Foundations, retaining structures, geotechnical aspects | Feb 04 | Nov. 04 | Nov. 06 |
| 6: EN1998-6 | Towers, masts, chimneys | March 05 | June 05 | June 07 |

# EUROCODE PACKAGES & EC8:

- Self-sufficient packages of ENs for design of each type of construction works (building, bridge, etc.) with a specific construction material.
- EC0 (Basis of design), EC1 (Actions), EC7 (Geotechnical) & EC8:

  Not basis of any EC-package; in all packages as service items.
- Withdrawal of all conflicting national standards:

  5 years after publication by CEN of last EN in package.
- EC8 parts to be included in EC-packages:
  - EN1998-1, -5 & -3: in packages for concrete, steel, composite, etc., buildings
  - EN1998-1, -5 & -2: in packages for concrete, steel etc. bridges
  - EN1998-1, -5 & -4: in packages for Concrete liquid retaining structures and for Steel silos, tanks, pipelines
  - EN1998-1, -5 & -6: in package for Steel towers and masts

| EC-Package No. & subject | EC7 Parts 1 & 2: | EC8 Part: | | | | | |
|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 |
| 2/1 Concrete buildings | ☯ | ☯ | | ☯ | | ☯ | |
| 3/1 Steel buildings | ☯ | ☯ | | ☯ | | ☯ | |
| 4/1 Composite (steel-concrete) buildings | ☯ | ☯ | | ☯ | | ☯ | |
| 5/1 Timber buildings | ☯ | ☯ | | ☯ | | ☯ | |
| 6/1 Masonry buildings | ☯ | ☯ | | ☯ | | ☯ | |
| 7 Aluminium structures | ☯ | ☯ | | | | ☯ | |
| 2/2 Concrete bridges | ☯ | ☯ | ☯ | | | ☯ | |
| 3/2 Steel bridges | ☯ | ☯ | ☯ | | | ☯ | |
| 4/2 Composite bridges | ☯ | ☯ | ☯ | | | ☯ | |
| 5/2 Timber bridges | ☯ | ☯ | ☯ | | | ☯ | |
| 2/3 Concrete liquid retaining and containment structures | ☯ | ☯ | | | ☯ | ☯ | |
| 3/3 Steel silos, tanks and pipelines | ☯ | ☯ | | | ☯ | ☯ | |
| 3/4 Steel piling | ☯ | ☯ | | | | ☯ | |
| 3/5 Steel cranes | ☯ | ☯ | | | | ☯ | |
| 3/6 Steel towers and masts | ☯ | ☯ | | | | ☯ | ☯ |

# STRUCTURE OF EN 1998-1: 2004

1. General
2. <u>Performance Requirements and Compliance Criteria</u>
3. <u>Ground Conditions and Seismic Action</u>
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

# 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

# Part II:

# Performance Requirements and Seismic Actions in EC8

# From EN1990 (Eurocode – Basis of structural design):

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

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

$P$ **: Prestressing**

$\psi_{2,i} Q_{k,i}$ **: Variable actions (quasi-permanent values)**

$A_{Ed} = \gamma_I A_{Ek}$ **: Design Seismic action**

$A_{Ek}$ **: Characteristic Seismic action,** $\gamma_I$ **: Importance factor of structure**

# From EN1990 & EN1998-1(Eurocode 8 – General):

$A_{Ek}$ **: «Reference Seismic action»:**

**Reference Probability of Exceedance, $P_R$, in design life $T_L$ of structure (or Reference Return Period, $T_R$)**

# IMPORTANCE CLASSES - IMPORTANCE FACTORS FOR BUILDINGS

| Importance class | Building | Recommended $\gamma_I$ value (NDP) |
|---|---|---|
| I | Minor importance for public safety | 0.8 |
| II | Ordinary | 1.0 (by definition) |
| III | Large consequences of collapse (schools, assembly halls, cultural institutions etc.) | 1.2 |
| IV | Of vital importance for civil protection (hospitals, fire stations, power plants, etc.) | 1.4 |

## From EN1990 - Eurocode: Basis of structural design:

**Design working life**: the assumed period for which a structure is to be used for its intended purpose with anticipated maintenance but without major repair being necessary.

**For :**

- Definition of design actions (e.g. wind, earthquake)
- Determination of material property deterioration (f.i. fatigue, creep)
- Life cycle costing
- Development of maintenance strategies

## In EN1998-1 – Eurocode 8 – General:

- Presumed design working life $T_L$ : 50 years
- Different values can be considered through Importance factor of the structure (reliability differentiation).

# IN EUROPE, SINCE ’60s (also in seismic codes)

- Instead of “Performance Level”:
- “Limit State” (LS) = state of unfitness to (intended) purpose:
  - ULS (Ultimate LS): safety of people and/or structure;
  - SLS (Serviceability LS): operation, damage to property.
- LS concept:
  - Adopted in 1985 CEB seismic Model Code;
  - Continued & expanded in 1994 ENV (prestandard) Eurocode 8;
  - According to EN 1990 (Eurocode: Basis of structural design): LS-design is the basis for all Eurocodes (including EC8).

# In EN1990 - Eurocode: Basis of structural design:

- **Ultimate limit states concern:**
  - the safety of people
  - the safety of the structure
- **Serviceability limit states concern:**
  - the functioning of the structure
  - the comfort of people
  - the appearance of the structure

| Limit State / Design Situation | U.L.S. | | S.L.S. |
|---|---|---|---|
| | | • loss of equilibrium of the structure or any part of it, considered as a rigid body;<br>• failure by excessive deformation, transformation of the structure or any part of it into a mechanism, rupture, loss of stability of the structure or any part of it, including supports and foundations;<br>• failure caused by fatigue or other time-dependent effects. | |
| Persistent | ✓ | | ✓ |
| Transient | ✓ | | ✓ |
| Accidental | ✓ | | |
| Seismic | ✓ | | ✓ |

# EN 1998: Adaptation of L.S. Design of new buildings, towers, tanks, pipelines, chimneys or silos to Performance-based concept:

- Verify explicitly No-life-threatening-collapse requirement ("Life Safety" performance level) for "rare" Earthquake (recommended NDP-reference seismic action for structures of ordinary importance: 475 years).
- Limit damage through damage limitation check for "frequent" Earthquake (recommended NDP-reference EQ for structures of ordinary importance: 95 yrs).
- Prevent collapse under any conceivable Earthquake through "Capacity Design”

# EN 1998: Design of foundations, bridges, retaining structures, masts:

- Verify explicitly only No-(life-threatening) collapse requirement under "rare" Earthquake (recommended NDP-reference seismic action for structures of ordinary importance: 475 years).

- No Serviceability or Damage Limitation checks for "frequent" Earthquake

- For some types of structures: Prevent collapse under any conceivable Earthquake through "Capacity Design”

# EN 1998-3: Assessment and retrofitting of buildings: EXPLICIT PERFORMANCE-BASED APPROACH:

## Assessment & Retrofitting for different Limit States under different Seismic Hazard levels

- Limit States (Performance Levels)
  - Damage Limitation (: Immediate Occupancy)
  - Significant Damage (: Life Safety)
  - Near Collapse.
- Flexibility for countries, owners, designers:
  - How many & which Limit States will be met and for what Hazard Level:
    - to be decided by country, or
    - (if country doesn't decide in National Annex) by owner/designer
  - Hazard Levels: NDPs - No recommendation given

    Noted that Basic Objective for ordinary new buildings is:
    - Damage Limitation: Occasional EQ (225yrs)
    - Significant Damage: Rare EQ (475yrs)
    - Near Collapse: Very rare EQ (2475yrs)
- Safety-critical facilities: Enhanced Objective, via multiplication of seismic action by importance factor $\gamma_I$

# EN 1998: SEISMIC ACTION FOR DAMAGE LIMITATION CHECKS

- **Seismic action for "damage limitation": NDP.**

  **Recommended for ordinary structures: 10%/10yrs (95yr EQ); ~50% of "design seismic action" (475 yr seismic action).**
- **In buildings: Interstorey drift ratio calculated for "damage limitation" action via "equal displacement rule" (elastic response):**
  - **<0.005 for brittle nonstructural elements attached to structure;**
  - **<0.0075 for ductile nonstructural elements attached to structure;**
  - **< 0.01 for nonstructural elements not interfering w/ structural response.**
- **Although the recommended ~50% of 475 yr (design) seismic action is a low estimate of the 95 yr seismic action, in concrete, steel or composite frame buildings damage limitation checks control member sizes.**

# Conclusion: In EN1998-1: Eurocode 8 – General

The <u>Design Seismic action</u> is defined as the one for which the No-(life-threatening-)collapse requirement is verified

The Reference Return Period of the <u>Reference Seismic action</u> is a NDP, with a recommended value of 475 years (corrresponding Reference Probability of Exceedance in the structure's design life of 50 years: 10%)

The Reference Seismic action is described (through the national zonation maps) in terms of a single parameter: the <u>Reference Peak Ground Acceleration on Rock</u>, $a_{gR.}$

The <u>design ground acceleration</u> on rock, $a_g$, is the reference PGA times the importance factor: $a_g = \gamma_I a_{gR}$

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

# Definition of Elastic Spectra in Eurocode 8:

- Spectral shape: Defined in National Annex as NDP as function of:
  - Ground type (surface layers, a few tens of m)
  - Earthquake Magnitude
  - (possibly) deep geology below surface deposits.
- Spectral shape: consists of regions of:
  - Constant response spectral pseudo-acceleration
  - Constant response spectral pseudo-velocity
  - Constant response spectral displacement
- Recommended: Two types of horizontal spectra from S. European data:
  - Type 1: High & moderate seismicity regions (distant EQs, $M_s > 5.5$);
  - Type 2: Low seismicity; local EQs ($M_s < 5.5$).

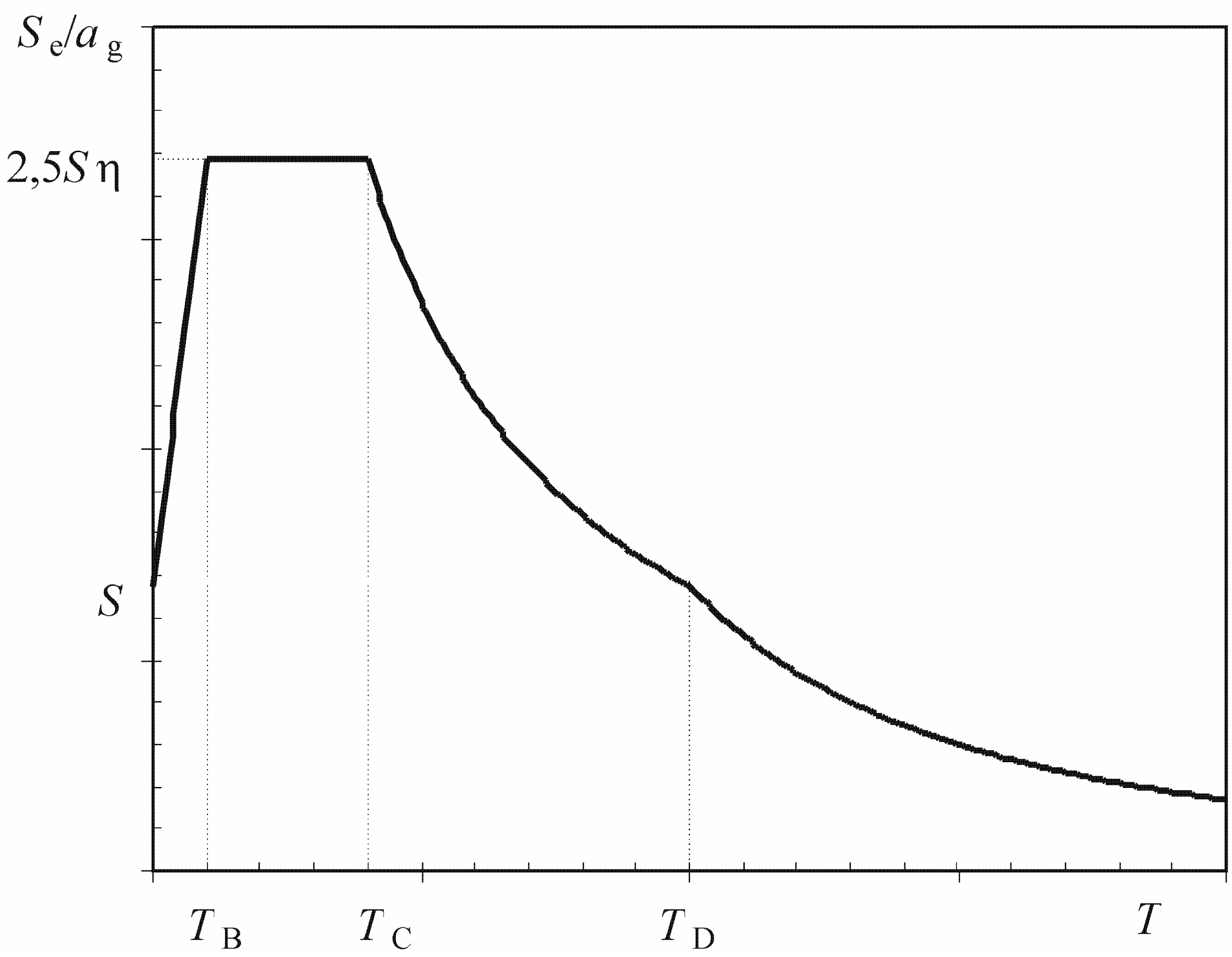
    (High amplification at low T; falls-off sooner with T).
- Detailed ground classification (5 standard ground types defined on the basis of shear-wave velocity in top 30m, plus 2 special ones)

# Standard Ground types

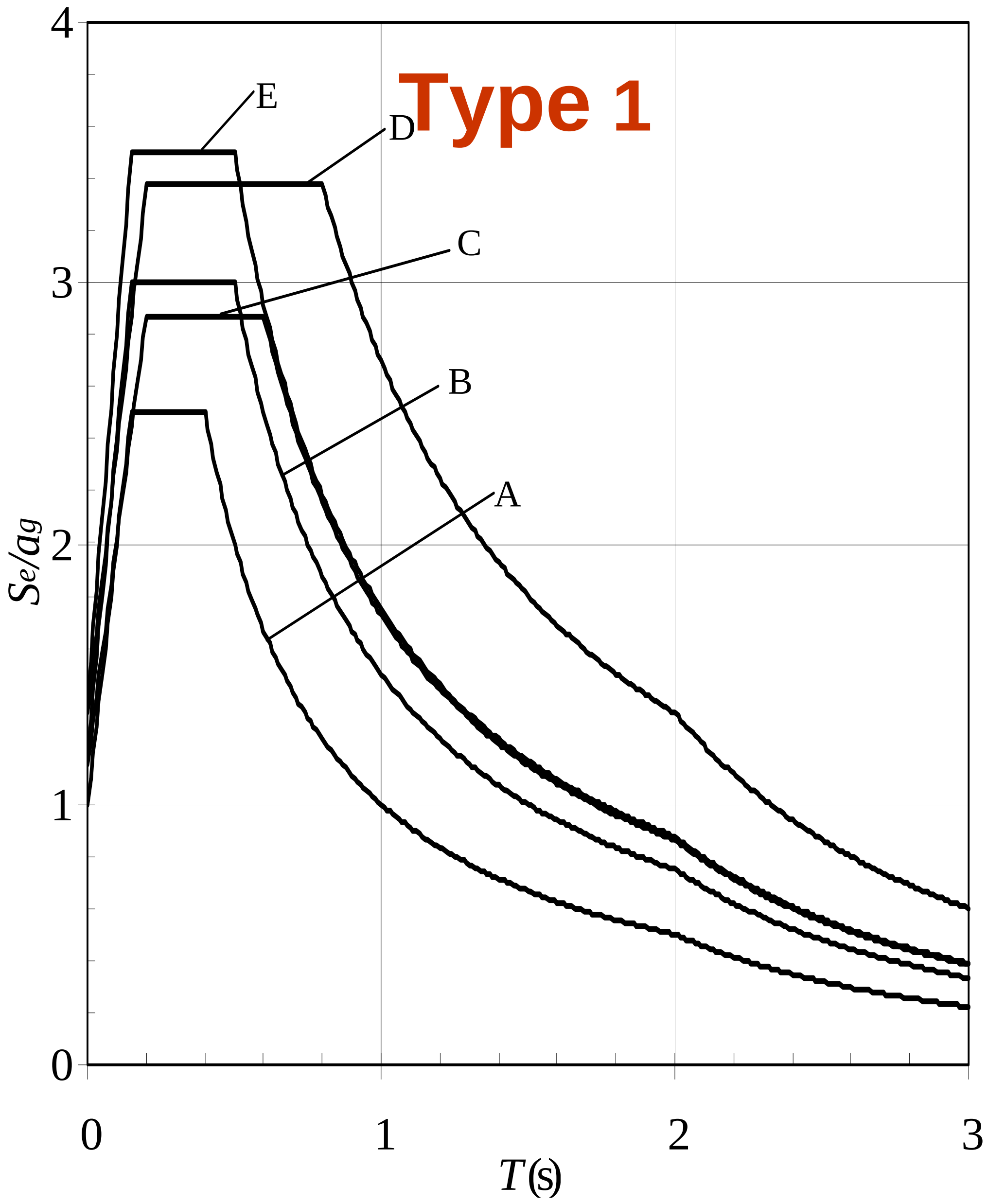
| | | $v_{s,30}$ (m/s) | $N_{SPT}$ | $c_u$ (kPa) |
|---|---|---|---|---|
| A | Rock with ≤5m weaker surface material | >800 | _ | _ |
| B | Very dense sand, gravel or very stiff clay, ≥ several tens of m | 360-800 | >50 | >250 |
| C | Dense or medium-dense sand, gravel or stiff clay, several tens to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

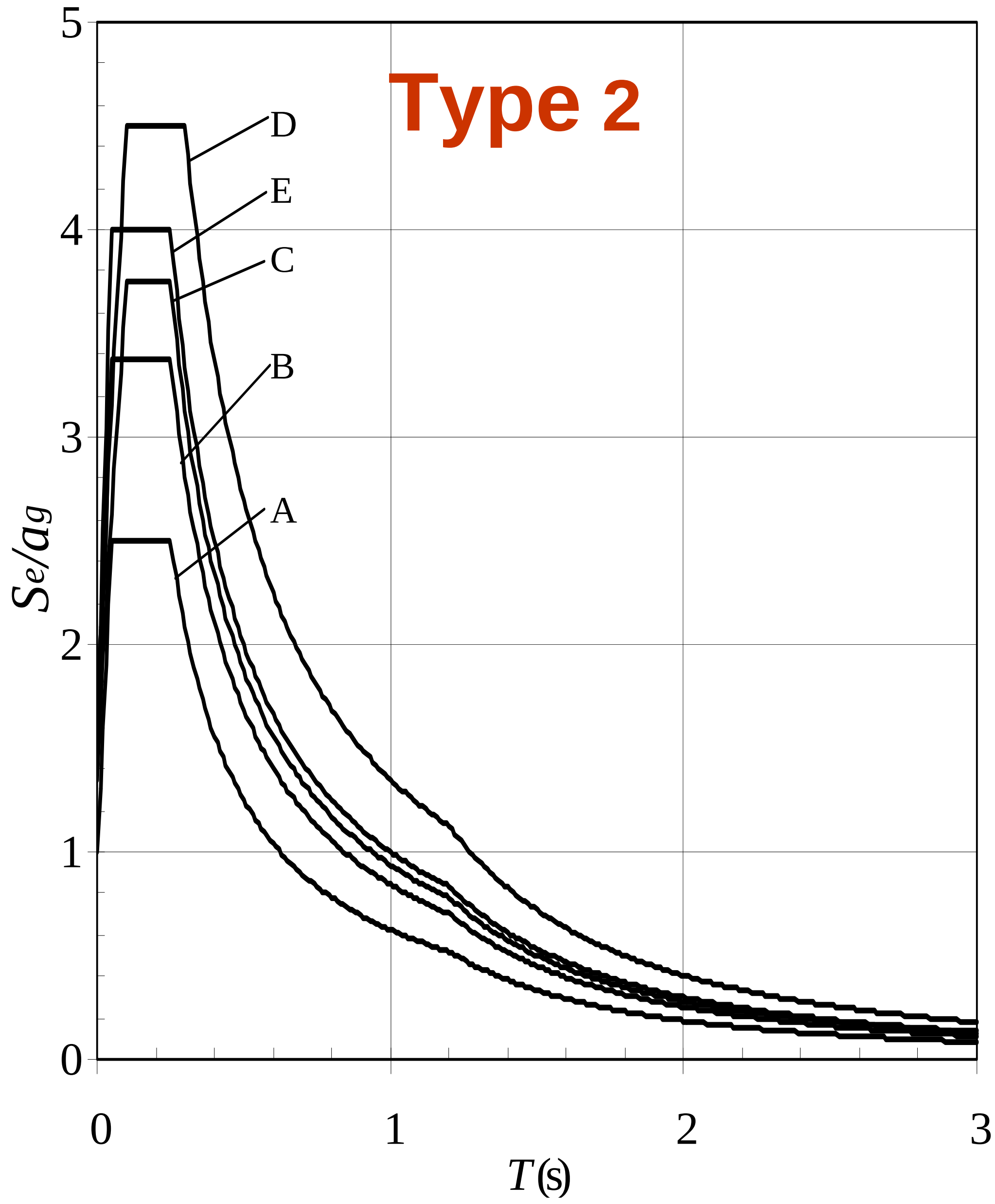
# Standard elastic response spectral shape

- Ranges of constant spectral pseudo-acceleration, pseudo-velocity, displacement, start at corner periods $T_B$, $T_C$, $T_D$.
- Uniform amplification of spectrum by soil factor S (incl. PGA at soil surface, $Sa_g$).
- Damping correction factor $\eta = \sqrt{10/(5+\xi)} \geq 0,55$
- Constant spectral acceleration = 2.5 times PGA at soil surface for horizontal, 3 times for the vertical.
- $T_B$, $T_C$, $T_D$, S: NDPs



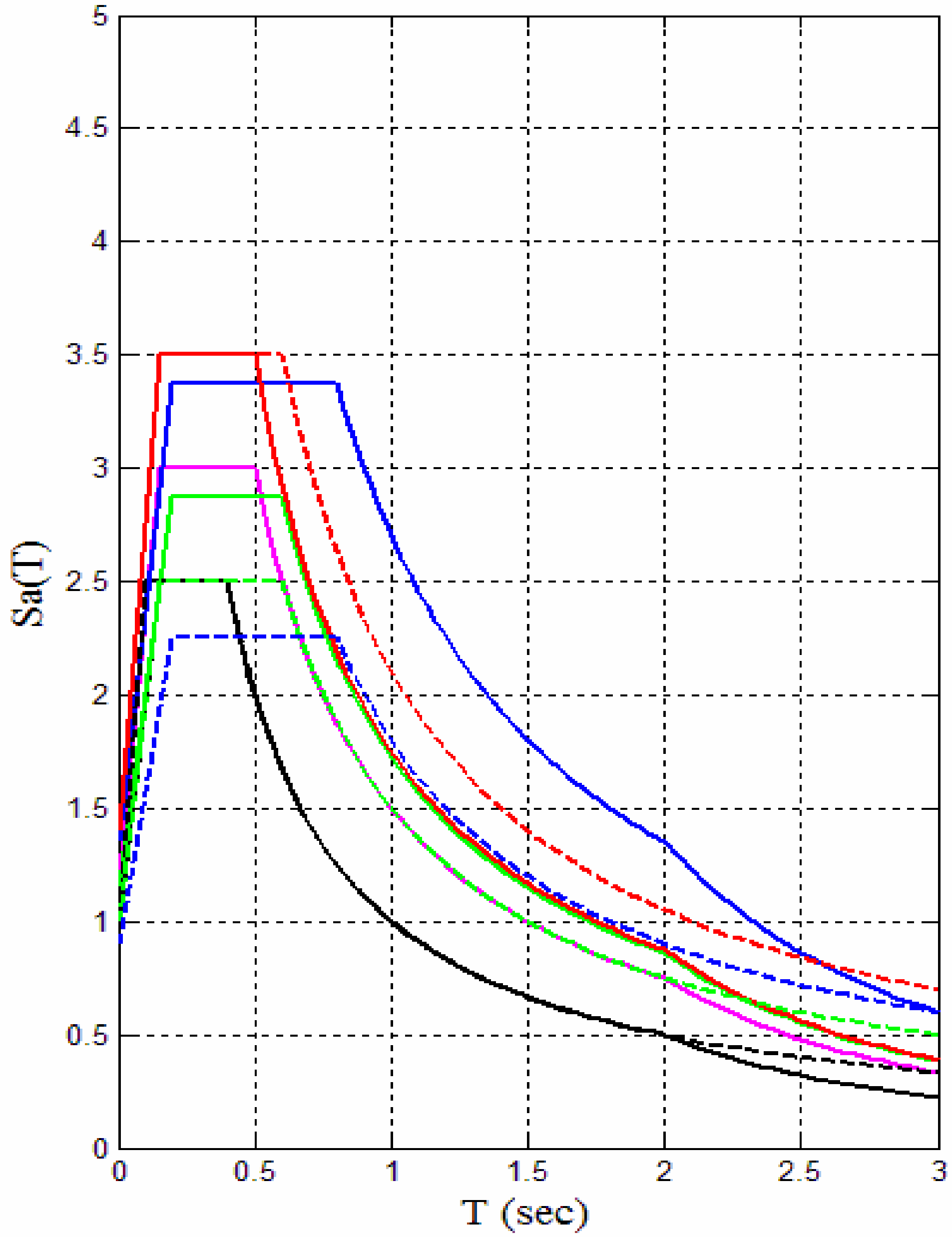
# Recommended horizontal elastic spectra for the standard ground types (5% damping, PGA on rock: 1g)



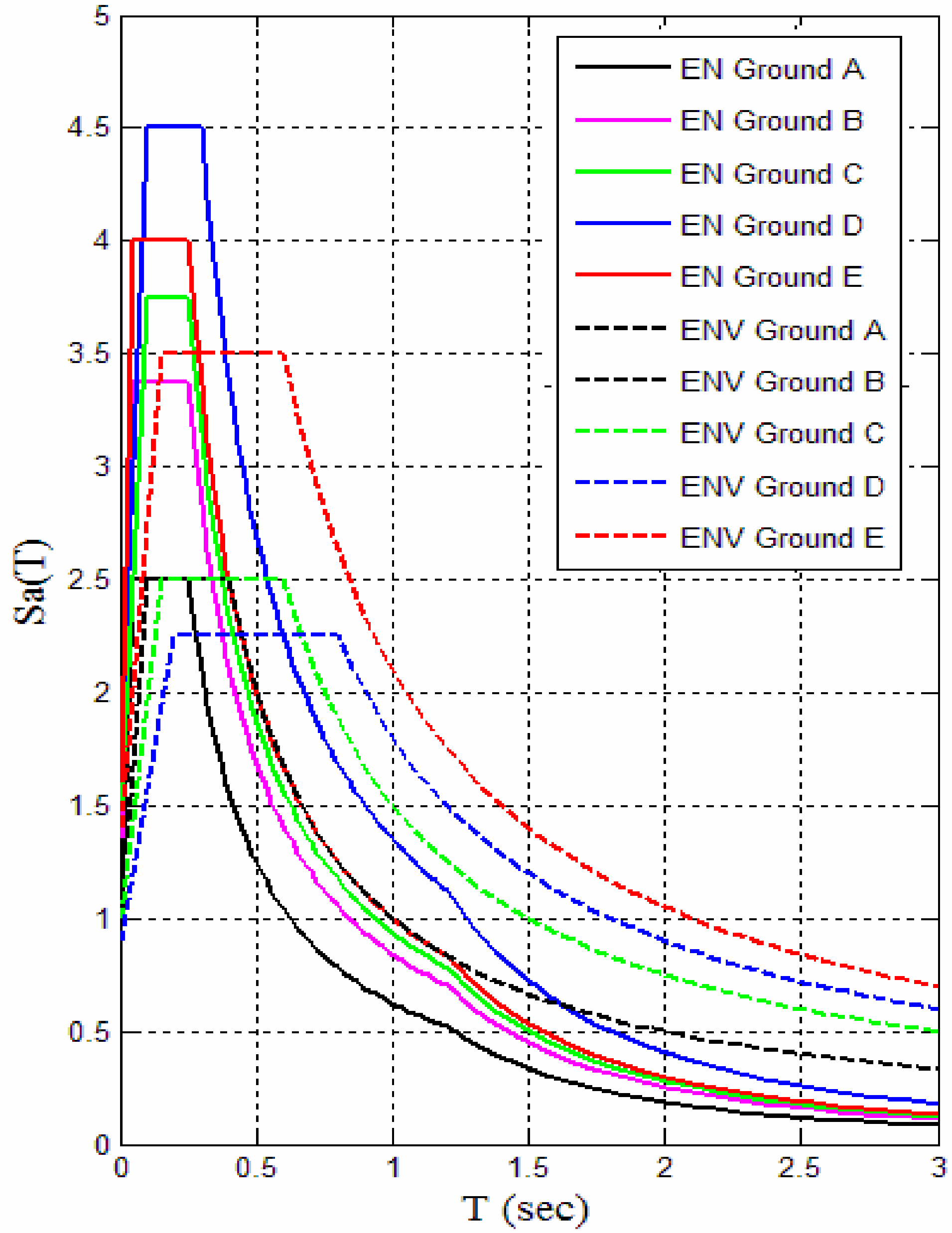



# EN vs. ENV: Elastic Spectrum for 5% damping

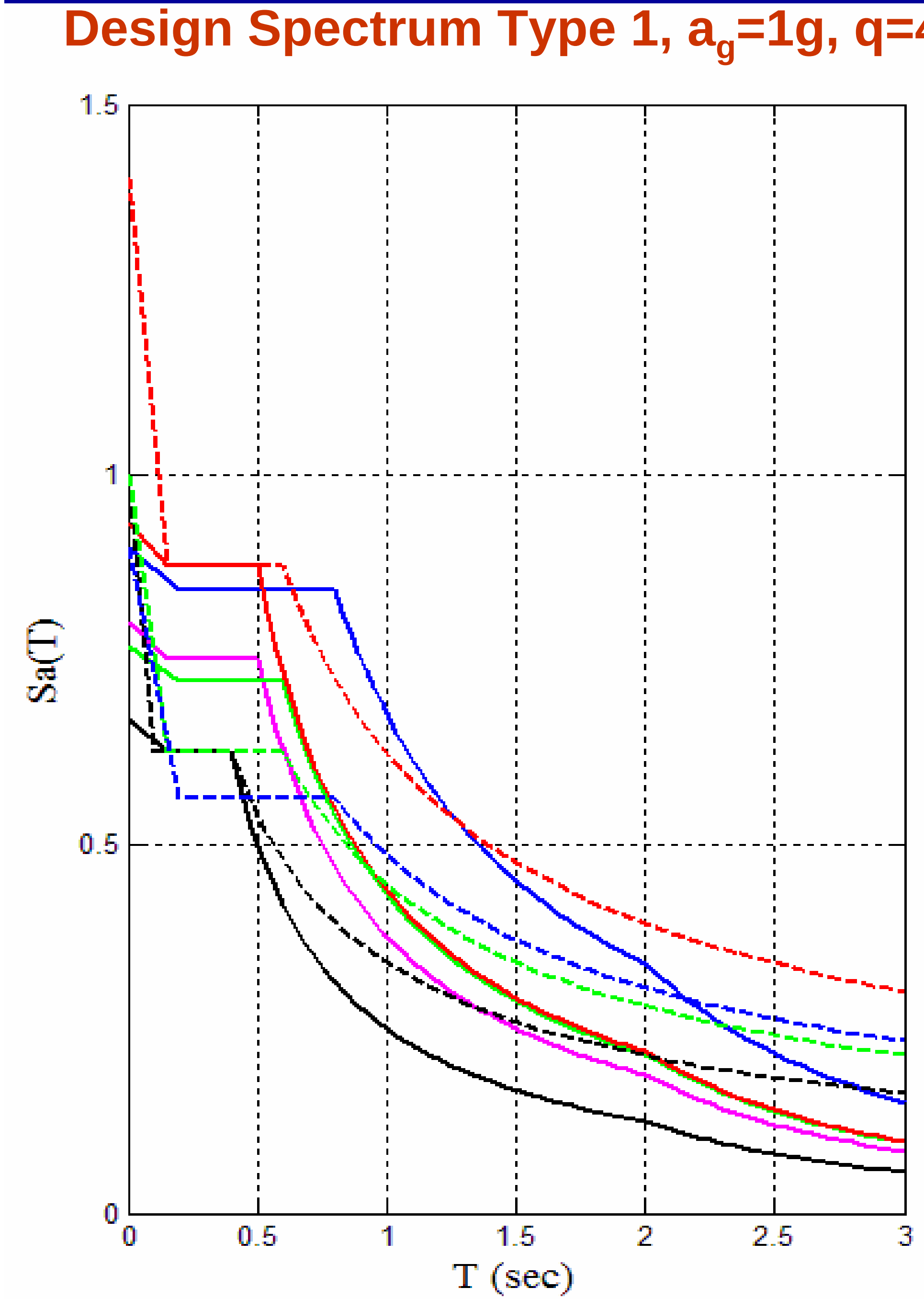
## Elastic Spectrum Type 1, $a_g$=1g

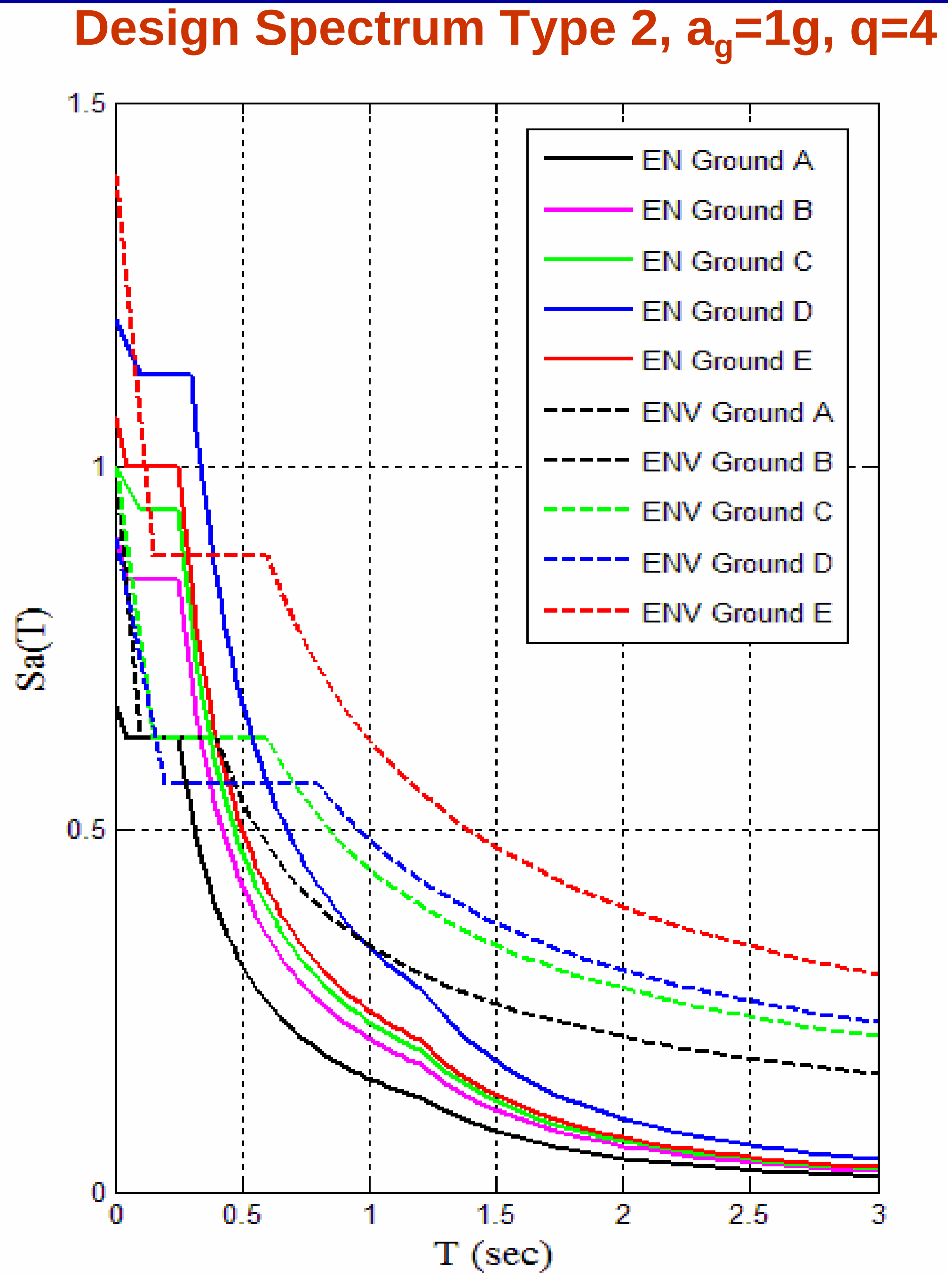



## Elastic Spectrum Type 2, $a_g$=1g



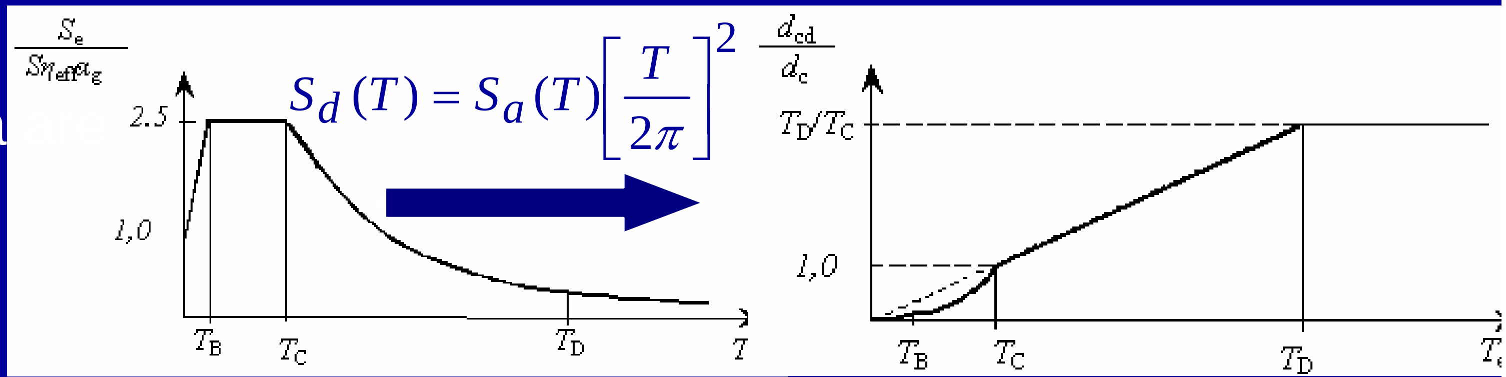
# Design Spectrum (: Elastic Spectrum divided by behaviour factor q) EN v. ENV for q=4



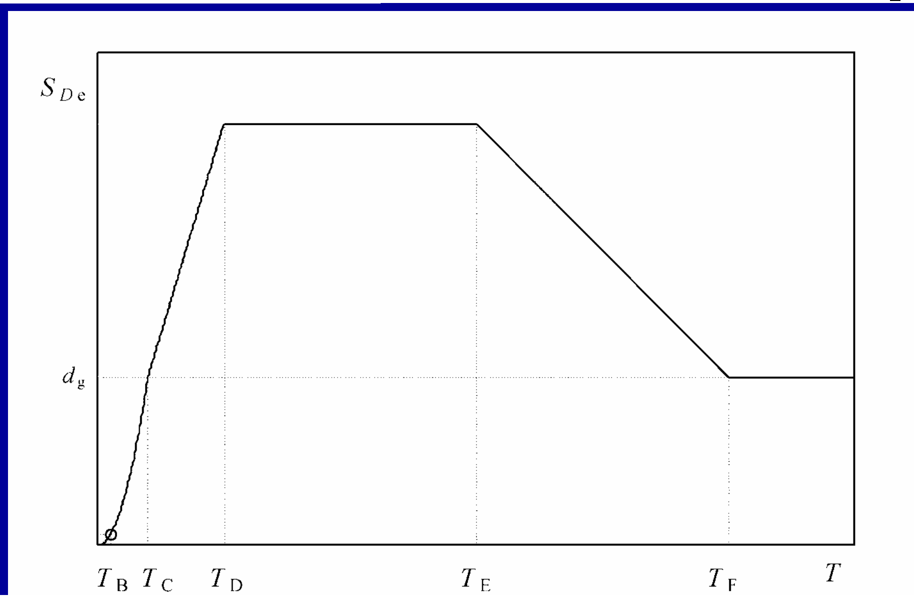



# Horizontal peak ground displacement & (elastic) displacement spectrum

- Peak ground displacement established on the basis of assumed displacement amplification factor of 2.5 in constant spectral displacement region: $$d_g = 0.025 a_g S T_C T_D$$
- Up to T~4s, elastic displacement spectra derived from the acceleration spectra (European data).

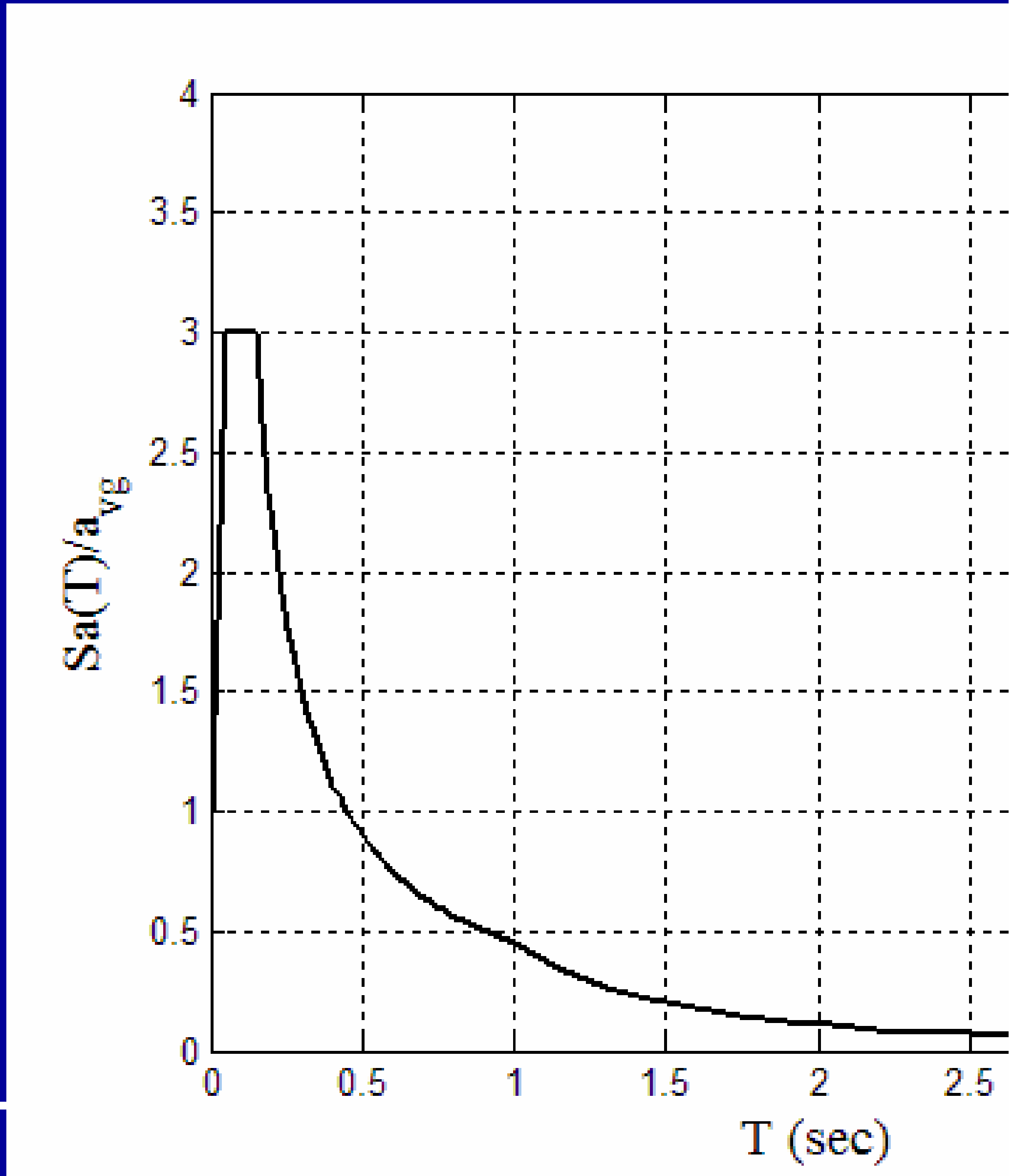



- Informative (non-binding) Annex:
  - Tail of displacement spectra for T>4s, on the basis of combination of data from Europe & Kobe:
  - New corner period $T_E$ depends on ground type;
  - $T_F$=10s.



# Vertical elastic spectra

- Corner periods $T_B$, $T_C$, $T_D$: NDPs
- Recommended:
  - Independent of ground type (insufficient data)
  - $T_B = 0.05s$
  - $T_C = 0.15s$
  - $T_D = 1.0s$
  - Peak vertical ground acceleration:
    - $a_{vg} = 0.9a_g$, if Type 1 spectrum appropriate;
    - $a_{vg} = 0.45a_g$, if Type 2 spectrum.



# Elastic response spectra for the two special ground types ($S_1$ and $S_2$)

- Through a special site-specific study.
- For $S_1$: Establish dependence of response spectrum on thickness and $v_s$ value of soft clay/silt layer and on its stiffness contrast with the underlying materials (low internal damping and abnormally long range of linear behaviour, conducive to anomalous site amplification).
- For $S_2$: Examine possibility of soil failure.

# Other special provisions for seismic actions

- **Topographic amplification** (at the top of ridges or isolated cliffs)
- **Near-source effects**: No general provisions;
  - site-specific spectra required, to take into account near-source effects for <u>bridges</u> <10km from known active fault that can produce Moment Magnitude >6.5
- **Spatial variability** of seismic action for <u>pipelines</u> & <u>bridges</u> with deck continuous over >2/3 of distance beyond which ground motion considered uncorrelated (:NDP, depending on ground type, recommended: from 600m for rock, to 300m for soft soil).
  - Simplified method superimposes (to seismic action effects that neglect motion spatial variability) static effects of postulated relative displacements of supports (in the same or opposite direction) that depend on:
    - peak ground displacement and
    - distance beyond which ground motion is considered uncorrelated.

# Part III:
# Design of new buildings for earthquake resistance, according to Eurocode 8-Part 1

(emphasis on concrete buildings)

# 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

# EN1998-1: DESIGN CONCEPTS FOR SAFETY UNDER DESIGN SEISMIC ACTION

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

   (Or: Foundation elements - incl. piles - designed & detailed for ductility)
2. Design w/o energy dissipation & ductility: $q\leq1.5$ for overstrength; design only according to EC2 - EC7 (Ductility Class “Low”– DCL) Only:
   - for Low Seismicity (NDP; recommended: PGA on rock $\leq$0.08g)
   - for superstructure of base-isolated buildings.

# Force-based design for energy-dissipation & ductility, to meet no-(life-threatening-)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*-*δ* curve, in monotonic loading.
- For given period, *T*, of elastic SDoF system, inelastic spectrum relates:
  - ratio $q = F_{el}/F_y$ of peak force, $F_{el}$, that would develop if the SDoF system was linear-elastic, to its yield force, $F_y$, (“behaviour factor”)

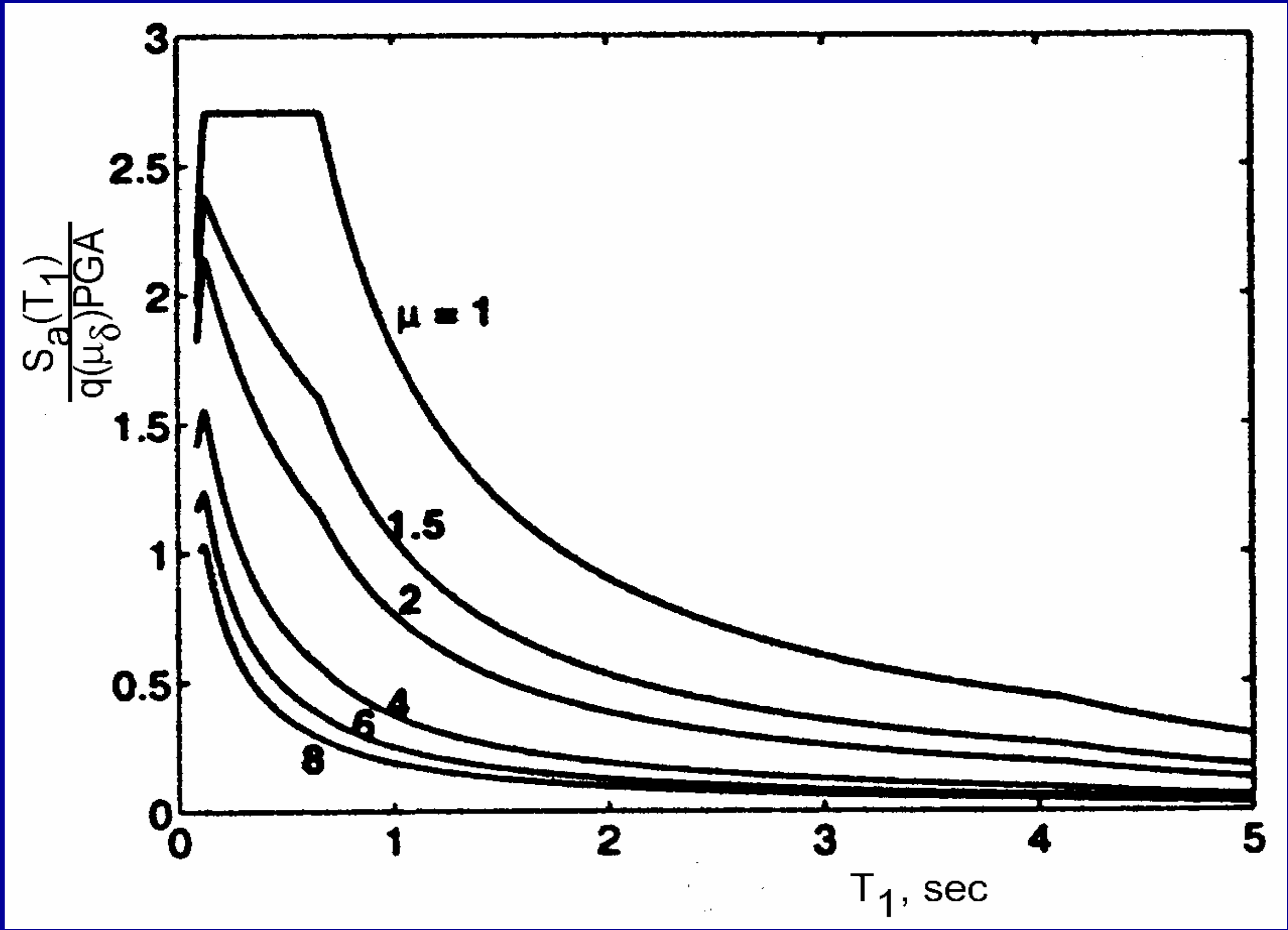
  to

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

# Inelastic spectra (Vidic et al) adopted in Eurocode 8

$$\mu_\delta = q \qquad \text{if } T \geq T_C$$

$$\mu_\delta = 1 + (q-1)\frac{T_C}{T} \qquad \text{if } T < T_C$$




Inelastic spectra for $T_C$=0.6s normalised to peak ground acceleration, PGA

# Trading-off ductility against strength in earthquake-resistant design (ductility as an alternative to strength)

$$\mu_\delta = 1 + (q-1)\frac{T_C}{T} \qquad \text{if } T < T_C$$

$$\mu_\delta = q \qquad \text{if } T \geq T_C$$

- The higher the value of $\mu_\delta$, the lower is the required strength

## Control of inelastic seismic response: Soft-storey collapse mechanism, to be avoided through proper structural configuration:

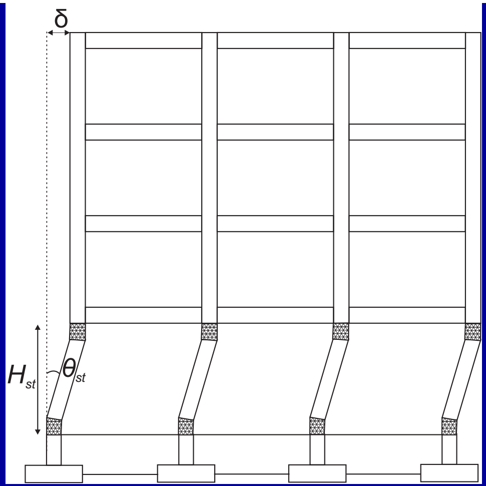
– Strong-column/weak beam frames, with beam-sway mechanisms, involving:

plastic hinging at all beam ends, and
either plastic hinging at column bottoms, or
rotations at the foundation.

– Wall-equivalent dual frames, with beam-sway mechanism, involving:

plastic hinging at all beam ends, and
either plastic hinging at wall & column bottoms, or
rotations at the foundation.

Soft-storey collapse mechanism, to be avoided through proper structural configuration:




–Strong-column/weak beam frame, with beam-sway mechanism, involving:
plastic hinging at all beam ends, and
either plastic hinging at column bottoms, or
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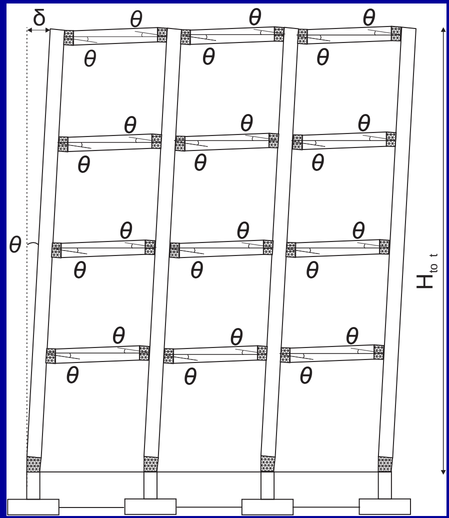
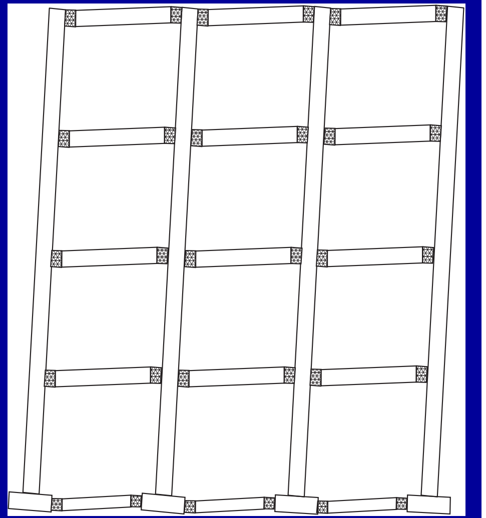
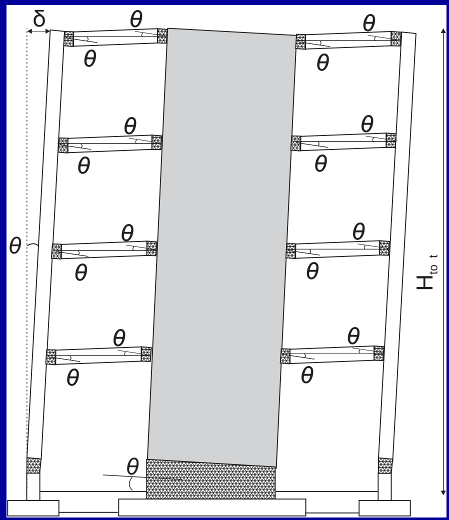
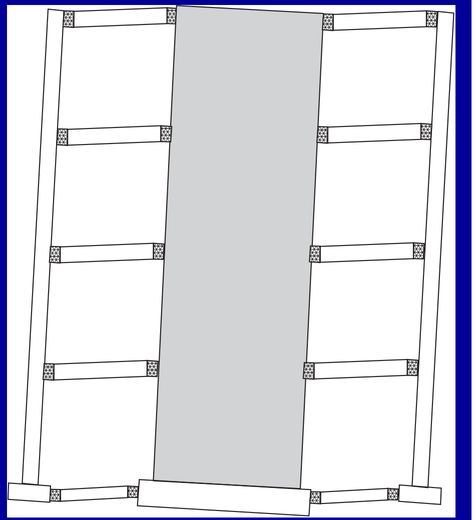


–Wall-equivalent dual frame, with beam-sway mechanism, involving:
plastic hinging at all beam ends, and
either plastic hinging at wall & column bottoms, or
rotations at the foundation.



# Control of inelastic seismic response through capacity design

- 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, $R_d$, 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, $q$)

## Criteria for the selection of elements where inelastic deformations are allowed to take place, instead of being capacity-designed to stay in the elastic range:

- “Ductility”: the inherent capacity of the element to develop large inelastic deformations & dissipate energy under cyclic loading, without substantial loss of its force-resistance.
- The importance of the element for the stability of other elements & the integrity of the whole (greater importance of vertical elements compared to the horizontal ones; importance increases from the top of the building to its foundation).
- The accessibility of the element and the difficulty to inspect & repair any damage.

# EC8-PART 1: FOR ALL MATERIALS:

- 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)

# ANALYSIS METHODS

## (& CORRESPONDING MEMBER VERIFICATION CRITERIA)

- Reference method:

  Linear modal response spectrum procedure, with elastic spectrum reduced by (behaviour-factor) q:
  - Applicable in all cases, except in base-isolated structures w/ (strongly) nonlinear isolation devices.
- If building heightwise regular & higher-modes unimportant ($T<4T_c$, $T<2s$):

  (Linear) Lateral force procedure, emulating response-spectrum method:
  - T from mechanics; reduction of forces by 15% if >2 storeys & $T<2T_c$
- Nonlinear analysis, static (pushover) or dynamic (t-history), for:
  - Evaluation of system overstrength factor in redundant systems;
  - Performance evaluation of existing or retrofitted buildings;
  - Design with direct check of deformations of ductile members, w/o q-factor.
- Member verification at the ULS (for “Life-Safety” EQ):
  - In terms of forces (resistances), except:
  - If nonlinear analysis ductile failure modes checked in terms of deformations

# EC8-Part 1: REGULARITY OF BUILDINGS IN ELEVATION
## (FOR APPLICABILITY OF LATERAL FORCE PROCEDURE & FOR VALUE OF BEHAVIOUR FACTOR, q)

- 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.*

- Heightwise irregular buildings: q-factor reduced by 20%

# EC8-Part 1: REGULARITY OF BUILDINGS IN PLAN
(FOR ANALYSIS OF TWO SEPARATE PLANAR/2D MODELS)

Criteria can be checked before any analysis:

- *K* & *m* ~ symmetric w.r.to 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:
  - $r$ *(torsional radius of struct. system)* $\geq l_s$ *(radius of gyration of floor plan)*:
    Translational fundamental T(s) > torsional.
  - $e_o$ *(eccentricity between floor C.S. & C.M.)* $\leq 0.3\ r$:
    Conservative bound to satisfactory performance (element ductility demands ~ same as in torsionally balanced structure).

Alternative for buildings ≤ 10m tall:

- In both horizontal directions: $r^2 \geq l_s^2 + e_o^2$

# 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 $\leq$ 15%;
    - Regularity classification does not change.

# LINEAR ANALYSIS FOR DESIGN SEISMIC ACTION –
## ULS MEMBER VERIFICATION - COMPLIANCE CRITERIA FOR LIFE SAFETY

- 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 in seismic isolation with very nonlinear devices)
- If:
  - building regular in elevation &
  - higher modes unimportant

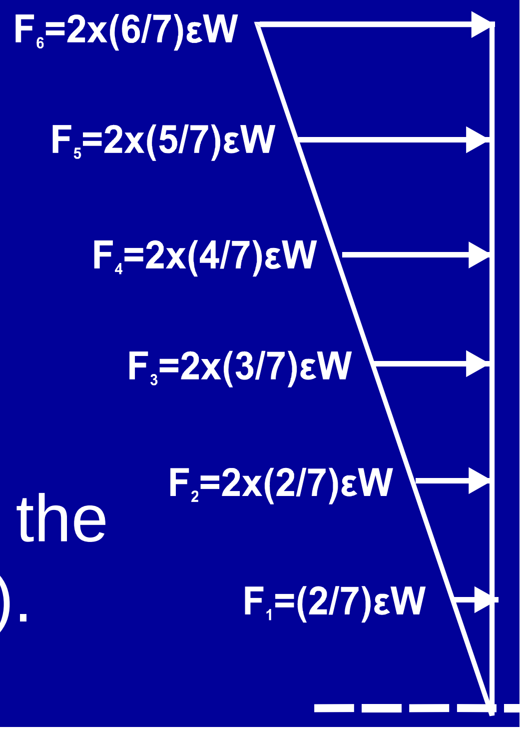
    (fundamental T <4$T_c$ & <2sec, $T_C$: T at end of constat spectral acceleration plateau):

  (linear) Lateral force procedure emulating response-spectrum method:
  - T from mechanics (Rayleigh quotient);
  - Reduction of forces by 15% if >2 storeys & T<2$T_c$

– Member verification at the Ultimate Limit State (ULS) for “Life-Safety” EQ in terms of forces (resistances)

# LINEAR ANALYSIS FOR DESIGN SEISMIC ACTION *Cont'd*

- Reference approach is modal response spectrum analysis, with design spectrum:
  - Number of modes taken into account:
    - All those with modal mass ≥ 5% of total in one of the directions of application of the seismic action;
    - Sufficient to collectively account for ≥ 90% of total mass in each direction of application of the seismic action.
  - Combination of modal responses:
    - CQC (Complete Quadratic Combination);
    - SRSS (Square-Root-of-Sum-of-Squares) if ratio of successive modal periods < 0.9 & > 1/0.9.
- Lateral force procedure:
  - Static lateral forces on storey or nodal masses proportional to the mass times its distance from the base (inverted triangular heightwise distribution).



# ANALYSIS FOR ACCIDENTAL TORSION

- Accidental displacement of masses in the direction normal to the horizontal seismic action component, by:
  - $e_i = \pm 0.05 L_i$ ($\pm 0.1 L_i$ if there are irregular-in-plan masonry infills), where $L_i$ : plan dimension normal to the horizontal seismic action component and parallel to $e_i$
- Taken into account by means of:

1. Linear static analysis under torques (w.r.to vertical axis) on storey or nodal masses equal to the storey or nodal forces of the lateral force procedure, times $e_i = 0.05 L_i$ (same sign at all storeys or nodes)
2. Superposition of the action effects due to the analysis in 1, to the seismic action effects due to the horizontal seismic action components w/o the accidental eccentricity (from lateral force or modal response spectrum procedure), with the same sign as the seismic action effect due to the horizontal seismic action component.

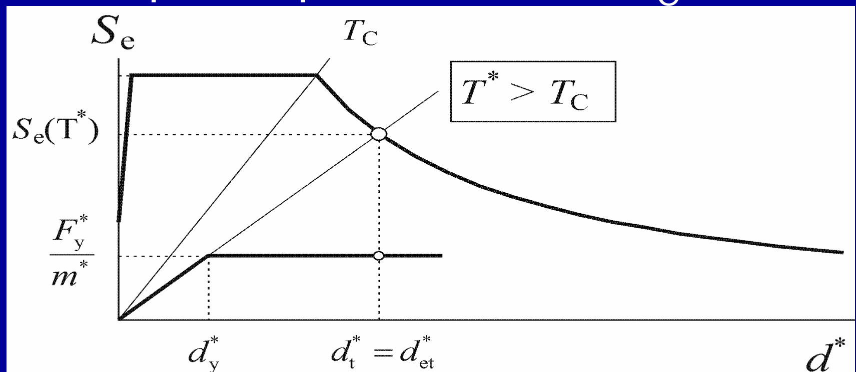
# $2^{nd}$-ORDER (P-Δ) EFFECTS IN ANALYSIS

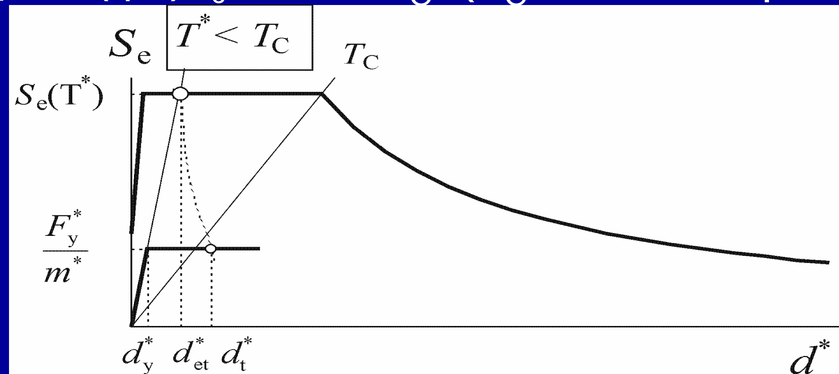
- $2^{nd}$-order effects taken into account at the storey level (index: i) through their ratio to the $1^{st}$-order effects of the seismic action (in terms of storey moments): $\theta_i = N_{tot,i}\Delta\delta_i / V_i H_i$
  - $N_{tot,i}$= total vertical load at and above storey i in seismic design situation;
  - $\Delta\delta_i$ = interstorey drift at storey i in seismic design situation, equal to that calculated from the linear analysis for the design spectrum, times the behaviour factor q;
  - $V_i$ = storey shear in storey i in seismic design situation;
  - $H_i$ = height of storey i.
- If $\theta_i \leq 0.1$ at all storeys, $2^{nd}$-order effects may be neglected (this is normally the case, as indirect consequence of interstorey drift limitation under damage-limitation seismic action);
- If $\theta_i > 0.1$ at any storey, $2^{nd}$-order effects are taken into account by dividing all $1^{st}$-order effects from the linear analysis by $(1-\theta_i)$;
- $\theta_i > 0.2$ at any storey to be avoided (never the case, thanks interstorey drift limitation under damage-limitation seismic action).
- In buildings designed for the seismic action, $2^{nd}$-order effects in the persistent-and-transient design situation are always negligible.

# NONLINEAR ANALYSIS FOR DESIGN SEISMIC ACTION – ULS MEMBER VERIFICATION - COMPLIANCE CRITERIA FOR LIFE SAFETY

- Allowed: Displacement-based design, w/o q-factor:

– Nonlinear analysis, static (pushover) or dynamic (t-history)
  – Fairly detailed rules for calculation of deformation demands.
  – For pushover analysis (N2 method):
    - Target displacement from 5%-damped elastic spectrum (Vidic et al, ’94):
      – equal displacement if $T>T_C$ $\mu=1+(q-1)T_c/T$, if $T<T_C$ ($T_C$: transition period)




– Member verification at the ULS (for “Life-Safety” EQ) in terms of:
  – deformations in ductile members/mechanisms (no deformation limits given);
  – forces (resistances) for brittle members/mechanisms

– Gap: Deformation capacities delegated to National Annexes
  →Part 3 (Assessment & retrofit) fills the gap (National Annex may refer there).

# COMBINATION OF ACTION EFFECTS OF INDIVIDUAL SEISMIC ACTION COMPONENTS

- For linear analysis, or nonlinear static (Pushover) analysis:
  - Rigorous approach : SRSS-combination of seismic action effects EX, EY, EZ of individual components X, Y, Z: $E=\pm\sqrt{(EX^2+EY^2+EZ^2)}$
    - Very convenient for modal response spectrum analysis (single analysis for all components X, Y, Z and combination done simultaneously with that of modal contributions).
  - Approximation:

$$E=\pm\max(|EX|+0.3|EY|+0.3|EZ|;\ |EY|+0.3|EX|+0.3|EZ|;\ |EZ|+0.3|EX|+0.3|EY|).$$

  - In nonlinear static (Pushover) analysis, component Z is always neglected and internal forces from above combinations cannot exceed member force resistances
- For time-history nonlinear analysis:
  - Seismic action components X, Y, Z applied simultaneously.

# 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-$\Delta$ effects (governs sizes of frame members).

# Implementation of Eurocode 8 seismic design philosophy

1. 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.
2. Member verification for the Ultimate Limit State (ULS) in bending under the "design seismic action", with elastic spectrum reduced by the behaviour factor q.
3. 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.
4. Capacity design of members and joints in shear.
5. Detailing of plastic hinge regions, on the basis of the value of the curvature ductility factor that corresponds to the q-factor value.

# EC8-PART 1: DAMAGE LIMITATION CHECK

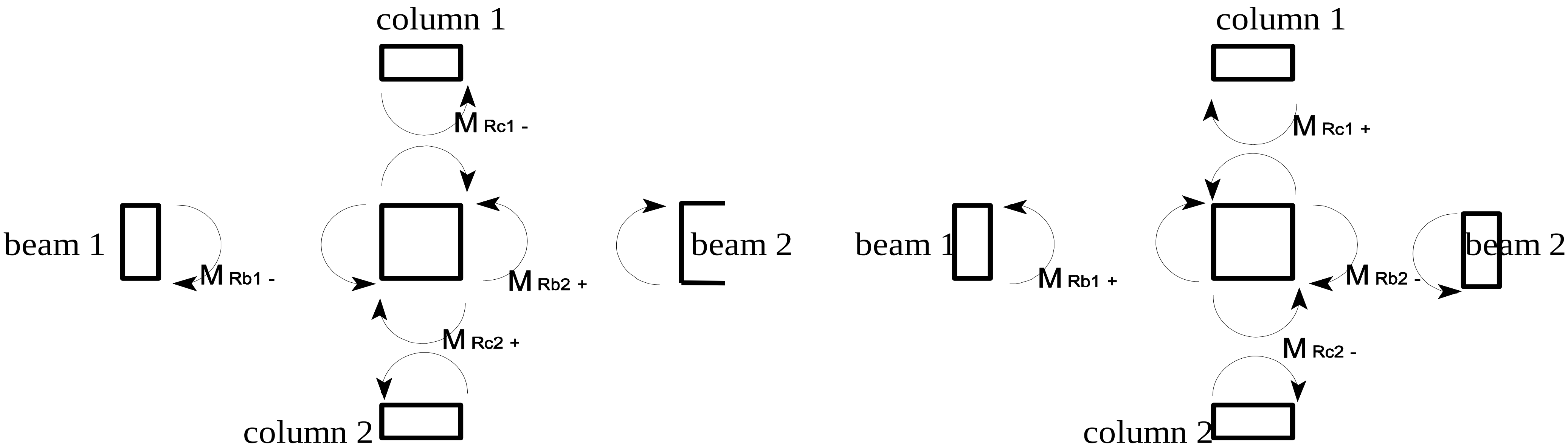
- Seismic action for “damage limitation”: NDP.
  Recommended for ordinary buildings: 10%/10yrs (95yr EQ);
  ~50% of “design seismic action” (475yr EQ).
- Interstorey drift ratio calculated for “damage limitation” action via “equal displacement rule” (elastic response):
  - <0.5% for brittle nonstructural elements attached to structure;
  - <0.75% for ductile nonstructural elements attached to structure;
  - < 1% for nonstructural elements not present or not interfering w/ 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.

**Fulfilment of strong column/weak beam capacity design rule, with overstrength factor $\gamma_{Rd}$ on beam strengths:**

$$\sum M_{Rc} \geq \gamma_{Rd} \sum M_{Rb}$$

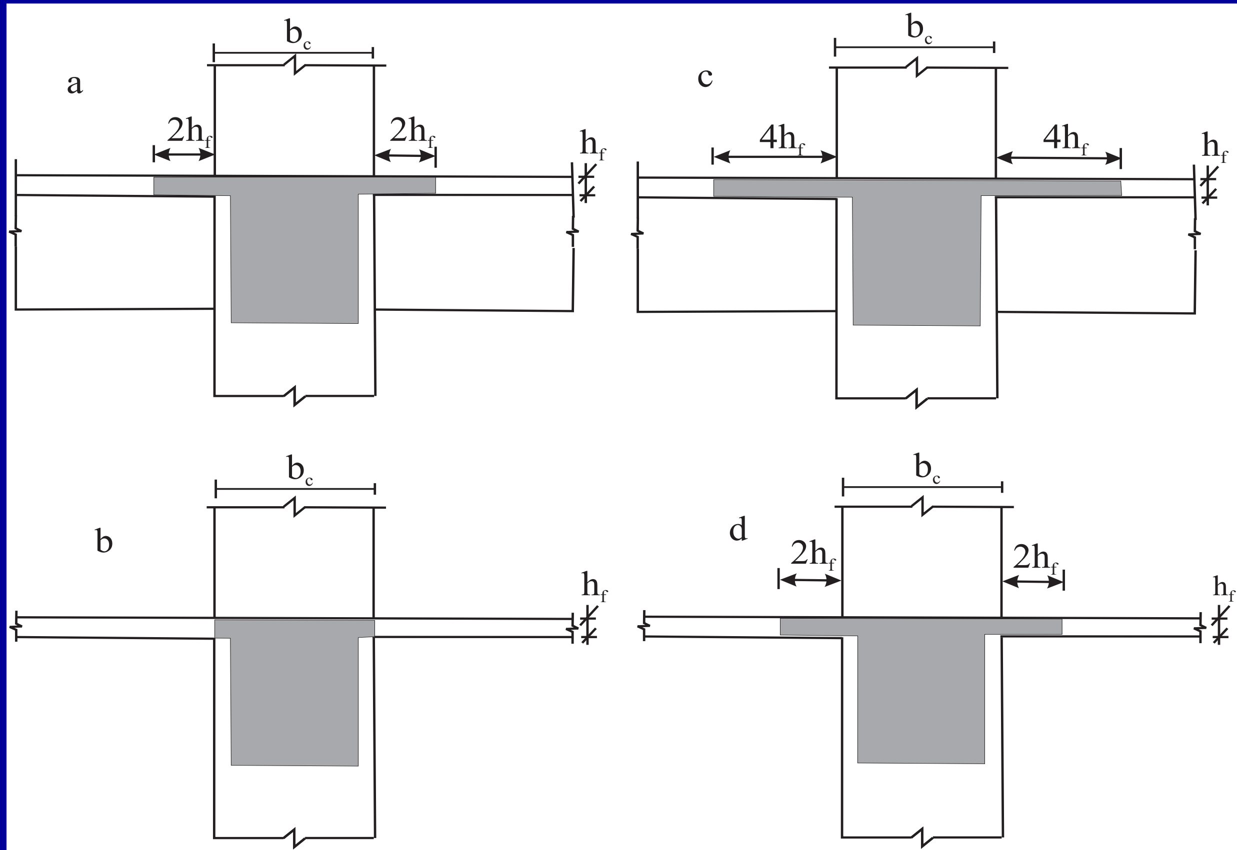
- Eurocode 8: $\gamma_{Rd}$ = 1.3; strong column/weak beam capacity design required only in frames or frame-equivalent dual systems (frames resist >50% of seismic base shear) above two storeys (except at top storey joints).

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



**But:**

Width of slab effective as tension flange of beams at the support to a column:




Eurocode 8 (a, b: at exterior column; c, d: at interior column): small – is it safe for capacity design?

# NDP-partial factors for materials, in ULS verifications:

- Except for timber buildings:
  - Recommended: use same values as for persistent & transient design situations (i.e. in concrete buildings: $\gamma_c$=1.5, $\gamma_s$=1.15);
- Timber buildings:
  - In DC L (Low): Same values as for persistent & transient design situations;
  - In DC M (Medium), or H (High): Same values as for accidental design situations.

# 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 $\gamma_{Rd}(R_{di}/E_{di})\leq q$, where $R_{di}$ force capacity in the dissipative zone or element controlling the seismic action effect of interest, $E_{di}$ the seismic action effect there from the elastic analysis and $\gamma_{Rd}$=1.2
    - For individual spread footings of walls or columns of moment-resisting frames, $R_{di}/E_{di}$ is the minimum value of $M_{Rd}/M_{Ed}$ in the two orthogonal principal directions at the lowest cross-section of the vertical element where a plastic hinge can form in the seismic design situation;
    - For individual spread footings of columns of concentric braced frames, $R_{di}/E_{di}$ is the minimum value of $N_{pl.Rd}/N_{Ed}$ among all diagonals which are in tension in the particular seismic design situation; for eccentric braced frames, $R_{di}/E_{di}$ is the minimum value of $V_{pl.Rd}/V_{Ed}$ and $M_{pl.Rd}/M_{Ed}$ among all seismic links of the frame;
    - For common foundations of more than one elements, $\gamma_{Rd}(R_{di}/E_{di})$ =1.4.

# STRUCTURE OF EN1998-1:2004

1 General
2 Performance Requirements and Compliance Criteria
3 Ground Conditions and Seismic Action
4 Design of Buildings
5 <u>Specific Rules for Concrete Buildings</u>
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

**buildings**

# Eurocode 8 definitions:

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

Eurocode 2 definition of wall: ≠ column in that cross-section is

# Seismic Design Philosophy for RC buildings according to Eurocode 8

- **Ductility Classes (DC)**
  - Design based on **energy dissipation** and **ductility**:
    - **DC** (**M**) **Medium** q=3 x system overstrength factor (≈1.3).
    - **DC** (**H**) **High** q= 4-4.5 x system overstrength factor (≈1.3).
- The aim of the design is to control the **inelastic seismic response**:
  - Structural **configuration** & relative **sizing** of members to ensure a beam-sway mechanism.
  - **Detailing** of plastic hinge regions (beam ends, base of columns) to sustain inelastic deformation demands.
- Plastic hinge regions are detailed for **deformation demands** related to **behaviour factor q**:
  - $\mu_\delta = q$ if $T > T_c$
  - $\mu_\delta = 1 + (q-1) T_c / T$ if $T \le T_c$

# Material limitations for "primary seismic elements"

| Ductility Class | DC L (Low) | DC M (Medium) | DC H (High) |
|---|---|---|---|
| Concrete grade | No limit | ≥ C16/20 | ≥ C16/20 |
| Steel class per EN 1992-1-1, Table C1 | B or C | B or C | only C |
| longitudinal bars | | only ribbed | only ribbed |
| Steel overstrength: | No limit | No limit | $f_{yk,0.95} \leq 1.25 f_{yk}$ |

# Basic value, $q_o$, of behaviour factor for regular in elevation concrete buildings in Eurocode 8

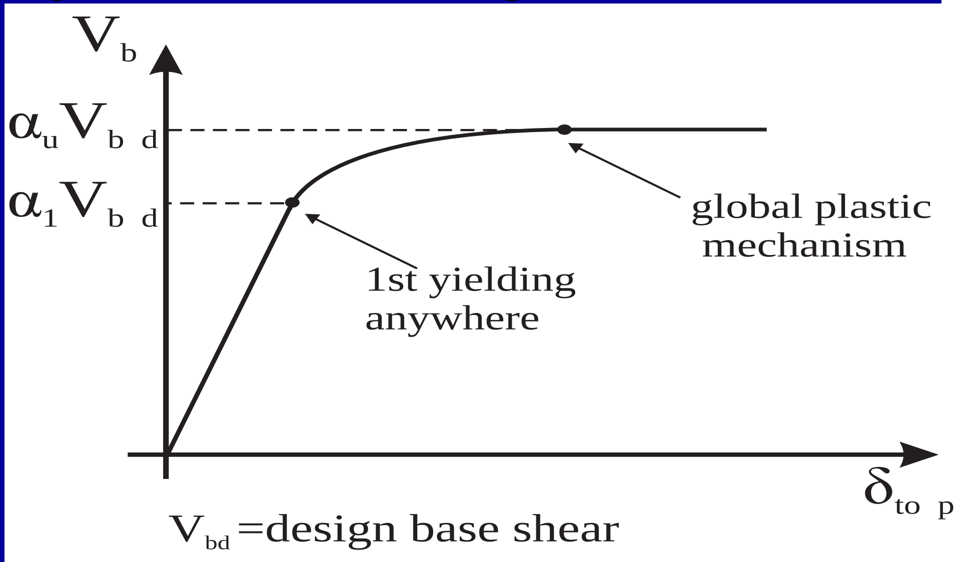
| **Lateral-load resisting structural system** | **DC M** | **DC H** |
|---|---|---|
| Inverted pendulum system* | 1.5 | 2 |
| Torsionally flexible structural system** | 2 | 3 |
| Uncoupled wall system (> 65% of seismic base shear resisted by walls; more than half by uncoupled walls) not belonging in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

*** :** at least 50% of total mass in upper-third of the height, or with energy dissipation at base of a single element (except one-storey frames w/ all columns connected at the top via beams in both horizontal directions in plan & with max. value of normalized axial load $\nu_d$ in combination(s) of the design seismic action with the concurrent gravity loads ≤ 0.3).

**** :** 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 $(1+a_o)/3 \leq 1$, ($a_o$: prevailing wall aspect ratio = $\Sigma H_i/\Sigma l_{wi}$).

# $\alpha_u/\alpha_1$ in behaviour factor of buildings designed for ductility: due to system redundancy & overstrength




Normally:

$\alpha_u$ & $\alpha_1$ from base shear - top displacement curve from pushover analysis.

- $\alpha_u$: seismic action at development of global mechanism;
- $\alpha_1$: seismic action at 1st flexural yielding anywhere.

- $\alpha_u/\alpha_1 \leq 1.5$;
- 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 value of buildings regular in plan and 1.0

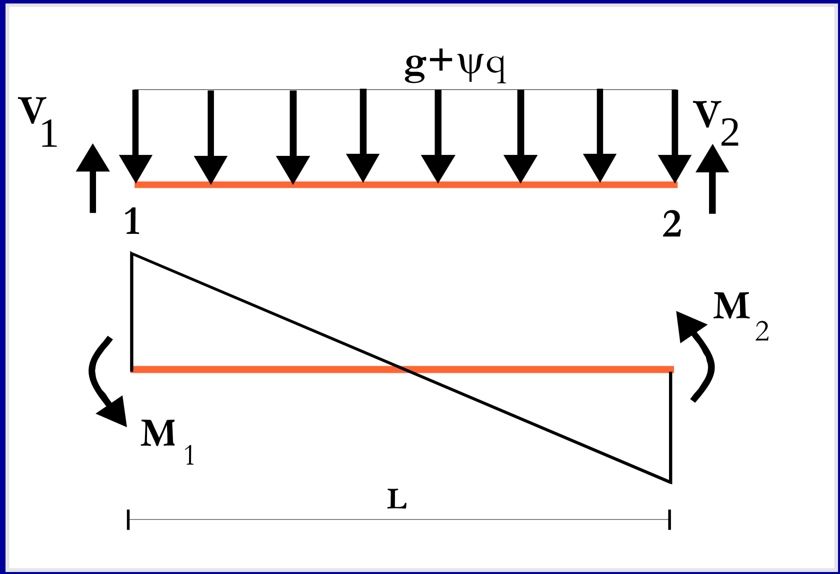
# Capacity design of members, against pre-emptive shear failure

# I. Beams

Equilibrium of forces and moments on a beam

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

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

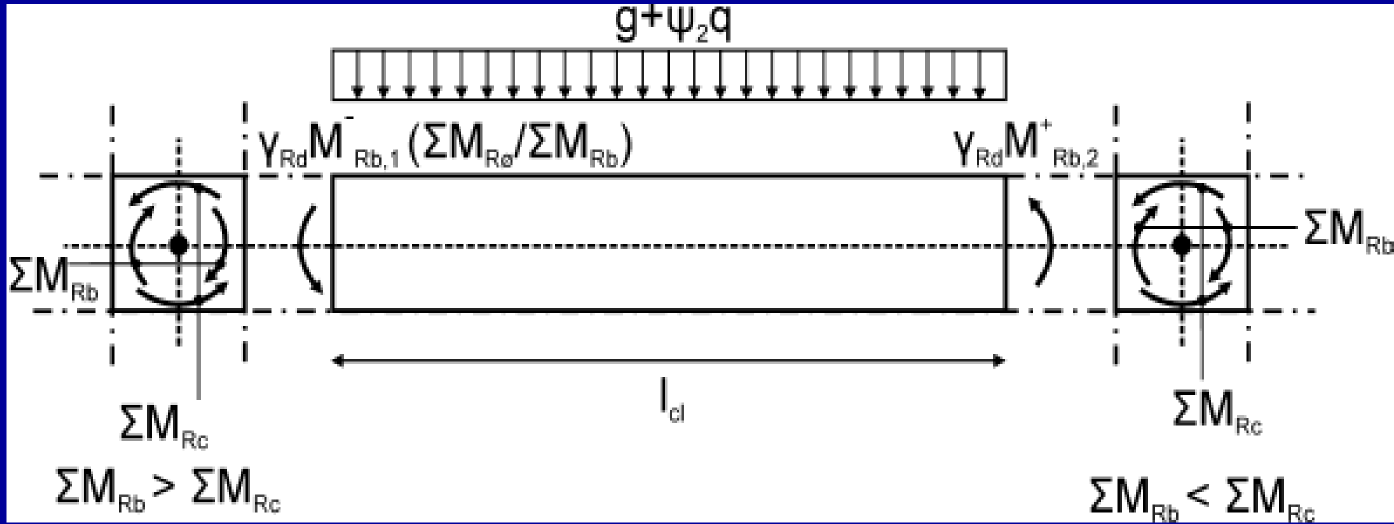



**Capacity-design shear in a beam weaker than the columns:**

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

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

# Capacity-design shear in beams (weak or strong) - Eurocode 8




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

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

Eurocode 8:

- in DC M $\gamma_{Rd}$=1.0,
- in DC H $\gamma_{Rd}$=1.2 & reversal of V accounted for, depending on: $\zeta_i = \frac{\min V_{i,d}(x_i)}{\max V_{i,d}(x_i)}$

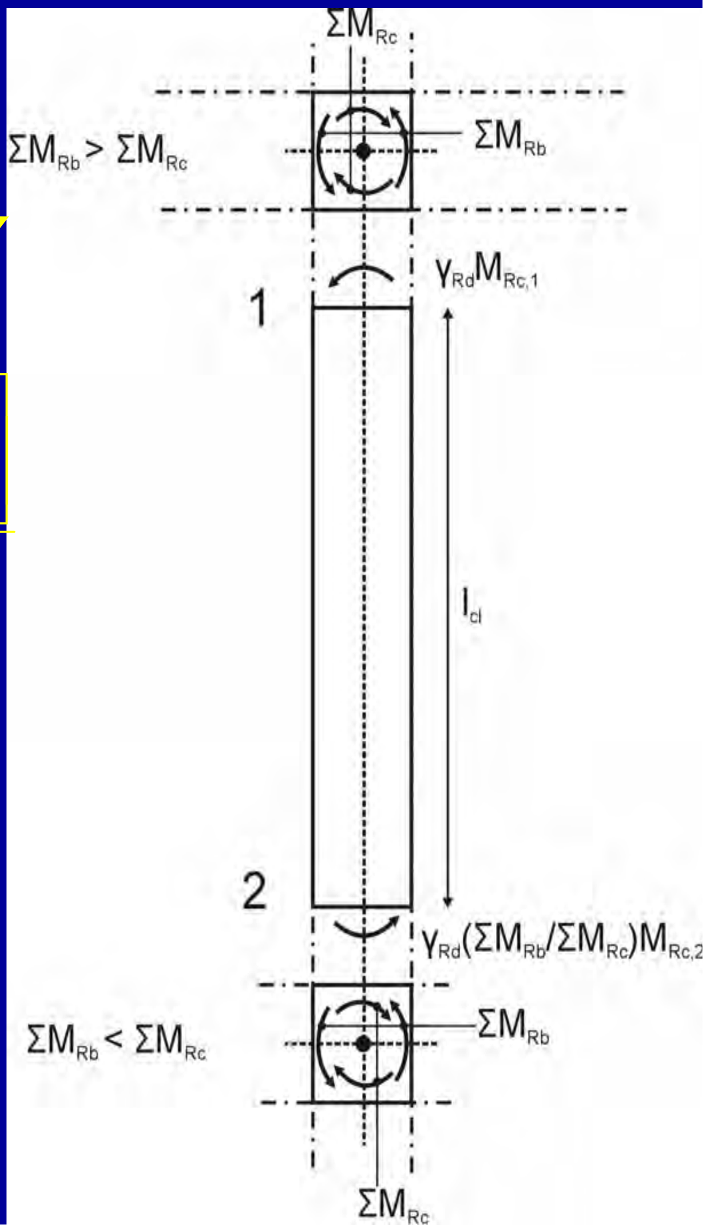
# II. Columns

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

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

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

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




Eurocode 8:

- in DC M $\gamma_{Rd}$=1.1,
- in DC H $\gamma_{Rd}$=1.3

# III. Walls

**Eurocode 8:**

Over-design in shear, by multiplying shear forces from the analysis for the design seismic action, $V'_{Ed}$, by factor $\varepsilon$:

**DC M** walls:

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

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

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

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

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

$\gamma_{Rd}$=1.2

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

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

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

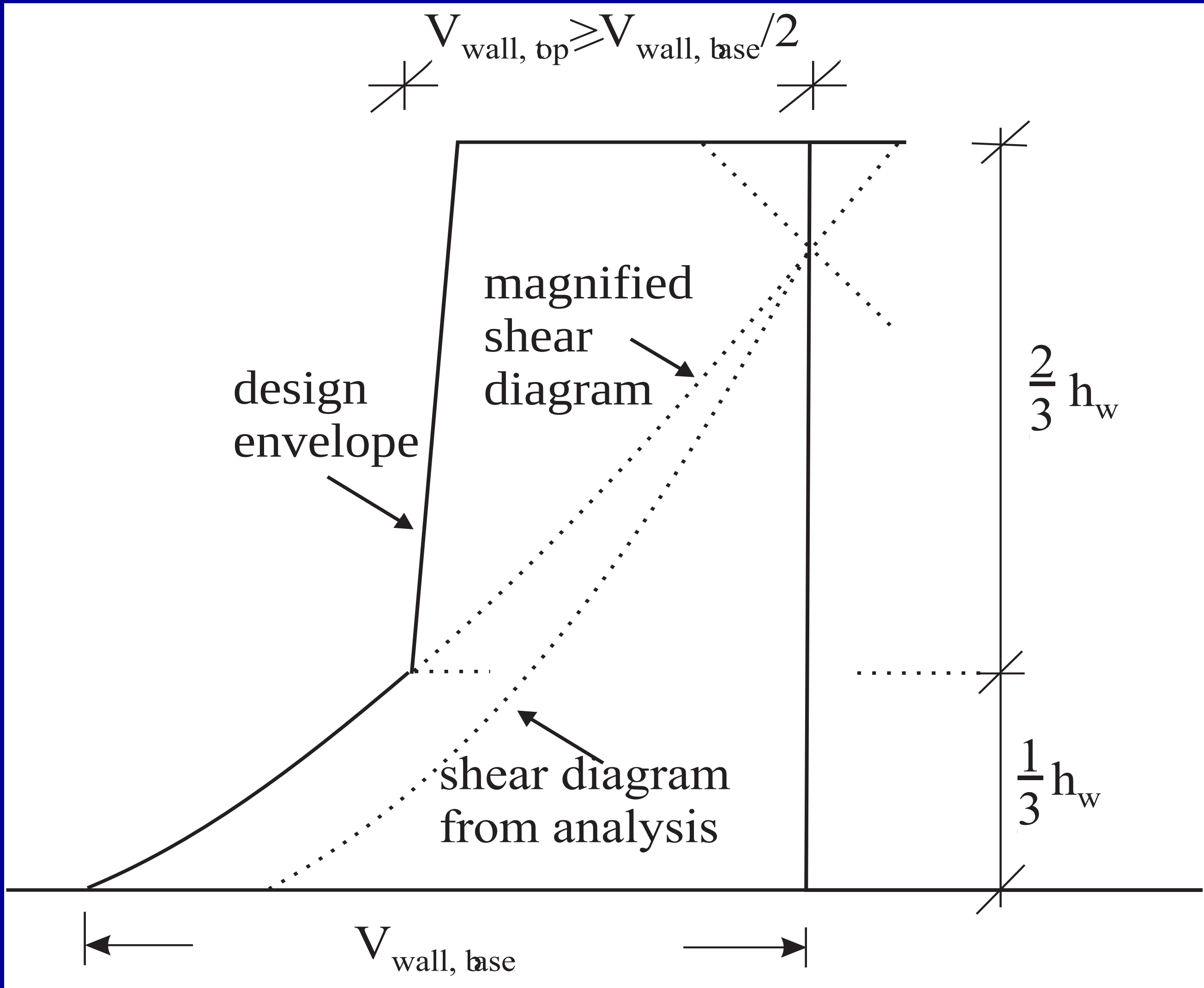
$S_e(T)$: ordinate of elastic response spectrum

$T_C$: upper limit T of const. spectral acc. region

$T_1$: fundamental period.

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

# Design shear forces in “ductile wall” of dual structural systems per Eurocode 8




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

## DETAILING OF DISSIPATIVE ZONES (FLEXURAL PLASTIC HINGES) FOR CURVATURE DUCTILITY FACTOR $\mu_\varphi$ CONSISTENT w/ q-FACTOR

- $\mu_\varphi=2q_o-1$ if $T_1 \geq T_c$
- $\mu_\varphi=1+2(q_o-1)T_c/T_1$ if $T_1<T_c$
  - $T_1$: fundamental period of building,
  - $T_c$: T at upper limit of constant spectral acceleration region,
  - $q_o$ : q-factor unreduced for irregularity in elevation (multiplied w/ $M_{Ed}/M_{Rd}$ at wall base).
- Derivation:
  - Relation between $\mu_\varphi$ & $L_{pl}/L_s$ ($L_{pl}$: plastic hinge length, $L_s$: shear span) & $\mu_\delta$ (: top displacement ductility factor) in buildings staying straight due to walls or strong columns: $\mu_\delta=1+3(\mu_\varphi-1)L_{pl}/L_s(1-0.5L_{pl}/L_s)$;
  - Relation $q$-$\mu_\delta$-$T$ :

    $\mu_\delta= q$ if $T_1 \geq T_c$, $\mu_\delta= 1+(q-1)T_c/T_1$ if $T_1<T_c$;
  - Relation of $L_{pl}$ & $L_s$ for typical RC beams, columns & walls (for EC2 confinement model: $\varepsilon^*_{cu}=0.0035+0.1\alpha\omega_w$):

    $L_{pl}\approx 0,3L_s$ & for (safety) factor 2: $L_{pl}=0,15L_s$ . Then: $\mu_\varphi \approx 2\mu_\delta-1$
- For steel B ($\varepsilon_u$: 5-7.5%, $f_t/f_y$: 1.08-1.15) increase $\mu_\varphi$-demand by 50%

# MEANS TO ACHIEVE $\mu_\varphi$ IN PLASTIC HINGES

- Members w/ axial load & symmetric reinforcement, $\omega=\omega'$ (columns, ductile walls):
  - Confining reinforcement (for walls: in boundary elements) with (effective) mechanical volumetric ratio:

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

    - $v_d=N_d/b_chf_{cd}$; $\varepsilon_{yd}=f_{yd}/E_s$;
    - $b_c$: width of compression zone; $b_o$: width of confined core;
    - $\omega_\nu$: mechanical ratio of longitudinal web reinforcement $=\rho_\nu f_{yd,\nu}/f_{cd}$
  - Columns meeting strong-column/weak-beam rule ($\Sigma M_{Rc}>1.3\Sigma M_{Rb}$), provided w/ full confining reinforcement only at (building) base;
  - DC H strong columns ($\Sigma M_{Rc}>1.3\Sigma M_{Rb}$) also provided w/ confining reinforcement for 2/3 of $\mu_\varphi$ in all end regions above base;
- Members w/o axial load & w/ unsymmetric reinforcement (beams):
  - Max. mechanical ratio of tension steel:

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

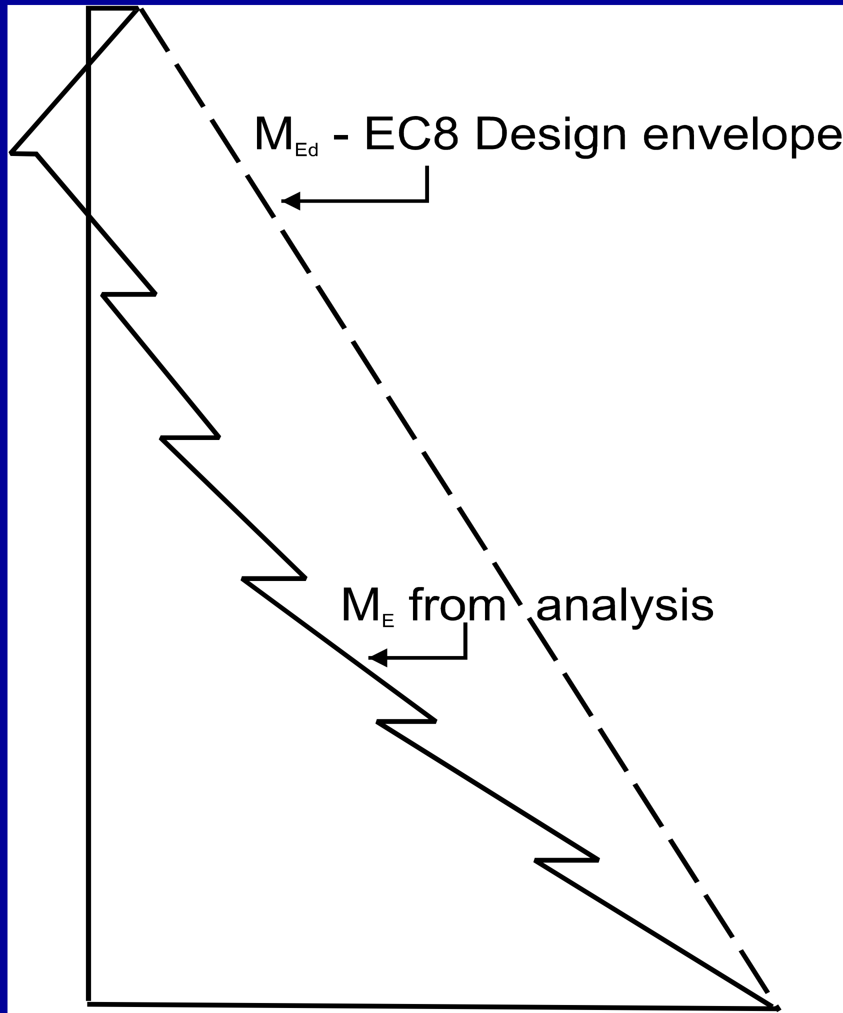
# EC8 - SPECIAL FEATURE:
# TWO TYPES OF DISSIPATIVE CONCRETE WALLS

- Ductile wall:
  - Fixed at base, to prevent rotation there w.r.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 $l_w \geq$ 4m, expected to develop during design EQ 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, or connectivity with transverse walls preventing pl. hinge rotation at base, wall cannot be designed for energy dissipation in pl. hinge at base.

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

But:
design of ductile walls in flexure, to ensure that plastic hinge develops only at the base:




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

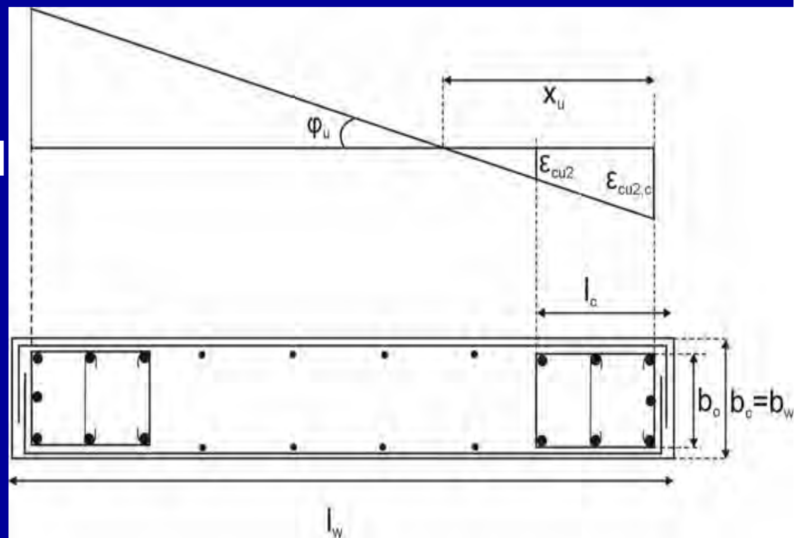
# DESIGN & DETAILING OF DUCTILE WALLS

- Inelastic action limited to plastic hinge at base, so that cantilever relation between q & $\mu_\varphi$ 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:

    *1.5* for DC M

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

    - $M_{Ed}$: design moment at base (from analysis),
    - $M_{Rd}$: design flexural resistance at base,
    - $S_e(T)$: ordinate of elastic response spectrum,
    - $T_c$: upper limit T of const. spectral acc. region
    - $T_1$ fundamental period.




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

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

over part of compression zone depth: $x_u=(v_d+\omega_\nu)l_w\varepsilon_{yd}b_c/b_o$

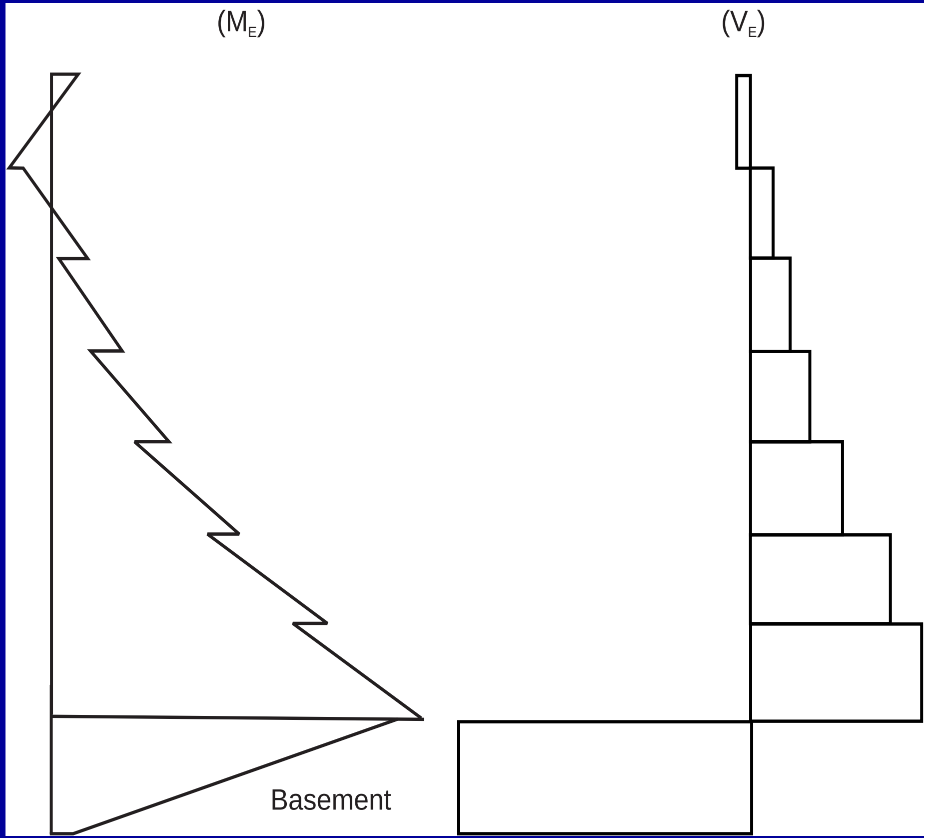
where strain between: $\varepsilon^*_{cu}=0.0035+0.1\alpha\omega_w$ & $\varepsilon_{cu}=0.0035$

# Foundation problem for ductile walls

- To form plastic hinge at wall base → Need fixity there:
  - Very large & heavy footing; adds own weight to N & does not uplift; or
  - Fixity of wall in a "box type" foundation system:

1. Wall-like deep foundation beams along entire perimeter of foundation (possibly supplemented w/ interior ones across full length of foundation system) = main foundation elements transferring seismic action effects to ground.

   In buildings w/ basement: perimeter foundation beams may double as basement walls.
2. Slab designed to act as rigid diaphragm, at the level of top flange of perimeter foundation beams (e.g. basement roof).
3. Foundation slab, or two-way tie-beams or foundation beams, at level of bottom of perimeter foundation beams.

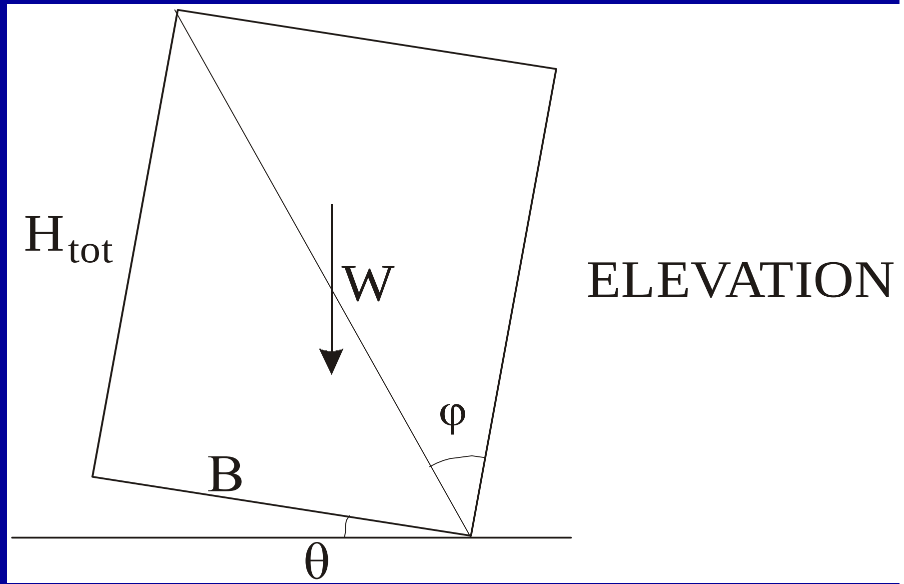



Fixity of interior walls provided by couple of horizontal forces between **2** & **3** → High reverse shear in part of the wall within the basement

# The problem of the foundation of a large wall

- Large $l_w$(=*h*) $\longrightarrow$
  - large moment at base
  - (for given axial load) low normalized axial force $v=N/(bhf_c)$~0.05.
- Footing of usual size w/ tie-beams of usual size: insufficient:
  - Max normalized moment $\mu=M/(bh^2f_{cd})$ that can be transferred to ground:
  - $\mu$ ~0.5$v$, i.e. ~wall cracking moment! $\longrightarrow$

Impossible to form plastic hinge at wall base. Wall will uplift & rock as rigid body.




~Rigid large walls on large footing:

Rocking $\rightarrow$ radiation damping in the soil.

Rotation of rocking wall:

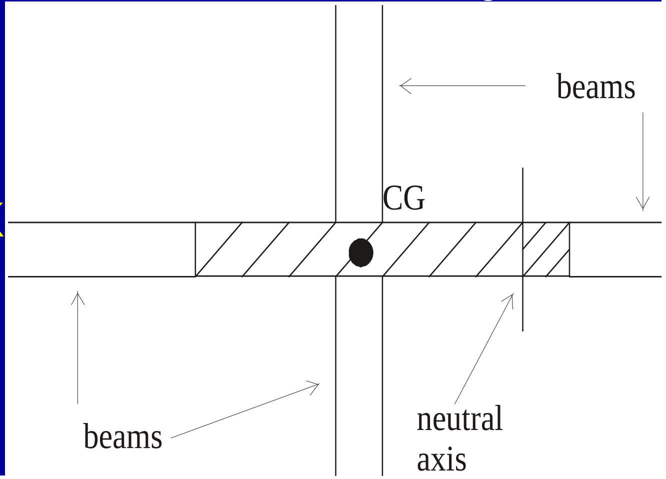
$\theta$~$S_v^2/Bg$ << $\varphi$=arctan($B/H_{tot}$) $\longrightarrow$

Very stable nonlinear-elastic behaviour; but hard to address in design

# Geometric effects in large walls, due to rocking or plastic hinging

- Rotation of uplifting/rocking wall takes place about a point close to the toe of its footing.
- Rotation at wall plastic hinge at base takes place about a neutral axis close to edge of wall section.
- In both cases centroid of wall section is raised at every rotation:
  - Centre of Gravity (CG) of masses supported by wall raised too → (temporary) harmless increase in potential energy, instead of damaging deformation energy;
  - Ends of beams framing into wall move upwards → beam moments & shears: stabilizing for the wall.

Plan view: beams framing into wall




- Wall responds as a "stack" of rigid blocks, uplifting at the base & at hor. sections that crack & yield (storey bottom). The favourable effects are indirectly taken into account in design → q-factor

# EXAMPLES OF LARGE WALLS

# LARGE LIGHTLY REINFORCED CONCRETE WALLS

- Wall system classified as one of large lightly reinforced walls if, in horizontal direction of interest:
  - at least 2 walls with $l_w$>4 m, supporting together >20% of gravity load above (: sufficient no. of walls / floor area & significant uplift of masses); if just one wall, q=2
  - fundamental period $T_1$<0.5 s 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.

# DESIGN & DETAILING OF LARGE LIGHTLY REINFORCED WALLS

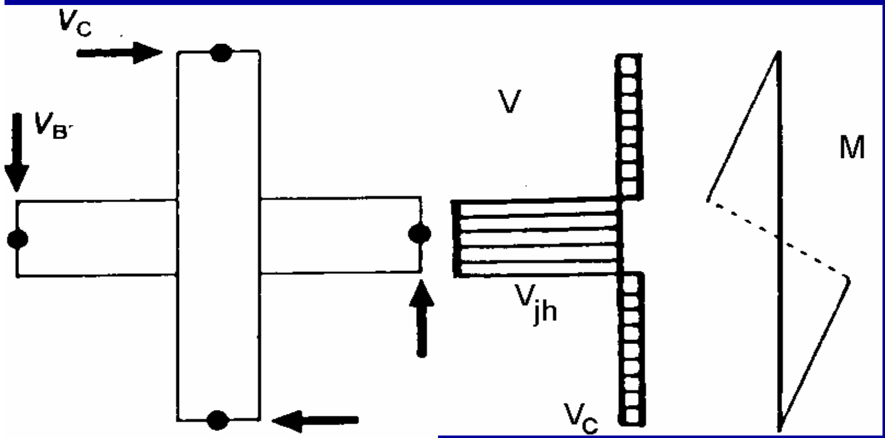
- *Vertical steel* tailored to demands due to M & N from analysis
  - Little excess (minimum) reinforcement, to minimise flexural overstrength.

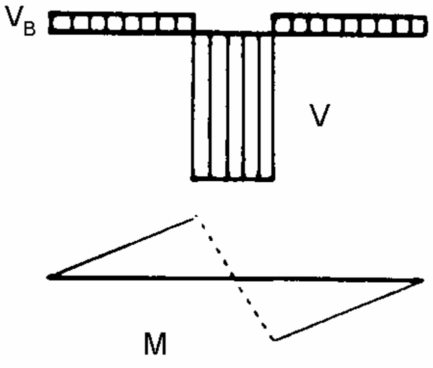
- Shear verification for V from analysis times (1+q)/2 ~2:
  - If so-amplified shear demand is less than (design) shear resistance w/o shear reinforcement:

    No (minimum) *horizontal reinforcement.* Reason:
    - Inclined cracking prevented (horizontal cracking & yielding due to flexure mainly at construction joints);
    - If inclined cracking occurs, crack width limited by deformation-controlled nature of response (vs. force-controlled non-seismic actions covered in EC2), even w/o min horizontal steel.

# BEAM-COLUMN JOINTS IN DC H FRAMES

# Shear forces in joints




shear forces within joint

max possible joint shear force & stress

**If $\sum M_{Rb} < \sum M_{Rc}$:**

$$V_{jh} = \left(A_{sb1} + A_{sb2}\right) f_y - V_C = \sum M_{Rb} \left( \frac{1}{z_b} - \frac{1}{h_{st}} \frac{L_b}{L_{bn}} \right)$$

$$\approx \left(A_{sb1} + A_{sb2}\right) f_y \left( 1 - \frac{z_b}{h_{st}} \frac{L_b}{L_{bn}} \right)$$

$$v_j = \frac{V_{jh}}{b_j h_{jc}}$$

If $b_c > b_w \rightarrow b_j = \min\{b_c; (b_w + 0.5h_c)\}$ If $b_c \leq b_w \rightarrow b_j = \min\{b_w; (b_c + 0.5h_c)\}$

# Shear failures of exterior beam-column joints - Left & right: reinforced joints; centre: unreinforced joint

# Principal stress approach for joint shear strength

<u>Diagonal cracking</u> of **unreinforced** joint if principal tensile stress due to:

- joint shear stress, $v_j$ &
- mean vertical compressive stress from column above, $\nu_{top} f_c$,

exceeds concrete tensile strength, $f_{ct}$.

$$v_j \geq v_{cr} = f_{ct}\sqrt{1 + \frac{\nu_{top} f_c}{f_{ct}}}$$

Eurocode 8: <u>Diagonal cracking</u> of **reinforced** joint if principal tensile stress due to:

- joint shear stress, $v_j$ &
- mean vertical compressive stress from column above, $\nu_{top} f_c$, <u>and</u>
- horizontal confining stress due to horiz. joint reinforcement, $-\rho_{jh} f_{yw}$:

exceeds concrete tensile strength, $f_{ct}$.

$$\rho_{jh} f_{yw} \geq \frac{v_j^2}{f_{ct} + \nu_{top} f_c} - f_{ct}$$

Joint <u>ultimate</u> shear stress $v_{ju}$ : if $nf_c$ ($n$: reduction due to transverse tensile strain) reached in principal stress direction:

$$v_j < v_{ju} = nf_c\sqrt{1 - \frac{\nu_{top}}{n}}$$

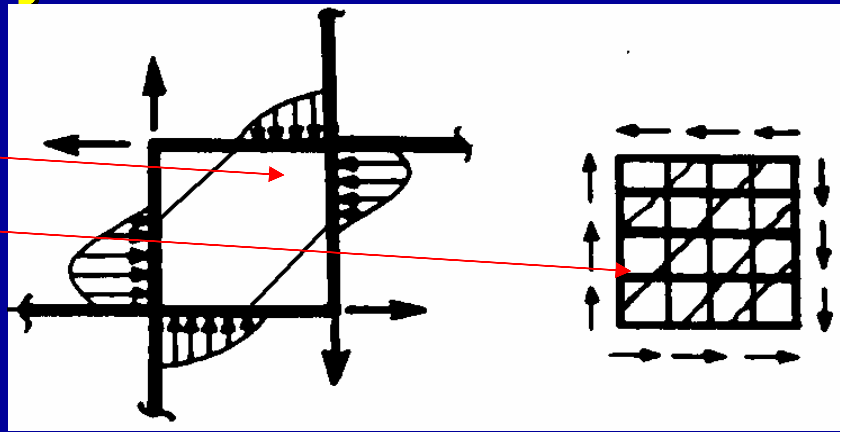
# Alternative approach in EC 8 for joint reinforcement

Diagonal strut

Truss of:

horizontal & vertical bars &

diagonal compressive field.



**Interior joints:**

$$A_{sh} f_{yw} \geq \left(A_{sb1} + A_{sb2}\right) f_y \left(1 - \frac{6}{5}\nu\right)$$

**Exterior joints:**

$$A_{sh} f_{yw} \geq A_{sb2} f_y \left(1 - \frac{6}{5}\nu\right)$$

# Detailing & dimensioning of primary seismic beams (secondary as in DCL)

| | DCH | DCM | DCL |
|---|---|---|---|
| "critical region" length | $1.5h_w$ | $h_w$ | |
| *Longitudinal bars (L):* | | | |
| $\rho_{min}$, tension side | $0.5f_{ctm}/f_{yk}$ | | $0.26f_{ctm}/f_{yk}$, 0.13%[0] |
| $\rho_{max}$, critical regions[1] | $\rho'+0.0018f_{cd}/(\mu_{\phi}\varepsilon_{sy,d}f_{yd})$[1] | | 0.04 |
| $A_{s,min}$, top & bottom | 2Φ14 ($308mm^2$) | - | |
| $A_{s,min}$, top-span | $A_{s,top\text{-}supports}/4$ | - | |
| $A_{s,min}$, critical regions bottom | $0.5A_{s,top}$[2] | | - |
| $A_{s,min}$, supports bottom | $A_{s,bottom\text{-}span}/4$[0] | | |
| $d_{bL}/h_c$ - bar crossing interior joint[3] | $\le \dfrac{6.25(1+0.8\nu_d)}{(1+0.75\dfrac{\rho'}{\rho_{max}})}\dfrac{f_{ctm}}{f_{yd}}$ | $\le \dfrac{7.5(1+0.8\nu_d)}{(1+0.5\dfrac{\rho'}{\rho_{max}})}\dfrac{f_{ctm}}{f_{yd}}$ | - |
| $d_{bL}/h_c$ - bar anchored at exterior joint[3] | $\le 6.25(1+0.8\nu_d)\dfrac{f_{ctm}}{f_{yd}}$ | $\le 7.5(1+0.8\nu_d)\dfrac{f_{ctm}}{f_{yd}}$ | - |
| *Transverse bars (w):* | | | |
| (i) outside critical regions | | | |
| spacing $s_w\le$ | 0.75d | | |
| $\rho_w\ge$ | $0.08(f_{ck}(MPa))^{1/2}/f_{yk}(MPa)$[0] | | |
| (ii) in critical regions: | | | |
| $d_{bw}\ge$ | 6mm | | |
| spacing $s_w\le$ | $6d_{bL}$, $\dfrac{h_w}{4}$, $24d_{bw}$, 175mm | $8d_{bL}$, $\dfrac{h_w}{4}$, $24d_{bw}$, 225mm | - |
| *Shear design:* | | | |
| $V_{Ed}$, seismic[4] | $1.2\dfrac{\sum M_{Rb}}{l_{cl}}\pm V_{o,g+\psi_2 q}$[4] | $\dfrac{\sum M_{Rb}}{l_{cl}}\pm V_{o,g+\psi_2 q}$[4] | From the analysis for the "seismic design situation" |
| $V_{Rd,max}$ seismic[5] | As in EC2: $V_{Rd,max}=0.3(1-f_{ck}(MPa)/250)b_{wo}zf_{cd}\sin2\theta$[5], with $1\le\cot\theta\le2.5$ | | |
| $V_{Rd,s}$, outside critical regions[5] | As in EC2: $V_{Rd,s}=b_wz\rho_wf_{ywd}\cot\theta$[5], with $1\le\cot\theta\le2.5$ | | |
| $V_{Rd,s}$, critical regions[5] | $V_{Rd,s}=b_wz\rho_wf_{ywd}$ ($\theta=45^o$) | As in EC2: $V_{Rd,s}=b_wz\rho_wf_{ywd}\cot\theta$, with $1\le\cot\theta\le2.5$ | |
| If $\zeta\equiv V_{Emin}/V_{Emax}$[6] <-0.5: inclined bars at angle $\pm\alpha$ to beam axis, with cross-section $A_s$/direction | If $V_{Emax}/(2+\zeta)f_{ctd}b_wd>1$: $A_s=0.5V_{Emax}/f_{yd}\sin\alpha$ & stirrups for $0.5V_{Emax}$ | - | |

# Footnotes to Table on detailing & dimensioning primary seismic beams (previous page)

(0) NDP (Nationally Determined Parameter) according to EC2. The Table gives the value recommended in EC2.

(1) $\mu_\phi$ is the value of the curvature ductility factor that corresponds to the basic value, $q_o$, of the behaviour factor used in the design

(2) The minimum area of bottom steel, $A_{s,min}$, is in addition to any compression steel that may be needed for the verification of the end section for the ULS in bending under the (absolutely) maximum negative (hogging) moment from the analysis for the "seismic design situation", $M_{Ed}$.

(3) $h_c$ is the column depth in the direction of the bar, $\nu_d = N_{Ed}/A_c f_{cd}$ is the column axial load ratio, for the algebraically 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: $\sum M_{Rb} > \sum M_{Rc}$, $M_{Rb}$ is replaced in the calculation of the design shear force, $V_{Ed}$, by $M_{Rb}(\sum M_{Rc}/\sum M_{Rb})$

(5) z is the internal lever arm, taken equal to 0.9d or to the distance between the tension and the compression reinforcement, $d-d_1$.

(6) $V_{Emax}$, $V_{E,min}$ are the algebraically maximum and minimum values of $V_{Ed}$ resulting from the ± sign; $V_{Emax}$ is the absolutely largest of the two values, and is taken positive in the calculation of $\zeta$; the sign of $V_{Emin}$ is determined according to whether it is the same as that of $V_{Emax}$ or not.

# Detailing & dimensioning of primary seismic columns (secondary as in DCL)

| | DCH | DCM | DCL |
|---|---|---|---|
| Cross-section sides, $h_c$, $b_c \geq$ | 0.25m; $h_v/10$ if $\theta=P\delta/Vh>0.1$ (1) | - | |
| "critical region" length (1) $\geq$ | $1.5\max(h_c,b_c)$, 0.6m, $l_c/5$ | $\max(h_c,b_c)$, 0.6m, $l_c/5$ | - |
| *Longitudinal bars (L):* | | | |
| $\rho_{min}$ | 1% | | $0.1N_d/A_cf_{yd}$, 0.2% (0) |
| $\rho_{max}$ | 4% | | 4% (0) |
| $d_{bL}\geq$ | 8mm | | |
| bars per side $\geq$ | 3 | | 2 |
| Spacing between restrained bars | $\leq$150mm | $\leq$200mm | - |
| distance of unrestrained to nearest restrained bar | $\leq$150mm | | |
| *Transverse bars (w):* | | | |
| Outside critical regions: | | | |
| $d_{bw}\geq$ | 6mm, $d_{bL}/4$ | | |
| Spacing $s_w \leq$ | $20d_{bL}$, $\min(h_c, b_c)$, 400mmm | | |
| $s_w$ in splices $\leq$ | $12d_{bL}$, $0.6\min(h_c, b_c)$, 240mm | | |
| Within critical regions: (2) | | | |
| $d_{bw}\geq$ (3) | 6mm, $0.4(f_{yd}/f_{ywd})^{1/2}d_{bL}$ | 6mm, $d_{bL}/4$ | |
| $s_w\leq$ (3),(4) | $6d_{bL}$, $b_o/3$, 125mm | $8d_{bL}$, $b_o/2$, 175mm | - |
| $\omega_{wd}\geq$ (5) | 0.08 | - | |
| $\alpha\omega_{wd}\geq$ (4),(5),(6),(7) | $30\mu_\phi^{*}\nu_d\varepsilon_{sy,d}b_c/b_o-0.035$ | - | |
| In critical region at column base: | | | |
| $\omega_{wd}\geq$ | 0.12 | 0.08 | - |
| $\alpha\omega_{wd}\geq$ (4),(5),(6),(8),(9) | $30\mu_\phi\nu_d\varepsilon_{sy,d}b_c/b_o-0.035$ | | - |
| *Capacity design check at beam-column joints:* (10) | $1.3\sum M_{Rb}\leq\sum M_{Rc}$<br>No moment in transverse direction of column | | - |
| *Verification for* $M_x$-$M_y$-N: | Truly biaxial, or uniaxial with ($M_z/0.7$, N), ($M_y/0.7$, N) | | |
| *Axial load ratio* $\nu_d=N_{Ed}/A_cf_{cd}$ | $\leq$ 0.55 | $\leq$ 0.65 | - |
| *Shear design:* | | | |
| $V_{Ed}$ seismic (11) | $1.3\dfrac{\sum M_{Rc}^{ends}}{l_{cl}}$ (11) | $1.1\dfrac{\sum M_{Rc}^{ends}}{l_{cl}}$ (11) | From the analysis for the "seismic design situation" |
| $V_{Rd,max}$ seismic (12), (13) | As in EC2:<br>$V_{Rd,max}=0.3(1-f_{ck}(MPa)/250)\min[1.25; (1+\nu_d); 2.5(1-\nu_d)]b_{wo}zf_{cd}\sin2\theta$,<br>with $1\leq\cot\theta\leq2.5$ | | |
| $V_{Rd,s}$ seismic (12), (13), (14) | As in EC2: $V_{Rd,s}=b_wz\rho_wf_{ywd}\cot\theta+N_{Ed}(h-x)/l_{cl}$ (13) with $1\leq\cot\theta\leq2.5$ | | |

# Footnotes to Table of detailing & dimensioning primary seismic columns (previous page)

(0) NDP (Nationally Determined Parameter) according to EC2. The Table gives the value recommended in EC2.

(1) $h_v$ is the distance of the inflection point to the column end further away, for bending within a plane parallel to the side of interest; $l_c$ is the column clear length.

(2) For DCM: If a value of q not greater than 2 is used for the design, the transverse reinforcement in critical regions of columns with axial load ratio $\nu_d$ not greater than 0.2 may just follow the rules applying to DCL columns.

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

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

(5) $\omega_{wd}$ is the ratio of the volume of confining hoops to that of the confined core to the centreline of the hoops, times $f_{yd}/f_{cd}$.

(6) $\alpha$ is the "confinement effectiveness" factor, computed as $\alpha = \alpha_s\alpha_n$; where: $\alpha_s = (1\text{-}s/2b_o)(1\text{-}s/2h_o)$ for hoops and $\alpha_s = (1\text{-}s/2b_o)$ for spirals; $\alpha_n = 1$ for circular hoops and $\alpha_n=1-\{b_o/[(n_h-1)h_o]+h_o/[(n_b-1)b_o]\}/3$ for rectangular hoops with $n_b$ legs parallel to the side of the core with length $b_o$ and $n_h$ legs parallel to the one with length $h_o$.

(7) For DCH: at column ends protected from plastic hinging through the capacity design check at beam-column joints, $\mu_\phi^*$ is the value of the curvature ductility factor that corresponds to 2/3 of the basic value, $q_o$, of the behaviour factor used in the design; at the ends of columns where plastic hinging is not prevented because of the exemptions listed in Note (10) below, $\mu_\phi^*$ is taken equal to $\mu_\phi$ defined in Note (1) of the Table for the beams (see also Note (9) below); $\varepsilon_{sy,d}= f_{yd}/E_s$.

(8) Note (1) of the Table for the beams applies.

(9) For DCH: The requirement applies also in the critical regions at the ends of columns where plastic hinging is not prevented, because of the exceptions listed in Note (10) below.

(10) The capacity design check does not need to be fulfilled at beam-column joints: (a) of the top floor, (b) of the ground storey in two-storey buildings with axial load ratio $\nu_d$ not greater than 0.3 in all columns, (c) if shear walls resist at least 50% of the base shear parallel to the plane of the frame (wall buildings or wall-equivalent dual buildings), and (d) in one-out-of-four columns of plane frames with columns of similar size.

(11) At a member end where the moment capacities around the joint satisfy: $\sum M_{Rb}<\sum M_{Rc}$, $M_{Rc}$ is replaced by $M_{Rc}(\sum M_{Rb}/\sum M_{Rc})$.

(12) z is the internal lever arm, taken equal to 0.9d or to the distance between the tension and the compression reinforcement, $d\text{-}d_1$.

(13) The axial load, $N_{Ed}$, and its normalized value, $\nu_d$, are taken with their most unfavourable value in the seismic design situation for the shear verification (considering both the demand, $V_{Ed}$, and the capacity, $V_{Rd}$).

(14) x is the compression zone depth at the end section in the ULS of bending with axial load.

# Detailing & dimensioning of ductile walls (cont'd next page)

| | DCH | DCM | DCL |
|---|---|---|---|
| Web thickness, $b_{wo}\geq$ | max(150mm, $h_{storey}/20$) | | - |
| critical region length, $h_{cr}\geq$ | $\geq$ max($l_w$, $H_w/6$) (1)<br>$\leq$ min($2l_w$, $h_{storey}$) if wall ≤6 storeys<br>$\leq$ min($2l_w$, 2 $h_{storey}$) if wall > 6 storeys | | - |
| *Boundary elements:* | | | |
| a) in critical region: | | | |
| - length $l_c$ from edge ≥ | $0.15l_w$, $1.5b_w$, length over which $\varepsilon_c> 0.0035$ | | where $\rho_L>2\%$ |
| - thickness $b_w$ over $l_c$ ≥ | 200mm, $h_{st}/15$, if $l_c\leq max(2b_w, l_w/5)$,<br>200mm, $h_{st}/10$, if $l_c>max(2b_w, l_w/5)$ | | - |
| - vertical reinforcement: | | | |
| $\rho_{min}$ over $A_c=l_cb_w$ | 0.5% | | 0.2%(0) |
| $\rho_{max}$ over $A_c$ | 4% (0) | | |
| - confining hoops (w) (2): | | | |
| $d_{bw}\geq$ | 8mm | if $\rho_L$ over $A_c=l_cb_w$ >2%:apply DCL rule for $\rho_L>2\%$ | 6mm, $d_{bL}/4$ |
| spacing $s_w\leq$(3) | min($25d_{bh}$, 250mm) | | min($20d_{bL}$, $b_{wo}$ 400mm)(0) |
| $\omega_{wd}\geq$(2) | 0.12 | 0.08 | - |
| $\alpha\omega_{wd}\geq$(3),(4) | $30\mu_\phi(\nu_d+\omega_v)\varepsilon_{sy,d}b_w/b_o-0.035$ | | - |
| b) storey above critical region | as is critical region, but with required $\alpha\omega_{wd}$, $\omega_{wd}$ reduced by 50% | $\rho_v\geq0.5\%$ wherever $\varepsilon_c>0.2\%$; elsewhere $\rho_v\geq0.2\%$ | |
| c) over the rest of the wall: | No boundary elements. $\rho_v\geq0.5\%$ wherever $\varepsilon_c>0.2\%$; elsewhere $\rho_v\geq0.2\%$ | | - |
| *Web:* | | | |
| - vertical bars (v): | | | |
| $\rho_{v,min}$ | 0.2% | 0.2%(0) | |
| $\rho_{v,max}$ | 4% | | |
| $d_{bv}\geq$ | 8mm | - | |
| $d_{bv}\leq$ | $b_{wo}/8$ | - | |
| spacing $s_v\leq$ | min($25d_{bv}$, 250mm) | Min($3b_{wo}$, 400mm) | |
| - horizontal bars: | | | |
| $\rho_{hmin}$ | 0.2% | max(0.1%, $0.25\rho_v$)(0) | |
| $d_{bh}\geq$ | 8mm | - | |
| $d_{bh}\leq$ | $b_{wo}/8$ | - | |
| spacing $s_h\leq$ | min($25d_{bh}$, 250mm) | 400mm | |
| *axial load ratio* $\nu_d= N_{Ed}/A_cf_{cd}$ | ≤0.35 | ≤0.4 | - |
| *Design moments $M_{Ed}$:* | If $H_w/l_w\geq2$, design moments from linear envelope of maximum moments $M_{Ed}$ from analysis for the "seismic design situation", shifted up by the "tension shift" $a_l$ | | From analysis for "seismic design situation" |

# Detailing & dimensioning of ductile walls (cont'd from previous page)

| | DCH | DCM | DCL |
|---|---|---|---|
| *Shear design:* | | | |
| Multiplicative factor ε on the shear force $V'_{Ed}$ from the analysis for "seismic design situation": | if $H_w/l_w \le 2$(5): $\varepsilon = 1.2M_{Rdo}/M_{Edo} \le q$<br>if $H_w/l_w > 2$(5), (6): $\varepsilon = \sqrt{\left(1.2\frac{M_{Rdo}}{M_{Edo}}\right)^2 + 0.1\left(q\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \le q$ | ε=1.5 | ε=1.0 |
| Design shear force in walls of dual systems with $H_w/l_w>2$, for z between $H_w/3$ and $H_w$: (7) | $V_{Ed}(z) = \left(\frac{0.75z}{H_w} - \frac{1}{4}\right)\varepsilon V_{Ed}(0) + \left(1.5 - \frac{1.5z}{H_w}\right)\varepsilon V_{Ed}\left(\frac{H_w}{3}\right)$ | | From analysis for "seismic design situation" |
| $V_{Rd,max}$ outside critical region | As in EC2: $V_{Rd,max}=0.3(1-f_{ck}(MPa)/250)b_{wo}(0.8l_w)f_{cd}\sin 2\theta$, with $1 \le \cot\theta \le 2.5$ | | |
| $V_{Rd,max}$ in critical region | 40% of EC2 value | As in EC2 | |
| $V_{Rd,s}$ outside critical region | As in EC2: $V_{Rd,s}=b_{wo}(0.8l_w)\rho_h f_{ywd}\cot\theta$ with $1 \le \cot\theta \le 2.5$ | | |
| $V_{Rd,s}$ in critical region; web reinforcement ratios. $\rho_h$, $\rho_v$ | | | |
| (i) if $\alpha_s=M_{Ed}/V_{Ed}l_w \ge 2$ : $\rho_v=\rho_{v,min}$, $\rho_h$ from $V_{Rd,s}$: | As in EC2: $V_{Rd,s}=b_{wo}(0.8l_w)\rho_h f_{ywd}\cot\theta$ with $1 \le \cot\theta \le 2.5$ | | |
| (ii) if $\alpha_s<2$: $\rho_h$ from $V_{Rd,s}$: (8) | $V_{Rd,s}=V_{Rd,c}+b_{wo}\alpha_s(0.75l_w)\rho_h f_{yhd}$ | As in EC2: $V_{Rd,s}=b_{wo}(0.8l_w)\rho_h f_{ywd}\cot\theta$ with $1 \le \cot\theta \le 2.5$ | |
| $\rho_v$ from: (9) | $\rho_v f_{yvd} \ge \rho_h f_{yhd} - N_{Ed}/(0.8l_w b_{wo})$ | | |
| Resistance to sliding shear: via bars with total area $A_{si}$ at angle $\pm\phi$ to the horizontal (10) | $V_{Rd,s} = A_{si}f_{yd}\cos\phi + A_{sv}\min(0.25f_{yd}, 1.3(f_{yd}f_{cd})^{1/2}) + 0.3(1-f_{ck}(MPa)/250)b_{wo}xf_{cd}$ | | |
| $\rho_{v,min}$ at construction joints (9),(11) | $0.0025, \frac{1.3f_{ctd} - \frac{N_{Ed}}{A_c}}{f_{yd} + 1.5\sqrt{f_{cd}f_{yd}}}$ | - | |

# Footnotes to Table on detailing & dimensioning ductile walls (previous pages)

(0) NDP (Nationally Determined Parameter) according to EC2. The Table gives the value recommended in EC2.

(1) $l_w$ is the long side of the rectangular wall section or rectangular part thereof; $H_w$ is the total height of the wall; $h_{storey}$ is the storey height.

(2) For DC M: If for the maximum value of axial force in the wall from the analysis for the "seismic design situation" the wall axial load ratio $\nu_d = N_{Ed}/A_c f_{cd}$ satisfies $\nu_d \leq 0.15$, the DCL rules may be applied for the confining reinforcement of boundary elements; the waiver applies also if this value of the wall axial load ratio is $\nu_d \leq 0.2$ but the value of q used in the design of the building is not greater than 85% of the q-value allowed when the DC M confining reinforcement is used in boundary elements.

(3) Notes (4), (5), (6) of the Table for columns apply for the confined core of boundary elements.

(4) $\mu_\phi$ is the value of the curvature ductility factor that corresponds to the product of the basic value $q_o$ of the behaviour factor times the value of the ratio $M_{Edo}/M_{Rdo}$ at the base of the wall (see Note (5)); $\varepsilon_{sy,d} = f_{yd}/E_s$, $\omega_{vd}$ is the mechanical ratio of the vertical web reinforcement.

(5) $M_{Edo}$ is the moment at the wall base from the analysis for the "seismic design situation"; $M_{Rdo}$ is the design value of the flexural capacity at the wall base for the axial force $N_{Ed}$ from the analysis for the same "seismic design situation".

(6) $S_e(T_1)$ is the value of the elastic spectral acceleration at the period of the fundamental mode in the horizontal direction (closest to that) of the wall shear force multiplied by ε; $S_e(T_c)$ is the spectral acceleration at the corner period $T_C$ of the elastic spectrum.

(7) A dual structural system is one in which walls resist between 35 and 65% of the seismic base shear in the direction of the wall shear force considered; z is distance from the base of wall.

(8) For $b_w$ and d in m, $f_{ck}$ in MPa, $\rho_L$ denoting the tensile reinforcement ratio, $N_{Ed}$ in kN, $V_{Rd,c}$ (in kN) is given by:

$$V_{Rd,c} = \left\{ \min\left[ \frac{180}{\gamma_c}(100\rho_L)^{1/3},\ 35\sqrt{1+\sqrt{\frac{0.2}{d}}}\ f_{ck}^{1/6} \right]\left(1+\sqrt{\frac{0.2}{d}}\right) f_{ck}^{1/3} + 0.15\frac{N_{Ed}}{A_c} \right\} b_w d$$

$N_{Ed}$ is positive for compression and its minimum value from the analysis for the "seismic design situation" is used; if the minimum value is negative (tension), $V_{Rd,c}=0$.

(9) The minimum value of the axial force from the analysis for the "seismic design situation" is used as $N_{Ed}$ (positive for compression).

(10) $A_{sv}$ is the total area of web vertical bars and of any additional vertical bars placed in boundary elements against shear sliding; x is the depth of the compression zone.

(11) $f_{ctd} = f_{ctk,0.05}/\gamma_c$ is the design value of the (5%-fractile of) tensile strength of concrete.

# Overall effect of masonry infills

- Field experience & numerical/experimental research show that:
  - masonry infills attached to the structural frame, in general have a beneficial effect on seismic performance, especially if the building structure has little engineered earthquake resistance.
- If effectively confined by the surrounding frame, regularly distributed infill panels:
  - reduce, through their in-plane shear stiffness, storey drift demands & deformations in structural members
  - increase, via their in-plane shear strength, storey lateral force resistance,
  - contribute, through their hysteresis, to the global energy dissipation.
- In buildings designed for earthquake resistance, non-structural masonry infills may be a 2$^{nd}$ line of defence & a source of significant overstrength.

# Current position of EC8 on masonry infills

- Eurocode 8 does not encourage designers to profit from the beneficial effects of masonry infills by reducing the seismic action effects for which the structure is designed.
- Eurocode 8 warns against the adverse effects of infills & requires prevention measures for them.
- If there is structural connection between the masonry infill & the surrounding frame (by shear connectors, or other ties, belts or posts), the building is considered/designed as a confined masonry building, instead of a concrete structure with masonry infills.

# Possible adverse effects of masonry infills

- Infills that are too strong & stiff relative to the concrete structure itself

  → may override its seismic design, including the efforts of the designer & intent of codes to control inelastic response by spreading inelastic deformation demands throughout structure

  (e.g. when ground storey infills fail → soft storey).
- Infills non-uniformly distributed in plan or in elevation:

  → concentration of inelastic deformation demands in part of the structure.
- Adverse local effects on structural frame

  → pre-emptive brittle failures.

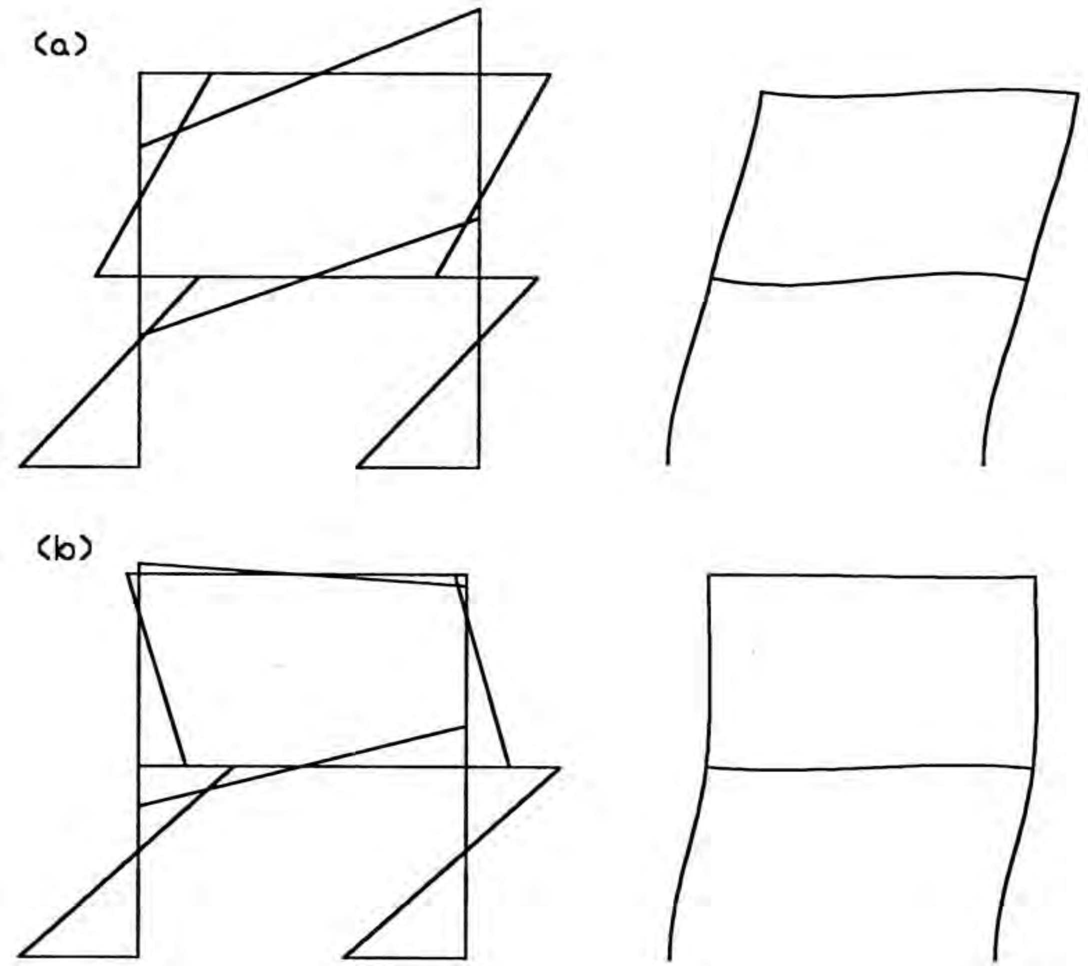
## Possible adverse effects of masonry infills (cont'd)

- Best way to protect concrete building from adverse effects of irregular masonry infilling:

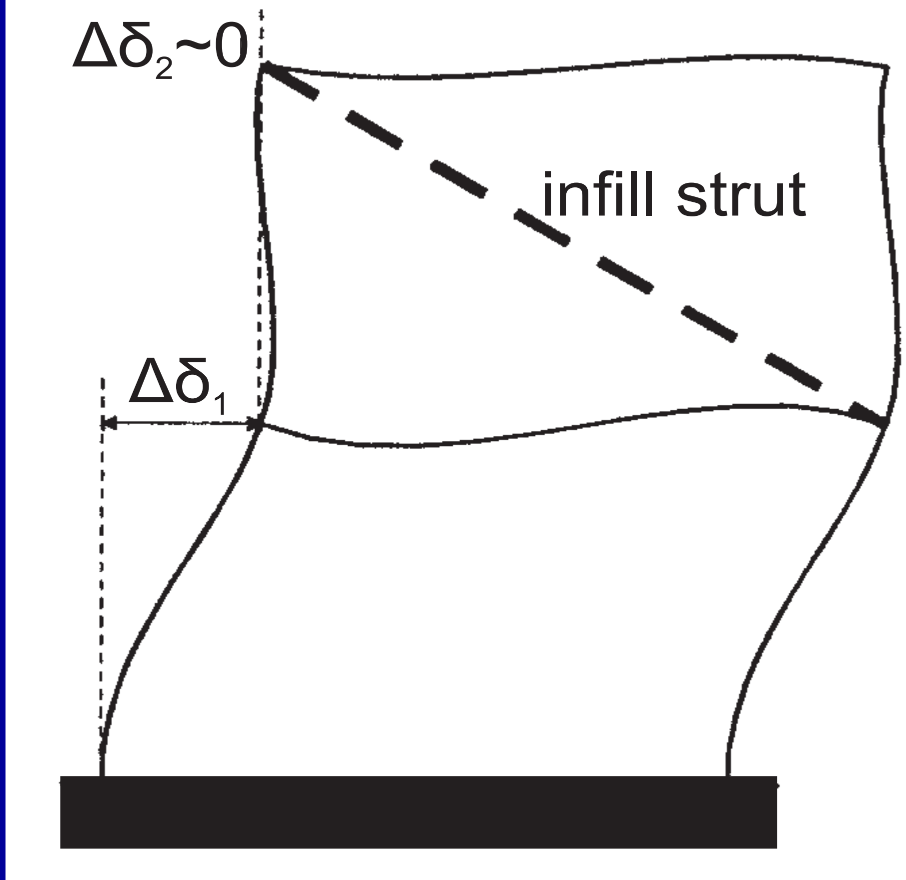
  shear walls sufficiently strong/stiff to overshadow infilling.
- Eurocode 8:
  - Shear walls that resist at least 50% of the seismic base shear: sufficient for waiving special requirements for buildings with infills.

# Possible adverse effects of masonry infills (cont'd)

## Worst possible effect: Open ground storey → soft-storey




(c)




2-storey frame: Protection of elements in infilled storey from large moments & deformations - overloading of ground storey columns:

(a) bending moments & deformation in frame w/o infills;

(b), (c) bending moments & deformation in frame w/ stiff infills in 2nd storey.

# Open ground storey


(a)

(b)

Collapse of ground storey due to reduction of infills:
(a) Olive View Hospital, San Fernando, Ca, 1971; (b) Aegio (GR) 1995

# EC8 design for infill heightwise irregularity

- Eurocode 8: design columns of storey where infills are reduced relative to overlying storey, to remain elastic till infills in storey above reach their ultimate force resistance:
  - Deficit in infill shear strength in a storey is compensated by increase in resistance of the frame (vertical) members there:
  - In DC H frame or frame-equivalent dual buildings, seismic internal forces in the columns from the analysis for the design seismic action are multiplied by:

$$\eta = \left(1 + \Delta V_{Rw} / \Sigma V_{Ed}\right) \leq q$$

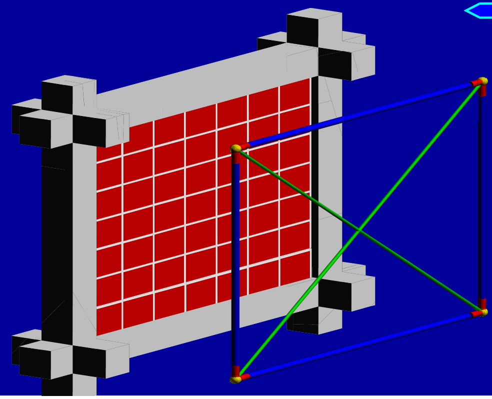
    - $\Delta V_{Rw}$ : total reduction of resistance of masonry walls in storey concerned w.r.to storey above,
    - $\Sigma V_{Ed}$ : sum of seismic shear forces in all vertical primary seismic members of storey (storey design shear force).
  - If $\eta < 1.1$, magnification of seismic action effects may be omitted.

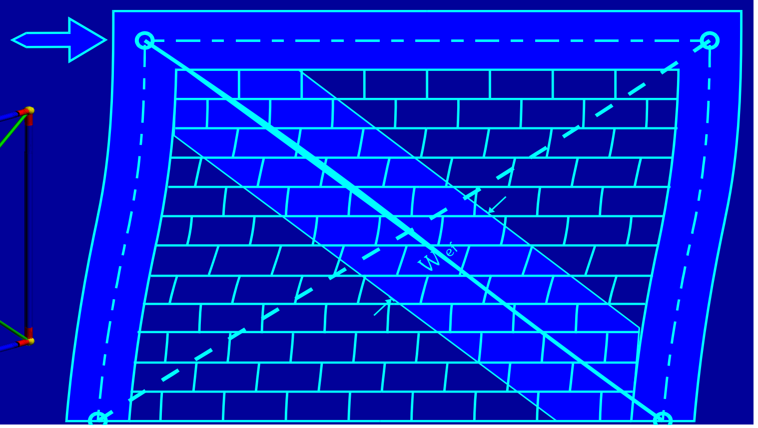
# Asymmetry of infills in plan

- Asymmetric distribution of infills in plan → torsional response to translational horizontal components of seismic action:
  - Members on side with fewer infills (“flexible” side) have larger deformation demands & fail first.
- The increase in global lateral strength & stiffness due to the infills makes up for an uneven distribution of interstorey drift demands in plan:
  - Maximum member deformation demands for planwise irregular infilling do not exceed peak demands anywhere in plan, in a similar structure w/o infills.

# EC8 design against infill planwise asymmetry

- Eurocode 8: doubles accidental eccentricity (from 5 to 10%) in the analysis, if infills are planwise irregular.
- Doubling of accidental eccentricity: is not enough for “severely irregular” arrangement of infills in plan →
  – analysis of 3D structural model explicitly including the infills,
  – sensitivity analysis of the effect of stiffness & position of infills (disregarding one out of 3-4 infill panels per planar frame, especially on flexible sides).
- <u>But:</u>
  – No guidance is given for in-plane modelling of infills.
  – Simplest modelling of solid panel (without openings):
    - two diagonal struts.
  – Effect of openings?





# Adverse local effects on structural frame

Shear failure of weak columns due to interaction with strong infills

# EC8 design against local effect of strong infills

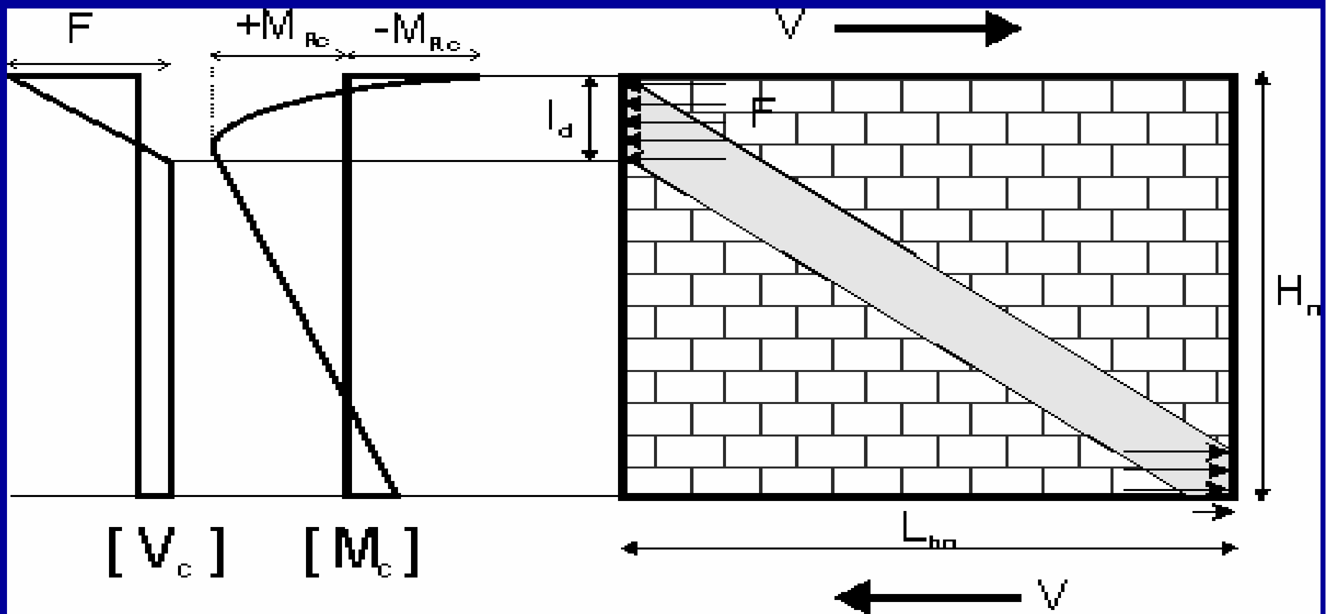
## Shear loading of column by infill strut force:

- Eurocode 8: verify in shear the length $l_c = w_{inf}/\cos\theta$, at top & bottom of column where diagonal strut force of infill may be applied, for the smaller of the two design shear forces:
  - Horizontal component of infill strut force, equal to the horizontal shear strength of the panel (shear strength of bed joints times horizontal cross-sectional area of panel); or
  - Capacity design shear: 2x(design value of column flexural capacity, $M_{Rd,c}$) divided by contact length, $l_c$

Width of strut:

$$w_{inf} = \frac{0.175 L_{bn}}{\cos\theta(\lambda H)^{0.4}}$$

$$\lambda = \left(\frac{E_w b_w \sin 2\theta}{4 E_c I_c H_n}\right)^{\frac{1}{4}}$$




Eurocode 8: fraction (~15%) of panel diagonal, $L_{bn}/\cos\theta$

# **Adverse local effects on structural frame** (cont'd)

## Shear failures of short (captive) columns



# EC8 design of squat “captive” columns

- Capacity-design calculation of design shear force, w/:
  - clear length of column, $l_{cl}$ = length of column not in contact to the infills &
  - plastic hinging assumed to take place at column section at the termination of the contact with the infill wall.
- Transverse reinforcement required to resist the design shear force is placed not just along clear length of column, $l_{cl}$; also into the column part which is in contact to the infills (over length equal to column depth, $h_c$, within plane of infill).
- Entire length of column taken as critical region, with stirrups as in “critical” regions.

# Part IV:
# Seismic assessment and retrofitting of existing buildings, according to Eurocode 8- Part 3
# (emphasis on concrete buildings)

- In seismic regions, existing substandard buildings:

  Largest threat to human life & property.

- From cost-benefit point of view:

  Unless triggered by earthquake, change in use, etc., seismic retrofitting normally is not worthwhile.

- Obstacle to upgrading, in addition to economic factors:
  - lack of standards & guidelines;
  - technical difficulty of design of retrofitting;
  - long disruption of occupancy and use of facility

- Problem technically more challenging in RC than in masonry buildings:
  - Diversity due to wider typology & continuous evolution of codes;
  - short history of exposure to seismic hazard

# EN 1998-3:2005
# Assessment and Retrofitting of buildings

| | | No. of NDPs |
|---|---|---|
| 1. | General | _ |
| 2. | Performance Requirements and Compliance Criteria | 3 |
| 3. | Information for Structural Assessment | 2 |
| 4. | Assessment | 2 |
| 5. | Decisions for Structural Intervention | _ |
| 6. | Design of Structural Intervention | _ |
| Annex A (Informative): | Concrete Structures | 1 |
| Annex B (Informative): | Steel or Composite Structures | 1 |
| Annex C (Informative): | Masonry Buildings | 1 |
| | Total: | 10 |

# Part 3 of EC8:
# Assessment and retrofitting of buildings

- The only part in the whole set of 58 EN-Eurocodes that deals w/ existing structures
- 1st standard in Europe on seismic assessment and retrofitting of buildings – No experience in European practice w/ codified seismic assessment and retrofitting.

➢ Part 3 of EC8 is an experiment. Not known yet whether and how it will work in practice.

# STRUCTURE OF EN 1998-3

- Normative part: General rules on:
  - Performance requirements & criteria (LSs),
  - Analysis methods & applicability conditions,
  - Format of verifications,
  - Information for assessment & implications, etc.

- All material-specific aspects:

  In 3 Informative (nonbinding) Annexes:
  - Concrete structures
  - Steel or composite structures
  - Masonry buildings

# EC8-PART 3, PERFORMANCE-BASED APPROACH:

- Assessment & Retrofitting for different Performance Levels ("Limit States") under different Seismic Hazard levels
- "Limit States" (Performance Levels)

  Damage Limitation (: Immediate Occupancy)

  Significant Damage (: Life Safety)

  Near Collapse

- Flexibility for countries, owners, designers:
  - How many & which Limit States will be met and for what Hazard Level:
    - to be decided by country, or
    - (if country doesn't decide in National Annex) by owner/designer
  - Hazard Levels: NDPs - No recommendation given

    Noted that Basic Objective for ordinary new buildings is:
    - Damage Limitation: Occasional EQ (225yrs)
    - Significant Damage: Rare EQ (475yrs)
    - Near Collapse: Very rare EQ (2475yrs)
- Safety-critical facilities: Enhanced Objective, via multiplication of seismic action by importance factor $\gamma_I$

# EN 1998-3 “Assessment & retrofitting”

- **Fully displacement-based approach:**
  - Capacity-demand-comparisons for verification of ductile elements (existing, retrofitted or new): in terms of deformations.
  - Main deformation measure:
    - <u>Chord rotations at member ends</u>
  - Retrofit aims at reducing deformation demands on existing members below their capacities

    *(global stiffening by addition of new elements easier than local modification of existing members to increase their deformation capacities).*
  - End result:

    More cost-effective assessment & retrofitting

# Seismic Assessment according to EC8-Part 3

# EC8: Detailed seismic assessment of individual buildings:

- Necessary first step for design of the retrofitting.
- Identifies deficiencies to be corrected.
- Assessment criterion in ENV EC8 for strengthening & repair (ENV1998-1-4:1996):

  Compliance with EC8 for new structures.
- Existing structures do not comply with detailing, configuration, regularity, etc. rules of modern codes: according to that criterion, all members need to be retrofitted.

# Information for the Assessment

1. "limited knowledge":
   - Only for linear analysis;
   - "Confidence factor", equal to 1.35, corrects mean material strengths from in-situ tests etc. (division or multiplication, whatever is less favorable).
2. "normal knowledge":
   - For linear or nonlinear analysis;
   - "Confidence factor", equal to 1.2, corrects mean material strengths from in-situ tests etc. (as above).
3. "full knowledge":
   - For linear or nonlinear analysis;
   - Mean material strengths from in-situ tests etc. used w/o "confidence factor".

# Information for the assessment (cont'd)

1. "limited knowledge":
   - Structural geometry from:
     - ✓ original <u>drawings</u> & in-situ spot checks; or
     - ✓ full campaign of <u>in-situ measurements</u>, if original drawings not available.
   - Default assumptions for materials, verified with <u>1 sample</u> /floor /type of member.
   - Reinforcement from <u>simulation of original design</u> (with checks in ~20% of members / type of member).
2. "normal knowledge":
   - Structural geometry & reinforcement from:
     - ✓ original <u>drawings</u> & in-situ checks in <u>~20%</u> of members / type of member; or
     - ✓ full in-situ measurements & reinforcement exposure in <u>≥ 50%</u> of members / type of member, if drawings not available.
   - Materials from:
     - ✓ original <u>specifications</u>, verified in-situ w/ <u>1 sample</u> /floor / type of member; or
     - ✓ <u>2 samples</u> / floor / type of member.
3. "full knowledge":
   - Structural geometry & reinforcement from:
     - ✓ original <u>drawings</u> & in-situ checks in <u>≥ 20%</u> of members / type of member; or
     - ✓ full in-situ measurements & reinf. exposure in <u>≥ 80%</u> of members / type of member
   - Materials from:
     - ✓ original <u>test reports</u>, verified in-situ w/ <u>1 sample</u> /floor / type of member; or
     - ✓ <u>3 samples</u> / floor / type of member.

# “Ductile” vs. “Brittle” elements

- Ductile elements (in RC: columns, beams, walls in bending):

  Verification on the basis of deformations (regardless of analysis procedure).

- Brittle elements (in RC: columns, beams, walls, joints in shear):

  Verification on the basis of forces.

# “Primary” & “Secondary” seismic elements

- Engineer may designate elements as “primary” or “secondary”, depending on which ones he relies upon for lateral stiffness & resistance:
  – lateral stiffness & strength of “secondary” elements neglected in model, or included as degrading w/ cyclic deformations;
  – criteria on their EQ-induced deformations are less strict than for primary elements.

# EC8-PART 3: ANALYSIS METHODS FOR DEFORMATION DEMANDS IN DUCTILE ELEMENTS

- 4 types of analysis for deformation demands, all w/ seismic action defined by 5%-damped elastic spectrum:
  1. Linear static (equivalent lateral forces);
  2. Linear dynamic (modal response spectrum);
  3. Nonlinear static ("pushover") – Reference method;
  4. Nonlinear dynamic (time-histories: $\geq 3$, $\geq 7$ for mean results).

For 1 & 2: Equal displacement rule, w/o correction coefficients.

For 3: N2-method (target displacement: Equal displacement rule w/ correction due to short-T only).

For 3: If higher-modes important ($T>4T_c$, or $T>2$sec): "Modal pushover" or nonlinear dynamic analysis.

For 3 & 4: Simple nonlinear member models encouraged;

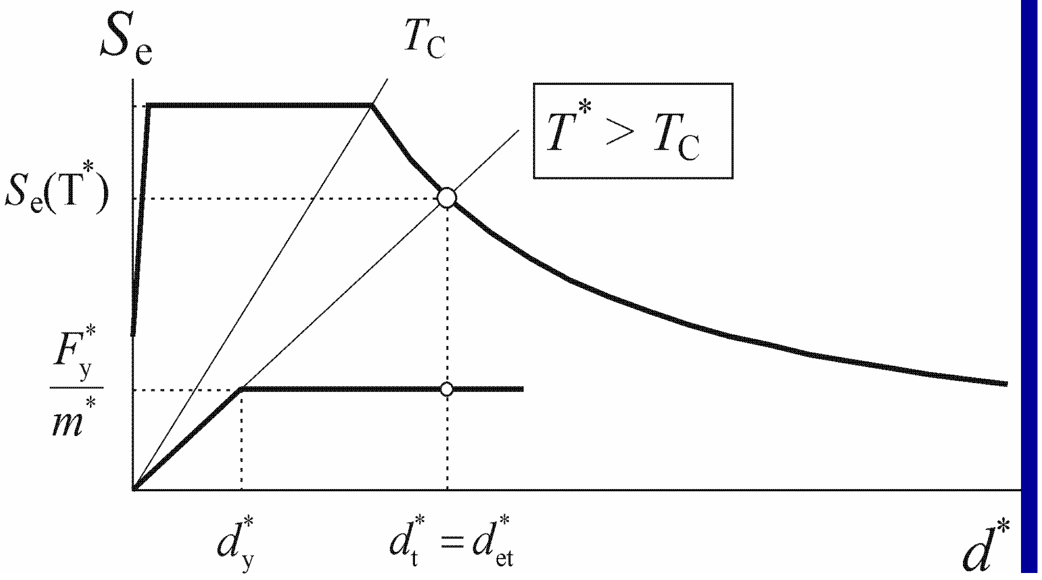
- More important than sophistication of model: ability to represent effective stiffness up to yielding, to capture dominant periods.

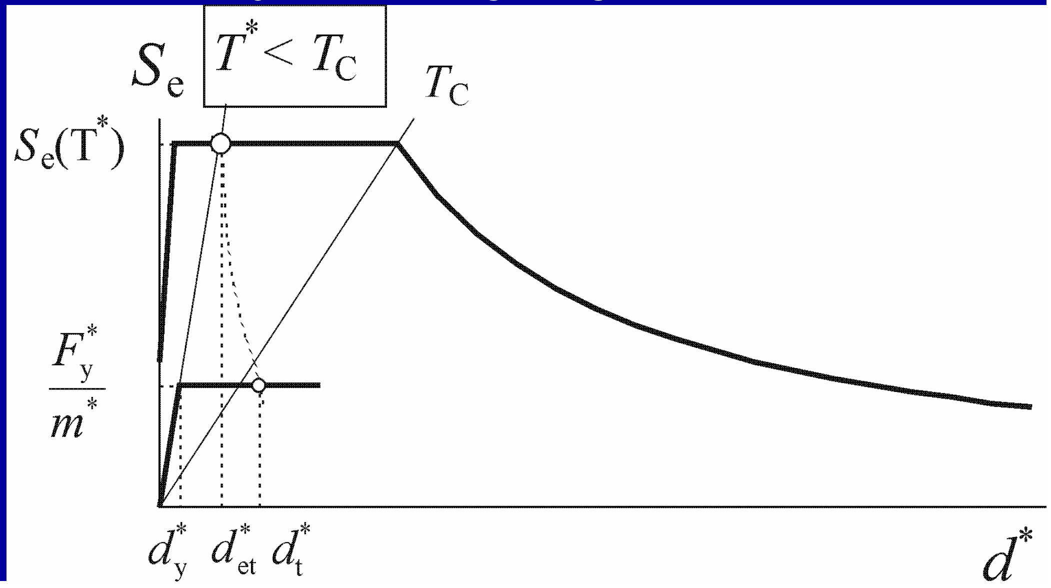
# Nonlinear static analysis (pushover)

- Basis: Fajfar's N2 method:
  - Lateral forces on masses $m_i$ follow postulated pattern of horizontal displacements, $\Phi_i$, with $\Phi_n$=1 at the "control node": $F_i = \alpha m_i \Phi_i$
  - Use a "uniform pattern" $\Phi_i$=1 **and** a (fundamental) "modal pattern" $\Phi_i$
  - Equivalent Single-Degree-of-Freedom System:

$$m^* = \sum m_i \Phi_i \qquad F^* = \frac{F_b}{\Gamma} \qquad d^* = \frac{d_n}{\Gamma} \qquad \Gamma = \frac{m^*}{\sum m_i \Phi_i^2}$$

  - Target displacement from 5%-damped elastic spectrum
    - equal displacement if $T>T_C$ $\qquad$ $\mu=1+(q-1)T_c/T$, if $T<T_C$ ($T_C$: transition period)



# EC8-PART 3: APPLICABILITY OF LINEAR ANALYSIS

Under seismic action (hazard level) of interest:

- Uniform distribution of inelasticity:

  DCR: Ratio of (elastic) moment demand to capacity (~member displacement ductility ratio).

  Criterion:

  Ratio of Max. to Min. value of DCR over all ductile members that go inelastic (ends of strong elements framing into joint excluded)

  < limit value, between 2 and 3 (NDP; recommended value: 2.5).

  (Fairly restrictive; linear analysis only for buildings w/ fairly uniform distribution of overstrengths).

If

(a) criterion above is satisfied,

(b) building is heightwise regular &

(c) higher-modes are unimportant ($T<4T_c$, $T<2sec$), then:

- Linear static analysis w/ triangular distribution of lateral forces

# **REGULARITY IN ELEVATION IN EN1998-1** (applies to Part 3) (FOR APPLICABILITY OF LATERAL FORCE PROCEDURE)

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.*

# Effective elastic stiffness, EI (in linear or nonlinear analysis)

- Part 1of EC8 (for design of new buildings):
  - EI = secant stiffness at yielding;
  - RC: EI = 50% of uncracked gross-section stiffness.
- 50% of uncracked gross-section stiffness:
  - OK in force-based design of new buildings (conservative for force demands);
  - Not OK in displacement-based assessment (unconservative for displacement demands).
- More realistic, esp. in damage limitation check, $\theta_E \leq \theta_y$:
  - $EI = M_y L_s / 3\theta_y$ : secant stiffness at yielding of both ends in antisymmetric bending

# Annex A: RC member verification criteria

| Limit State (LS): | Damage Limitation | Significant Damage (SD) | Near Collapse (NC) | |
|---|---|---|---|---|
| Member: | | | linear analysis | nonlinear analysis |
| ductile primary | $\theta_E \leq \theta_y$ | $\theta_E \leq 0.75\theta_{u,m\text{-}\sigma}$ | $\theta_E \leq \theta_{u,m\text{-}\sigma}$ | |
| ductile secondary | | $\theta_E \leq 0.75\theta_{um}$ | $\theta_E \leq \theta_{um}$ | |
| brittle primary | Check only if NC LS not checked. Then use NC criteria w/ $V_E$ (or $V_{E,CD}$ for SD LS w/ linear analysis) | | $V_{E,CD} \leq V_{Rd,EC2}$, $\leq V_{Rd,EC8}/1.15$ | $V_E \leq V_{Rd,EC2}$, $\leq V_{Rd,EC8}/1.15$ |
| brittle secondary | | | $V_{E,CD} \leq V_{Rm,EC2}$, $\leq V_{Rm,EC8}$ | $V_E \leq V_{Rm,EC2}$, $\leq V_{Rm,EC8}$ |

$\theta_E$, $V_E$: chord-rotation & shear force demand from analysis; $V_{E,CD}$ : from capacity design;

$\theta_y$: chord-rotation at yielding; $\theta_{um}$: expected value of ultimate chord rotation;

$\theta_{u,m\text{-}\sigma}$: mean-minus-sigma ult. chord rotation $=\theta_{um}/1.5$, or $=\theta_y+\theta^{pl}_{um}/1.8$;

$V_{Rd}$, $V_{Rm}$: shear resistance, w/ or w/o material safety & confidence factors;

$V_{R,EC2}$: shear resistance in mon. loading; $V_{R,EC8}$: shear resistance in cyclic loading after flex. yielding.

**<u>If analysis is linear:</u> Shear force $V_{E,CD}$ from equilibrium, under end moments consistent with plastic hinging there or (in beams or columns) around joint**

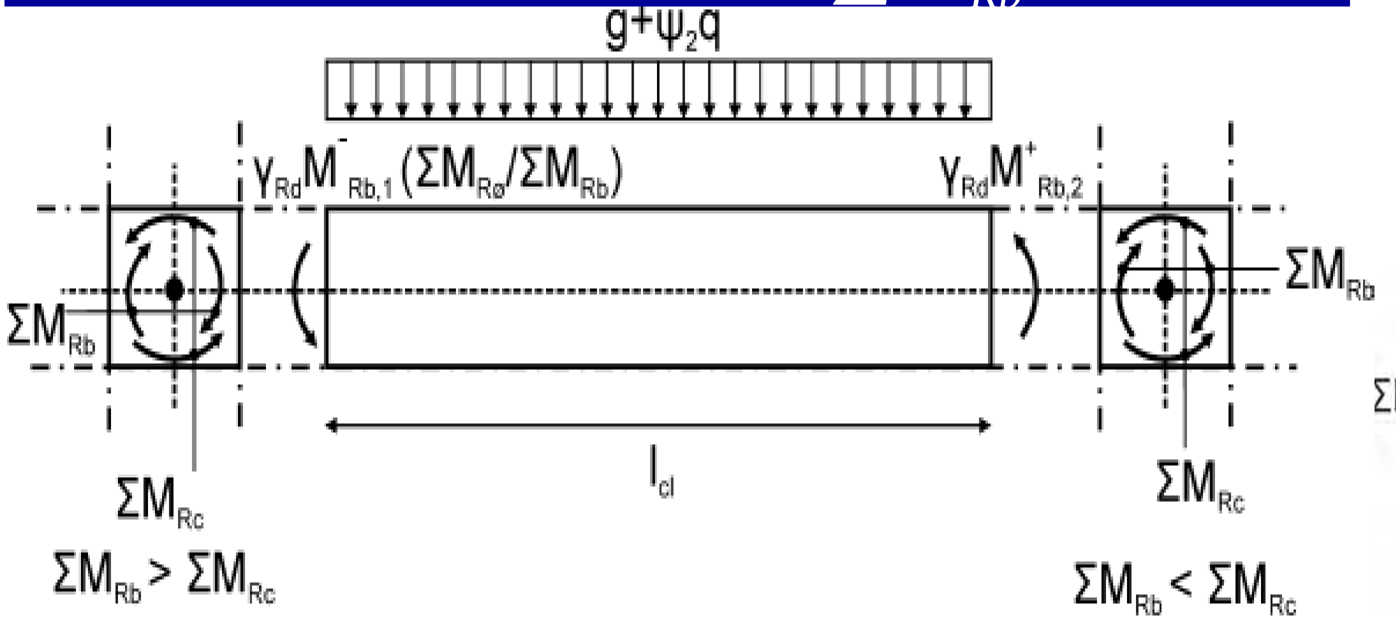
**<u>Walls:</u>**

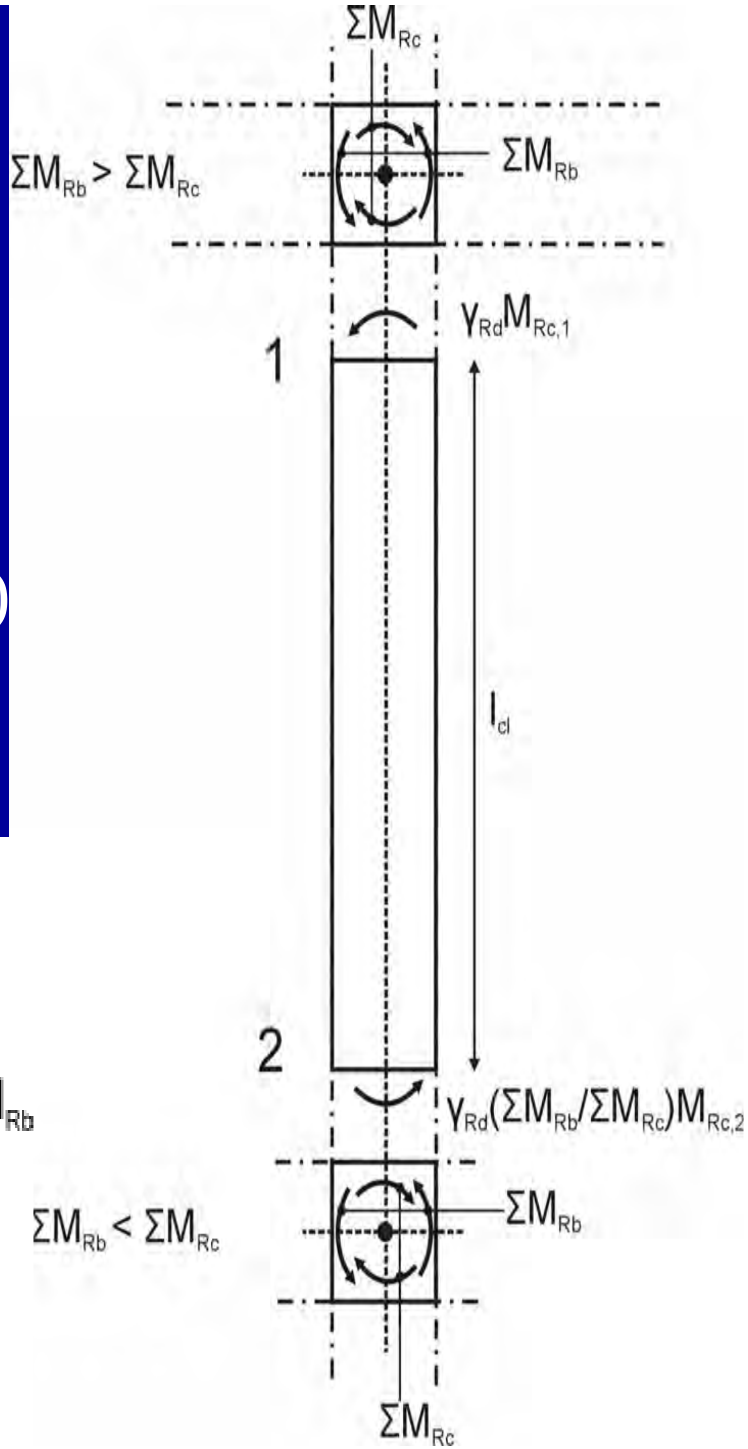
$$V_{Sd} = \frac{M_{Rw}}{M_{EW}} V_E$$ **($M_{RW}$, $M_{EW}$ : at the base)**

**<u>Columns:</u>**

$$M_{i,d} = M_{Rc,i} \min\left(1, \frac{\sum M_{Rb}}{\sum M_{Rc}}\right)$$

**<u>Beams:</u>** $$M_{i,d} = M_{Rb,i} \min\left(1, \frac{\sum M_{Rc}}{\sum M_{Rb}}\right)$$



# Annex A: Chord-rotation at RC member yielding

Beams, rect. columns:

$$\theta_y = \phi_y \frac{L_s + z}{3} + 0.0013\left(1 + 1.5\frac{h}{L_s}\right) + 0.13\phi_y \frac{d_b f_y}{\sqrt{f_c}}$$

Walls:

$$\theta_y = \phi_y \frac{L_s + z}{3} + 0.002\left(1 - \max[1, \frac{L_s}{8h}]\right) + 0.13\phi_y \frac{d_b f_y}{\sqrt{f_c}}$$

- $\phi_y$: yield curvature (via 1st principles, adapted to median $M_y$);
- $L_s$ = M/V: shear span at member end (~L/2);
- z~0.9d: tension shift (= 0 if member not diagonally cracked by shear at flexural yielding: $M_y/L_s$);
- h : section depth (diameter D for circular piers);
- $f_y$, $f_c$: MPa;
- $d_b$: bar diameter;
- Last term: Due to bar slip from anchorage zone beyond member end (omitted if such slippage not possible)

# ANNEX A: Seismically-detailed RC members w/ rect. web

## Expected value of ultimate chord rotation (20% drop in resistance)

$$\theta_{um} = \alpha_{st}\left(1-\frac{3a_{wall}}{8}\right)\left(0.3^{\nu}\right)\left[\frac{\max(0.01,\omega')}{\max(0.01,\omega)}f_c\right]^{0.225}\left(\frac{L_s}{h}\right)^{0.35}25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)}\left(1.25^{100\rho_d}\right)$$

or:

$$\theta_{um} = \theta_y + \alpha_{st,pl}\left(1-0.4a_{wall}\right)\left(0.25^{\nu}\right)\left[\frac{\max(0.01,\omega')}{\max(0.01,\omega)}\right]^{0.3} f_c^{0.2}\left(\frac{L_s}{h}\right)^{0.35}25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)}\left(1.275^{100\rho_d}\right)$$

| | |
|---|---|
| $\alpha_{st}$: | 0.016 for hot-rolled ductile steel or heat-treated (tempcore);<br>0.01 for brittle cold-worked steel; |
| $\alpha_{st,pl}$: | 0.0145 for hot-rolled ductile steel or heat-treated (tempcore);<br>0.0075 for brittle cold-worked steel; |
| $\alpha_{wall}$: | 1 for shear walls; |
| $\omega$, $\omega'$: | mechanical ratio of tension (including web) & compression steel; |
| $\nu$: | $N/bhf_c$ (b: width of compression zone; N>0 for compression); |
| $L_s/h$ : | M/Vh: shear span ratio; |
| α : | confinement effectiveness factor : $\alpha = \left(1-\frac{s_h}{2b_c}\right)\left(1-\frac{s_h}{2h_c}\right)\left(1-\frac{\sum b_i^2}{6b_ch_c}\right)$ |
| $\rho_{sx}$: | $A_{sh}/b_ws_h$: transverse steel ratio // direction (x) of loading; |
| $\rho_d$: | ratio of diagonal reinforcement. |

## Non-seismically detailed members w/o lap splices - cyclic loading

- Plastic part, $\theta^{pl}_{um}=\theta_{um}-\theta_y$, of ultimate chord rotation is multiplied by 0.825.

# Seismically detailed RC beams, columns, walls w/ rect. web in cyclic loading:

## Expected value of ultimate chord rotation (Alternative)

$$\theta_u = \theta_y + \theta_u^{pl} = \theta_y + (\varphi_u - \varphi_y) L_{pl} \left( 1 - \frac{0.5 L_{pl}}{L_s} \right)$$

- $\phi_y$: yield curvature (from 1st principles);
- $\phi_u = \min\left( \frac{\varepsilon_{cu,c}}{\xi_{cu,c} d_c}, \frac{\varepsilon_{su}}{(1-\xi_{su})d} \right)$, with: $\varepsilon_{cu,c} = 0.004 + 0.5\left( \alpha \rho_s f_{yw} / f_{cc} \right)$

  where $\rho_s$: stirrup ratio, index c: confined
- $L_{pl} = \frac{L_s}{30} + 0.02h + 0.11 \frac{d_b f_y}{\sqrt{f_c}}$
- $L_s$ = M/V: shear span at member end;
- $h$: section depth;
- $f_y$, $f_c$: MPa;
- $d_b$: bar diameter.

# RC members w/ or w/o seismic detailing, w/ ribbed bars lap-spliced over $l_o$ in plastic hinge region

- Compression reinforcement counts as double.

- For yield properties $M_y$, $\phi_y$ , $\theta_y$: $f_y$ of tension steel multiplied x $l_o/l_{oy,min}$ if $l_o \le l_{oy,min}=(0.3f_y/\sqrt{f_c})d_b$

- For ultimate chord rotation $\theta_{um}=\theta_y+\theta^{pl}_{um}$: $\theta^{pl}_{um}$ x $l_o/l_{ou,min}$ if $l_o \le l_{ou,min}=d_b f_y/[(1.05+14.5\alpha_{rs}\omega_{sx})\sqrt{f_c}]$,
  - $f_y$, $f_c$ in MPa, $\omega_{sx}=\rho_{sx}f_{yw}/f_c$: mech. transverse steel ratio // loading,
  - $\alpha_{rs}=(1-s_h/2b_o)(1-s_h/2b_o)n_{restr}/n_{tot}$ ($n_{restr}/n_{tot}$ restrained-to-total lap-spliced bars).

# Cyclic shear resistance of RC members (reduction w/ cyclic displacements)

- Shear resistance after flexural yielding, as <u>controlled by stirrups</u>

(linear degradation of both $V_c$ and $V_w$ with displacement ductility demand $\mu_\Delta{}^{pl}=(\theta-\theta_y)/\theta_y$ )

$$V_R = \frac{h-x}{2L_s}\min\left(N, 0.55A_c f_c\right) + \left(1-0.05\min\left(5, \mu_\Delta^{pl}\right)\right)\left[0.16\max(0.5, 100\rho_{tot})\left(1-0.16\min\left(5, \frac{L_s}{h}\right)\right)\sqrt{f_c}A_c + V_w\right]$$

$V_w$: contribution of web reinf. = $\rho_w b_w z f_{yw}$ ($b_w$: web width, z: internal lever arm; $\rho_w$: web reinf. ratio)

$\rho_{tot}$: total longitudinal reinforcement ratio

h: section depth

x : depth of compression zone

$A_c = b_w d$

- Shear resistance as <u>controlled by web crushing</u> (diagonal compression)
  - **Walls**, before flexural yielding ($\mu_\Delta{}^{pl} = 0$), or after flexural yielding (cyclic $\mu_\Delta{}^{pl} > 0$):

$$V_R = 0.85\left(1-0.06\min\left(5, \mu_\Delta^{pl}\right)\right)\left(1+1.8\min\left(0.15, \frac{N}{A_c f_c}\right)\right)\left(1+0.25\max(1.75, 100\rho_{tot})\right)\left(1-0.2\min\left(2, \frac{L_s}{h}\right)\right)\sqrt{f_c}b_w z$$

  - **Squat columns** ($L_s/h \leq 2$) after flexural yielding (cyclic $\mu_\Delta{}^{pl} > 0$):

$$V_R = \frac{4}{7}\left(1-0.02\min\left(5, \mu_\Delta^{pl}\right)\right)\left(1+1.35\frac{N}{A_c f_c}\right)\left(1+0.45\cdot 100\rho_{tot}\right)\sqrt{\min\left(f_c, 40\right)}b_w z\sin 2\delta$$

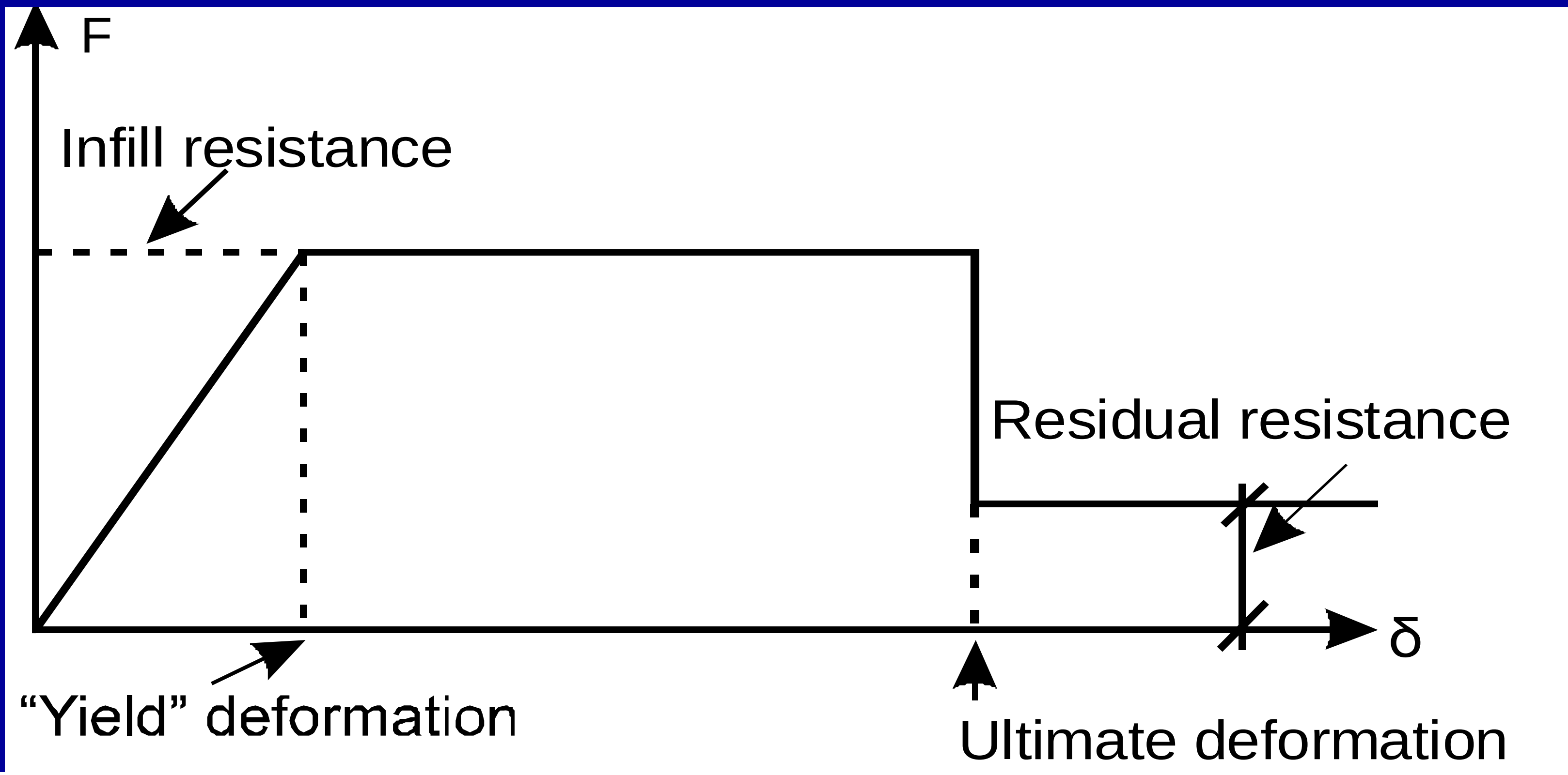
δ: angle between axis and diagonal of column ($\tan\delta = h/2L_s$)

# Conclusion: EC8-Assessment approach for RC

- Estimation of displacement/deformation demands independent from deformation capacities: deformation demands and capacities estimated and compared <u>at the member level</u> (chord rotations).
- <u>Analysis</u> for the estimation of displacement/deformation demands may be simple (even linear, if inelasticity is uniformly distributed within structure);
  - the basis for estimation of displacement/deformation demands is the <u>equal displacement rule (</u>except in nonlinear dynamic analysis);
  - simple member models are encouraged;
  - more important than the sophistication of the model is the ability to reproduce the <u>effective stiffness to yielding</u>, in order to capture the dominant periods of vibration.
- Simple, yet fairly accurate semi-empirical models given for estimation of member <u>deformation capacities</u>, as controlled by flexure, shear or lap-splicing.
- Approach simple, but practical.

# Major gap for assessment & retrofitting existing buildings

- Although effect of infills on seismic performance is largest, if the structural frame has little engineered earthquake resistance:
  - Lack of specific rules for modelling, verification & retrofitting masonry infills (possibly into semi-structural components).



# Seismic Retrofitting (EC8-Part 3, Annex A)

# General rules:

- Detailed assessment should guide selection of retrofit strategy & extent of intervention:
  - *Deficiencies in few scattered elements*:

    local modification of elements
  - *Deficiencies in one part of the structure:*

    possible irregularity (weak storey, unbalanced structure, etc.) to be removed (by adding new elements, strengthening or even weakening existing members, etc.)
  - *Generalized deficiency:*

    add new elements (walls or bracings) to increase stiffness & reduce deformation demands;

    or upgrade most (if not all) elements (costly, inconvenient)

# Concrete Jackets

# ***Concrete Jackets*** (continued/anchored in joint) **EN1998-3**

Calculation assumptions:

- Full composite action of jacket & old concrete assumed (jacketed member: monolithic”), even for minimal shear connection at interface (roughened interface, steel dowels epoxied into old concrete: useful but not essential);
- $f_c$ of “monolithic member”= that of the jacket (avoid large differences in old & new $f_c$)
- Axial load considered to act on full, composite section;
- Longitudinal reinforcement of jacketed column: mainly that of the jacket. Vertical bars of old column considered at actual location between tension & compression bars of composite member (~ “web” longitudinal reinforcement), with its own $f_y$;
- Only the transverse reinforcement of the jacket considered for confinement;
- For shear resistance, the old transverse reinforcement taken into account only in walls, if anchored in the (new) boundary elements.

Then:

✓ $M_R$ & $M_y$ of jacketed member: ~100% of

✓ $\theta_y$ of jacketed member for pre-yield (elastic) stiffness:

if roughening of interface ~105%,
if no roughening ~120% of

✓ Shear resistance of jacketed member: ~90% of

✓ Flexure-controlled ultimate deformation $\theta_u$: ~100% of

those of “monolithic member” calculated w/ assumptions above.

## Concrete Jackets w/ bars not continued/anchored in joint:

Jacket considered only to confine the full old section.

# Steel Jackets

## ***Steel Jackets*** (not continued/anchored in joint): **EN1998-3**

Jacket stops ahead of joint (several mm gap to joint face)

- Flexural resistance, pre-yield (elastic) stiffness & flexure-controlled ultimate deformation of RC member : not enhanced by jacket (flexural deformation capacity ~same as in “old” member inside jacket, w/o effect of confinement);
- 50% of shear resistance of steel jacket, $V_j=A_j f_{yj} h$, can be relied upon for shear resistance of retrofitted member (suppression of shear failure before or after flexural yielding);
- Lap-splice clamping effected via friction mechanism at jacket-member interface, if jacket extends to ~1.5 times splice length and is bolt-anchored to member at end of splice region & ~1/3 its height from joint face (anchor bolts at third-point of side)

# FRP Jackets

## ***FRP Jackets*** (not continued/anchored in joint): **EN1998-3**

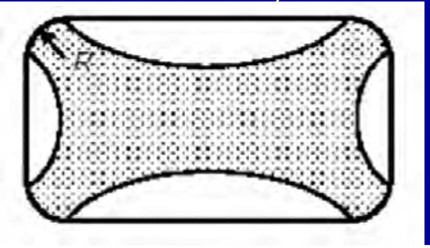
Rectangular X-section w/ continuous longitudinal bars (no lap splices):

- $M_R$ & $M_y$, pre-yield (elastic) stiffness $EI_{eff}$ of RC member: not significantly enhanced by FRP jacket (increase neglected);
- Flexure-controlled ultimate deformation, $\theta_u$: confinement factor due to stirrups enhanced due to FRP confinement by $\alpha\rho_f f_{f,e}/f_c$
  - $\rho_f=2t_f/b_w$ : FRP ratio;
  - $f_{f,e}$: FRP effective strength: $f_{f,e}=\min\left(f_{u,f}, \varepsilon_{u,f}E_f\right)\left(1-0.7\min\left(f_{u,f}, \varepsilon_{u,f}E_f\right)\frac{\rho_f}{f_c}\right)$

    where:

    $f_{u,f}$, $E_f$ : FRP tensile strength & Modulus;

    $\varepsilon_{u,f}$: FRP limit strain; CFRP, AFRP: $\varepsilon_{u,f}$=0.015; GFRP: $\varepsilon_{u,f}$=0.02; polyacetal FRP: $\varepsilon_{u,f}$= 0.032;
  - confinement effectiveness: $\alpha=\left(1-\frac{(h-2R)^2+(b-2R)^2}{3bh}\right)$ *b, h:* sides of X-section; *R*: radius at corner



## ***FRP Jackets*** (not continued/anchored in joint): **EN1998-3**

Rectangular X-section w/ longit. bars lap-spliced **over $l_o$** in plastic hinge:

- Compression reinforcement counts as double.
- For yield properties $M_y$, $\phi_y$, $\theta_y$: $f_y$ of tension steel multiplied x $l_o/l_{oy,min}$ if $l_o<l_{oy,min}=(0.2f_y/\sqrt{f_c})d_b$
- For ultimate chord rotation $\theta_{um}=\theta_y+\theta^{pl}_{um}$: $\theta^{pl}_{um}$ calculated on the basis of confinement by the stirrups alone, multiplied x $l_o/l_{ou,min}$ if $l_o<l_{ou,min}=d_b f_y/[(1.05+14.5\alpha_{rs}\rho_f f_{f,e}/f_c)\sqrt{f_c}]$,
  - $f_c$ in MPa, $\rho_f=2t_f/b_w$: FRP ratio, $f_{f,e}$: effective FRP strength in MPa,
  - $\alpha_{rs}=4/n_{tot}$ ($n_{tot}$ : total lap-spliced bars, only the 4 corner ones restrained).

# *FRP Jackets* – EN 1998-3/Annex A (cont'd)

- Shear resistance of FRP-jacketed member:

$$V_R = \frac{h-x}{2L_s}\min\left(N, 0.55A_c f_c\right) + \left(1-0.05\min\left(5, \mu_\theta^{pl}\right)\right)\left[0.16\max(0.5, 100\rho_{tot})\left(1-0.16\min\left(5, \frac{L_s}{h}\right)\right)\sqrt{f_c}A_c + V_w\right] + V_f$$

  $V_f = \min(\varepsilon_{u,f}E_{u,f}, f_{u,f})\rho_f b_w z/2$

  contributes to member shear resistance as controlled by <u>diagonal tension</u>

  – $\rho_f$ :FRP ratio, $\rho_f = 2t_f/b_w$;

  – $f_{u,f}$:FRP tensile strength;

  – z : internal lever arm.

- Total shear resistance of retrofitted member as controlled by diagonal tension, should not exceed shear resistance of old RC member as controlled by <u>web crushing</u>.

# SPEAR building assessment & retrofitting

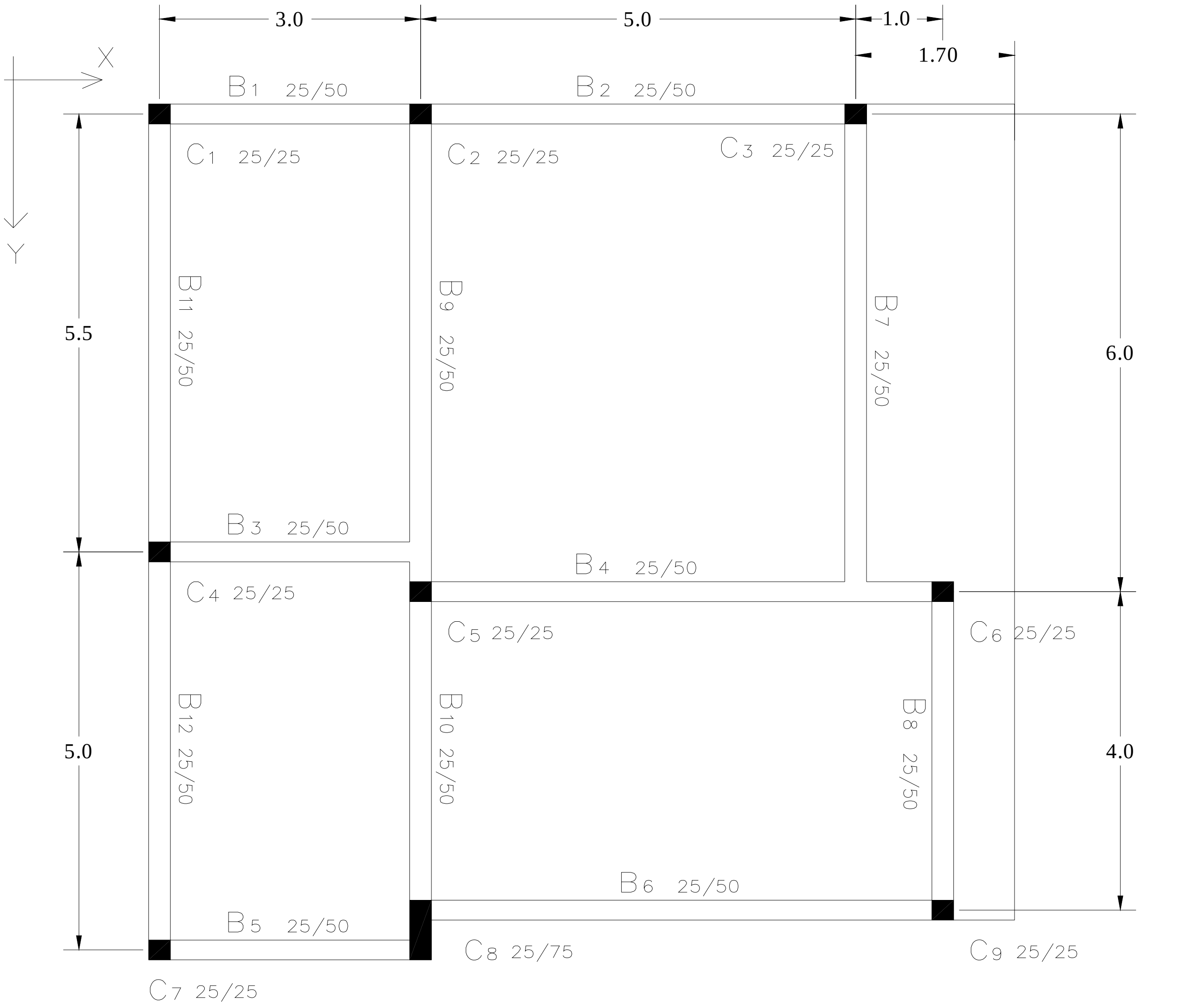
PRE-TEST NONLINEAR DYNAMIC ANALYSIS OF TORSIONALLY UNBALANCED 3-STOREY FULL-SCALE BUILDING, PsD-TESTED AT JRC-ISPRA UNDER BIDIRECTIONAL MOTION.

MODELLING, MEMBER STIFFNESS, RESISTANCE & ULTIMATE DEFORMATIONS ACCORDING TO EC8-PART 3 (by UPatras, Structures Lab)

STRUCTURE TESTED BEFORE RETROFITTING (Jan. 2004) & AFTER REPAIR & RETROFITTING (Sept. 2004, March 2005)

# TORSIONALLY UNBALANCED 3-STOREY SPEAR TEST BUILDING

- Representative of buildings of the 60's in Greece w/o engineered EQ-resistance
  - eccentric beam-column connections
  - smooth/hooked bars lap-spliced at floor levels




# Analysis of frame response & assessment of its performance w/ models accepted/proposed by EN 1998-3

- Nonlinear dynamic analysis:
  - columns fixed at foundation level
  - finite size of beam-column joints
  - P-Δ effects in columns
  - Members:
    1. Point-hinge model;
    2. (simplified) Takeda model (bilinear envelope, no strength degradation);
    3. Elastic stiffness EI = $M_yL_s/3\theta_y$: secant at yielding in antisymmetric bending;
    4. Flexure-controlled ultimate chord rotation (mean capacity);
    5. Shear resistance as reduced by post-yield cyclic deformations.

    3.-5. w/ modifications due to:
    - poor detailing of unretrofitted columns (including splicing of smooth/hooked bars);
    - FRP-wrapping or RC jacketing of columns.

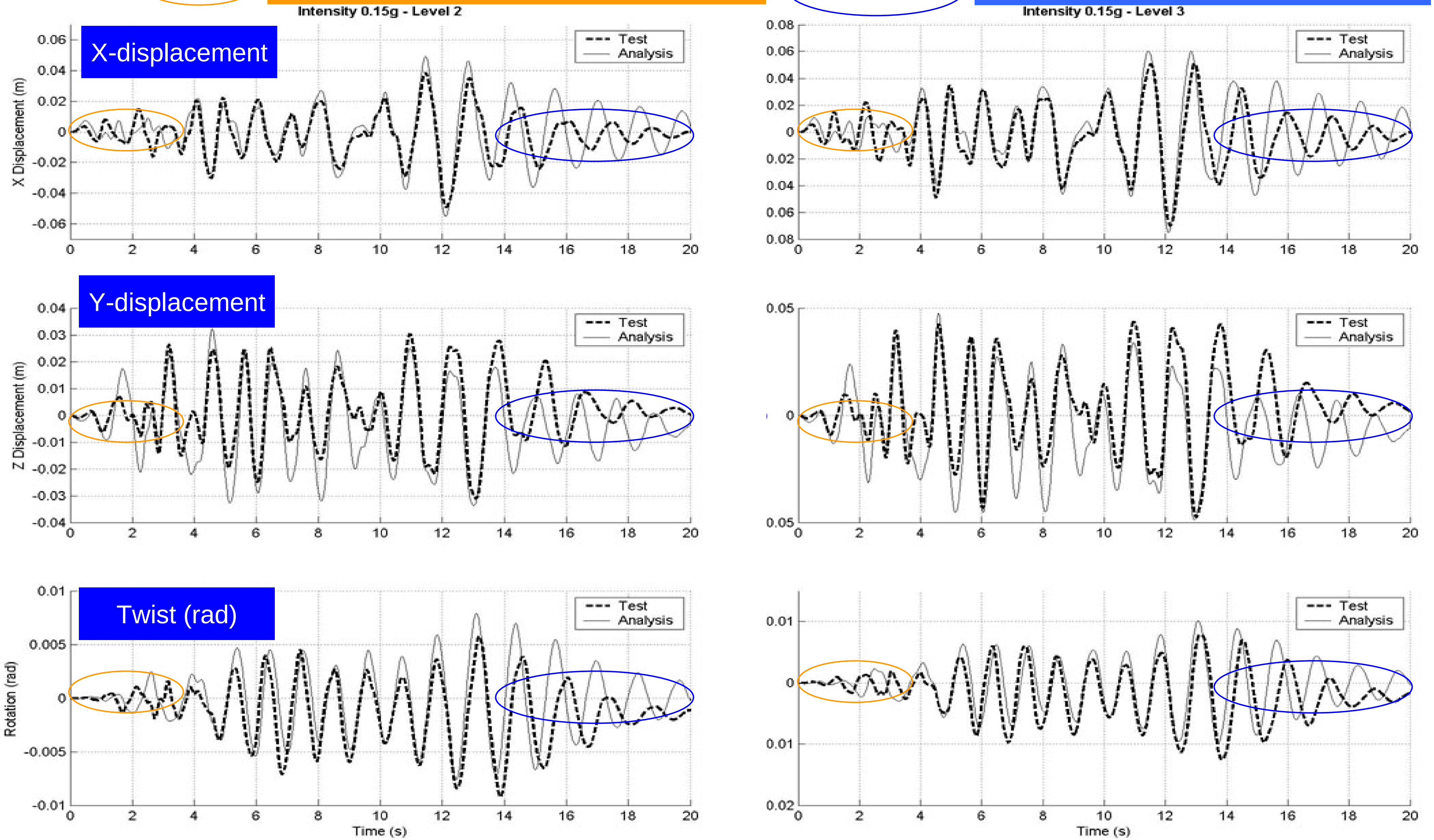
- Performance evaluated in terms of chord rotation demand-to-capacity (damage) ratio:
  - At “ultimate deformation” of the member (: resistance becomes < 80% of peak resistance) Demand-to-capacity (damage) ratio = 1.

PsD test at 0.15g

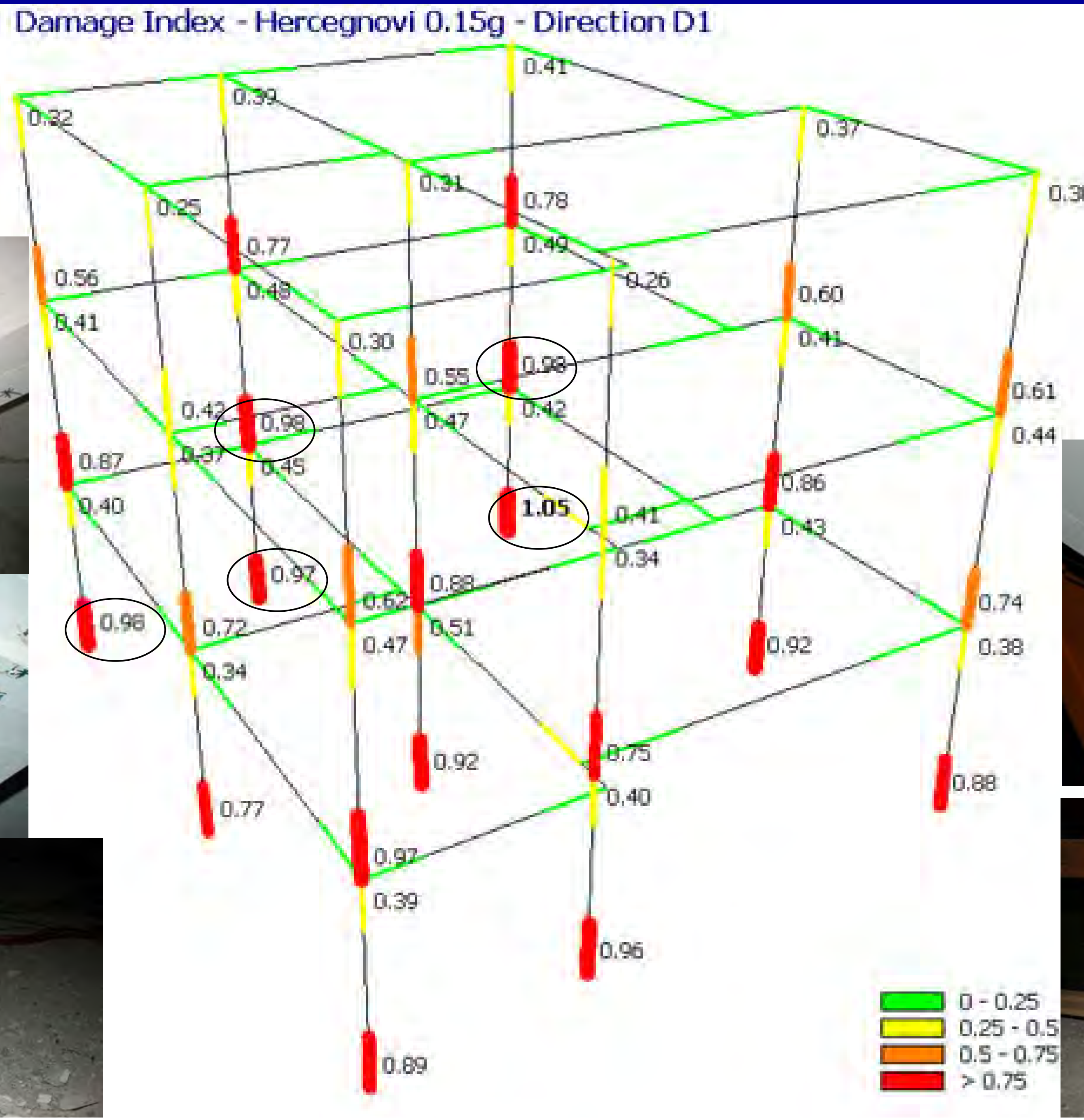
# **Unretrofitted frame, 0.15g:** t-histories of hor. displacements & twist at CM, floors 2 & 3 (continuous line: pre-test calculations; dotted line: test)

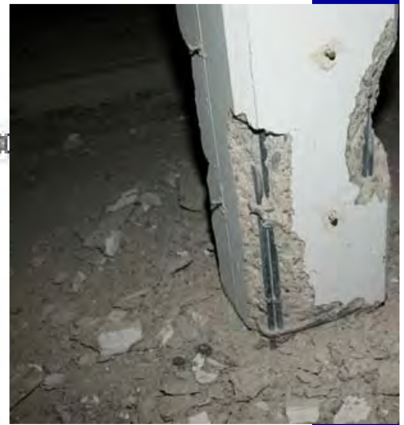
: initial T overestimated by secant-to-yield stiffness

:post-ultimate strength degradation → real T ↑



# Column & beam demand-to-capacity (damage) ratio:



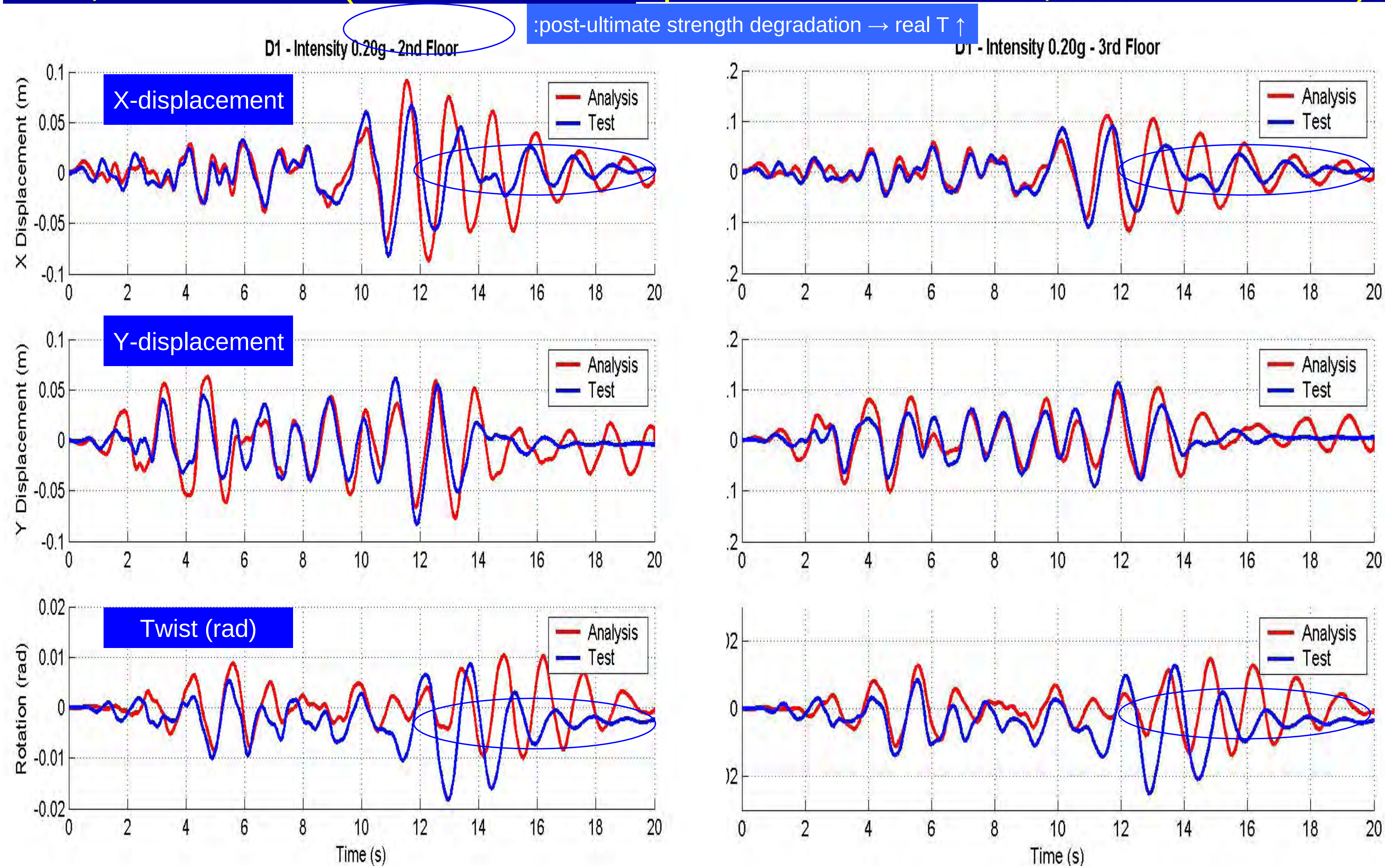
# FRP-retrofitting of frame: Analysis of response & performance evaluation according to EC8-Part 3

- Ends of all 0.25m-square columns in all 3 storeys wrapped with 2 layers of uni-directional GFRP over 0.6m from face of joint, for confinement.
- Full-height wrapping of large (0.25x0.75 m) column with 2 layers of bi-directional Glass FRP for confinement & shear strengthening.
- 2 layers of bi-directional Glass FRP applied on (two) exterior faces of corner joints for shear strengthening (also over end of adjacent beams); no continuity w/ FRP wrapping of member ends.
- Retrofitted frame re-tested (at PGA of 0.2g or 0.3g).
- Pre-test analysis of response to 0.2g bidirectional motions, with modelling assumptions & evaluation criteria (including the FRP-wrapped members) according to EC8-Part 3.

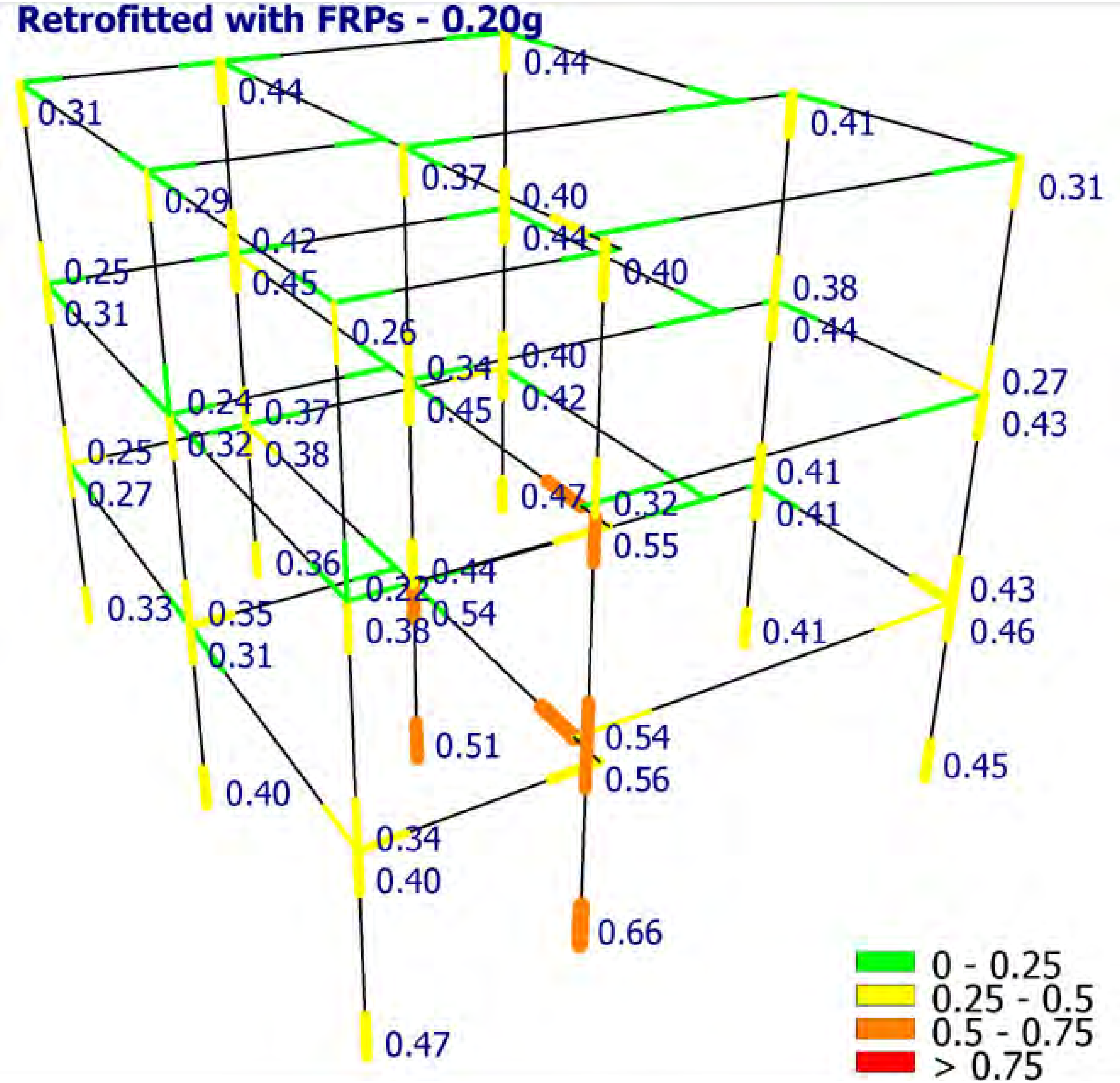
# FRP-retrofitting of frame

PsD test w/ FRPs at 0.2g

# FRP-retrofitted frame, 0.2g: t-histories of hor. displacements & twist at CM, floors 2 & 3 (continuous line: pre-test calculations; dotted line: test)

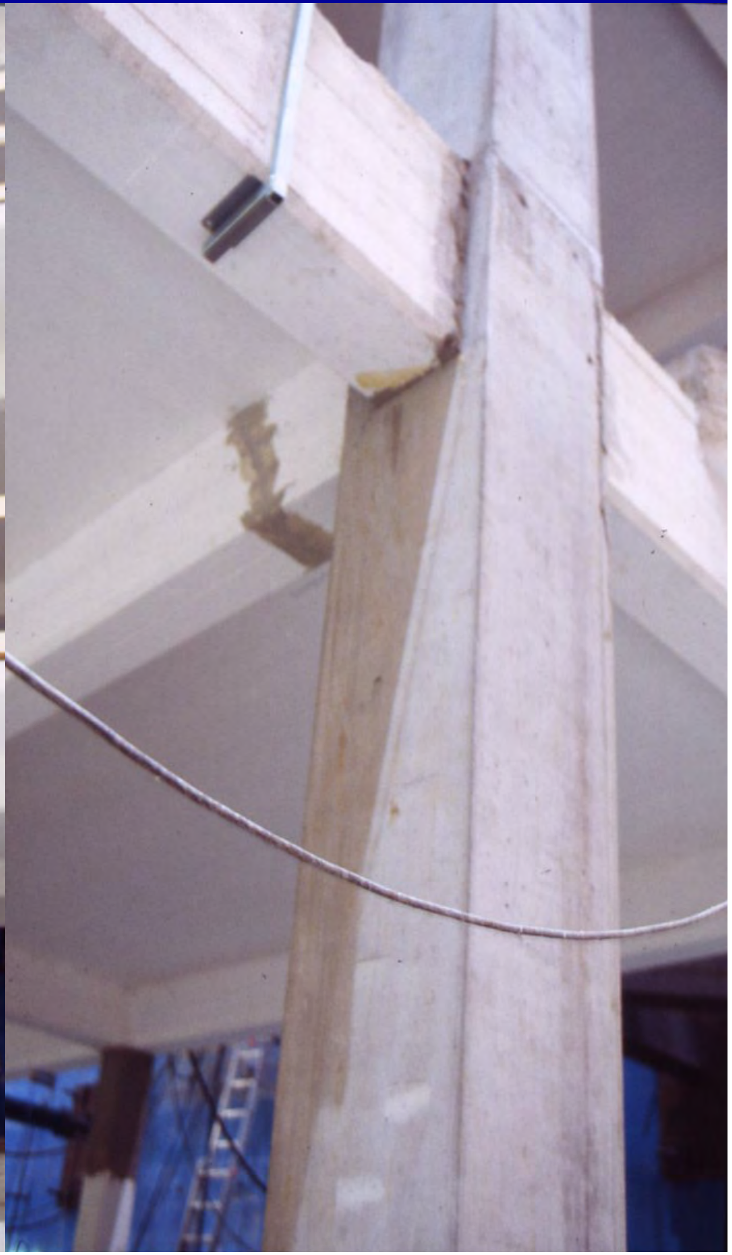


FRP-retrofitted frame, 0.2g: Predicted column & beam demand-to-capacity (damage) ratio



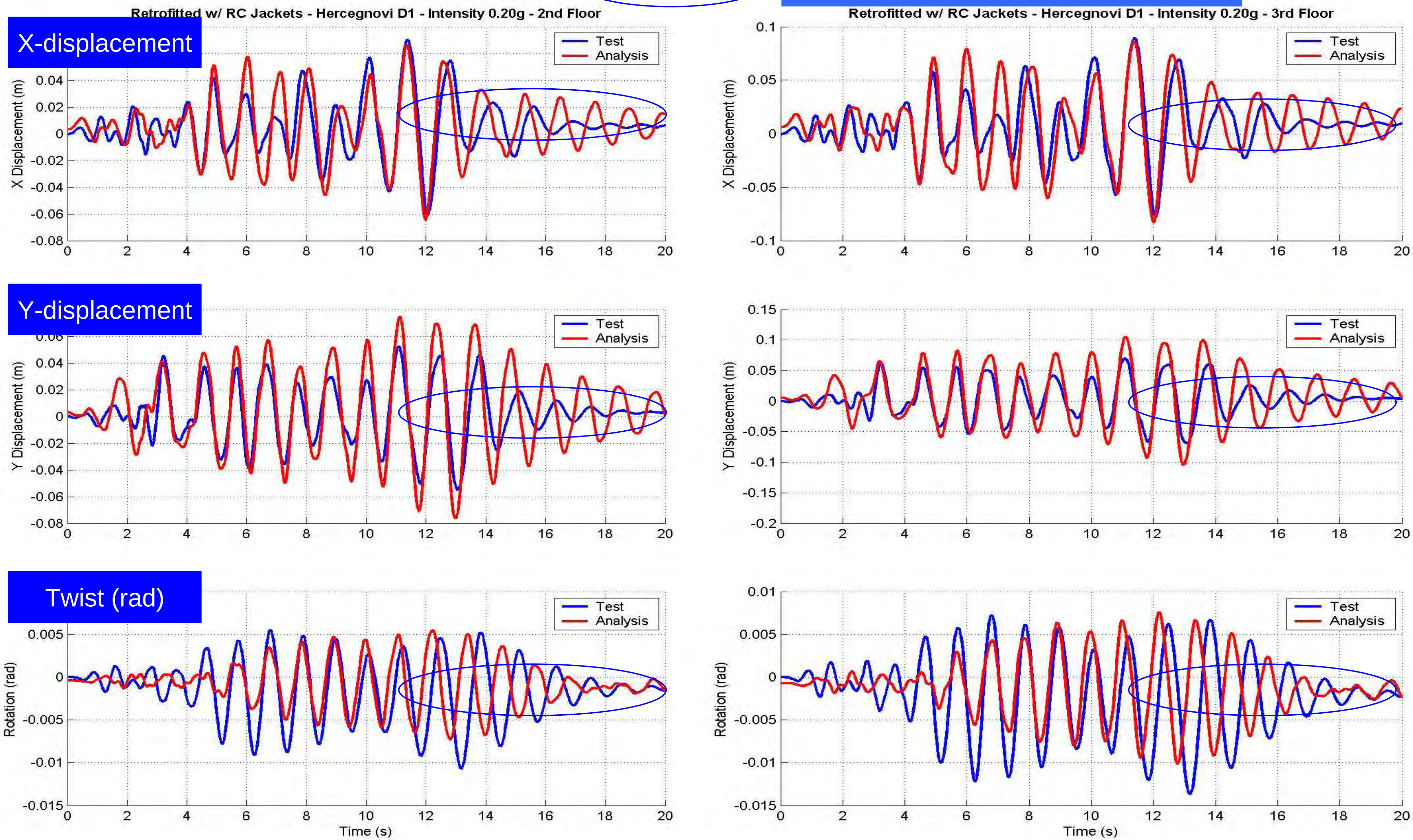
# Concrete-jacket retrofitting of frame: Analysis of response & performance evaluation per EN1998-3

- RC jacketing of the central columns on two adjacent flexible sides from 250mm- to 400mm-square, w/ 3 16mm bars along each side & a 10mm perimeter tie @ 100mm centres.
- FRP wrapping of all columns removed.
- Retrofitted frame retested at PGA of 0.2g or 0.3g.
- Pre-test analysis of response to 0.2g bidirectional motion w/ the modeling assumptions & evaluation criteria (including the RC-jackets) in EN1998-3.

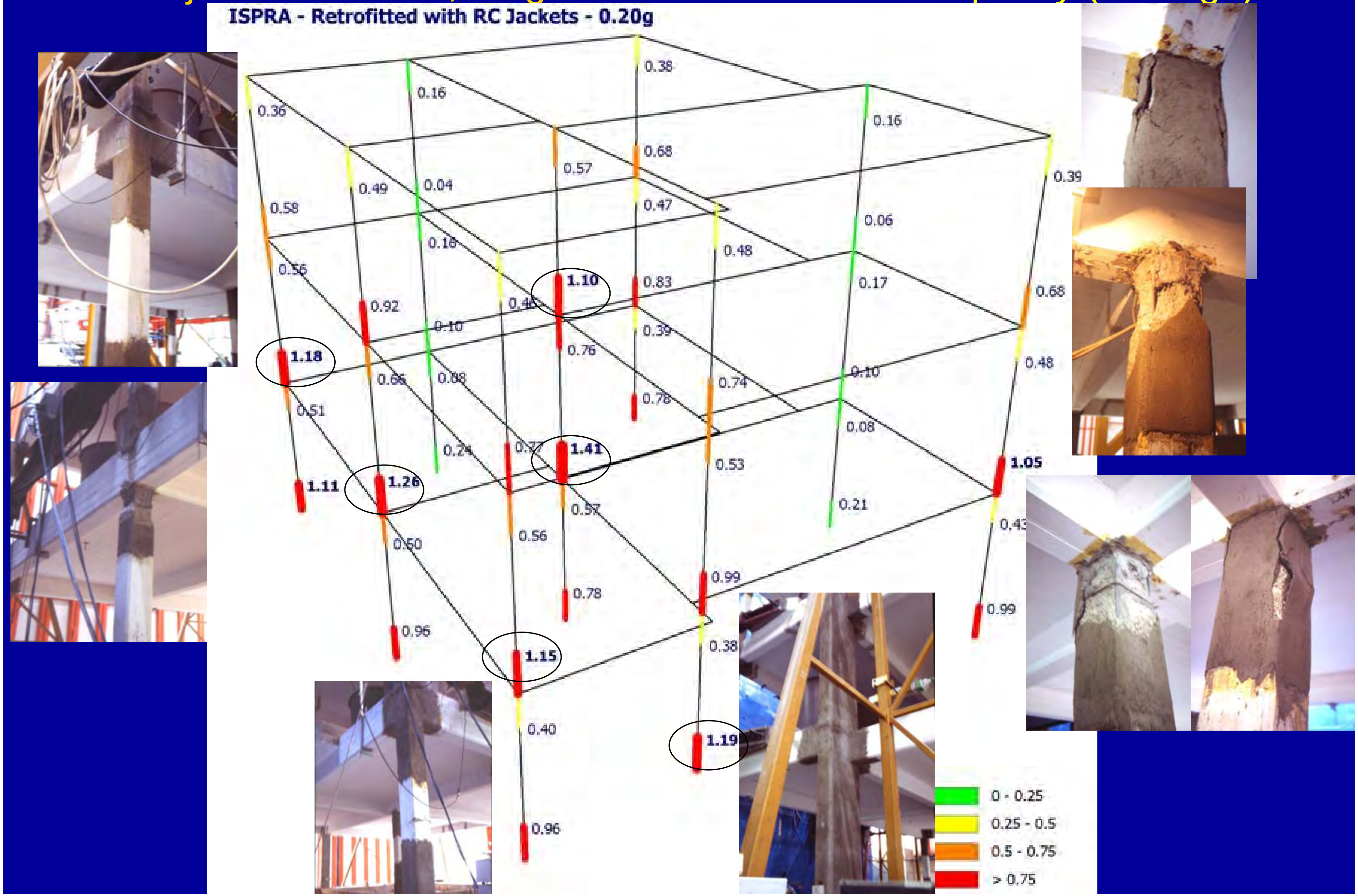


# Concrete-jacketed frame, 0.2g: t-histories of hor. displacement & twist at CM, floors 2 & 3 (continuous line: pre-test prediction; dotted line: test)

:post-ultimate strength degradation → real T ↑



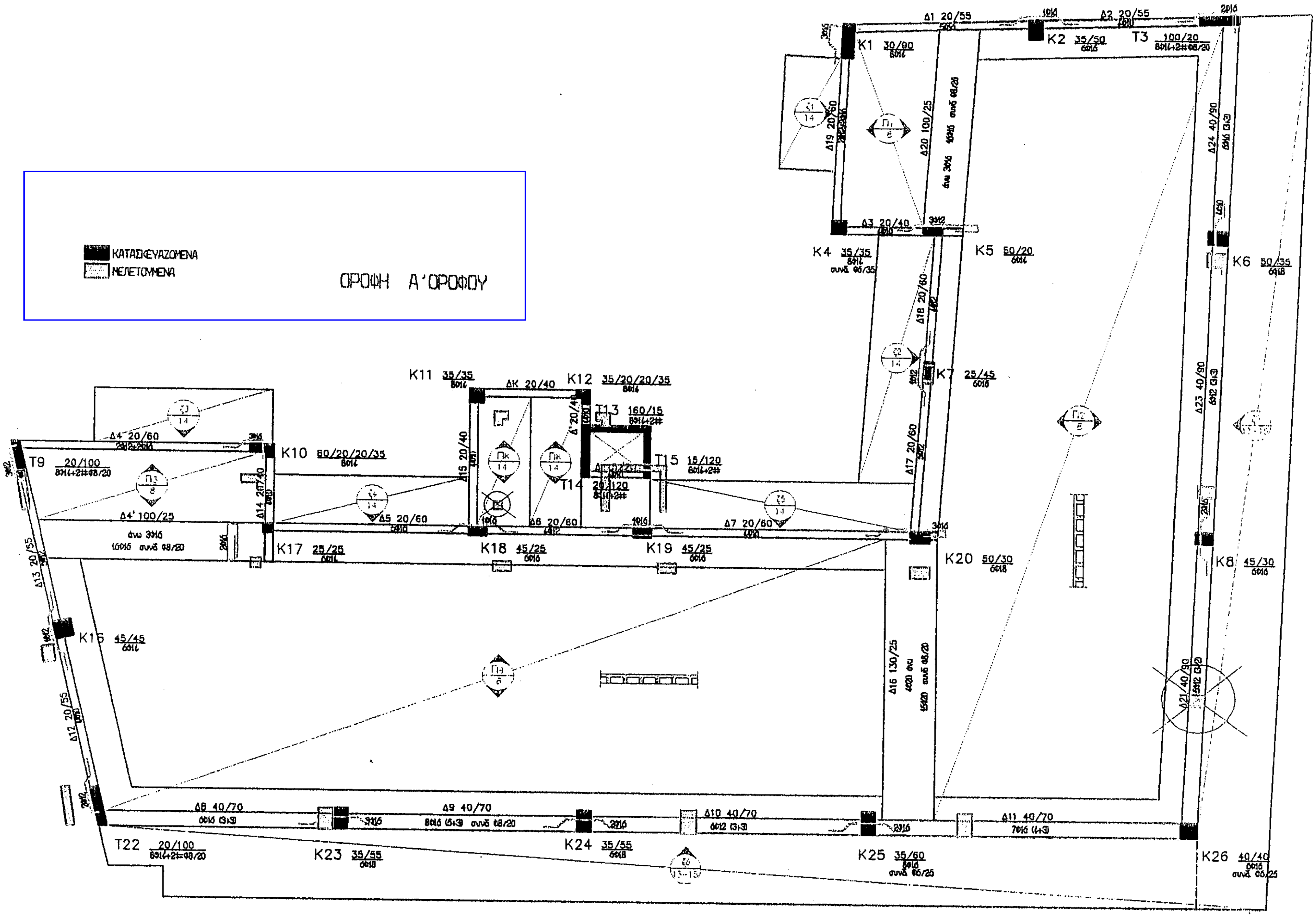
# Concrete-jacketed frame, 0.2g: member demand-to-capacity (damage) ratio



# Conclusions of Case Study on SPEAR test frame

- With the very simple RC member models and deformation properties given in Annex A of EC8-Part 3, displacement response history in 3D and extent & location of damage in <u>unretrofitted</u>, <u>FRP-retrofitted</u> and <u>RC-jacketed</u> test frame was predicted fairly well until ultimate deformation of most distressed member(s), despite complexities of the problem:
  - poor member detailing:
    - eccentric beam-column connections
    - lap- splicing of smooth/hooked bars;
  - bi-directional motion with evolutionary frequency content
    - (low-amplitude long-period component appeared in input at ~12sec, causing resonance);
  - strongly torsional response.

- 6-storey Athens building – Wing collapsed in 1999 earthquake
- Nonlinear dynamic analysis w/ “most likely” ground motions at site, to find collapse mechanism (by UPatras, Structures Lab)

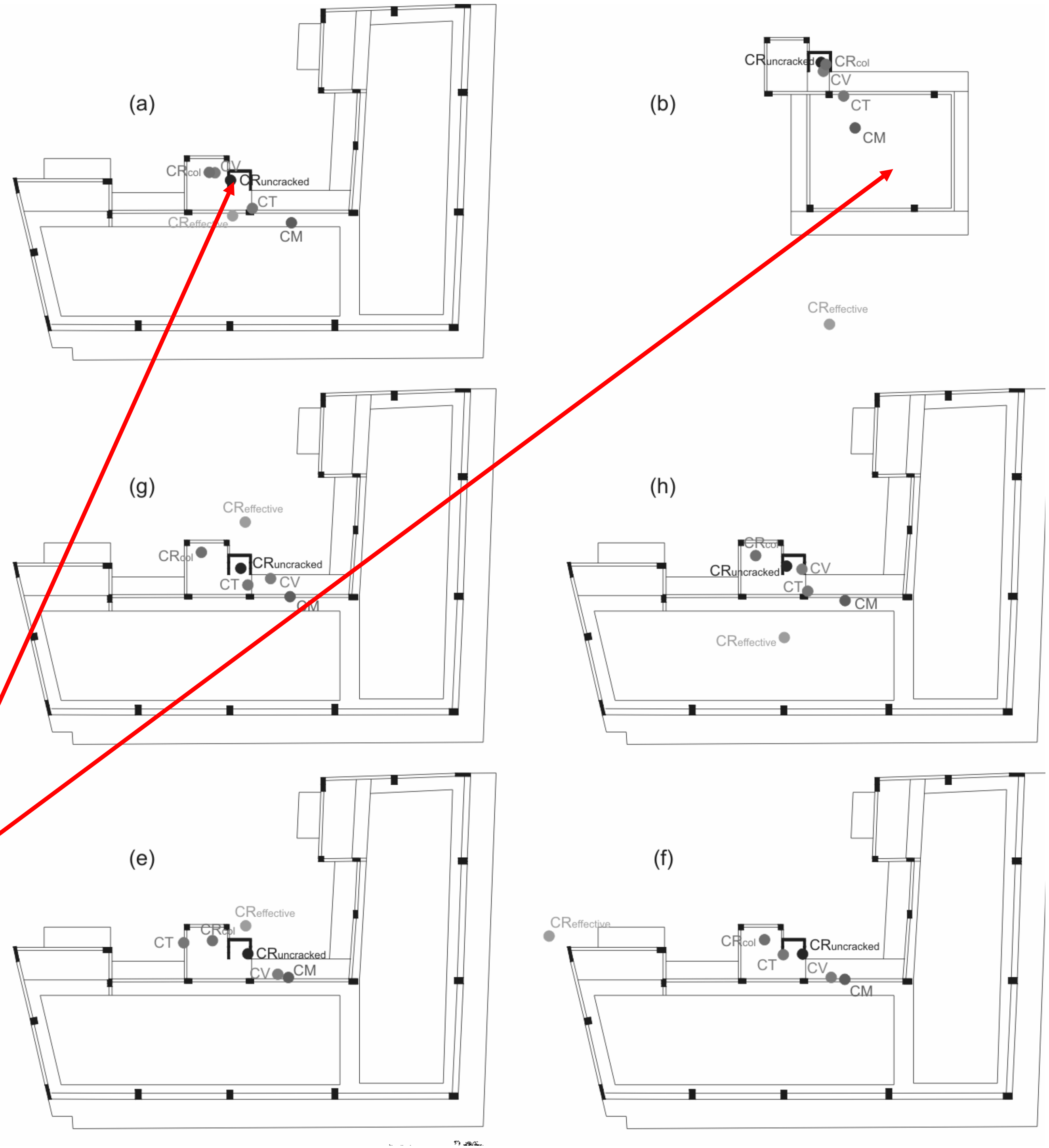


ΚΑΤΑΣΚΕΥΑΖΟΜΕΝΑ
ΜΕΛΕΤΟΥΜΕΝΑ
ΟΡΟΦΗ Α΄ΟΡΟΦΟΥ
Κ1 30/90
Κ2 35/50
Τ3 100/20
Κ4 35/35
Κ5 50/20
Κ6 50/35
Κ7 25/45
Κ8 45/30
Κ10 60/20/20/35
Κ11 35/35
Κ12 35/20/20/35
Τ13 160/15
Τ14 20/120
Τ15 15/120
Κ16 45/45
Κ17 25/25
Κ18 45/25
Κ19 45/25
Κ20 50/30
Τ22 20/100
Κ23 35/55
Κ24 35/55
Κ25 35/60
Κ26 40/40
Τ9 20/100
Δ1 20/55
Δ2 20/55
Δ3 20/40
Δ4 20/60
Δ4' 100/25
Δ5 20/60
Δ6 20/60
Δ7 20/60
Δ8 40/70
Δ9 40/70
Δ10 40/70
Δ11 40/70
Δ12 20/55
Δ13 20/55
Δ14 20/40
Δ15 20/40
Δ16 130/25
Δ17 20/60
Δ18 20/60
Δ19 20/60
Δ20 100/25
Δ21 40/90
Δ23 40/90
Δ24 40/90
ΔΚ 20/40

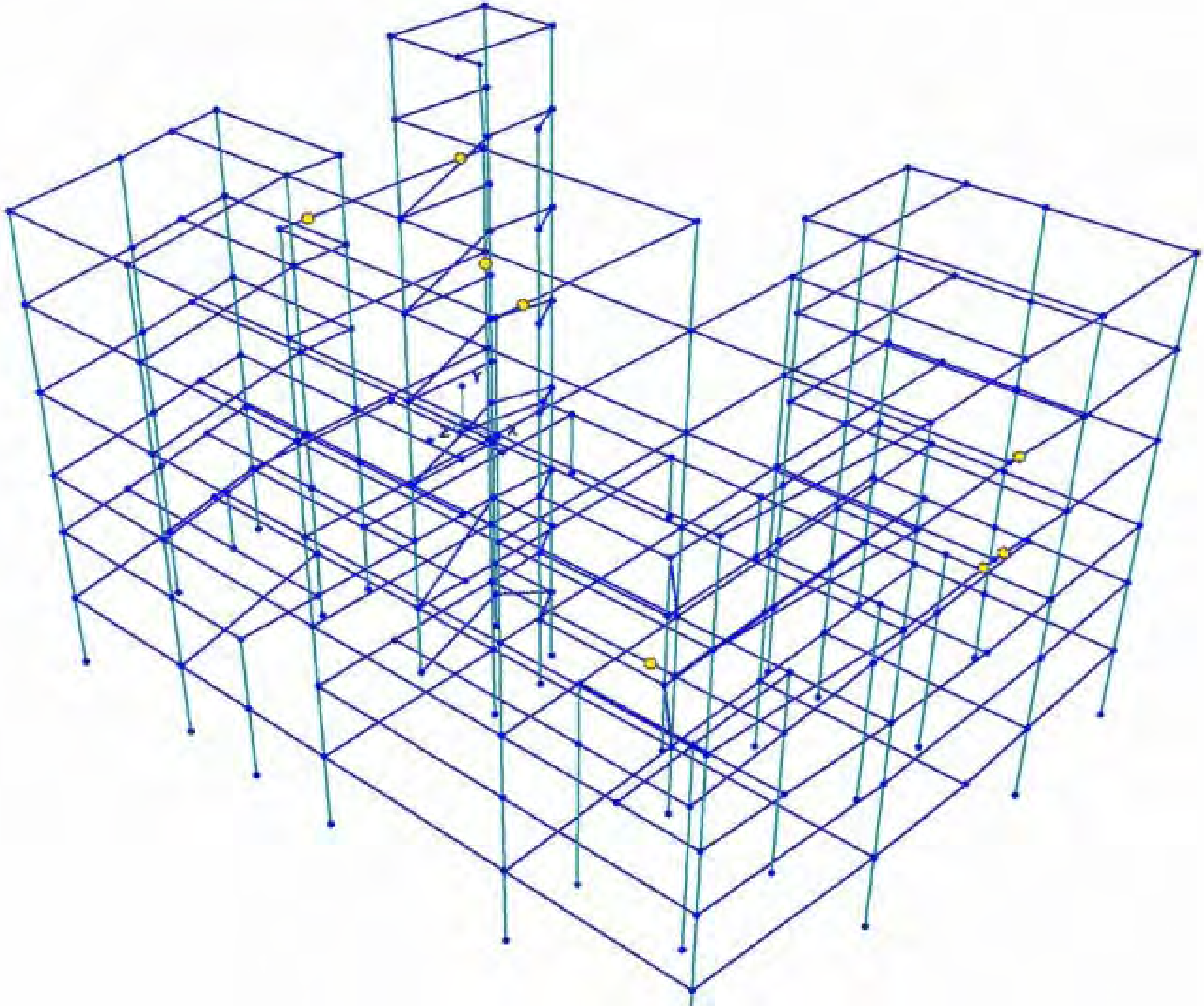
- Eccentricities between Centre of Mass (CM) & Centres of Rigidity (CR) or Strength (CV) or Twist (CT) in various storeys, induce torsional response.

  Higher modes are important.

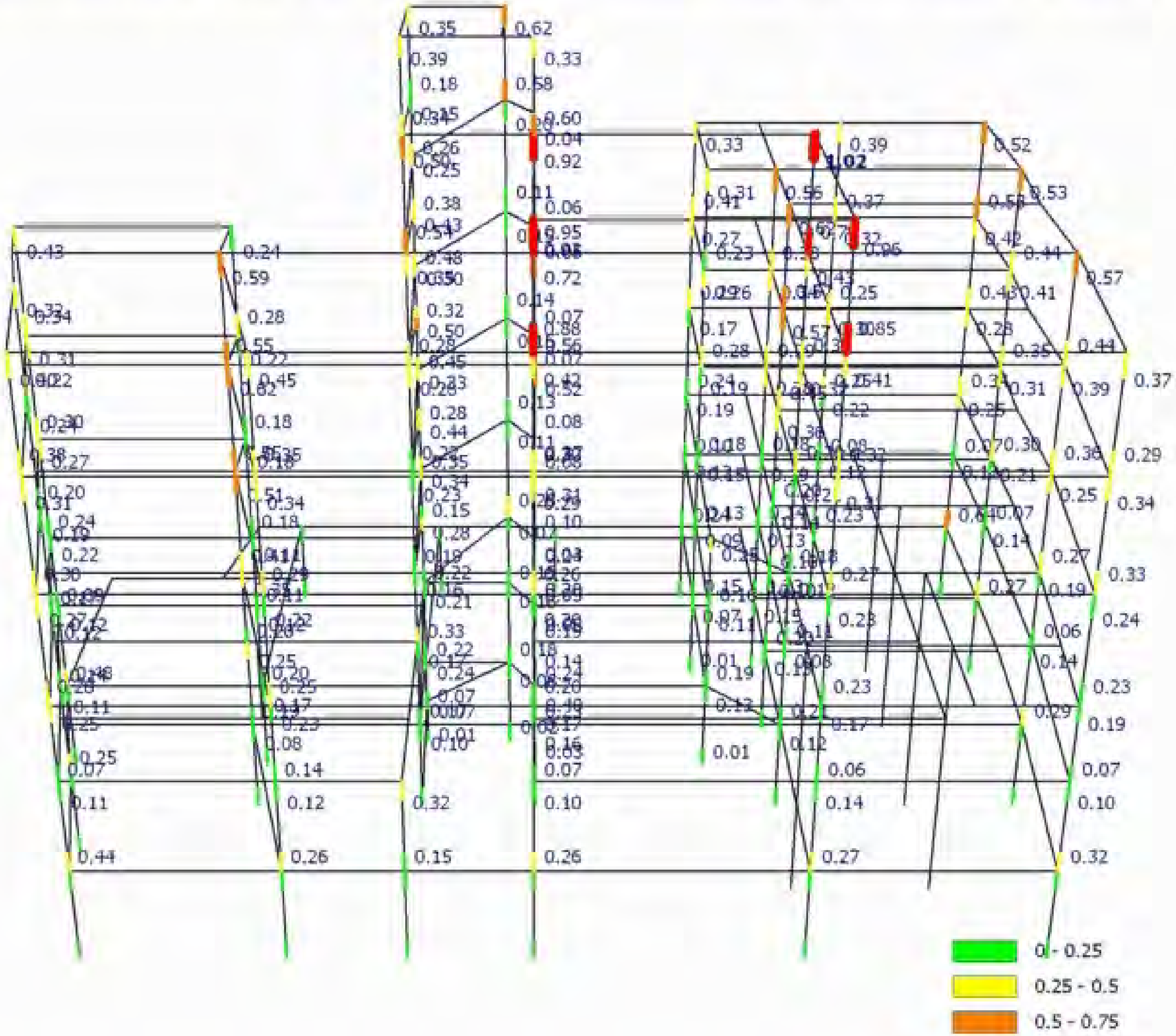
  Due to flexible diaphragms, elevator shaft & penthouse vibrate out of phase w.r.to the rest of the building



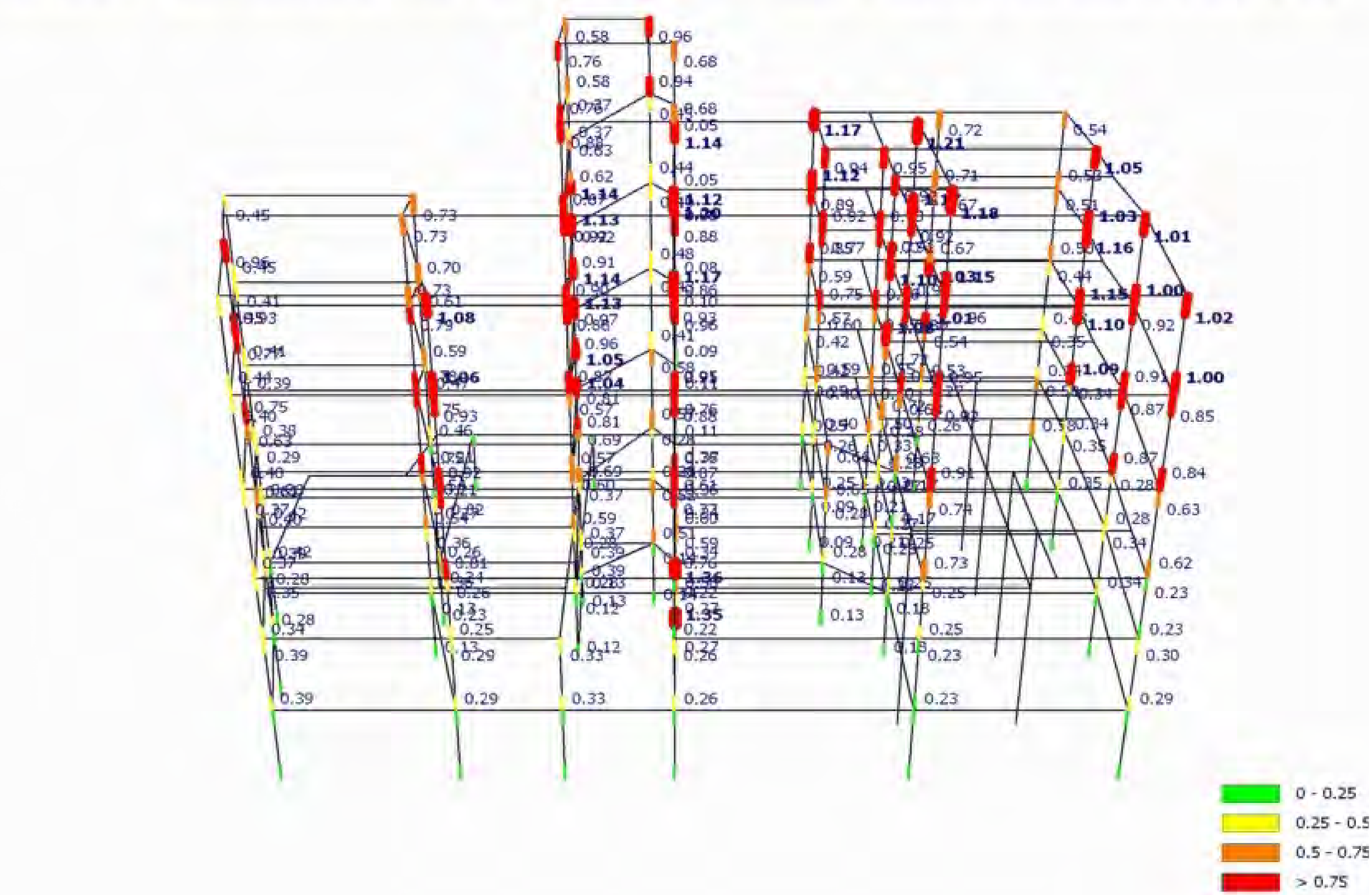
Yield Exceeded



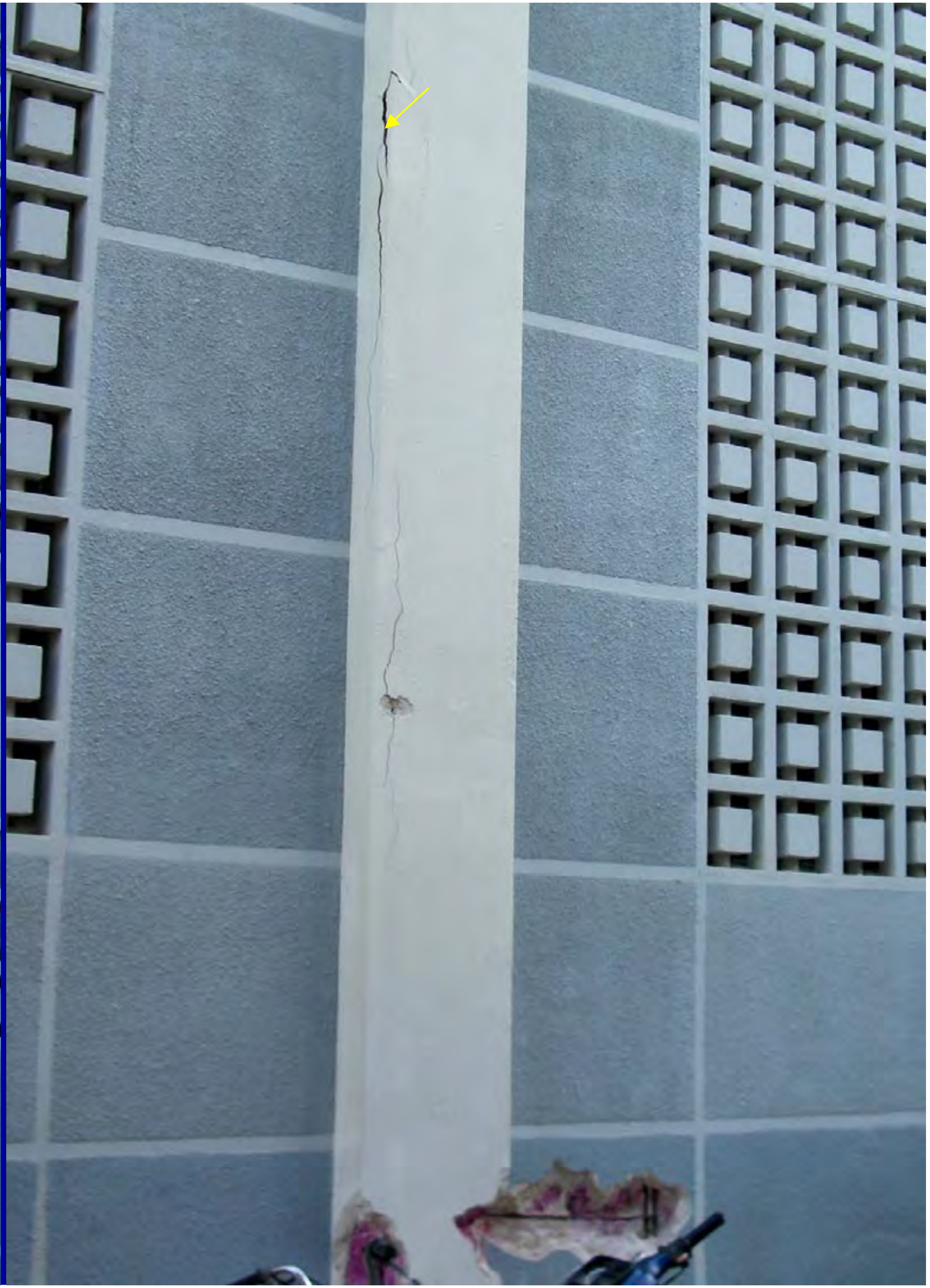
## ΚΑΤΑΣΚΕΥΗ - ΔΕΙΚΤΗΣ ΒΛΑΒΗΣ ΥΠΟΣΤΥΛΩΜΑΤΩΝ (ΚΑΜΨΗ - ΜΕΣΕΣ ΤΙΜΕΣ) ΑΠΟ ΑΝΑΛΥΣΕΙΣ ΧΡΟΝΟΪΣΤΟΡΙΑΣ



## ΚΑΤΑΣΚΕΥΗ - ΔΕΙΚΤΗΣ ΒΛΑΒΗΣ ΥΠΟΣΤΥΛΩΜΑΤΩΝ (ΔΙΑΤΜΗΣΗ - ΜΕΣΕΣ ΤΙΜΕΣ) ΑΠΟ ΑΝΑΛΥΣΕΙΣ ΧΡΟΝΟΪΣΤΟΡΙΑΣ

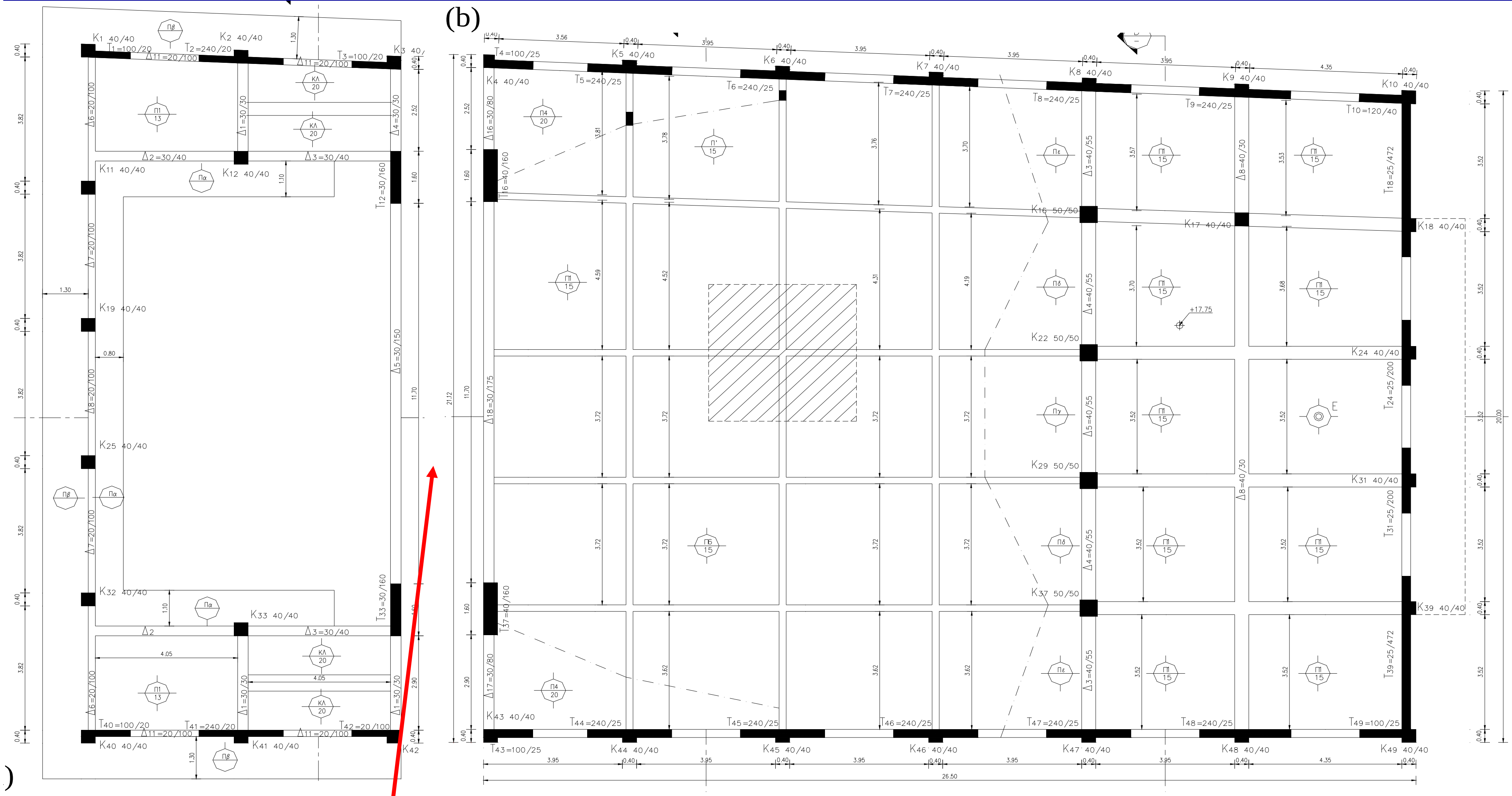


# SEISMIC ASSESSEMENT & RETROFITTING OF "KEFALOS" THEATRE IN CEPHALONIA ACCORDING TO EC 8 (by UPatras, Structures Lab)



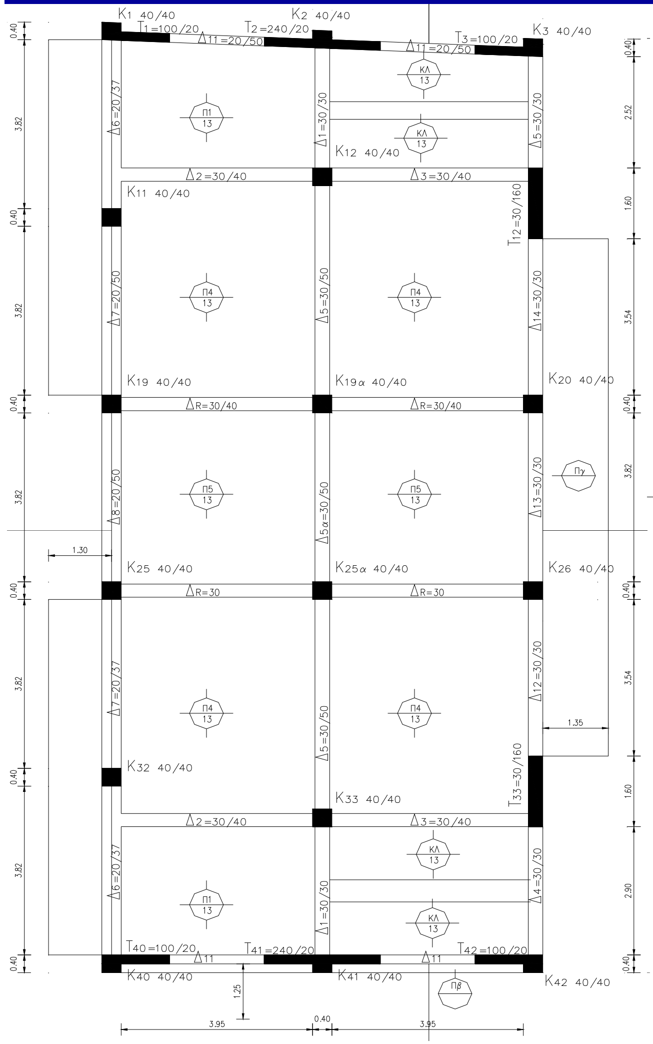


# Framing plan: Roof level

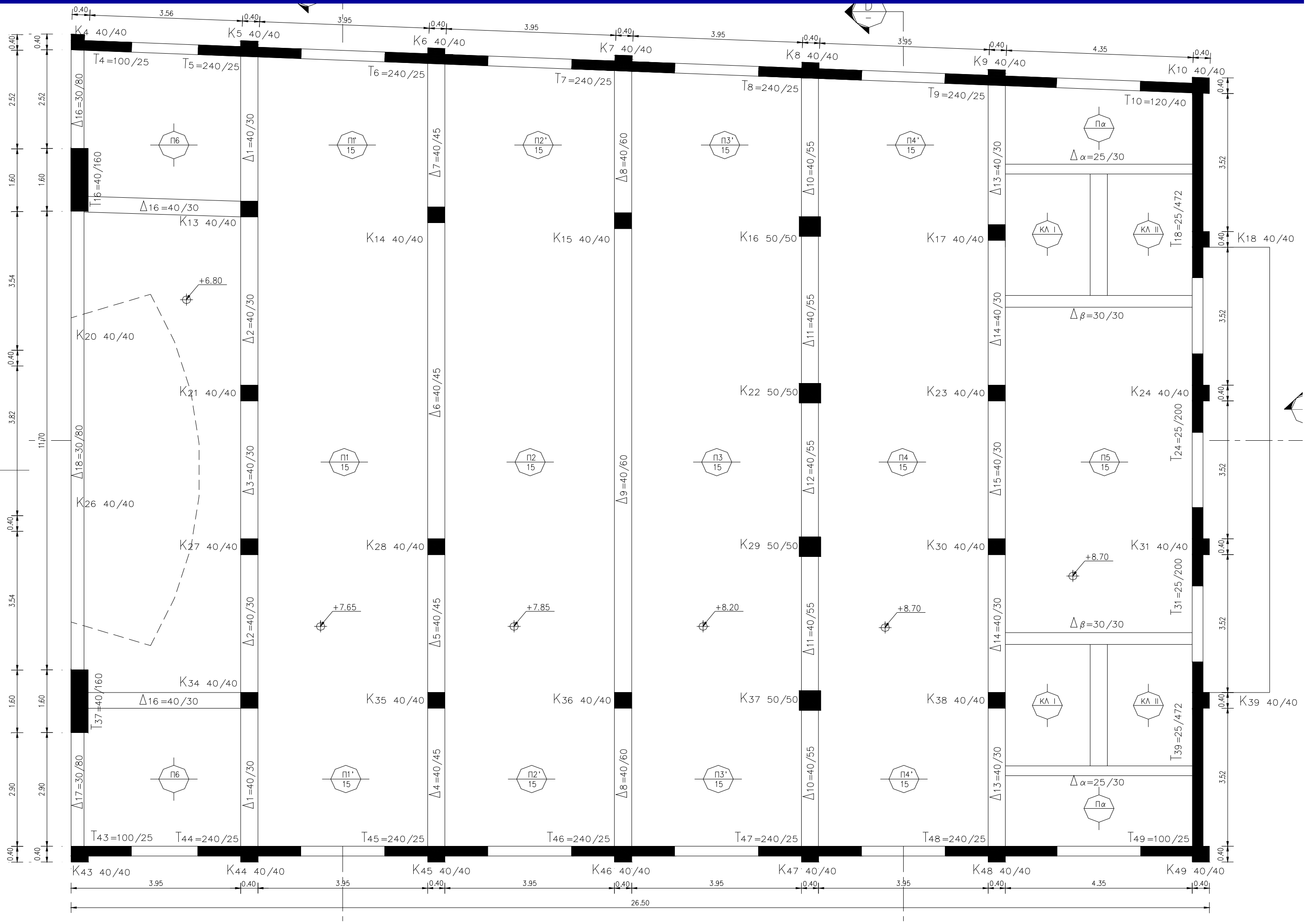


Expansion joint separates building to two independent parts ("Stage" & "Theatre"), both very irregular in plan and elevation

# Framing plan: Ground floor

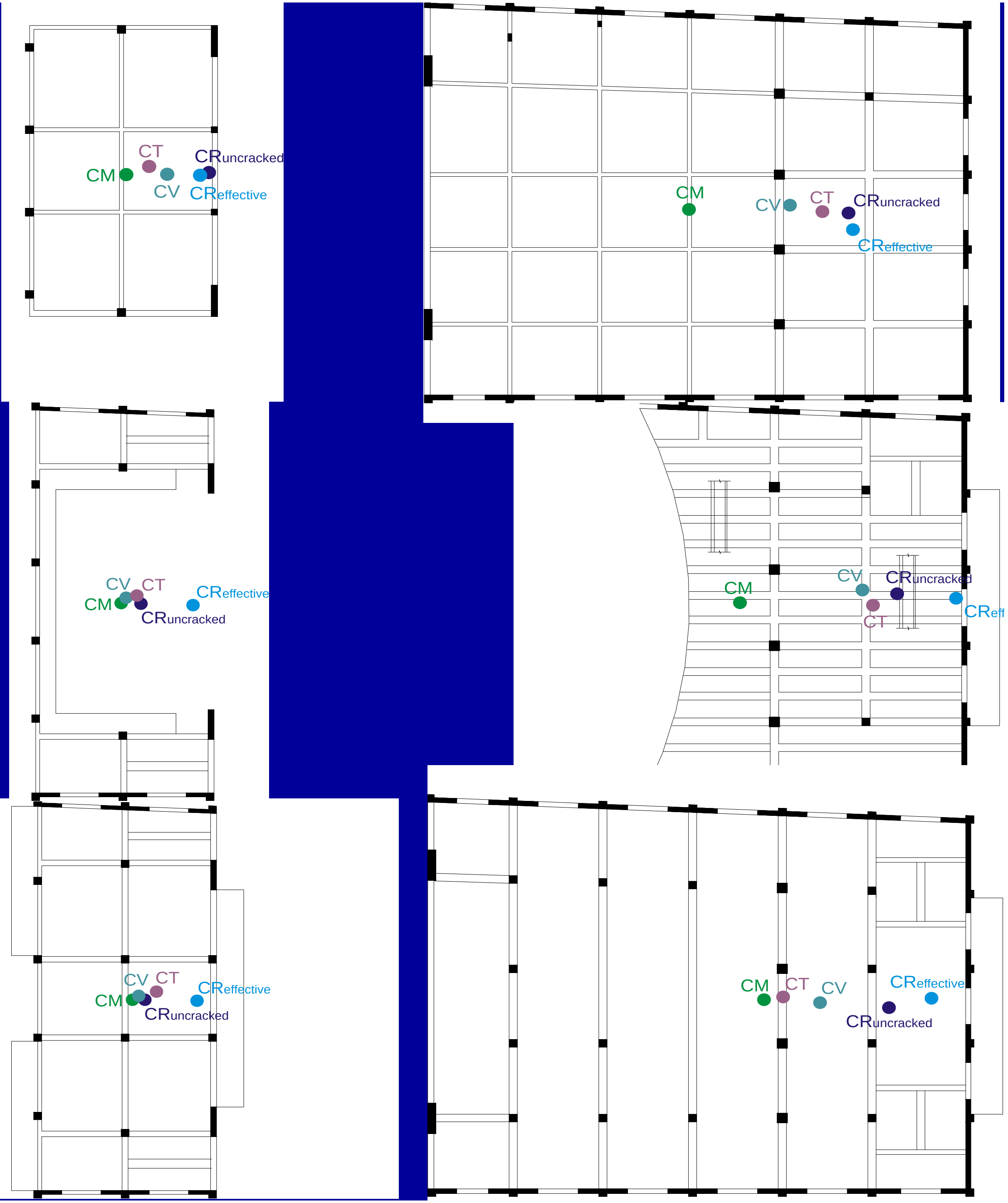


(c)

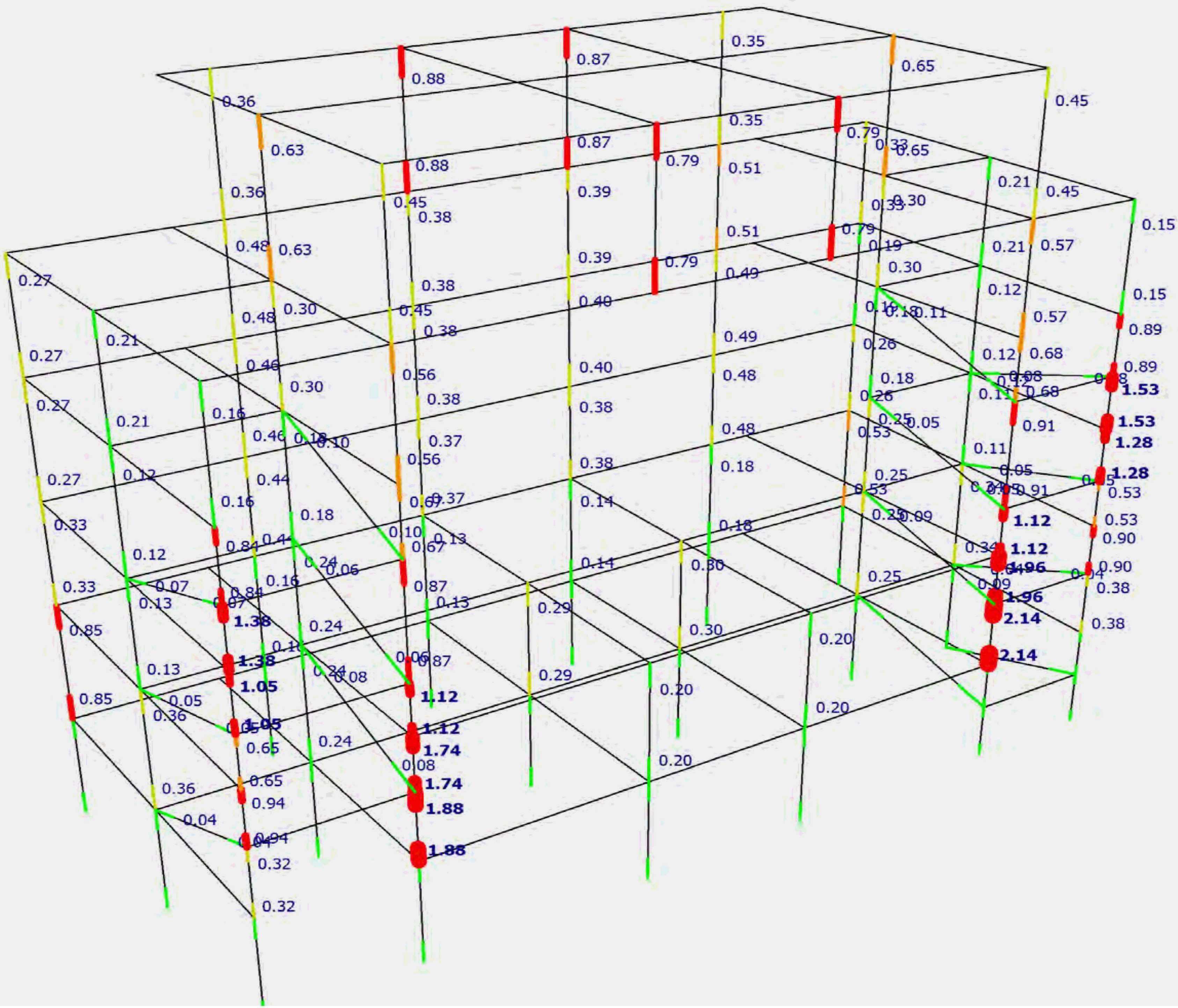


(d)

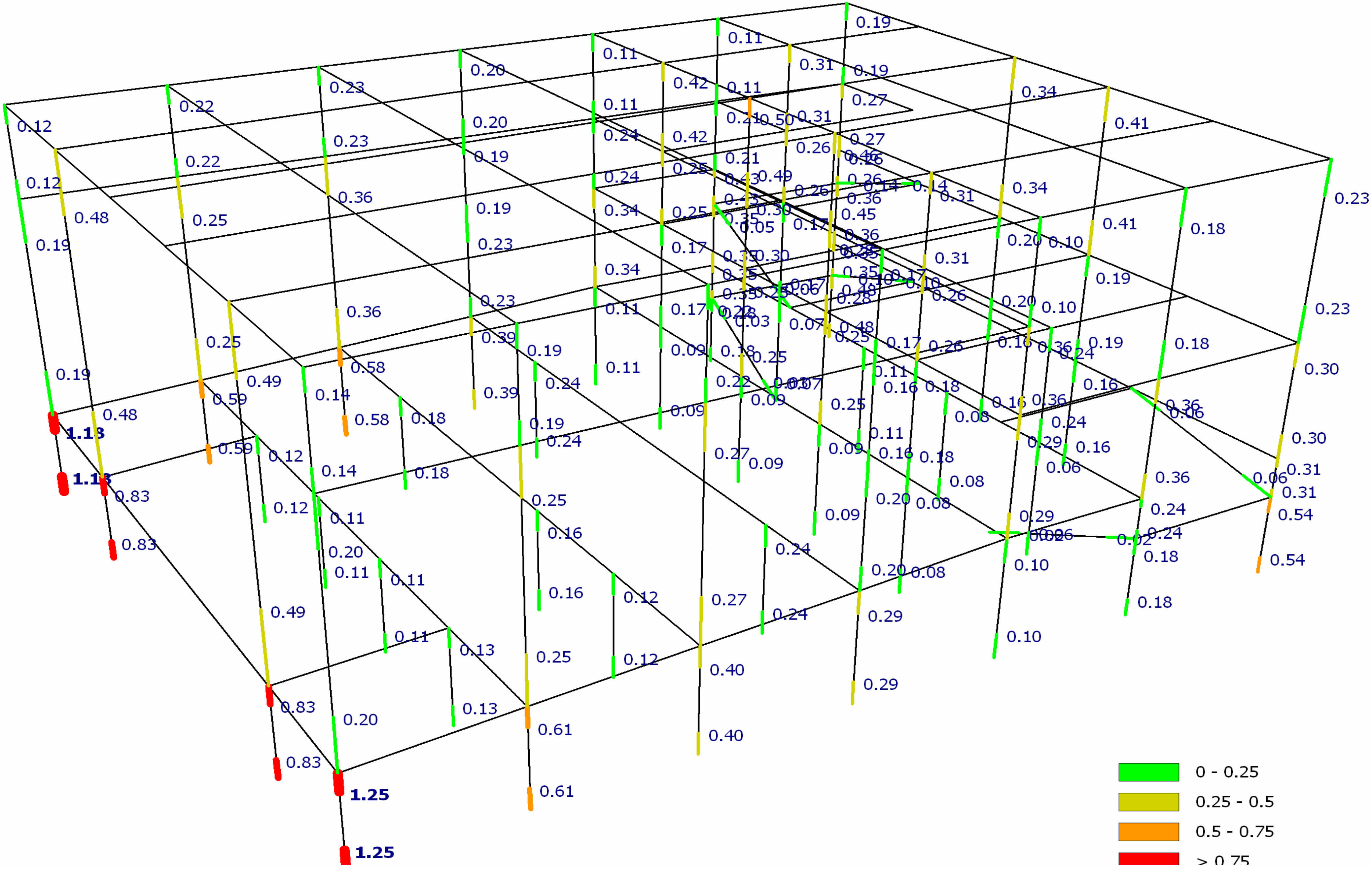
Eccentricities between Centre of Mass (CM) & Centres of Rigidity (CR) or Strength (CV) Twist (CT) in both parts of the building, induce torsional response & pounding of the two parts at the expansion joint



Demand-capacity ratios in **<u>shear</u>** of "Stage" part; mean values from nonlinear analyses under 56 bidirectional ground motions conforming to EC8 Soil C spectrum at **<u>PGA=0.1g</u>**



Demand-capacity ratios in **shear** of “Theatre” part; mean values from nonlinear analyses under 56 bidirectional ground motions conforming to EC8 Soil C spectrum at **PGA=0.1g**

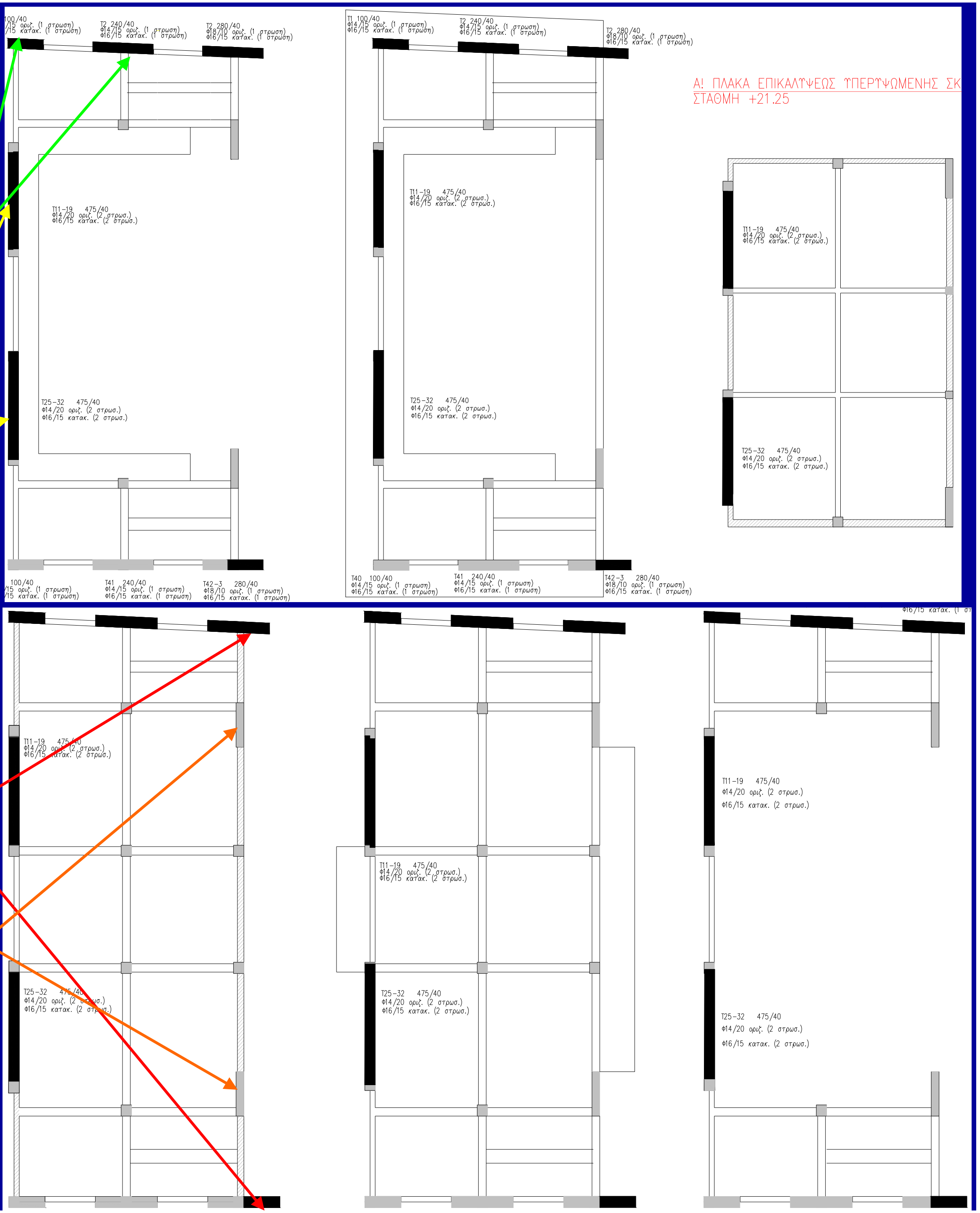


# Strengthening of “Stage” part

1. RC-jacketing of perimeter walls (also due to bar corrosion).

2. Two bays infilled w/ new RC walls, from foundation to rooftop.

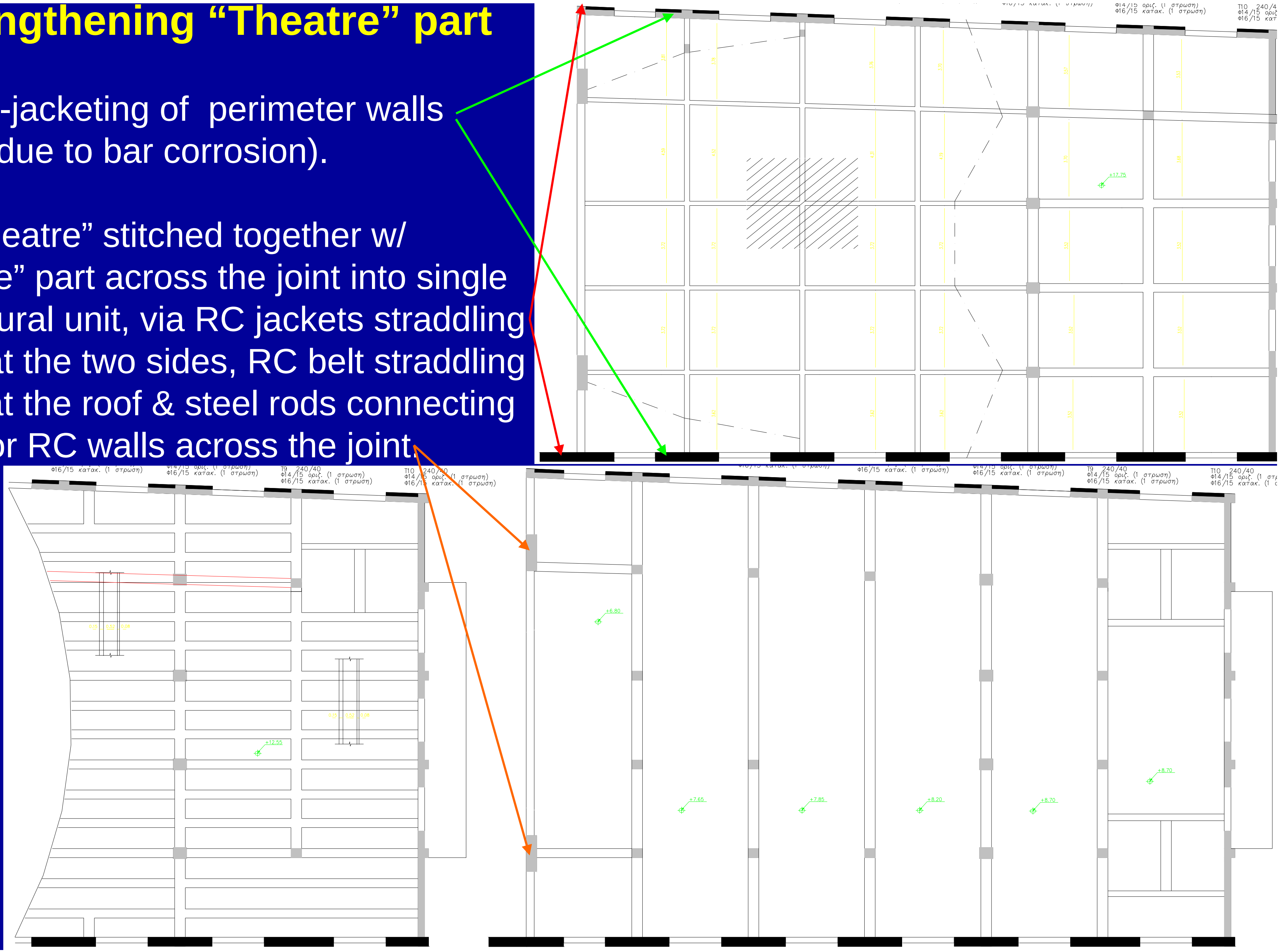
3. “Stage” stitched together w/ “Theatre” part across the joint into single structural unit (to eliminate torsional response & pounding) via RC jackets straddling joint at the two sides, RC belt straddling joint at the roof & steel rods connecting interior RC walls across the joint.

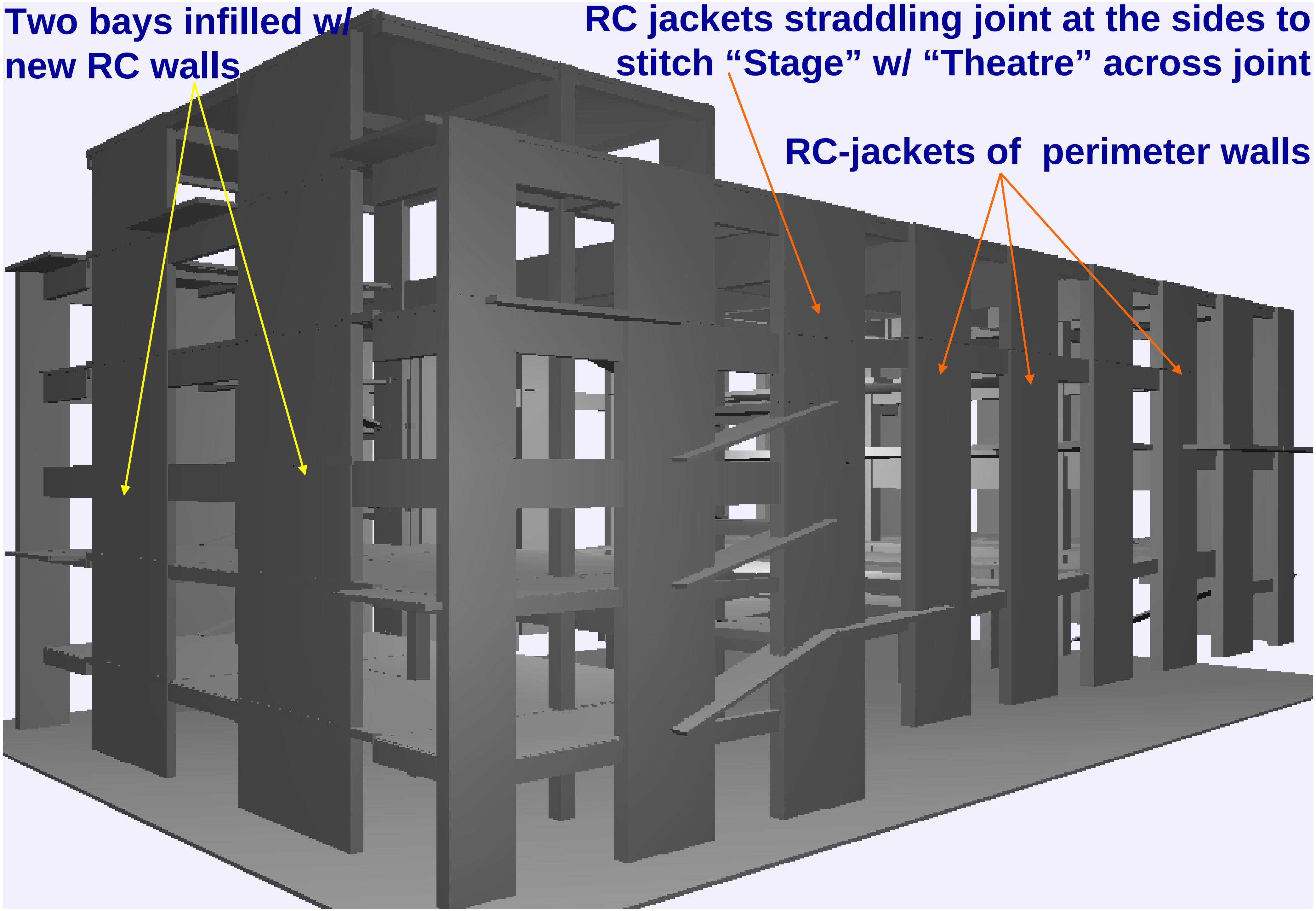


# Strengthening "Theatre" part

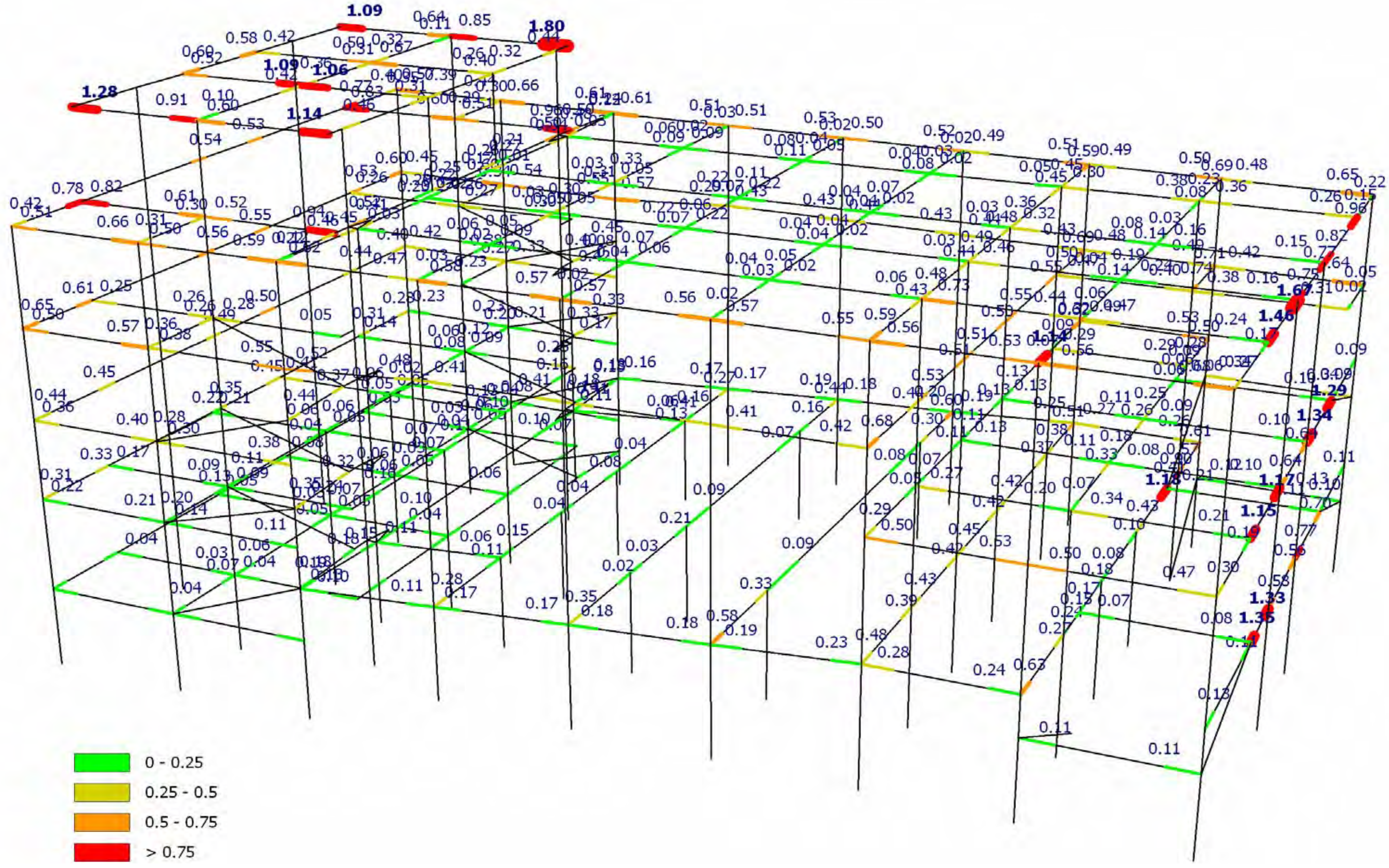
1. RC-jacketing of perimeter walls (also due to bar corrosion).

2. "Theatre" stitched together w/ "Stage" part across the joint into single structural unit, via RC jackets straddling joint at the two sides, RC belt straddling joint at the roof & steel rods connecting interior RC walls across the joint.

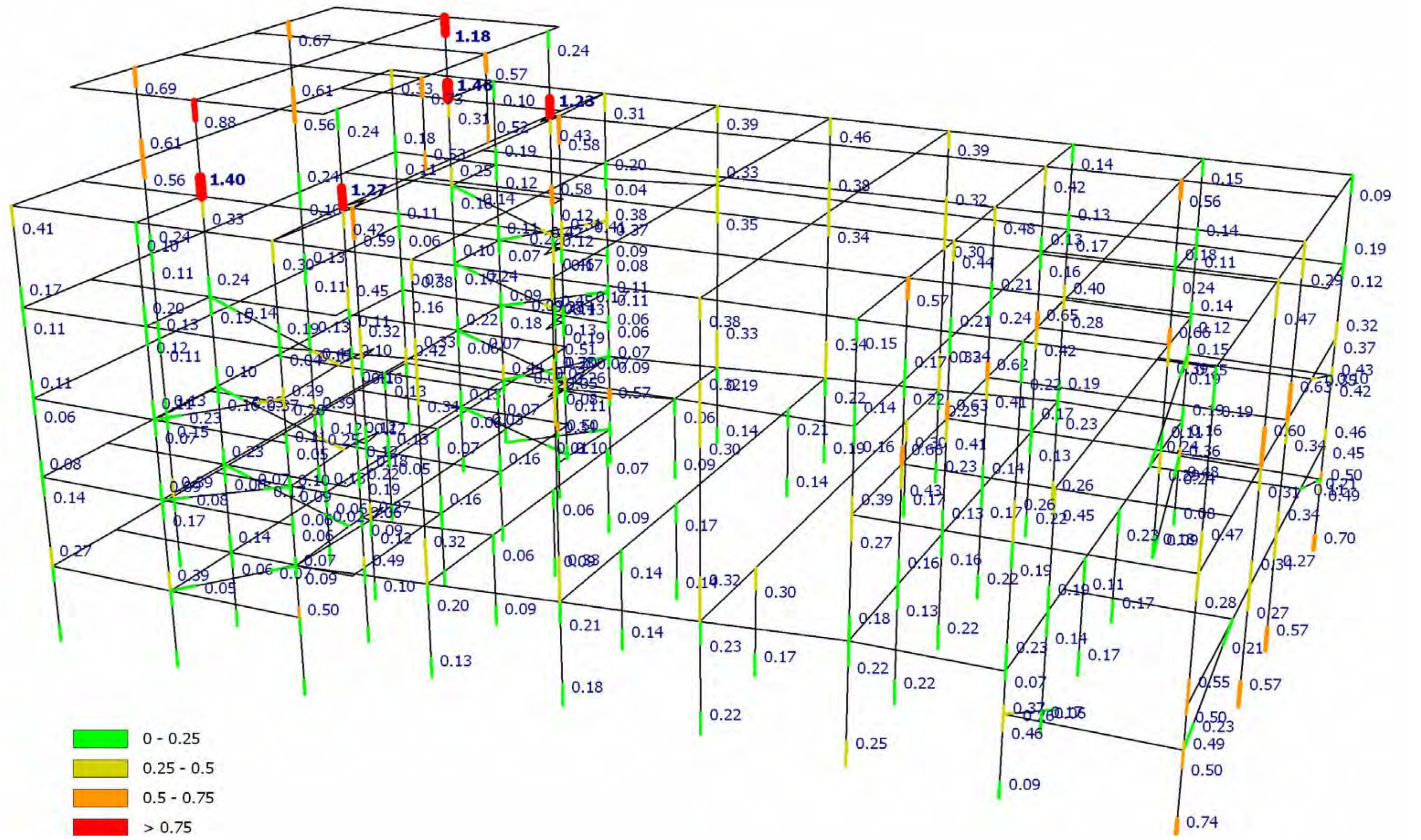



Two bays infilled w/ new RC walls
RC jackets straddling joint at the sides to stitch “Stage” w/ “Theatre” across joint
RC-jackets of perimeter walls

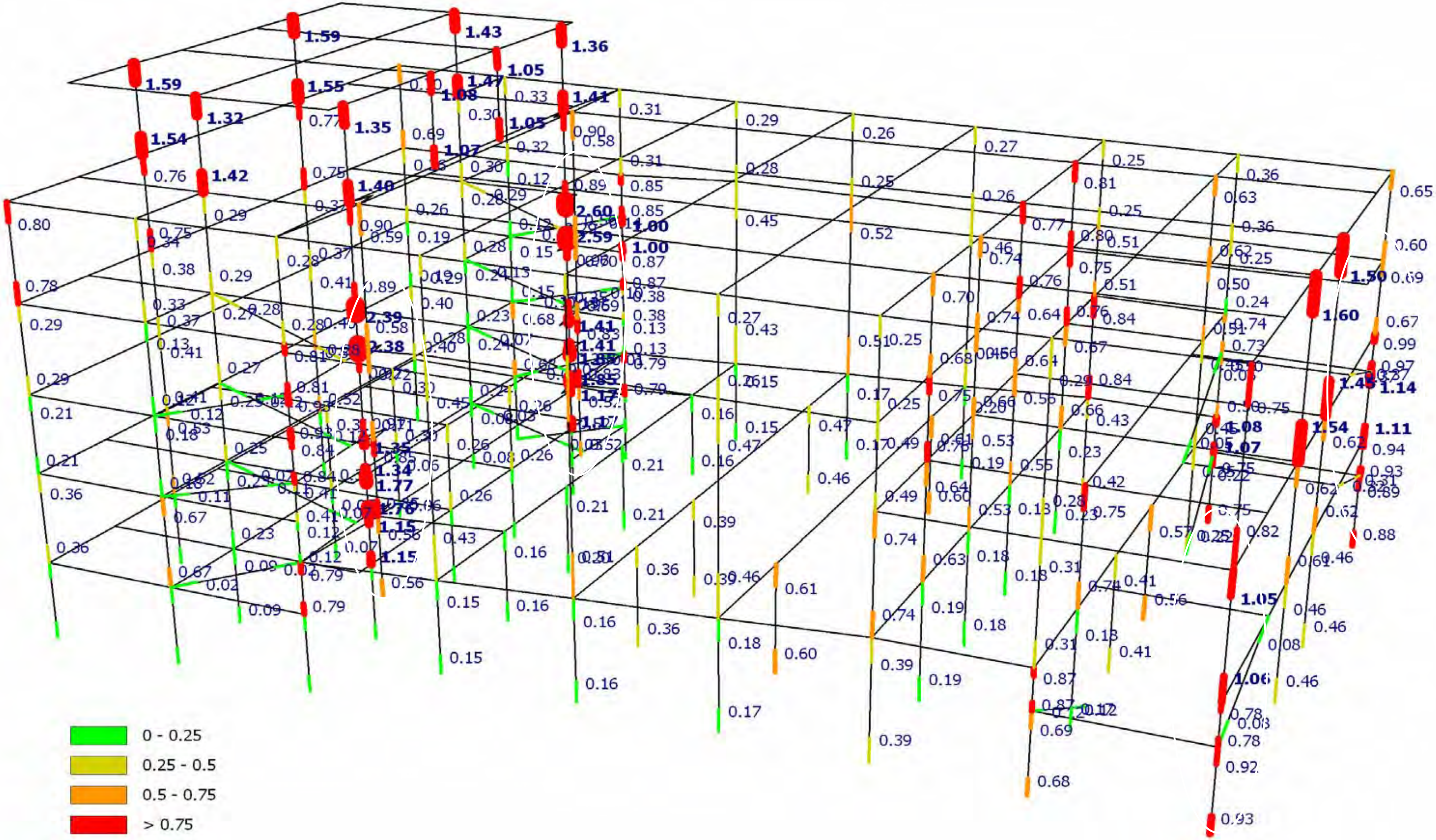
Demand-capacity ratios in **flexure** at the Near Collapse Limit State: **beams** of building **strengthened only by means of RC.** Mean values from nonlinear analyses for 56 bidirectional ground motions conforming to EC8 Soil C spectrum at **PGA=0.36g**



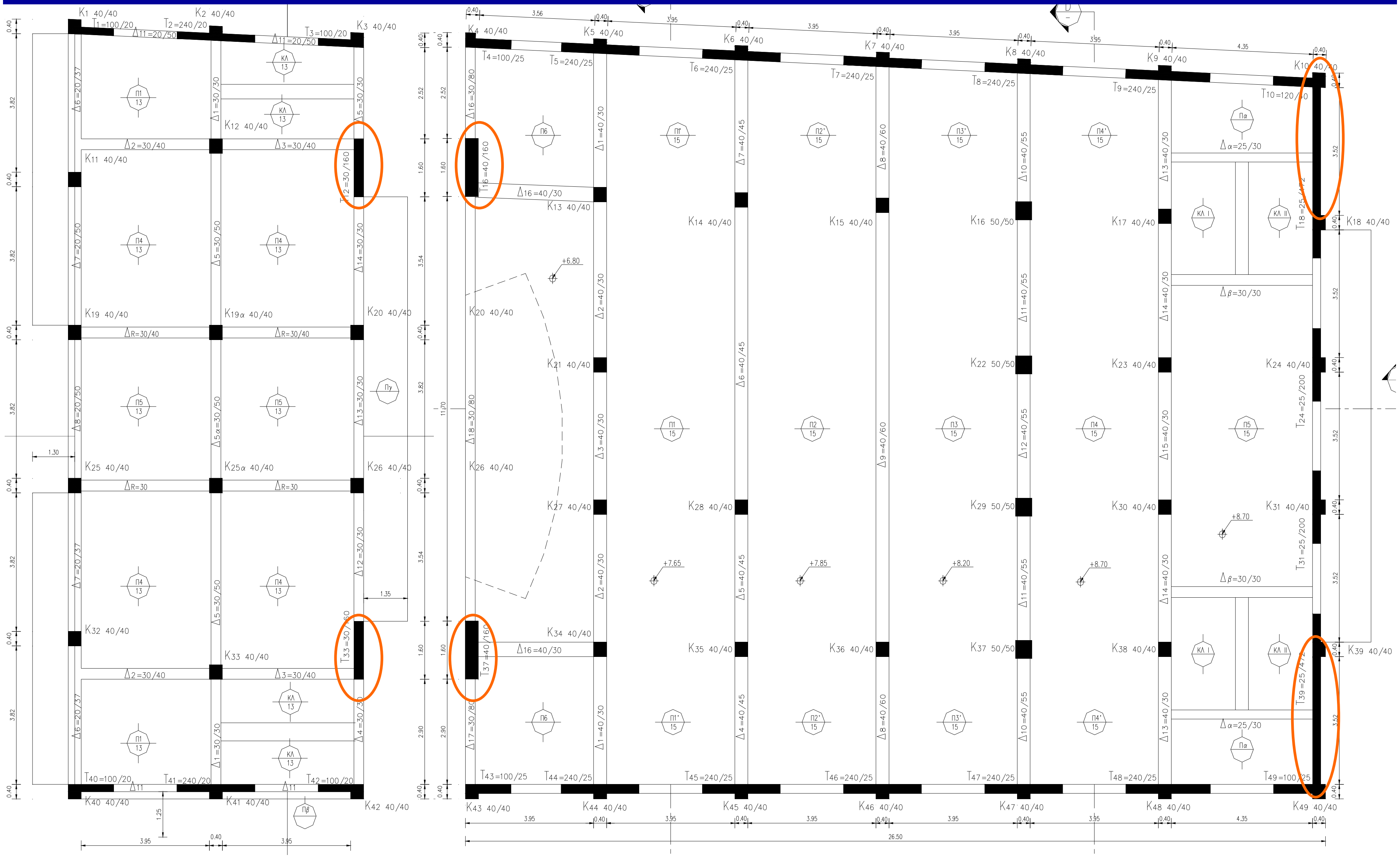
Demand-capacity ratios in **flexure** at the Near Collapse Limit State: **vertical members** of building **strengthened only by means of RC.** Mean values from nonlinear analyses for 56 bidirectional ground motions conforming to EC8 Soil C spectrum at **PGA=0.36g**



Demand-capacity ratios in **shear** - **vertical members** of building **strengthened only by means of RC** (before FRP strengthening). Mean values from nonlinear analyses for 56 bidirectional ground motions conforming to EC8 Soil C spectrum at **PGA = 0.36g**

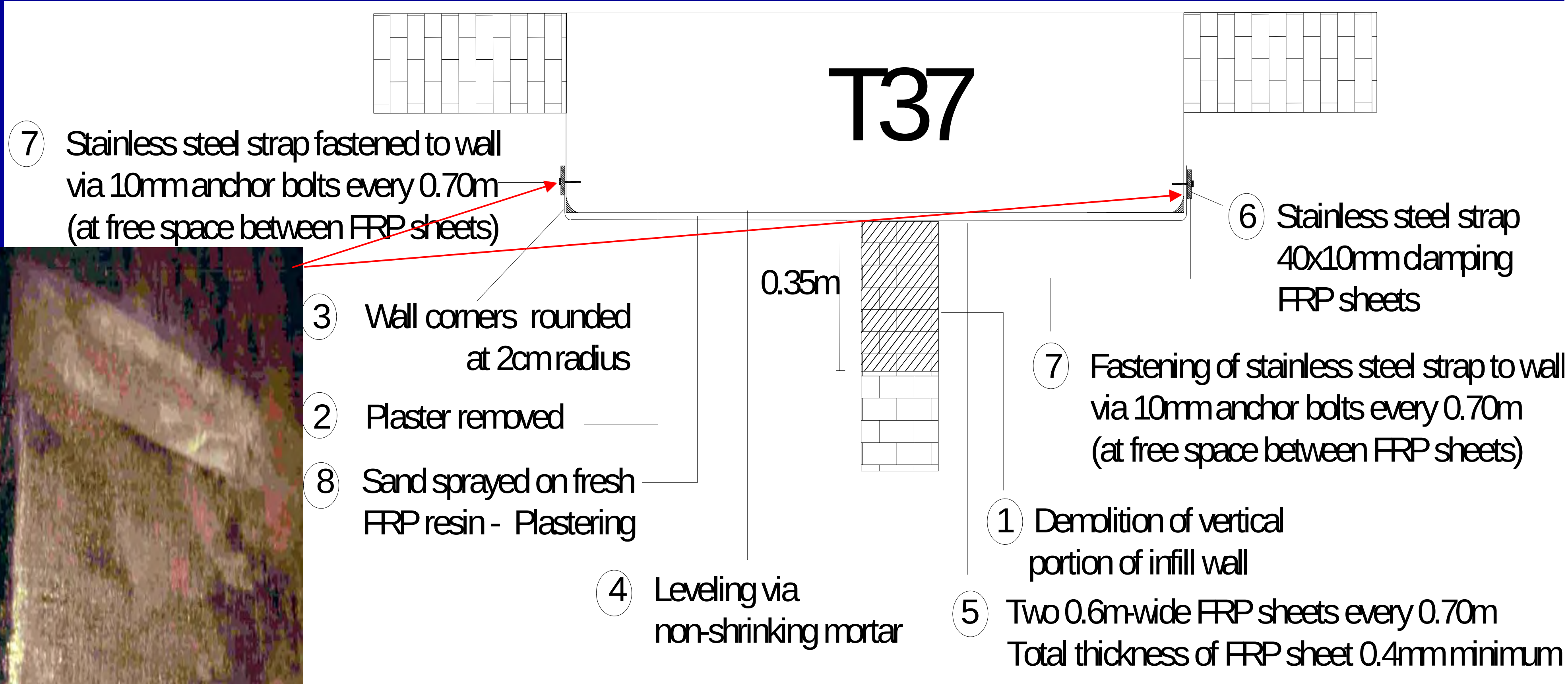


# Strengthening by one-sided Carbon FRP all shear-deficient walls



(c) (d)

# 1.6m-wide interior walls strengthened in shear w/ one-sided CFRP. Total thickness of Carbon fibre sheets: 0.4-0.5mm




# 3.5m-wide façade walls strengthened in shear w/ one-sided CFRP. Total thickness of Carbon fibre sheets: 0.4mm