Technical Chamber of Greece – Structural Eurocodes meeting

Development and basic aspects of EN 1992 and EN 1998

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EN 1992 : 2003
Eurocode 2: Design of concrete structures

GENERAL STRUCTURE OF EUROCODE 2

EN 1992-1-1	GENERAL RULES AND RULES FOR BUILDINGS
EN 1992-1-2	FIRE DESIGN
EN 1992-2	DESIGN ON CONCRETE BRIDGES
EN 1992-3	SILOS AND TANKS

Content of EN 1992-1-1

EN 1992-1-1 : Content (cont'd)

5. Structural analysis

- 5.1 General
- 5.2 Geometric imperfections
- 5.3 Idealisation of the structure
- 5.4 Linear elastic analysis
- 5.5 Linear analysis with limited redistribution
- 5.6 Plastic analysis
- 5.7 Non-linear analysis
- 5.8 Second order effects with axial load
- 5.9 Lateral instability of slender beams
- 5.10 Prestressed members and structures
- 5.11 Analysis for some particular structural members

6. Ultimate limit states (ULS)

- 6.1 Bending with or without axial force
- 6.2 Shear
- 6.3 Torsion
- 6.4 Punching
- 6.5 Design with strut and tie models

EN 1992-1-1 : Content (cont'd)

- 6.6 Anchorages and laps
- 6.7 Partially loaded areas
- 6.8 Fatigue

7. Serviceability limit states (SLS)

- 7.1 General
- 7.2 Stress limitation
- 7.3 Crack control
- 7.4 Deflection control

8 Detailing of reinforcement and prestressing tendons - General

- 8.1 General
- 8.2 Spacing of bars
- 8.3 Permissible mandrel diameters for bent bars
- 8.4 Anchorage of longitudinal reinforcement
- 8.5 Anchorage of links and shear reinforcement
- 8.6 Anchorage by welded bars
- 8.7 Laps and mechanical couplers
- 8.8 Additional rules for large diameter bars
- 8.9 Bundled bars
- 8.10 Prestressing tendons

EN 1992-1-1 : Content (cont'd)

EN 1992-1-1 : Content (cont'd)

Annexes

A (Informative)
Modification of partial factors for materials

B (Informative)
Creep and shrinkage strain

C (Normative)
Reinforcement properties

D (Informative)
Detailed calculation method for prestressing steel relaxation losses

E (Informative)
Indicative Strength Classes for durability

F (Informative)
Reinforcement expressions for in-plane stress conditions

G (Informative)
Soil structure interaction

H (Informative)
Global second order effects in structures

I (Informative)
Analysis of flat slabs and shear walls

J (Informative)
Examples of regions with discontinuity in geometry or action

EN 1992-1-1: General Overview

- Comparative studies show that the overall economy of construction of designs to EC2 are largely similar to those currently designed using actual national design standards
- There is little practical difference in results of design for bending
- The style of the Eurocodes and the way they are implemented are appreciably different, and there are some significant changes in aspects of the design process.
- There are associated changes arising through related Eurocodes and product standards. These make a suite of documents, including:
  - EN206 *Concrete: Performance, Production, Placing and Compliance Criteria,* and
  - EN13670 *Execution of Concrete Structures*

FUNDAMENTAL REQUIREMENTS

- SAFETY (STRUCTURAL RESISTANCE)
- SERVICEABILITY
- DURABILITY
  - Design working life
  - Inspection and maintenance levels
- *ECONOMY*
- *AESTHETICS*

Verification of safety and serviceabilty by the partial factor method for :

ULTIMATE LIMIT STATES	ULS
SERVICEABILITY LIMIT STATES	SLS

Basis of design – partial safety factors

Action	Comment	Symbol	Value
Shrinkage		γ_{SH}	1.00
Prestress	Favourable effect	$\gamma_{P,fav}$	1,00
	ULS with external prestressing	$\gamma_{P,unfav}$	1,30
	Unfavourable local effects	$\gamma_{P,unfav}$	1,20
Fatigue loads		$\gamma_{F,fat}$	1,00
Materials	**Comment**	**Symbol**	**Value**
Concrete	Persistent and transient design situations	γ_C	1.5
	Accidental design situation		1.2
Steel (reinforcement)	Persistent and transient design situations	γ_S	1.15
	Accidental design situation		1,00
Steel (prestressing)	Persistent and transient design situations	γ_S	1.15
	Accidental design situation		1,00

Structural Analysis

- Linear elastic analysis (ULS-SLS)
- Linear analysis with limited redistribution (ULS)
- Plastic analysis (ULS)
- Non-linear analysis (ULS-SLS)

Design value of prestressing forces

$$P_d = \gamma_P P_{m,t} \qquad \gamma_P = 1$$

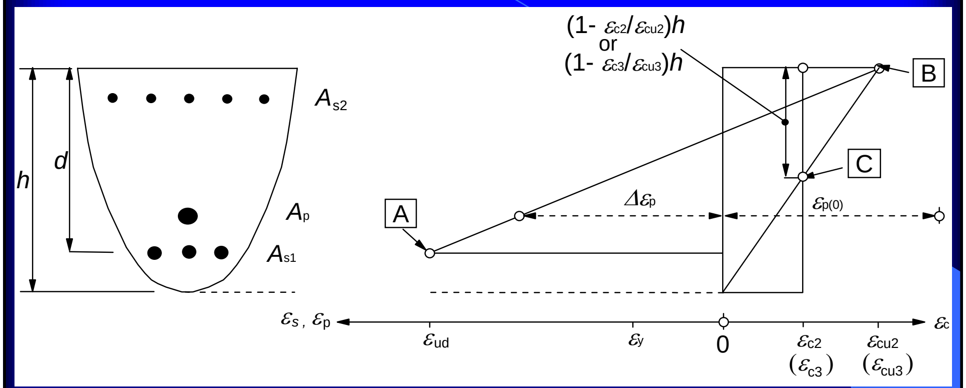
$P_{m,t}$ mean value at time t.

Ultimate Limit States : bending with or without axial force

Assumptions

- Plane sections remain plane
- Tensile strength of concrete ignored
- No relative slip between concrete and steel
- Possible strain distributions in cross-sections

Possible strain distributions in the Ultimate Limit State



A - Reinforcing steel tension strain limit

B - Concrete compression strain limit

C - Concrete pure compression strain limit

Ultimate limit state – Shear

$V_{Rd,c}$ Design shear resistance of the member without shear reinforcement

$V_{Rd,s}$ Design value of the shear force which can be sustained by the yielding shear reinforcement

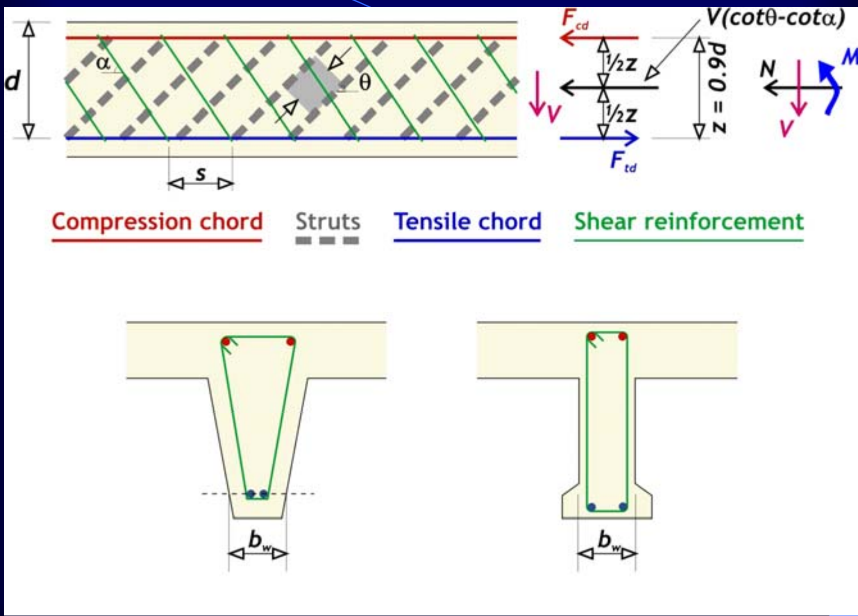
$V_{Rd,max}$ Design value of the maximum shear force which can be sustained by the member limited by crushing of the compression struts

General verification procedure : $V_{Ed} \leq V_{Rd}$

1) $V_{Ed} \leq V_{Rd,c}$

2) $V_{Ed} \leq V_{Rd,s}$ and $V_{Ed} \leq V_{Rd,max}$

Truss Model & Notation for Shear Reinforced Members



Ultimate limit state – Shear (contnd)

$V_{Rd,c}$ = [(0,18/γ_c)k(100 $\rho_l f_{ck}$)$^{1/3}$ + 0,15 σ_{cp}] $b_w d$

k = 1 + (200/d)$^{1/2}$

d effective depth of the cross-section in mm

$\rho_l = A_{sl} / b_w$ d < 0,02 A_{sl} area of the tensile reinforcement, b_w smallest width of the cross-section in the tensile area

$\sigma_{cp} = N_{Ed} / A_c$ (> 0 compression)

Min. value $V_{Rd,c}$ = (0, 035k$^{3/2}$.$f_{ck}^{1/2}$ + 0,15σ_{cp}) $b_w d$

Ultimate limit state – Shear (contnd)

Inf. of $V_{Rd,s} = (A_{sw}/s)\, z\, f_{ywd} \cot\theta$

and $V_{Rd,max} = b_w\, z\, \nu\, f_{cd} / (\cot\theta + \tan\theta)$

$\nu = 0{,}6\,[\,1 - f_{ck} / 250\,]$

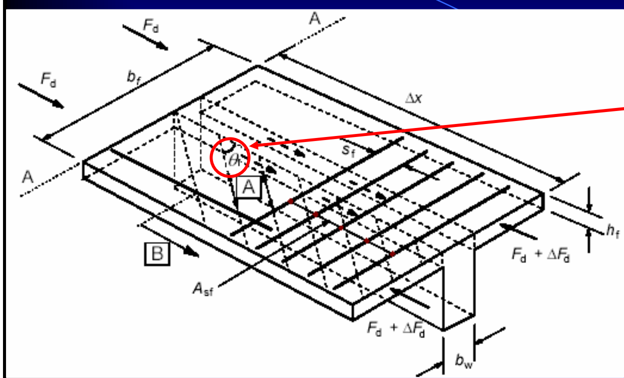
$1 < \cot\theta < 2{,}5$ or $45° > \theta > 22°$

In case of a compression axial force : $\alpha_{cw} V_{Rd,max}$

Increased resistance $1{,}25 > \alpha_{cw} > 1$ *where* $0 < \sigma_{cm} < 0{,}6 f_{cd}$

Reduced resistance $\alpha_{cw} < 1$ *where* $\sigma_{cm} > 0{,}6\, f_{cd}$

Shear between web and flanges of T-sections



$1{,}0 \le \cot\theta_f \le 2{,}0$ (compression flange)

$1{,}0 \le \cot\theta_f \le 1{,}25$ (tension flange)

Shear at the interface between concrete cast at different times

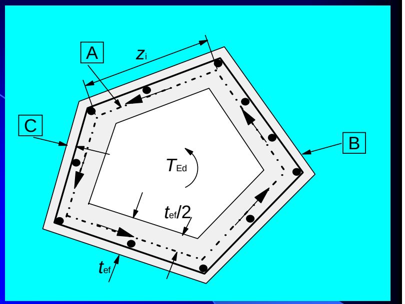
$v_{Rdi} = c\, f_{ctd} + \mu\, \sigma_n + \rho\, f_{yd}\, (\mu \sin\alpha + \cos\alpha) \le 0{,}5\, \nu\, f_{cd}$

C and μ are factors which depend on the roughness of the interface

Ultimate limit state verifications

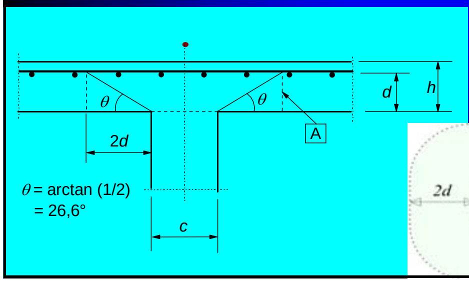
TORSION

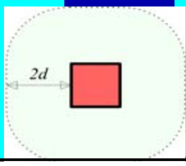
A - Centre-line
B - Outer edge of effective cross-section, circumference *u*,
C - Cover

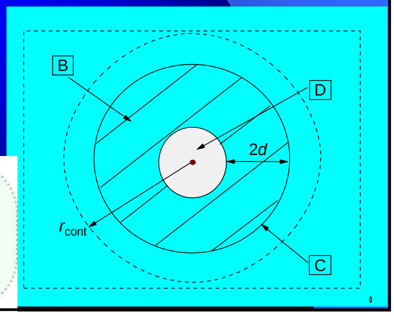


PUNCHING

A - Basic control section
B - Basic control area A_{cont}
C - Basic control perimeter u_1
D – Loaded area A_{load}







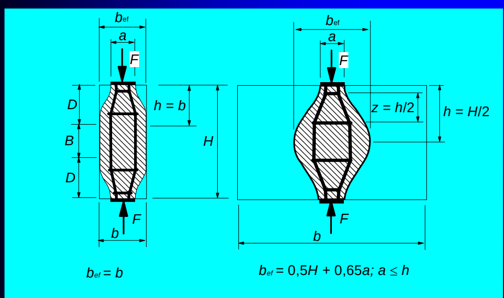
Design with strut and tie models

For zones where a non-linear strain distribution exists

Verification of struts (concrete)

⇨ struts without transverse tension $\sigma_{Rd,max} = f_{cd}$

⇨ struts with transverse tension $\sigma_{Rd,max} = \nu' f_{cd}$
(compressed and cracked zones)



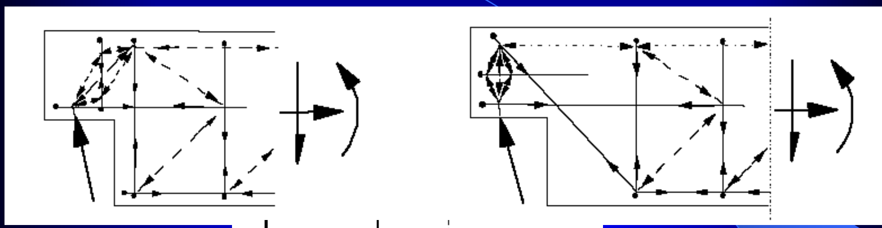
Verification of ties :

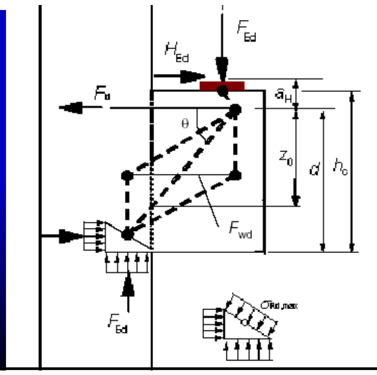
B – Continuity region

D – Discontinuity region

Examples of strut and tie models:

- for half joints (§ 10 – precast concrete elements & structures)
- for a corbel (Annex J- regions with discontinuities)





Serviceability limit state

- Functioning of the structure in normal use
- Comfort of people
- Appearance

The verification rules are deemed to ensure:

- the appropriate serviceability level
- the durability for the design working life

Serviceability criteria

$$E_d \leq C_d$$

The verifications relate to:

- **stress limitation**
- **limitation of crack width**
- **limitation of deformations**
- **limitation of vibrations**

Actions and material properties are taken into account with their representative values (partial factors equal to 1, unless otherwise specified)

EN 1998-1 : 2004
Eurocode 8 : Design of structures for earthquake resistance

Contents of EN 1998-1 : 2004
Eurocode 8: Design of structures for earthquake resistance

6 SPECIFIC RULES FOR STEEL BUILDINGS

- 6.1 GENERAL
- 6.2 MATERIALS
- 6.3 STRUCTURAL TYPES AND BEHAVIOUR FACTORS6.4 STRUCTURAL ANALYSIS
- 6.5 DESIGN CRITERIA AND DETAILING RULES FOR DISSIPATIVE STRUCTURAL BEHAVIOUR COMMON TO ALL STRUCTURAL TYPES
- 6.6 DESIGN AND DETAILING RULES FOR MOMENT RESISTING FRAMES
- 6.7 DESIGN AND DETAILING RULES FOR FRAMES WITH CONCENTRIC BRACINGS
- 6.8 DESIGN AND DETAILING RULES FOR FRAMES WITH ECCENTRIC BRACINGS
- 6.9 DESIGN RULES FOR INVERTED PENDULUM STRUCTURES
- 6.10 DESIGN RULES FOR STEEL STRUCTURES WITH CONCRETE CORES OR CONCRETE WALLS AND FOR MOMENT RESISTING FRAMES COMBINED WITH CONCENTRIC BRACINGS OR INFILLS
- 6.11 CONTROL OF DESIGN AND CONSTRUCTION

7 SPECIFIC RULES FOR COMPOSITE STEEL – CONCRETE BUILDINGS

- **7.1 GENERAL**
- **7.2 MATERIALS**
- **7.3 STRUCTURAL TYPES AND BEHAVIOUR FACTORS**
- **7.4 STRUCTURAL ANALYSIS**
- **7.5 DESIGN CRITERIA AND DETAILING RULES FOR DISSIPATIVE STRUCTURAL BEHAVIOUR COMMON TO ALL STRUCTURAL TYPES**
- **7.6 RULES FOR MEMBERS**
- **7.7 DESIGN AND DETAILING RULES FOR MOMENT FRAMES**
- **7.8 DESIGN AND DETAILING RULES FOR COMPOSITE CONCENTRICALLY BRACED FRAMES**
- **7.9 DESIGN AND DETAILING RULES FOR COMPOSITE ECCENTRICALLY BRACED FRAMES**
- **7.10 DESIGN AND DETAILING RULES FOR STRUCTURAL SYSTEMS MADE OF REINFORCED CONCRETE SHEAR WALLS COMPOSITE WITH STRUCTURAL STEEL ELEMENTS**
- **7.11 DESIGN AND DETAILING RULES FOR COMPOSITE STEEL PLATE SHEAR WALLS**
- **7.12 CONTROL OF DESIGN AND CONSTRUCTION**

8 SPECIFIC RULES FOR TIMBER BUILDINGS

- 8.1 GENERAL
- 8.2 MATERIALS AND PROPERTIES OF DISSIPATIVE ZONES
- 8.3 DUCTILITY CLASSES AND BEHAVIOUR FACTORS
- 8.4 STRUCTURAL ANALYSIS
- 8.5 DETAILING RULES
- 8.6 SAFETY VERIFICATIONS
- 8.7 CONTROL OF DESIGN AND CONSTRUCTION

9 SPECIFIC RULES FOR MASONRY BUILDINGS

- 9.1 SCOPE
- 9.2 MATERIALS AND BONDING PATTERNS
- 9.3 TYPES OF CONSTRUCTION AND BEHAVIOUR FACTORS
- 9.4 STRUCTURAL ANALYSIS
- 9.5 DESIGN CRITERIA AND CONSTRUCTION RULES
- 9.6 SAFETY VERIFICATION
- 9.7 RULES FOR “SIMPLE MASONRY BUILDINGS”

10 BASE ISOLATION

- 10.1 SCOPE
- 10.2 DEFINITIONS
- 10.3 FUNDAMENTAL REQUIREMENTS
- 10.4 COMPLIANCE CRITERIA
- 10.5 GENERAL DESIGN PROVISIONS
- 10.6 SEISMIC ACTION
- 10.7 BEHAVIOUR FACTOR
- 10.8 PROPERTIES OF THE ISOLATION SYSTEM
- 10.9 STRUCTURAL ANALYSIS
- 10.10 SAFETY VERIFICATIONS AT ULTIMATE LIMIT STATE

ANNEX A (INFORMATIVE) ELASTIC DISPLACEMENT RESPONSE SPECTRUM

ANNEX B (INFORMATIVE) DETERMINATION OF THE TARGET DISPLACEMENT FOR NONLINEAR STATIC (PUSHOVER) ANALYSIS

ANNEX C (NORMATIVE) DESIGN OF THE SLAB OF STEEL-CONCRETE COMPOSITE BEAMS AT BEAM-COLUMN JOINTS IN MOMENT RESISTING FRAMES

EUROCODE 8 (SEISMIC DESIGN):

SPECIFIC RULES FOR CONCRETE BUILDINGS

Ductility classes

- **New ductility classes (DC)**

  (changes dictated by national comments supported by a number of background studies)
  - DC 'H' (≈old 'M', increased q, CD for V_{Sd} in beams, ...)
  - DC 'M' (≈old 'L', increased q, CD for V_{Sd} in beams, ...)
  - DC 'L' (EC2, no brittle steel A, q≤1.5)
- Basic value of behaviour factor (q_0)

STRUCTURAL TYPE	DCH	DCM
Frame system, dual system, coupled wall system	$4{,}5\alpha_u/\alpha_1$	$3{,}0\alpha_u/\alpha_1$
Wall system	$4{,}0\alpha_u/\alpha_1$	3,0
Core system	3,0	2,0
Inverted pendulum system	2,0	1,5

- **Overstrength**
  - **α_1 :** seismic action at first yield (anywhere)
  - **α_u :** seismic action at development of overall structural instability (collapse mechanism)
  - → Obtained from pushover analysis ($\alpha_u/\alpha_1 \leq 1.5$), or *defaults:*
    - Frames (or frame-equivalent dual): $\alpha_u/\alpha_1=1.3$ (1.1 for one-storey, 1.2 for one-bay frames)
    - Wall (or wall-equivalent dual):
      - Wall systems with only two uncoupled walls per horizontal direction: $\alpha_u/\alpha_1=1.0$
      - Other uncoupled wall systems: $\alpha_u/\alpha_1=1.1$
      - Wall -equivalent dual, or coupled wall systems: $\alpha_u/\alpha_1=1.2$
- **Final behaviour factor** $q=q_o \cdot k_w \geq 1,5$

$$k_w = \begin{cases} 1{,}00, \text{ for frame and frame} - \text{equivalent dual systems} \\ (1+\alpha_o)/3 \leq 1, \text{but not less than } 0{,}5, \text{ for wall, wall - equivalent and torsionally} \\ \text{flexible systems} \end{cases}$$

New structural systems

- **Large lightly reinforced wall system:**
  - comprises at least two walls with horizontal dimension not less than 4m and $2h_w/3$, which collectively support at least 20% of the total gravity load above in the seismic design situation
  - has a fundamental period T_1, for assumed fixity at the base against rotation, less or equal to 0.5sec
  - If a structural system does not qualify as a system of large lightly reinforced walls, then ***all*** its walls should be designed and detailed as *ductile walls*
- Frame, dual or wall systems without a *minimum torsional rigidity* (e_o<0.3r) should be classified as ***torsionally flexible (core) systems***

Design criteria

- **Local resistance condition**: $E_d \leq R_d$
- **Capacity design rule**: E_d from equilibrium conditions, assuming plastic hinges with their possible overstrengths formed in adjacent areas

  → to avoid brittle or undesirable failure mechanisms
- **Local ductility condition**: high plastic rotational capacities in potential plastic hinge regions
  - sufficient curvature ductility (post-failure 85%-moment resistance level) in all critical regions of primary elements

    $\mu_\phi = 2q_o - 1$ if $T_1 \geq T_C$

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

  (based on $\mu_\phi = 2\mu_\delta - 1$ and $\mu_\delta = q$ if $T_1 \geq T_C$, $\mu_\delta = 1 + (q-1)T_C/T_1$ if $T_1 < T_C$)

  Note that $q < q_o$ for irregular structures (no reduction in $\mu_{\phi,req}$!)

- **Structural redundancy**: high degree of redundancy accompanied by redistribution capacity (otherwise lower q-factor)
- **Secondary seismic members and resistances**:
  - resistances or stabilising effects not explicitly taken into account (e.g. membrane reactions of slabs mobilised by upwards deflections of structural walls)
  - non-structural elements (esp. masonry infills!)
- **Specific additional measures** (to reduce uncertainty):
  - minimize geometric errors (min dimensions, max b/h etc.)
  - minimize ductility uncertainties (min μ_φ, minρ_l, ν_{max})

Safety verifications

- For ULS verifications, partial safety factors for materials γ_c and γ_s shall account for strength degradation due to the cyclic deformations
- $\gamma_c = 1.5$ and $\gamma_s = 1.15$ (as in EC2) can be taken (convenient for practice!) assuming that
  - due to local ductility provisions the ratio between the residual strength after degradation and the initial one is roughly equal to the ratio between the γ_M-values for accidental and fundamental load combinations
  - if strength degradation is appropriately accounted in the evaluation of the material properties, the γ_M-values adopted for the *accidental* design situation may be used

Design to Eurocode 2 (EN1992-1)

- Recommended only for *low seismicity* areas
- In *primary* elements, reinforcing steel of *class B or C* (table C.1 EN1992-1) shall be used
- Behaviour factor up to q=1.5 may be used in deriving the seismic actions, regardless of the structural system and of regularity in elevation

Properties of reinforcement (EC2 – Annex C)

Product form	Bars and de-coiled rods			Wire Fabrics			Requirement or quantile value (%)
Class	A	B	C	A	B	C	-
Characteristic yield strength f_{yk} or $f_{0,2k}$ (MPa)	400 to 600						5,0
Minimum value of $k = (f_t/f_y)_k$	≥1,05	≥1,08	≥1,15 <1,35	≥1,05	≥1,08	≥1,15 <1,35	10,0
Characteristic strain at maximum force, ε_{uk} (%)	≥2,5	≥5,0	≥7,5	≥2,5	≥5,0	≥7,5	10,0
Bendability	Bend/Rebend test			-			
Shear strength	-			0,3 A f_{yk} (A is area of wire)			Minimum
Maximum deviation from nominal mass (individual bar or wire) (%) — Nominal bar size (mm) ≤ 8	± 6,0						5,0
Nominal bar size (mm) > 8	± 4,5						

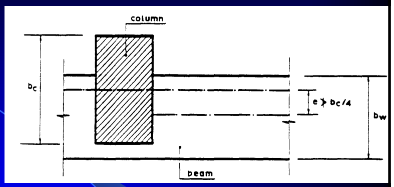
Note: The values for the fatigue stress range with an upper limit of βf_{yk} and for the minimum relative rib area for use in a Country may be found in its National Annex. The recommended values are given in Table C.2N. The value of β for use in a Country may be found in its National Annex. The recommended value is 0,6.

Design for DC M:
Geometrical constraints and materials

- ***Material requirements***
  - use of concrete <C16 not allowed in primary elements
  - use of concrete >C50 (HSC) for DC M is not covered
  - only *ribbed* bars are allowed as *longitudinal* reinforcing steel in critical regions of primary elements
  - in primary elements, reinforcing steel of class B or C (table C.1 EN1992-1) shall be used
  - welded wire meshes of steel B or C are allowed (should be ribbed if used as longitudinal reinforcement)

- ***Geometrical constraints***

  BEAMS
  - eccentricity of beam axis < $b_c/4$
  - width $b_w \leq \min\{b_c + h_w; 2b_c\}$



  COLUMNS
  - unless θ≤0.1, in primary columns b≥0.1l_o
    (l_o: distance from end to point of contraflexure)

  DUCTILE WALLS
  - web thickness $b_{wo} \geq \max\{150\text{mm}, h_s/20\}$
    (h_s: clear storey height)
  - additional requirements for confined boundary elements

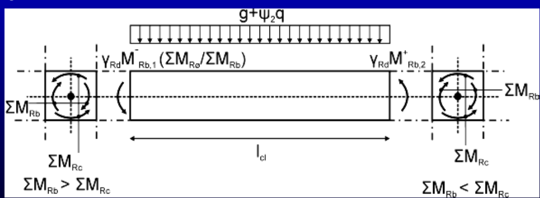
  LARGE LIGHTLY REINFORCED WALLS
  - web thickness $b_{wo} \geq \max\{150\text{mm}, h_s/20\}$

Design for DC M: Design action effects

- Moments and axial forces from analysis, except in primary ductile walls; redistribution of M permitted
- Shear forces from capacity design

  (shears $V_{max,i}$, $V_{min,i}$ calculated for end moments $M_{i,d}$)
  - ***Beams***
    (γ_{Rd}=1.0)

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

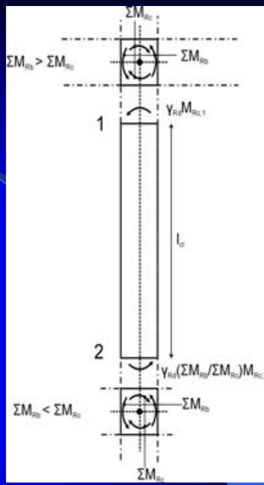


– ***Columns***

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

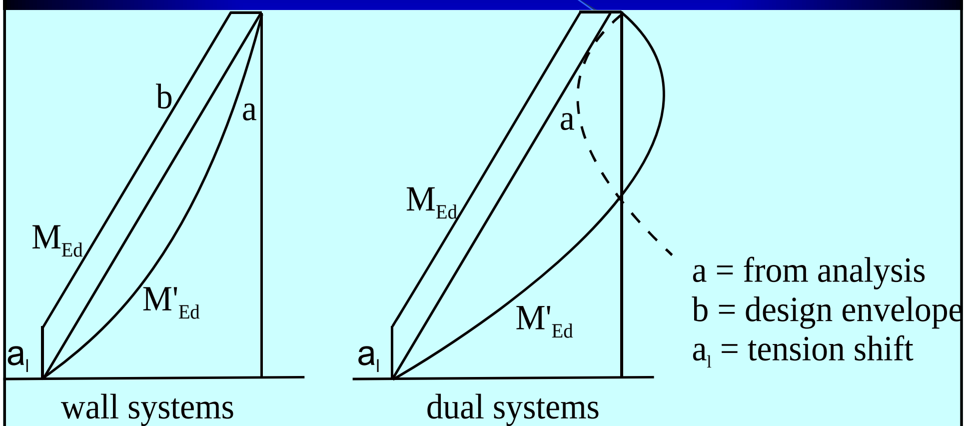
(γ_{Rd}=1.1)

to account for overstrength due to strain-hardening and confinement

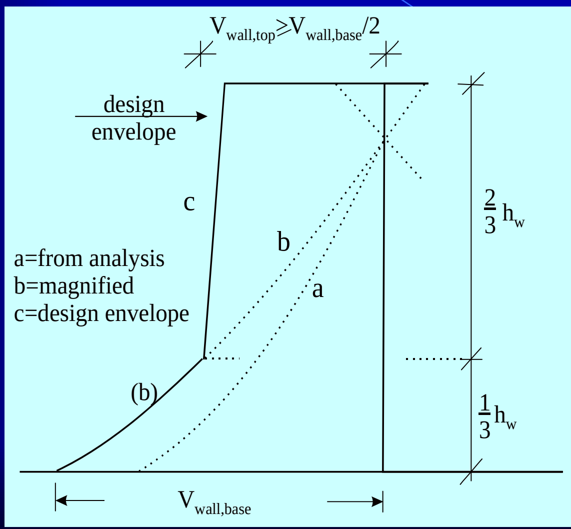


- ***Ductile walls***:
  - Redistribution between primary walls, up to 30%
  - Redistribution between coupling beams, up to 20%

– Design bending moment diagram (slender walls):



– Design shear force diagram (dual systems with slender walls):



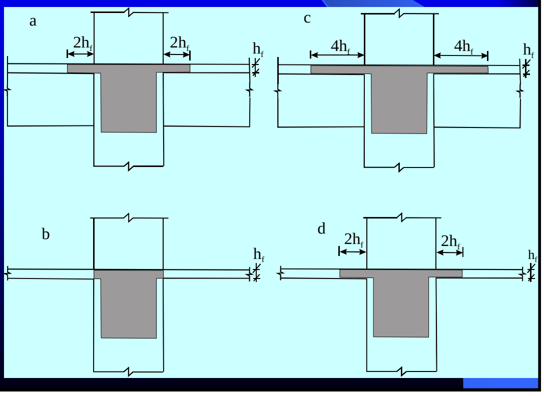
- Special provisions for ***large lightly reinforced walls***:
  - to ensure that flexural yielding precedes attainment of ULS in shear, shear force V'_{Ed} from analysis is increased

$$V_{Ed} = V'_{Ed} \frac{q+1}{2}$$

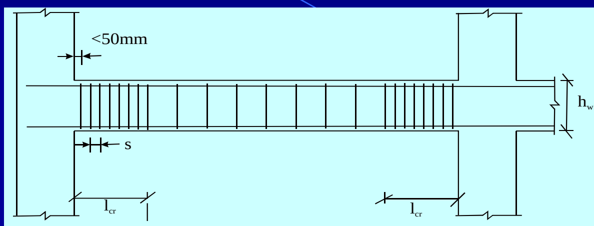
  - *additional* dynamic axial forces developed due to ***uplifting*** shall be taken into account in the ULS verification (M, N) → may be taken as 50% of the axial force in the wall due to the gravity loads ($g+\psi_2 q$)
  - if $q \leq 2$, these dynamic axial forces may be neglected

Design for DC M: ULS verifications and detailing

- ***Beams***
  - bending and shear resistances are computed according to EN1992-1
  - part of top-reinforcement in T-beams (& Γ-beams) may be placed outside the web, within effective flange width b_{eff}



- Detailing of DCM beams for local ductility
  - critical regions:



within l_{cr}, $\mu_{\varphi,req}$ is provided through:

0.018?

- additional $\rho' \geq ½\rho$ at bottom of supports
- tension reinforcement $\rho \leq \rho_{max} = \rho' + \frac{0{,}0018}{\mu_\phi \varepsilon_{sy,d}} \cdot \frac{f_{cd}}{f_{yd}}$
- within l_{cr}, hoops with: $\rho \geq \rho_{min} = 0{,}5\left(\frac{f_{ctm}}{f_{yk}}\right)$

  d_{bw} ≥6mm and spacing

  $s = \min\{h_w/4;\ 24d_{bw};\ 225\text{mm};\ 8d_{bL}\}$

- ***Columns***
  - bending and shear resistances are computed according to EN1992-1
  - simplified biaxial bending check with $0.7M_{Rd,uniax}$
  - in primary columns normalised axial force $\nu_d \leq 0.65$
  - Detailing of DCM columns for local ductility
    - long. reinforcement ratio $1\% \leq \rho_l \leq 4\%$
    - at least one intermediate bar (between corner bars)
    - critical (end) regions: $l_{cr} = \max\{h_c; l_{cl}/6;\ 450\,\text{mm}\}$
    - if $l_{cl}/h_c<3$ (short column), the entire height $l_{cl}=l_{cr}$
    - within l_{cr}, $\mu_{\varphi,req}$ (e.g. $=2q_o-1$) is provided
    - if $\mu_{\varphi,req}$ involves $\varepsilon_{cu} \geq 0.0035 \rightarrow$ confinement required!

- confinement reinforcement within l_{cr} *(DC M)*

$$\alpha \cdot \omega_{wd} \geq 30 \cdot \mu_\phi \cdot \nu_d \cdot \varepsilon_{sy,d} \cdot \frac{b_c}{b_o} - 0{,}035$$

$$\left[\omega_{wd} = \frac{\text{volume of confining hoops}}{\text{volume of concrete core}} \cdot \frac{f_{yd}}{f_{cd}}\right]$$

confinement effectiveness factor $\alpha=\alpha_n \cdot \alpha_s$

for rectangular cross sections:

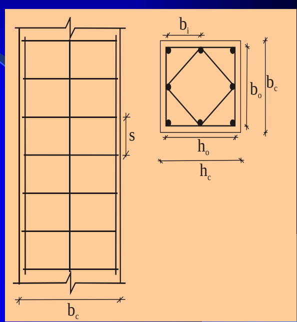
$$\alpha_n = 1 - \sum_n b_i^2 / 6 b_o h_o$$

$$\alpha_s = (1 - s/2b_o)(1 - s/2h_o)$$

for circular cross sections with spiral reinforcement:

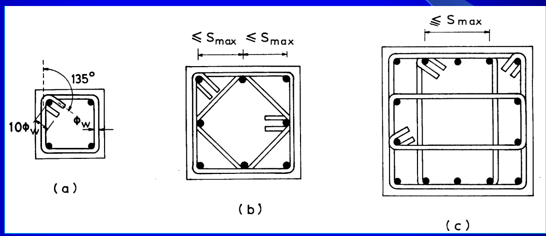
$$\alpha_n = 1$$

$$\alpha_s = (1 - s/2D_o)$$



min ω_{wd} = 0.08

- to prevent early local buckling of longitudinal bars
  - within l_{cr} : $s = min\{b_o/2; 175mm; 8d_{bL}\}$
  - distance between supported bars $s_{max} \leq 200$ mm



- transverse reinforcement within l_{cr} at the base of primary columns may be determined as specified in EN1992-1, provided that $v_d \leq 0.2$ and $q \leq 2.0$

- ***Beam-column joints***
  - horizontal confinement reinforcement in joints of *primary* beams with columns shall not be less than that provided within l_{cr} of columns
  - if beams with $b_w \geq b_c$ frame into all four sides of the joint, spacing of horizontal confinement reinforcement in the joint may be increased to *twice* that required above, but s≤150 mm
  - at least one intermediate (between column corner bars) vertical bar shall be provided at each side of a joint of primary beams and columns

- ***Ductile walls***
  - bending and shear resistances computed according to EN1992-1
  - in primary walls, normalised axial force $\nu_d \leq 0.4$
  - vertical web reinforcement shall be included in calculation of flexural resistance of wall sections
  - flexural resistance of composite sections (L, T, U, I or similar) based on effective flange width, ***min*** of:
    - actual flange width
    - ½ distance to adjacent web of the wall
    - 25% of total height of wall above the level considered

- Detailing of DCM walls for local ductility
  - height of critical region h_{cr} above the base

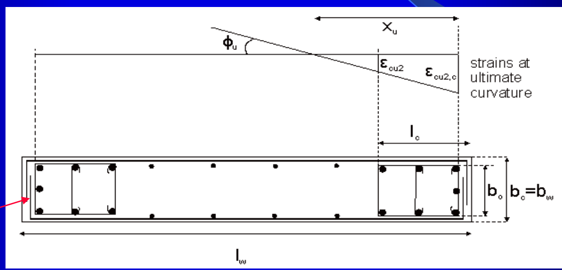
$$h_{cr} = \max\left[l_{w},\ H_w/6\right] \quad but\ h_{cr} \leq \begin{cases} 2 \cdot l_w \\ \begin{cases} h_s & for\ n \leq 6\ storeys \\ 2 \cdot h_s & for\ n \geq 7\ storeys \end{cases} \end{cases}$$

  - required μ_φ as in columns, but using q_o multiplied by M_{Ed}/M_{Rd} at base of wall (e.g. $\mu_\varphi = 2q_o M_{Ed}/M_{Rd} - 1$), to be provided by confinement of boundary elements
    - for walls with rectangular section

$$\alpha\omega_{wd} \geq 30\mu_\phi(\nu_d + \omega_v)\varepsilon_{sy,d}\frac{b_c}{b_o} - 0{,}035 \quad where\ \omega_v = \rho_v\frac{f_{yd,v}}{f_{cd}}$$

    - for barbelled walls, N and ω_v refer to $h_c b_c f_{cd}$ if $x_u \leq l_c$, otherwise analysis with confined concrete model needed

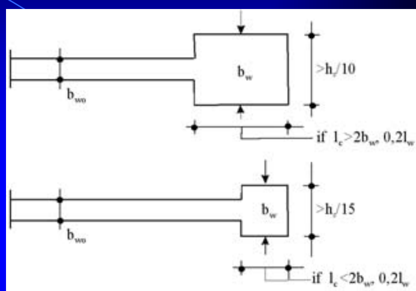
- confinement of boundary elements should extend
  - vertically: over h_{cr}
  - horizontally: over l_c (assuming ε_{cu2}=0.0035)



not good practice

  - min$l_c \geq \{0,15 \cdot l_w$ or $1,50.b_w\}$
- no confined boundary element is required over wall flanges with thickness $h_f > h_s/15$ and width $b_f > h_s/5$

- in boundary elements: minρ_l=0.5% (=½ min$\rho_{l,col}$)
- thickness $b_w \geq 200$, also:



- above h_{cr} EC2 applies, but if ε_c>0.002, minρ_l=0.5%
- ω_w in boundary elements may conform to EC2 only, if:
  - axial load $\nu_d \leq 0.15$
  - axial load $\nu_d \leq 0.20$ and q reduced by 15%

- ***Large lightly reinforced walls***
  - bending resistances computed according to EN1992-1
  - when $V_{Ed} \leq V_{Rd,c} = [C_{Rd,c} k (100 \rho_l f_{ck})^{1/3} + 0.15 \sigma_{cp}] b_{wd}$ $\rho_{w,min}$ in the web is not required
  - sliding shear check is done according to EN1992-1, but anchorage length of clamping bars increased by 50%
  - hoop and cross-tie vertical spacing $\leq \min\{100\text{mm}, 8d_{bL}\}$
  - vertical bars engaged by hoop or cross-tie with d≥6mm, within boundary elements with $l_c \geq \min\{b_w, 3b_w \sigma_{cm}/f_{cd}\}$, ($\sigma_{cm}$: mean value of concrete stress in compression zone)
  - horiz. + vert. ties according to EN1992-1 provided
    - along all intersections of walls
    - around openings in the wall
    - at all floor levels

Design for DC H

- generally similar to DCM, but more stringent detailing
- more detailed verification of beam-column ***joints***
- if $V_{Ed} > |V_E|_{\max} = (2+\zeta) \cdot f_{ctd} \cdot b_w \cdot d$, cross-inclined reinforcement required to resist shear in beams
- explicit calculation of joint resistance

$$V_{jhd} \leq \eta f_{cd} \sqrt{1 - \frac{\nu_d}{\eta}} b_j h_c$$

$$\frac{A_{sh} \cdot f_{ywd}}{b_j \cdot h_{jw}} \geq \frac{\left(\frac{V_{jhd}}{b_j \cdot h_{jc}}\right)^2}{f_{ctd} + \nu_d f_{cd}} - f_{ctd}$$

- explicit calculation of sliding shear resistance of walls

$$V_{Rd,S} = V_{dd} + V_{id} + V_{fd}$$

$$V_{id} = \Sigma A_{si} \cdot f_{yd} \cdot \cos\varphi$$

$$V_{dd} = \min \begin{cases} 1{,}3 \cdot \Sigma A_{sj} \cdot \sqrt{f_{cd} \cdot f_{yd}} \\ 0{,}25 \cdot f_{yd} \cdot \Sigma A_{sj} \end{cases}$$

$$V_{fd} = \min \begin{cases} \mu_f \cdot \left[\left(\Sigma A_{sj} \cdot f_{yd} + N_{Sd}\right) \cdot \xi + M_{Ed} / z\right] \\ 0{,}5\nu \cdot f_{cd} \cdot \xi \cdot l_w \cdot b_{wo} \end{cases}$$

Provisions for anchorages and splices

- ***hoops*** should be closed stirrups with 135° hooks and $10d_{bw}$ long extensions

- Anchorage of reinforcement

Columns

- anchorage length l_{bd} of column bars in critical regions based on $A_{s,req}/A_{s,prov} = 1$
- first $5d_{bL}$ of column bar within a joint not included in l_{bd}
- if N_{Ed} is tensile in a column, l_{bd} increased by 50%

Beams

- the part of beam bars bent in joints for anchorage should be placed *inside* the corresponding column hoops

---

- to prevent bond failure → limit d_{bL} passing through joints
  - interior beam-column joints $\frac{d_{bL}}{h_c} \le \frac{7{,}5 \cdot f_{ctm}}{\gamma_{Rd} \cdot f_{yd}} \cdot \frac{1+0{,}8 \cdot \nu_d}{1+0.75 k_D \cdot \rho'/\rho_{max}}$
  - exterior beam-column joints $\frac{d_{bL}}{h_c} \le \frac{7{,}5 \cdot f_{ctm}}{\gamma_{Rd} \cdot f_{yd}} \cdot (1+0{,}8 \cdot \nu_d)$

	DC H	DC M
k_D	1	2/3
γ_{Rd}	1.2	1.0

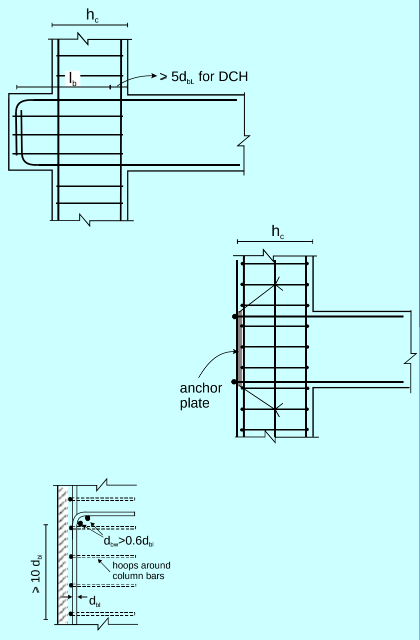
- if limit on d_{bL} difficult to satisfy, use special measures
- top or bottom bars passing through interior joints, shall terminate at distance $\geq l_{cr}$ from the face of the joint

Additional measures for anchorage in exterior beam-column joints

a) exterior stubs

b) plates welded to end of bars

c) transverse bars inside the bend



- Splicing of reinforcement
  - lap-splicing by ***welding*** not allowed within the l_{cr}
  - splicing by ***mechanical couplers*** allowed in columns and walls, if covered by appropriate (cyclic) testing
  - spacing of transverse reinforcement in the lap zone:

    s = min{b/4; 100mm}
  - required area of transverse reinforcement A_{st} within the lap zone

$$A_{st} = s\,(d_{bl}/50)(f_{yl,\,d}/f_{ywd})$$

area of one leg of transverse reinforcement

Design and detailing of secondary seismic elements

- designed/detailed to maintain bearing capacity, when subjected to max deformations under seismic actions
- does ***not*** apply to *non-seismic* members (e.g. slab ribs)
- max deformations calculated from analysis, in which the contribution of secondary elements to lateral stiffness is *neglected* and primary elements are modelled with their *cracked* flexural and shear stiffness
- verification: $M_d \leq M_{Rd}$ and $V_d \leq V_{Rd}$ where M_d, V_d calculated from above max deformations and cracked flexural and shear stiffness of *secondary* elements

Local effects due to masonry or concrete infills

- the entire length of columns in infilled ground floors considered as critical length and confined accordingly
- if $h_{inf} < l_{cl,col}$, $l_{cr} = l_{cl}$ plus special measures:
  - design shear calculated from CD based on l_{cl} and $\gamma_{Rd} M_{Rc}$
  - corresponding ties placed within $l_{cl} + h_c$
  - if ‘free’ length < $1.5h_c$, diagonal reinforcement needed
- if masonry infill on one side of column only, $l_{cr} = l_{cl}$
- length l_c of column over which the diagonal strut force of the infill is applied, should be verified in shear for min of horiz. component of strut force and CD shear

Seismic performance of multistorey R/C buildings designed to the prEN-1998-1:

- Trial application of the new provisions to **four** typical multi-storey buildings, **6-storey** and **10-storey**
  - with reinforced concrete (R/C) **frame** system
  - with **dual** (frame+wall) system
- Similar buildings previously designed (Kappos / Athana-ssiadou, EEE, 1997) for old ductility classes H and M
  - comparisons between the old and new designs
  - in terms of **cost** of materials and of seismic **performance**

Design of 6-storey buildings

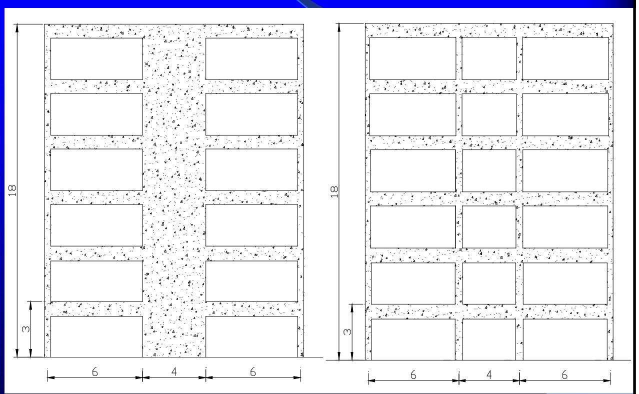
- Codes:

  EC2 , EC8 (prEN)
- Materials:

  C20/25 S400
- Design PGA:

  $\alpha_g=0.25$
- Code spectrum Type 1(M_s>5.5)
- Effective Stiffness:

  $EI_{eff}=0.5EI_g$

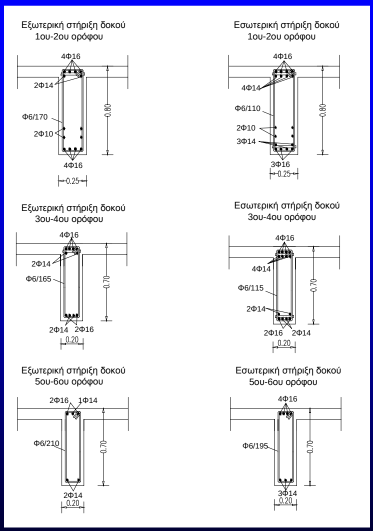


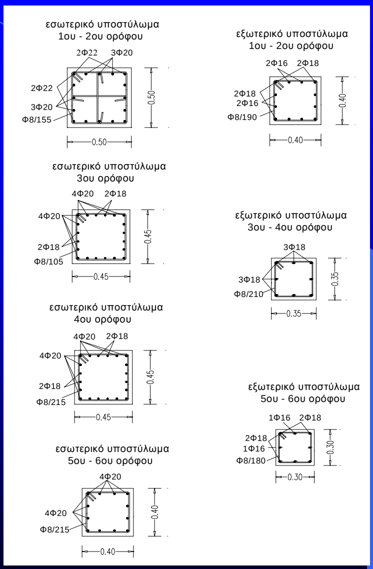
Behaviour factors q

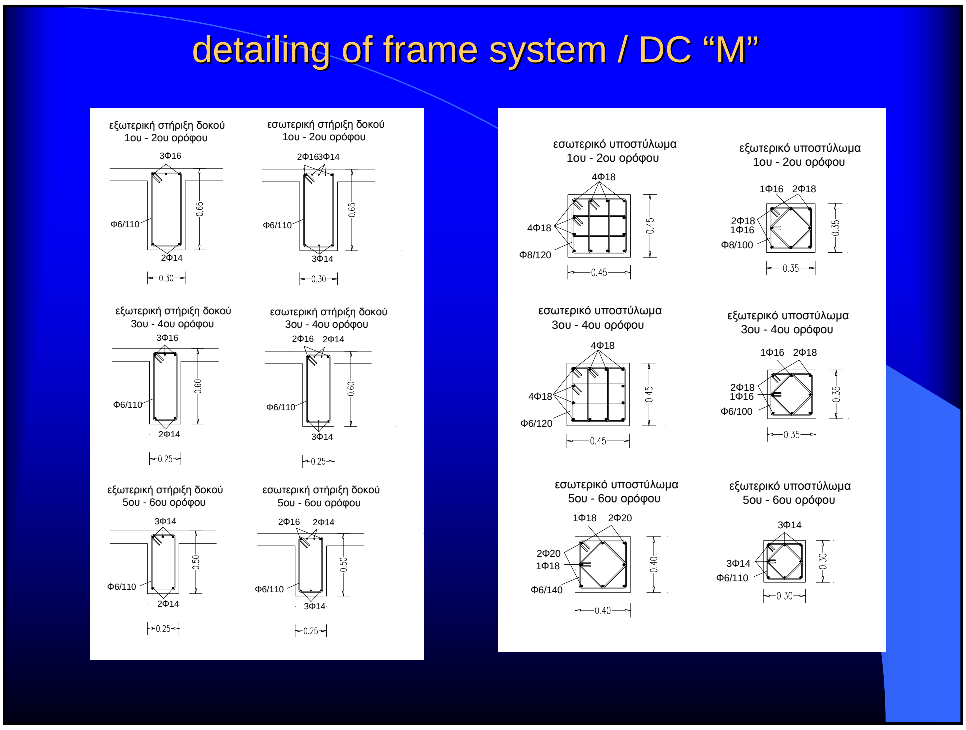
- q=1.5, for DC "L"
- q= $k_w \cdot q_o$, for DC "M" and "H"
  - frame system / DC "M": q=3.90
  - dual system / DC "M": q=3.6
  - frame system / DC "H": q=5.85
  - dual system / DC "H": q=5.40

→ *Very similar q-factors for both systems!*

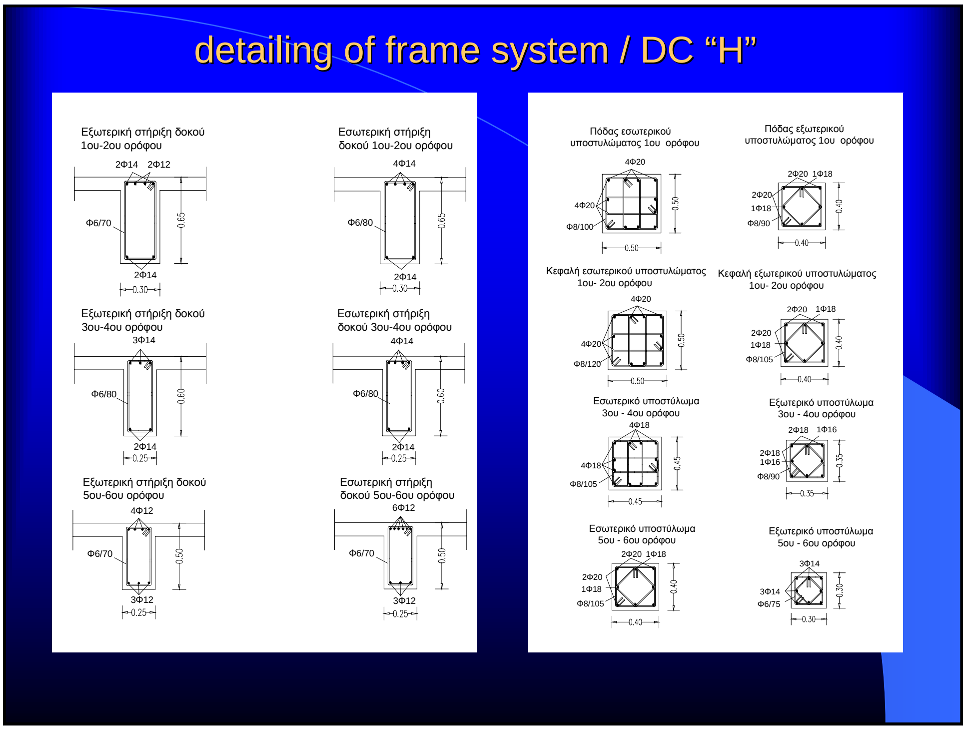
detailing of frame system / DC"L"





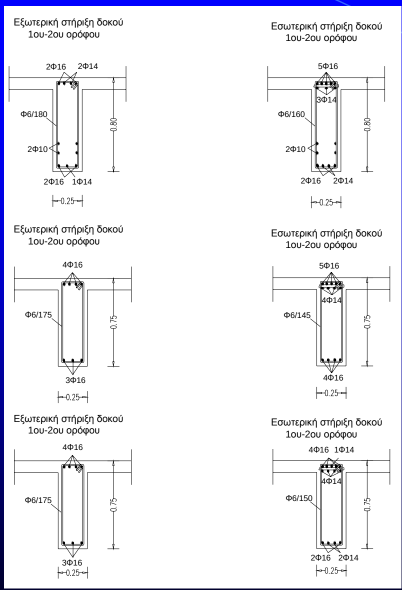


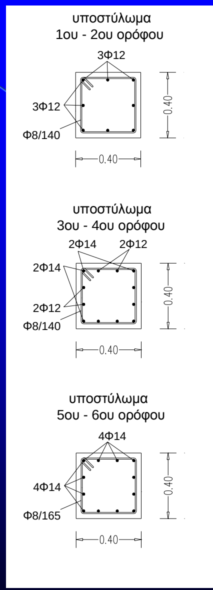
detailing of frame system / DC “M”
εξωτερική στήριξη δοκού 1ου - 2ου ορόφου
3Φ16
Φ6/110
2Φ14
0.65
0.30
εσωτερική στήριξη δοκού 1ου - 2ου ορόφου
2Φ16 3Φ14
Φ6/110
3Φ14
0.65
0.30
εξωτερική στήριξη δοκού 3ου - 4ου ορόφου
3Φ16
Φ6/110
2Φ14
0.60
0.25
εσωτερική στήριξη δοκού 3ου - 4ου ορόφου
2Φ16 2Φ14
Φ6/110
3Φ14
0.60
0.25
εξωτερική στήριξη δοκού 5ου - 6ου ορόφου
3Φ14
Φ6/110
2Φ14
0.50
0.25
εσωτερική στήριξη δοκού 5ου - 6ου ορόφου
2Φ16 2Φ14
Φ6/110
3Φ14
0.50
0.25
εσωτερικό υποστύλωμα 1ου - 2ου ορόφου
4Φ18
4Φ18
Φ8/120
0.45
0.45
εξωτερικό υποστύλωμα 1ου - 2ου ορόφου
1Φ16 2Φ18
2Φ18
1Φ16
Φ8/100
0.35
0.35
εσωτερικό υποστύλωμα 3ου - 4ου ορόφου
4Φ18
4Φ18
Φ6/120
0.45
0.45
εξωτερικό υποστύλωμα 3ου - 4ου ορόφου
1Φ16 2Φ18
2Φ18
1Φ16
Φ6/100
0.35
0.35
εσωτερικό υποστύλωμα 5ου - 6ου ορόφου
1Φ18 2Φ20
2Φ20
1Φ18
Φ6/140
0.40
0.40
εξωτερικό υποστύλωμα 5ου - 6ου ορόφου
3Φ14
3Φ14
Φ6/110
0.30
0.30



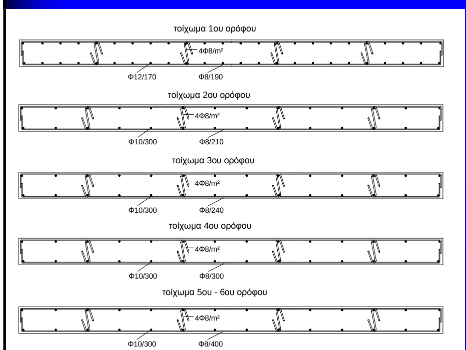
detailing of frame system / DC “H”
Εξωτερική στήριξη δοκού 1ου-2ου ορόφου
2Φ14 2Φ12
Φ6/70
2Φ14
0.65
0.30
Εσωτερική στήριξη δοκού 1ου-2ου ορόφου
4Φ14
Φ6/80
2Φ14
0.65
0.30
Εξωτερική στήριξη δοκού 3ου-4ου ορόφου
3Φ14
Φ6/80
2Φ14
0.60
0.25
Εσωτερική στήριξη δοκού 3ου-4ου ορόφου
4Φ14
Φ6/80
2Φ14
0.60
0.25
Εξωτερική στήριξη δοκού 5ου-6ου ορόφου
4Φ12
Φ6/70
3Φ12
0.50
0.25
Εσωτερική στήριξη δοκού 5ου-6ου ορόφου
6Φ12
Φ6/70
3Φ12
0.50
0.25
Πόδας εσωτερικού υποστυλώματος 1ου ορόφου
4Φ20
4Φ20
Φ8/100
0.50
0.50
Πόδας εξωτερικού υποστυλώματος 1ου ορόφου
2Φ20 1Φ18
2Φ20
1Φ18
Φ8/90
0.40
0.40
Κεφαλή εσωτερικού υποστυλώματος 1ου- 2ου ορόφου
4Φ20
4Φ20
Φ8/120
0.50
0.50
Κεφαλή εξωτερικού υποστυλώματος 1ου- 2ου ορόφου
2Φ20 1Φ18
2Φ20
1Φ18
Φ8/105
0.40
0.40
Εσωτερικό υποστύλωμα 3ου - 4ου ορόφου
4Φ18
4Φ18
Φ8/105
0.45
0.45
Εξωτερικό υποστύλωμα 3ου - 4ου ορόφου
2Φ18 1Φ16
2Φ18
1Φ16
Φ8/90
0.35
0.35
Εσωτερικό υποστύλωμα 5ου - 6ου ορόφου
2Φ20 1Φ18
2Φ20
1Φ18
Φ8/105
0.40
0.40
Εξωτερικό υποστύλωμα 5ου - 6ου ορόφου
3Φ14
3Φ14
Φ6/75
0.30
0.30

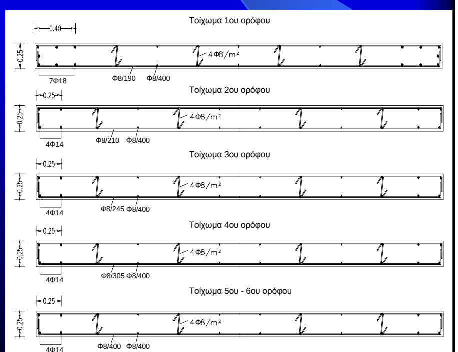
detailing of dual system / DC “L”



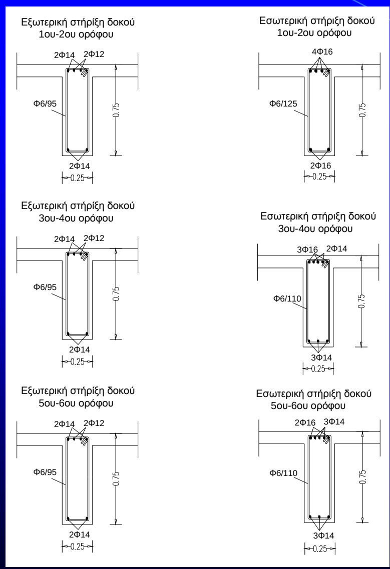


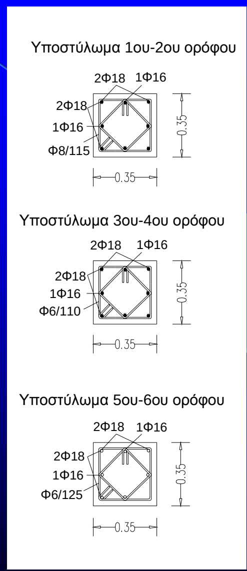
detailing of dual system / DC “L”



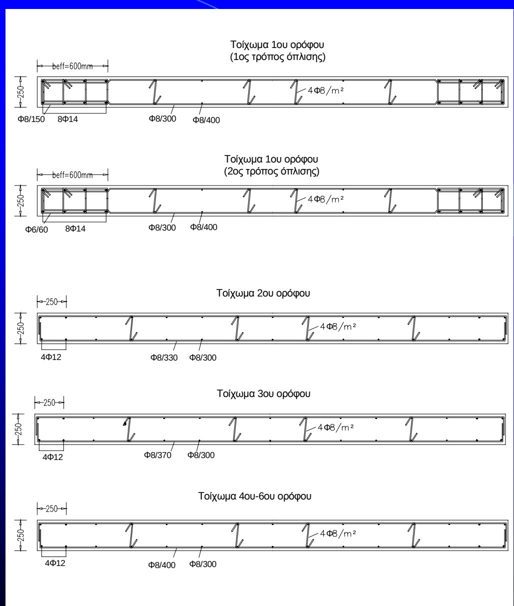


detailing of dual system / DC “M”

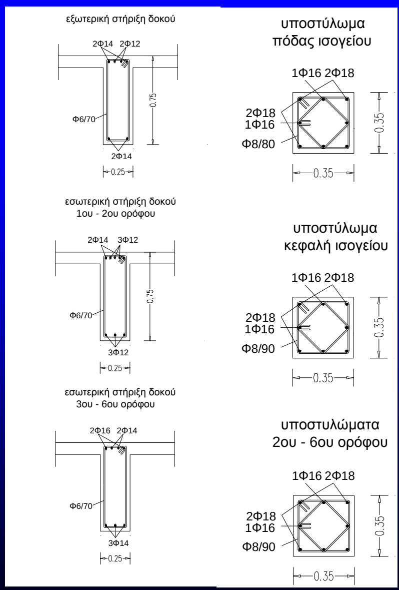


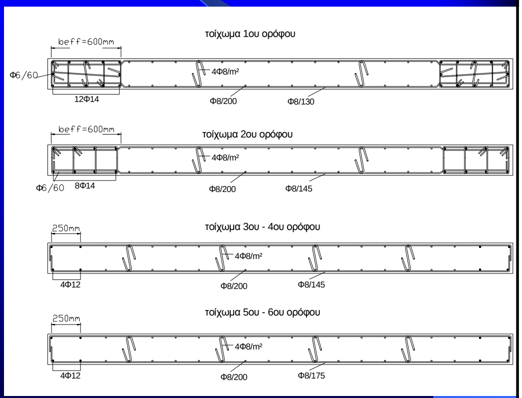


detailing of dual system / DC “M”

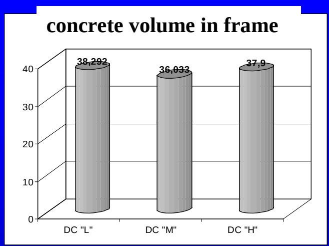


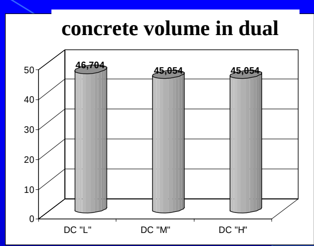
detailing of dual system / DC “H”

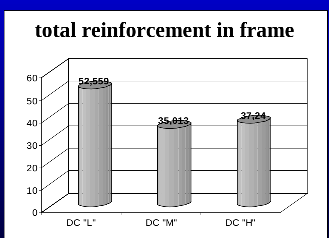


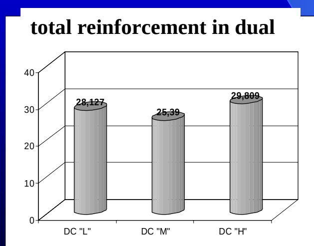


Quantities of materials



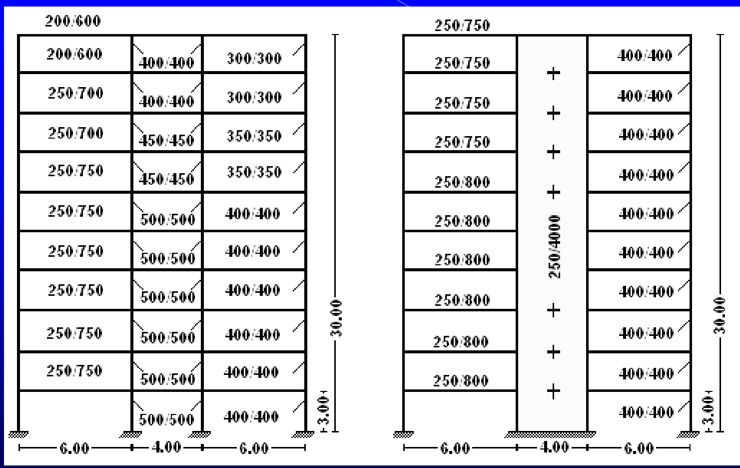






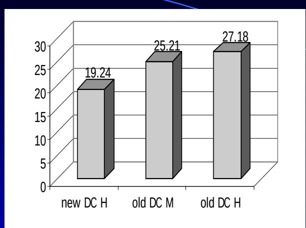
Design of 10-storey buildings for DC “H”

FR (T=0.96s) FW (T=0.64s)

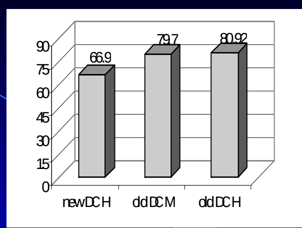


q=5.85 q=5.40

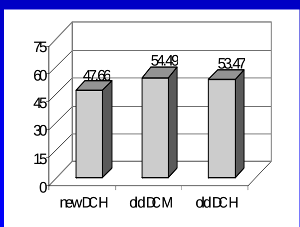
PGA=0.25g, C20/25 concrete, S400 steel



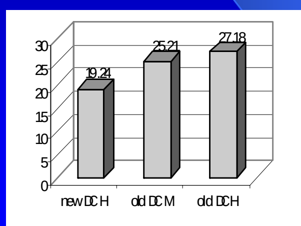
Concrete volume



Total weight of reinforcement

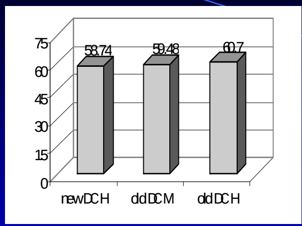


Weight of longit. reinforcement

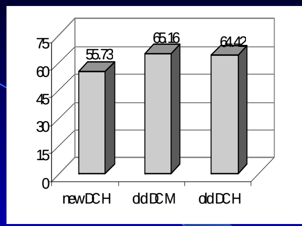


Weight of transv. reinforcement

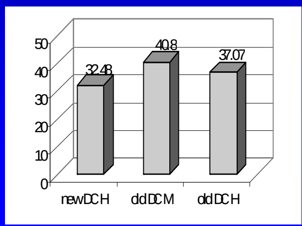
Required quantity of materials in the ***frame*** structures



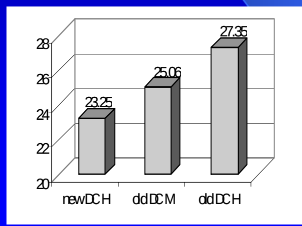
Concrete volume



Total weight of reinforcement



Weight of longit. reinforcement



Weight of transv. reinforcement

Required quantity of materials in the ***dual*** structures

Design to new DC'H' results in:

- 16% less reinforcement in the frame (FR)
- 14% less reinforcement in the dual (FW)
- Longitudinal reinforcement: 11 to 20% less
- Transverse reinforcement: 7 to 29% less
- 9% less concrete volume in the frame (FR)
- 2% less concrete volume in the dual (FW)

→ Main reason for reduced steel requirements: higher q-factors specified by the prEN

Seismic performance assessment

Modelling: Standard *point hinge* (DRAIN-2D/2000)

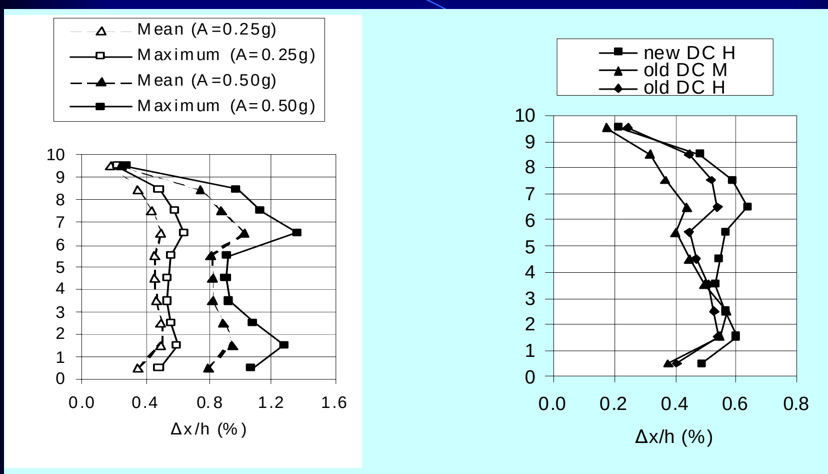
- Takeda model for members with N≅const.
- Bilinear with M_y-N interaction if N=n(t)

Failure criteria

- ***Local*** (member failure)
  - (i) Rotational capacity check: $\theta_p = k_V (\varphi_u - \varphi_y)(k_\mu l_{po})$
  - (ii) Shear force exceeding the corresponding capacity of the member at the maximum ductility level
- ***Global*** (storey failure): Dual criterion based on
  - (i) limiting interstorey drift of 2% and
  - (ii) simultaneous development of a sidesway collapse mechanism

Input motions: 6 records from Greece (from 3 earthquakes)
→ scaled to modified spectrum intensity (SI_m)

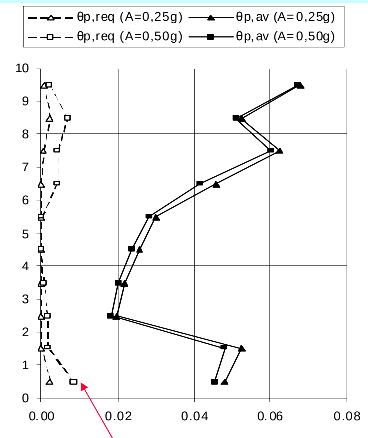
Interstorey drift ratios for frame structures



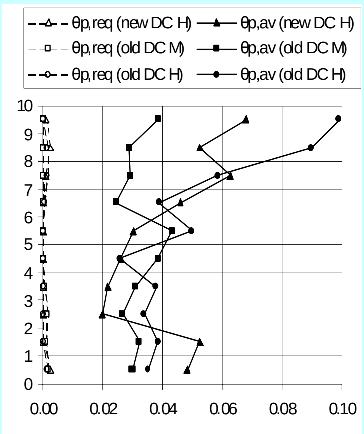
mean and max drifts for new DC H frame

comparison with the old EC8 (A=0.25g)

Required and available plastic rotations in the exterior columns of FR for the most critical motion



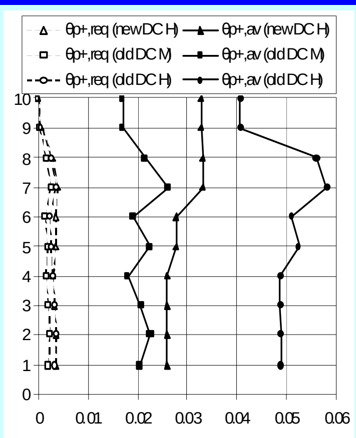
performance of the new DC H frame



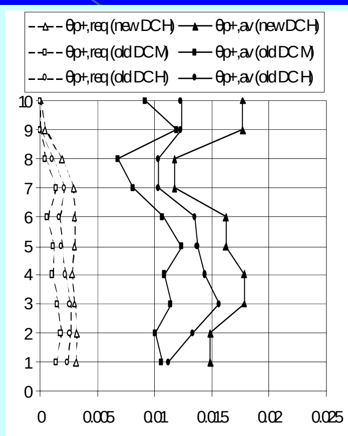
comparison with the 'old' EC8 (A=0.25g)

min $\theta_{p,av}/\theta_{p,req}$ = 5.4

Required and available plastic rotations in the interior beams of FR for the most critical motion

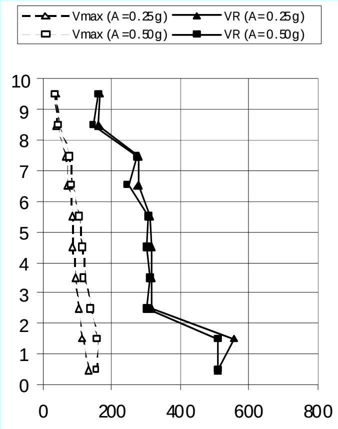


positive values

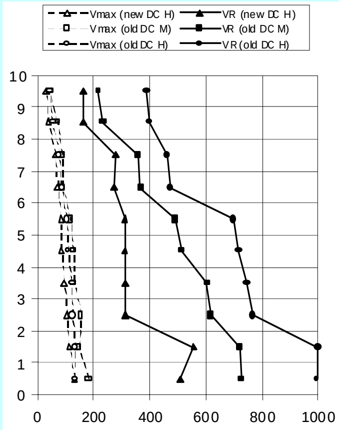


negative values

Required and available shear capacities (in kN) in the *columns* of FR for the most critical motion

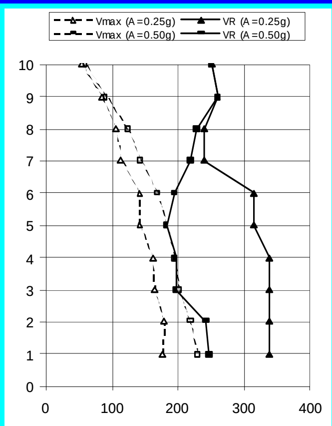


performance of the new DC H frame

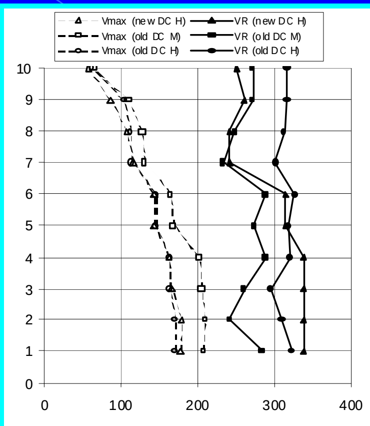


comparison with the old EC8 (A=0.25g)

Required and available shear capacities (in kN) in the *beams* of FR for the most critical motion

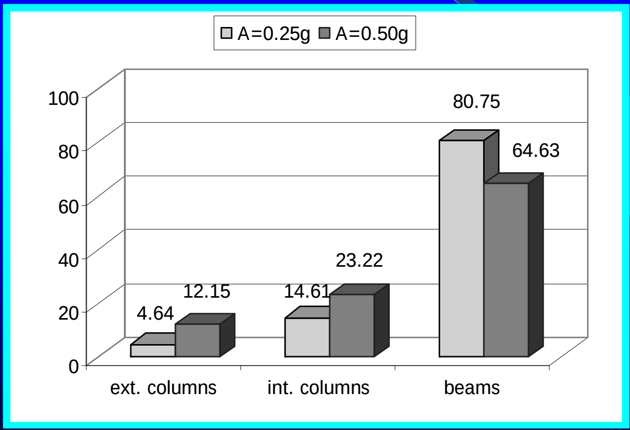


performance of the new DC H frame

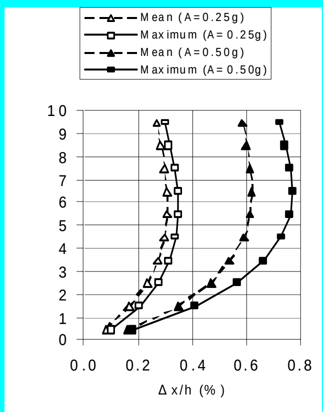


comparison with the old EC8 (A=0.25g)

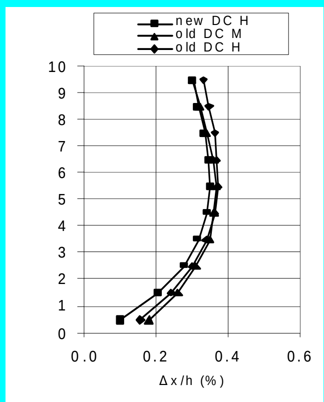
Percentage of the dissipated energy in the structural members of the frame structure



Interstorey drift ratios for **dual** structures

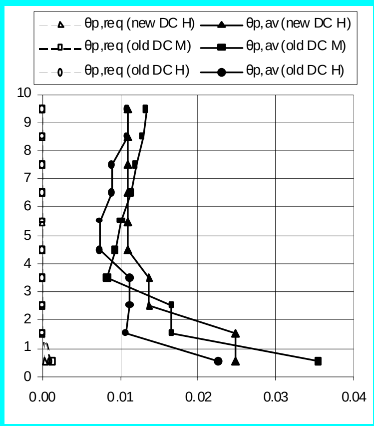


mean and maximum drifts for the new DC H frame

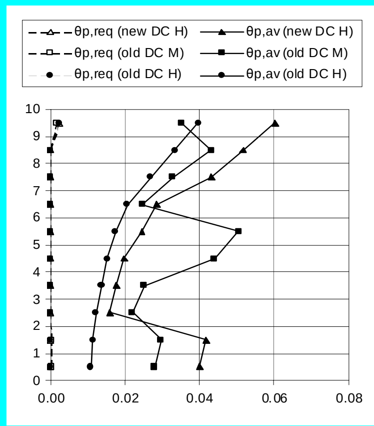


comparison with the old EC8 (A=0.25g)

Required and available plastic rotations in the vertical elements of FW for the most critical motion (A=0.25g)

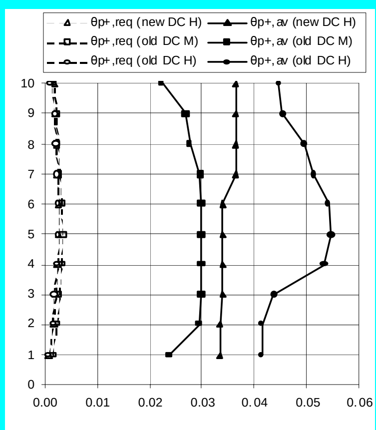


wall

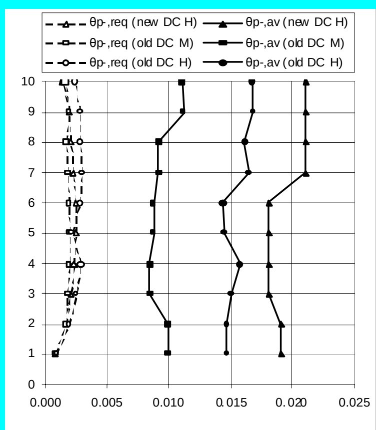


columns

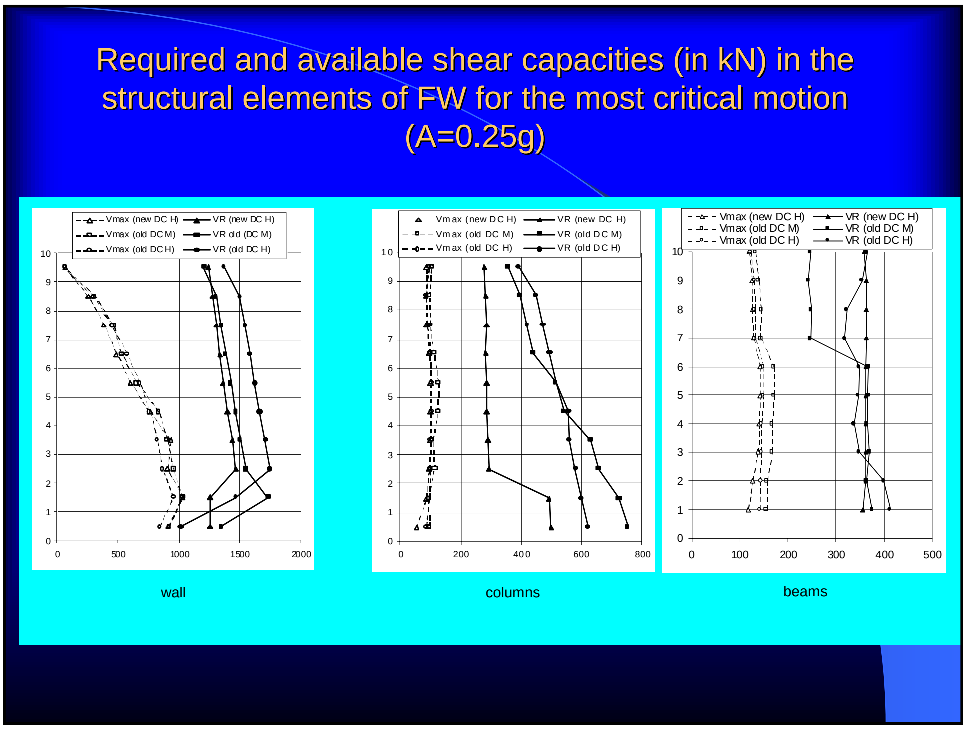
Required and available plastic rotations in the beams of FW for the most critical motion

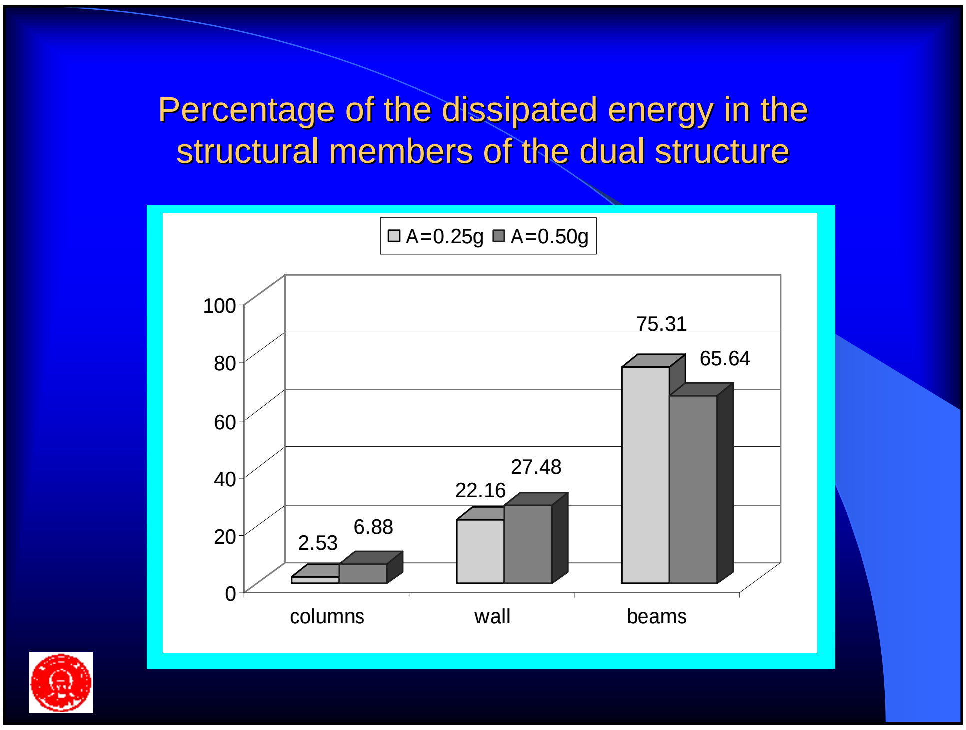


positive values



negative values


Required and available shear capacities (in kN) in the structural elements of FW for the most critical motion (A=0.25g)
Vmax (new DC H)
VR (new DC H)
Vmax (old DC M)
VR old (DC M)
Vmax (old DC H)
VR (old DC H)
wall
Vmax (new DC H)
VR (new DC H)
Vmax (old DC M)
VR (old DC M)
Vmax (old DC H)
VR (old DC H)
columns
Vmax (new DC H)
VR (new DC H)
Vmax (old DC M)
VR (old DC M)
Vmax (old DC H)
VR (old DC H)
beams


Percentage of the dissipated energy in the structural members of the dual structure
A=0.25g
A=0.50g
100
80
60
40
20
0
2.53
6.88
22.16
27.48
75.31
65.64
columns
wall
beams

CONCLUSIONS

- Design of R/C buildings for new DC H (prEN1998-1) appears generally adequate since:
  - seismic performance of both the **frame** and **dual** systems subjected to **design** earthquake was satisfactory with regard to all critical response parameters, i.e.
    - deflections and drifts
    - rotational ductility demands
    - shear capacity
  - the buildings behaved satisfactorily even for **twice** the design earthquake (related to collapse prevention requirement)
  - largest amount of input seismic **energy** dissipated in the **beams** even for the 'collapse prevention earthquake'

CONCLUSIONS (contnd.)

- **Seismic performance**: prEN version satisfactory, generally similar to that of ENV designs, **without being over-conservative** mainly with regard to the design of the vertical elements for the high ductility (DC H) class.
- **Economy**: designing to the new EC8 for DC'H' appears to be **more cost-effective** than to previous versions, since
  - volume of concrete was (slightly) lower
  - quantity of reinforcement (longitudinal, as well as transverse) were lower than in the ENV design;
  - ***main reason***: combination of higher q-factor with less stringent detailing requirements in the new DC H.