Greece – Bulgaria Seminar: Numerical Methods in Earthquake Engineering

ANALYTICAL METHODS FOR SEISMIC ASSESSMENT OF STRUCTURES

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➢ **Seismic assessment**: selection of method depends on the objectives of the assessment programme and the available funds and time-frame

- For assessment of seismic losses (loss scenarios) in metropolitan areas:
  → Empirical (classification methods)
- For prioritization with a view to retrofitting:
  → Empirical (rating methods)
- For the design of interventions (repair / strengthening) in specific buildings:
  → Analytical (+in-situ testing)

Analysis methods: General concepts

- Analytical methods are more accurate, but also more demanding and more expensive than empirical methods!
- Some typical situations, where seismic vulnerability assessment using analysis may be required include:
  - Specific buildings (or other structures) which are particularly important/valuable, or common (e.g. large-scale construction of identical structures).
  - Specific structures or types of structures for which no empirical data are available, because they are new (or even novel) and/or very complex to be assigned to typology classes.
- Typically, analytical assessment is used for specific buildings that should (possibly) be strengthened (post- or pre-earthquake situations)

Analysis methods: Basic elements

- The basic elements of an analytical assessment methodology could be summarised as follows:
  1. Determination of *hazard* parameters, to be used as input to the subsequent analyses
  2. Construction of an appropriate mechanical *model* of the specific structure (or the ‘generic’ structure)
  3. Analysis of the structural model, using the most appropriate computational procedure.
  4. Post-processing of the analysis results, to determine appropriate functionals of the response quantities, able to describe the *damage state* of each structural component
  5. (for loss assessment only): Correlation of the damage functionals determined in the previous step with *losses* (in monetary terms) associated with each component and with the structure as a whole

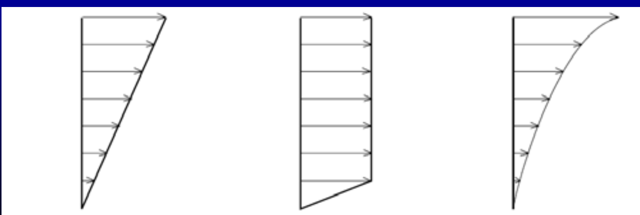
Determination of seismic input/actions

- For elastic analysis:
  - response spectra (for assessment)
    → site conditions etc. should be accounted for!
  - or: equivalent lateral loads (if proper conditions met)
- For assessment, it is common to adopt seismic actions lower than those in the design seismic code (↔new structures), e.g.
  - NEHRP (FEMA 178) Guidelines: $S_{asm} = 2/3\ S_{des}$
  - EC8 1-4 (1995): reduced a_g for redesign, based on:
    - remaining life of the structure
    - higher acceptable probability of exceeding a_g (for optimizing social, economic etc. objectives)

    → this approach is not adopted in EN 1998 -3 (2005)

Determination of seismic input/actions

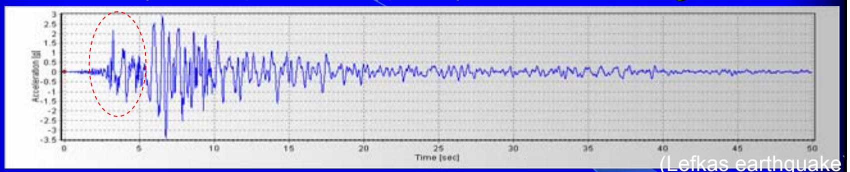
- For inelastic analysis:
  - Static – pushover analysis: distributions of lateral loading along the height required (loading profiles)
  - EC8 (and FEMA356): at least two vertical distributions of the lateral loads should be applied:
    - a “uniform” pattern: lateral forces proportional to mass regardless of elevation (uniform response acceleration);
    - a “modal” pattern: lateral forces consistent with the lateral force distribution (in the direction under consideration) determined in elastic analysis



a) Triangular profile b) Uniform profile c) Higher-mode profile

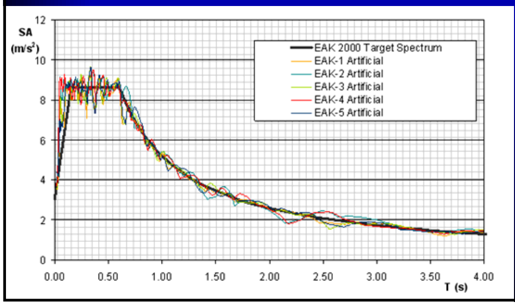
Determination of seismic input/actions

- Inelastic – dynamic (time history) analysis: base accelerograms



(Lefkas earthquake)

- available choices:
  - natural records (actual recordings): How to (*find and*) select?
  - artificial records



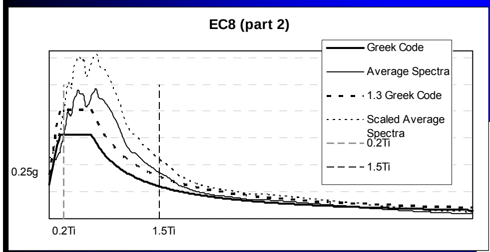
- compatible with (assessment) spectrum ('engineering' approach)
- from fault rupture models ('seismological' approach)

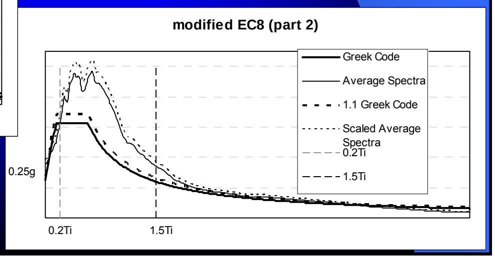
Determination of seismic input/actions

- Best (but not necessarily the most convenient) choice: **natural** records, selected (from databases) on the basis of
  - seismological criteria (magnitude M, distance R)
  - strong motion criteria, e.g. a_g/v_g (reflects frequency content)
  - min number of records required: n=3
  - if n≥7, statistics of the results can be used for assessment
  - records should be scaled ('normalised') to the level of the assessment seismic action – several techniques available:
    - ground motion parameters, usually a_g (convenient, but poor method – large scatter – for structures with T>0.5s), or v_g
    - spectral values: S_{pa}, S_{pv}, SI (spectrum intensity = area under the S_{pv} spectrum from T_1 to T_2)
    - different techniques work better for different period ranges…

Determination of seismic input/actions

- Scaling procedure adopted by EN1998-2 (Bridges)
  - Mean spectrum → Average of SRSS spectra of individual motions
  - Scaling factor → Calculated so that the mean spectrum is not lower than 1.1 times the 5%-damped elastic spectrum of the design seismic action in the period range $0.2T_1$ to $1.5T_1$
  - Different scaling factors for the two directions ($T_x \neq T_y$)
  - → Uniform scaling factor = average of the x and y scaling factors.



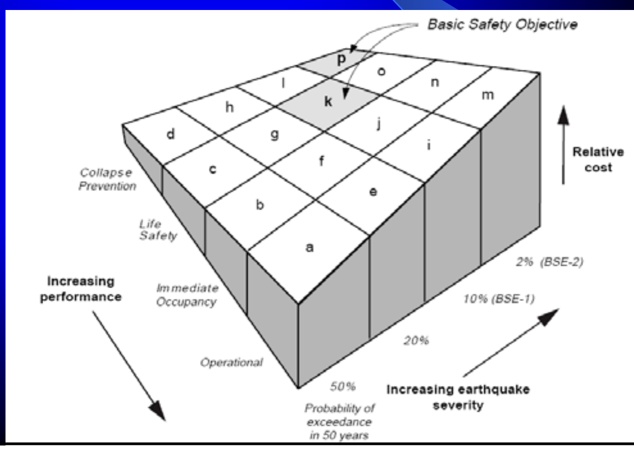


Determination of seismic input/actions

- Current trend: 'Performance-based' assessment
  - different performance requirements adopted (serviceability, life safety, non-collapse)
  - different seismic action levels considered for each performance level

FEMA 274:

Surface showing relative costs of various rehabilitation objectives



Types of analysis methods

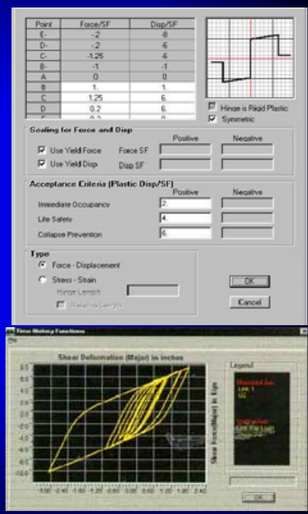
Linear Analysis
(covered elsewhere in this seminar)

- Static
- Response Spectrum
- Time History

Non-Linear Analysis

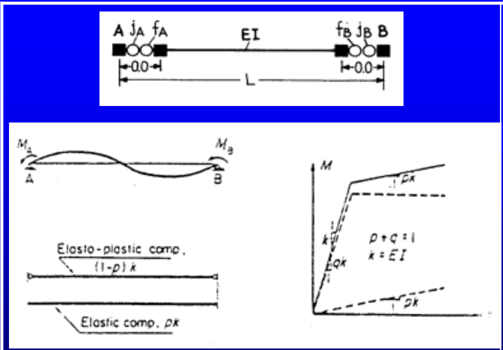
- **Static** (pushover – force or displacement control)

- **Dynamic** nonlinear time history analysis
- Wilson FNA method
- Ground acceleration excitation
- Multiple base excitation
- Load forcing functions
- Transient or steady state



- P-delta analysis
- Large displacement analysis

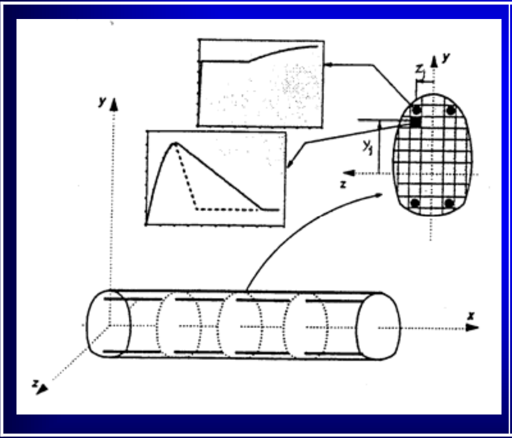
Categories of nonlinear element models



A. Line (1-D) beam-column elements with point (plastic) hinges:

- the most economic
- suitable for microscopic modelling of bond slip, dowel action, etc. (extra springs at the ends)

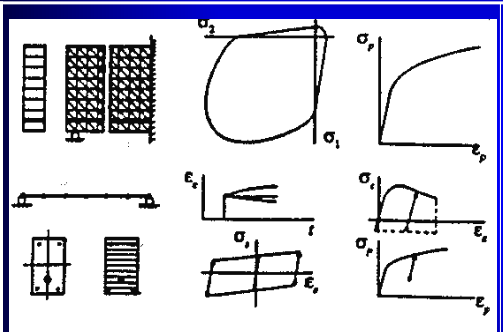
Nonlinear element models



B. Macroscopic finite elements (fibers/layers)

- based on the assumption of constant curvature in each sub-element
- can be applied for the analysis of frames with ~limited number of members

Nonlinear element models



C. Microscopic finite elements (1-D, 2-D, 3-D) “continuum” models

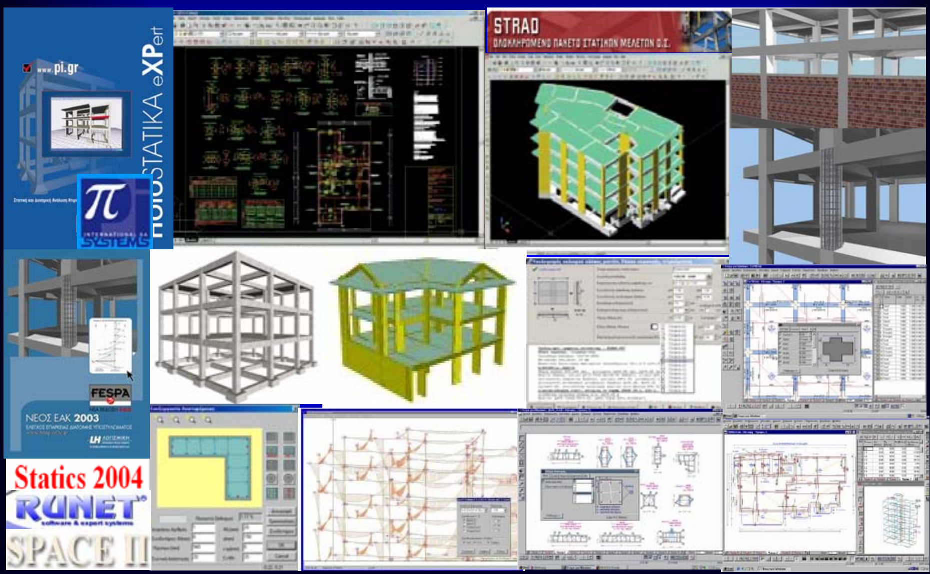
- the most accurate, but also the most expensive
- in general, not suitable for design office practice…

Inelastic static (pushover) analysis

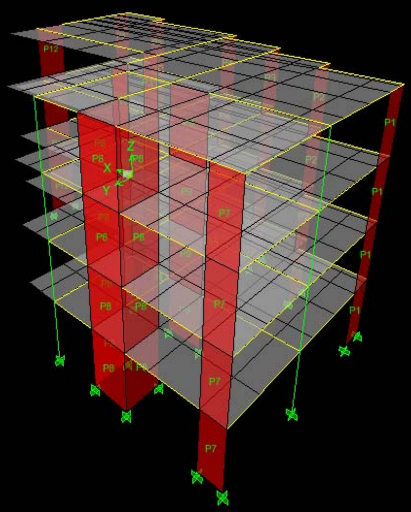
- Inelastic static (pushover) analysis has become a very popular tool for the seismic assessment of structures
- It is implemented in widespread/common assessment methodologies such as ATC40, FEMA273 & 356, HAZUS
- Modern seismic codes and design guidelines (EC8, ASCE-FEMA) introduce the use of inelastic analysis as an alternative to conventional elastic approaches
- The number of software packages supporting inelastic procedures is increasing rapidly

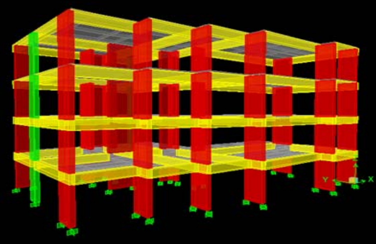
e.g. ETABS and SAP2000 support pushover analysis, mainly following the FEMA273 and ATC-40 guidelines

Professional Programs for Static Analysis & Design



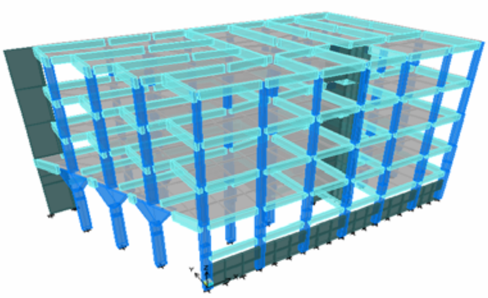
A multi-purpose FE program for buildings - Examples: a 4 storey building in Itea

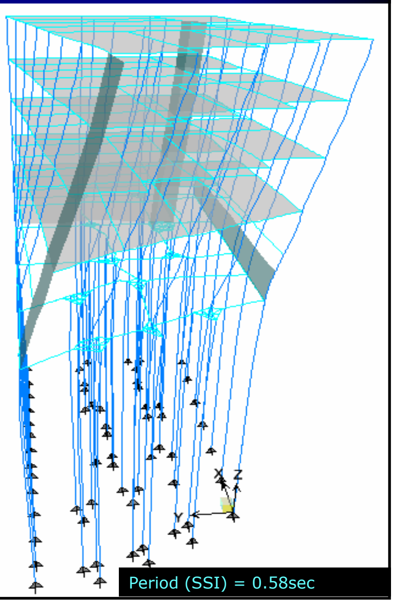




NEMISREF Research Project, Lab. of Soil Mechanics & Foundation Engineering

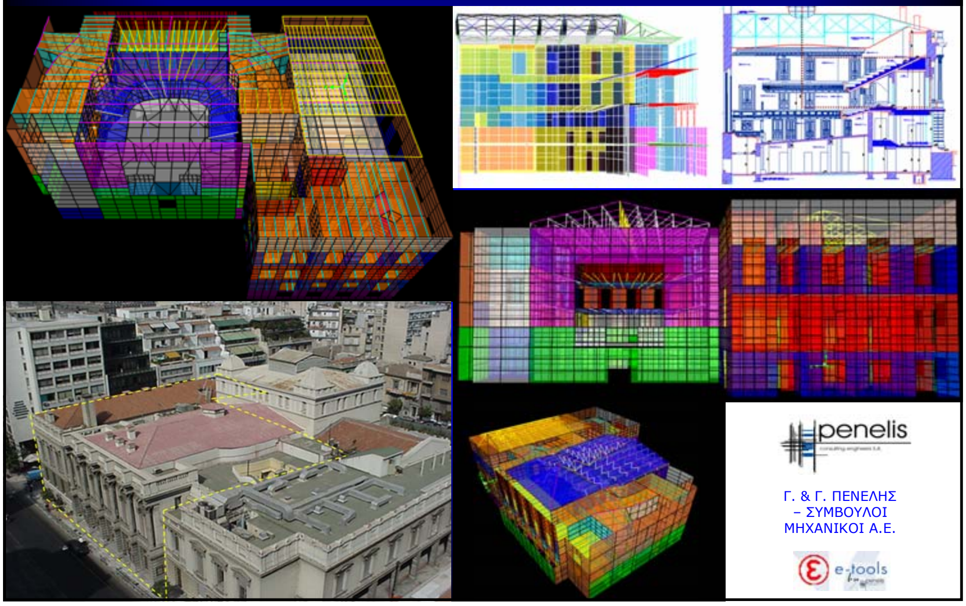
Examples: A 4 storey pile supported building damaged by the Lefkada (2003) earthquake



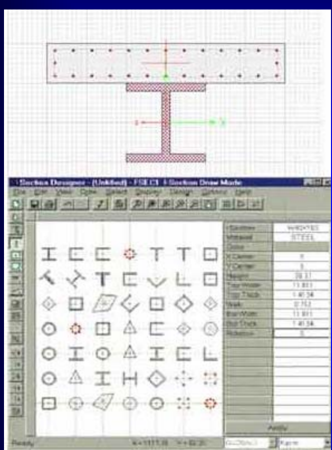


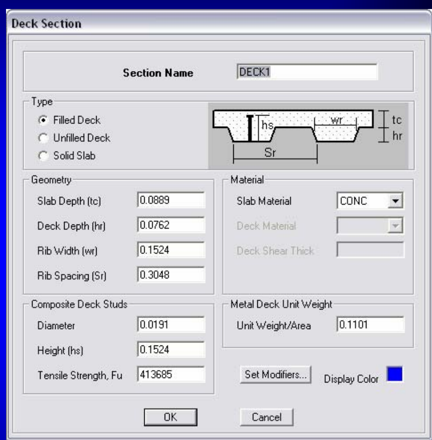
Sextos, Kirtas, Fotaki & Pitilakis (2005) 4th European Workshop on Irregular & Complex Structures

Example: National Theatre in Athens (masonry, concrete, wood & steel building)



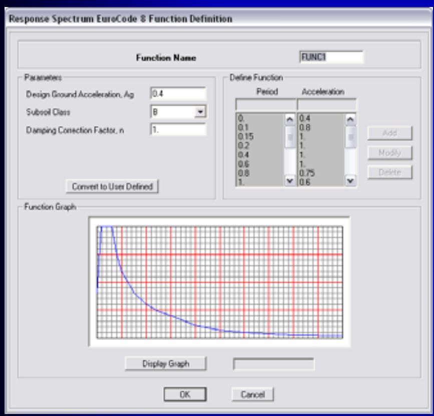
Various, composite and complex elements

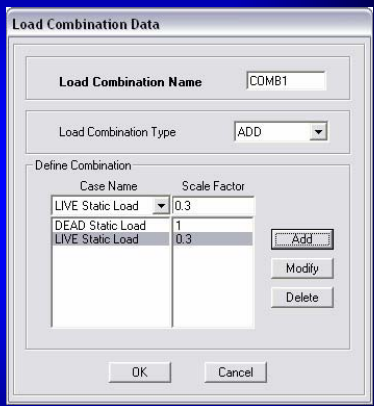




- integrated Section Designer allows definition of complex sections
- interactive composite beam member design for various design codes

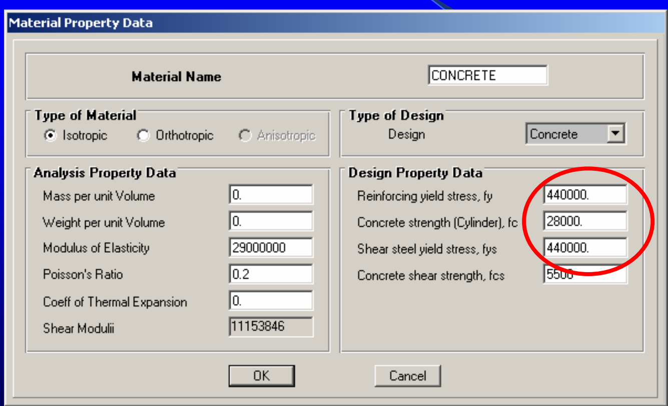
Variety of Loads and Load Combinations





Nonlinear member constitutive laws

ETABS and SAP2000 are able to estimate the moment – rotation (M – θ) curves of the structural elements provided that their material properties and reinforcement are known



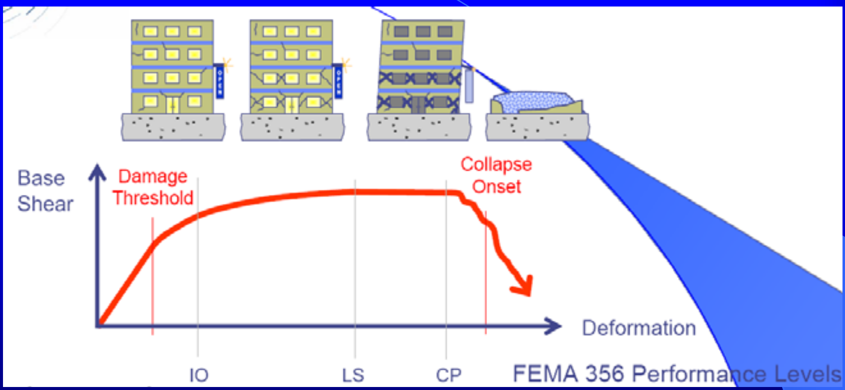
For inelastic analysis mean values of the material strengths are used

Inelastic static analysis: Advantages

Inelastic static (pushover) analysis is expected to provide information on many response characteristics that cannot be obtained from an elastic static or dynamic analysis such as:

- The realistic force demands on potentially brittle elements, such as axial force demands on columns, moment demands on beam-to-column connections, shear force demands in unreinforced masonry wall piers etc.
- Estimates of the deformation demands for yielding elements
- Consequences of the strength degradation of individual elements on the behaviour of the structural system
- Identification of the critical regions in which the deformation demands are expected to be high
- Identification of strength discontinuities in plan or elevation
- Verification of the completeness and adequacy of load path, considering all the elements of the structural system, all the connections (and optionally, if modelled, the stiff nonstructural elements of significant strength such as infill walls and the foundation system)

Definition of damage states in terms of pushover curve regions



- IO : Immediate Occupancy
- LS : Life Safety
- CP : Collapse Prevention

Inelastic static analysis: Limitations

It must be emphasized that the pushover analysis is approximate in nature and is based on static loading. As such it fails to represent dynamic phenomena with a large degree of accuracy

- Higher mode effects are not accounted for→ results may be very inaccurate if their influence is important (tall buildings and/or irregular configuration)
- The use of more than one lateral load pattern reduces but does not eliminate inaccuracy
- It is very difficult to properly include three-dimensional and torsional effects
- The progressive stiffness degradation, the changes in the modal characteristics, the period elongation and the different spectral amplifications are not considered
- Pushover analysis fails to identify failure mechanisms generated after the initial one

Inelastic static analysis: Critical features

Estimation of target displacement to ASCE-FEMA and Greek Code (2007)

$$\delta_t = C_0\, C_1\, C_2\, C_3\, (T_e^2 / 4\pi^2)\, S_{pa}$$

S_{pa}: elastic spectral pseudo-acceleration ↔ based on initial period T_e

C_0: coefficient for correlating $S_d=[T^2/4\pi^2]\cdot S_{pa}$) to δ_t at the top of the building

= 1.0, 1.2, 1.3, 1.4, 1.5, for no. of storeys 1, 2, 3, 5, and ≥10, respectively.

C_1: coefficient for correlating elastic to inelastic displacement ($C_1=\delta_{inel}/\delta_{el}$).

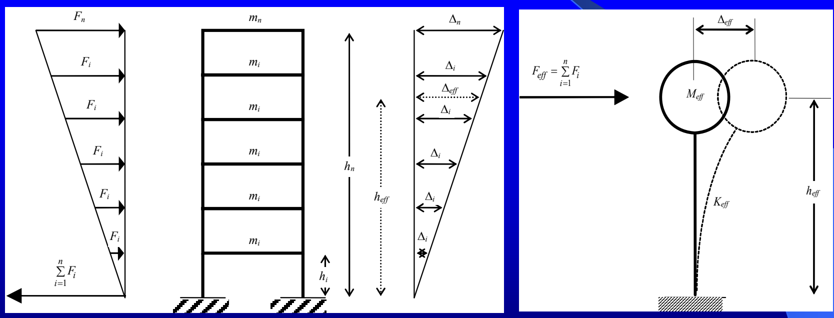
$C_1=1.0$ for $T_e \geq T_2$

$C_1=[1.0+(R-1)T_2/T_e]/R$ for $T_e < T_2$

where $R=V_{el}/V_y$ the ratio of elastic strength demand to the yield strength

$$R = \frac{S_{pa}/g}{V_y/W}\cdot\frac{1}{C_0}$$

Reduction of MDOF structure to equivalent SDOF system



$$M_{eff} = \sum_{i=1}^{n} m_i c_i = \sum_{i=1}^{n} m_i \left(\frac{\Delta_i}{\Delta_{eff}}\right)$$

$$\Delta_{eff} = \frac{\sum_{i=1}^{n} m_i \Delta_i^2}{\sum_{i=1}^{n} m_i \Delta_i}$$

$$a_{eff} = \frac{F_{eff}}{\sum_{i=1}^{n} m_i c_i}$$

$$T_{eff} = 2\pi \sqrt{\frac{\Delta_{eff}}{a_{eff}}}$$

C_2: coefficient to account for the effect of the hysteresis loop shape on the inelastic displacement.

→ Type 1 structures: low ductility members that have poorer hysteretic characteristics than those in Type 2 (high ductility) structures.

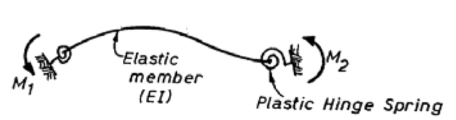
Values of C_2 coefficient in FEMA 273

Performance level	$T = 0.1s$		$T \geq T_2$	
	type 1 structures	type 1 structures	type 1 structures	type 1 structures
Immediate occupancy (serviceability)	1.0	1.0	1.0	1.0
Life safety	1.3	1.0	1.1	1.0
Collapse prevention	1.5	1.0	1.2	1.0

$C_3 = 1+5(\theta-0.1)/T$, where $\theta = M^{II}/M^{I}$ (for R/C structures usually $C_3=1$)

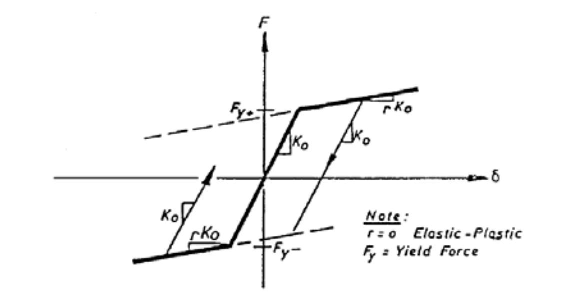
Inelastic **dynamic** analysis: Basic concepts

- Member-type models (1 member = 1 FE) typically used:
  - lumped plasticity, *or*
  - (less common) spread plasticity
  - for infills: diagonal struts *or* shear panels
- Various hysteresis models:
  - elastoplastic, bilinear
  - stiffness-degrading (Takeda, Otani, Q-model…)
- Various integration methods for time history response
  - Newmark β=1/4 (constant acceleration) *or* 1/6 (linear accln.)
  - Wilson θ
  - others…



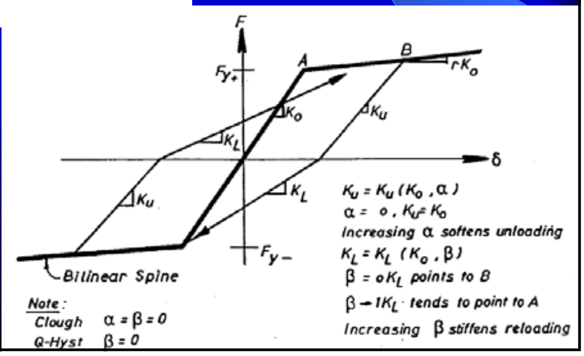
$$[M]\{\ddot{u}(t)+\Delta\ddot{u}\}+[C]\{\dot{u}+\Delta\dot{u}\}+[K]\{u(t)+\Delta u\} = \{P(t+\Delta t)\}$$

Common hysteresis models



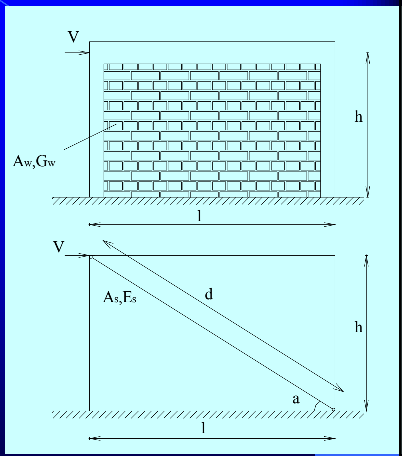
Elasto-plastic
and Bilinear
(not appropriate for R/C!)

Modified Takeda
Degrading Stiffness



Modelling of infill panels

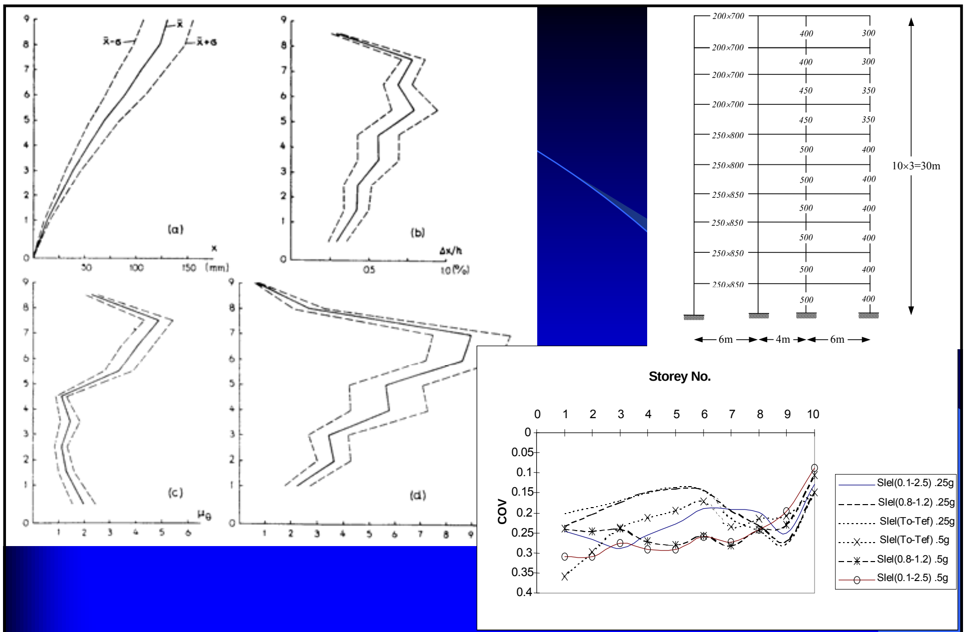
- equivalent diagonal struts *or*
- shear panels
- relationship between axial stiffness of strut (EA) and shear stiffness (GA) of panel $E_s A_s = \dfrac{G_w A_w}{\cos^2\alpha \ \sin\alpha}$
- diagonal struts give more realistic N_{col}
- panels with openings:
  - ignore completely if opening area >50% total
  - for areas between 50% και 0, modelling depends on arrangement of openings…



Inelastic dynamic analysis: Advantages and limitations

- The most accurate, but also the most ‘expensive’ method!
- Uncertainties involved :
  - Assumption made for the stiffness of the elastic part of lumped plasticity member models: calculated interstorey drifts may increase by more than 100 percent (Kappos, 1986).
  - Normalizing of input motions (e.g. to same SI): differences in main response quantities up to about 100 %, but COV≈ 30%, quite uniform along the height.
  - Other input parameters:
    - variability in material strengths (f_c, f_y)
    - assumptions regarding effective shear and axial stiffness etc.

    have smaller effect on calculated response of R/C frames.

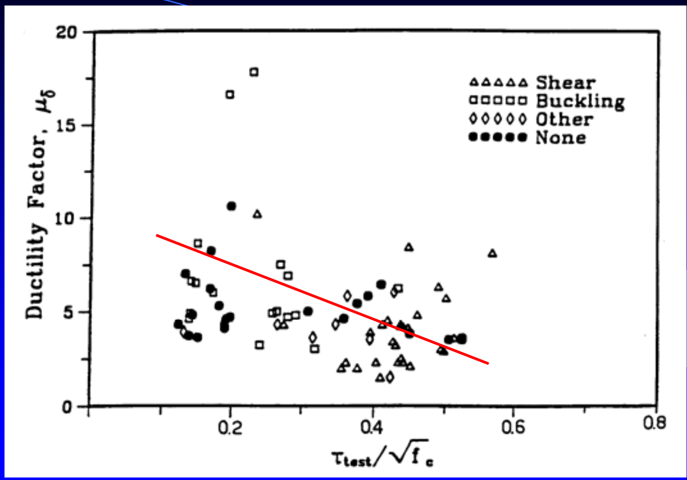


Response statistics for multistorey frame structures subjected to several input motions normalised to the same spectrum intensity

Inelastic static & dynamic analysis: Evaluation of **supplies**

- High degree of uncertainty in the deformational capacity of R/C members, even for the case of monotonic loading!
  - significant scatter in either ductility or drift ratios reflects both uncertainties in the load transfer mechanisms of R/C members under cyclic loading and differences in testing techniques.
  - single most important parameter affecting rotational capacity: level of shear stress (τ) $\rightarrow$ in general ductility decreases with increasing shear stress
  - R/C members subjected to cyclic loading generally fail due to a combination of
    - large deformation (θ_p)
    - low-cycle fatigue (hysteretic energy dissipated)

$$D = \frac{\theta_{max}}{\theta_u} + \beta \frac{\int dE_h}{M_y \theta_u}$$



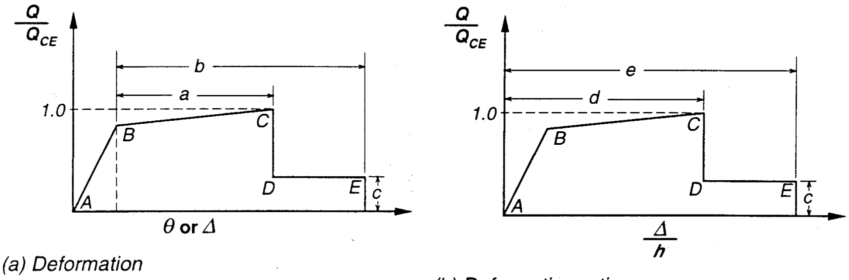
Displacement ductility factor of R/C beams subjected to cyclic loading as a function of shear stress τ=V/(bd) (French & Schultz 1991)

Modelling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures— **R/C Beams**

FEMA 356

Conditions			Modeling Parameters[3]			Acceptance Criteria[3]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.010	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.2	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.0020	0.0030	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.0	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.0020	0.0030	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]										
			0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

* $\sqrt{f_c}$ (psi) = 0.083$\sqrt{f_c}$ (MPa)



(a) Deformation

(b) Deformation ratio

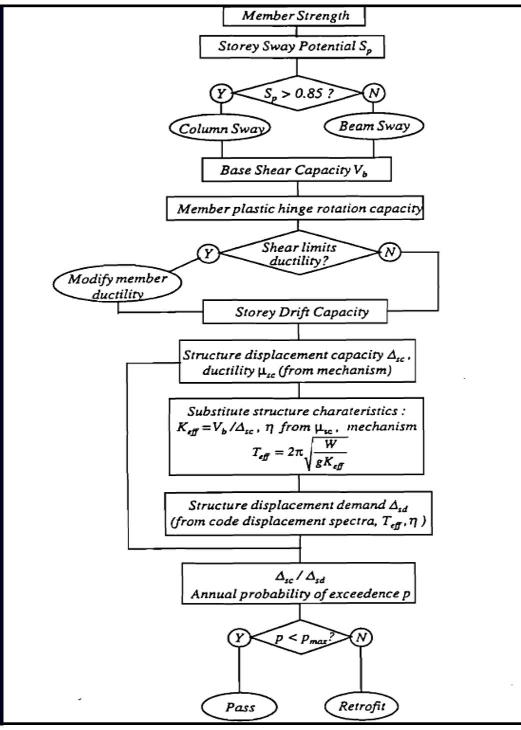
Generalised force vs. deformation relationship (FEMA 273 & 356)

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table shall be permitted.

Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures— **R/C columns**

FEMA 356

Conditions			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008
ii. Columns controlled by shear[1, 3]										
All cases[5]			—	—	—	—	—	—	.0030	.0040
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]										
Hoop spacing ≤ d/2			0.01	0.02	0.4	0.005	0.005	0.01	0.01	0.02
Hoop spacing > d/2			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01
iv. Columns with axial loads exceeding $0.70P_o$[1, 3]										
Conforming hoops over the entire length			0.015	0.025	0.02	0.0	0.005	0.01	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0



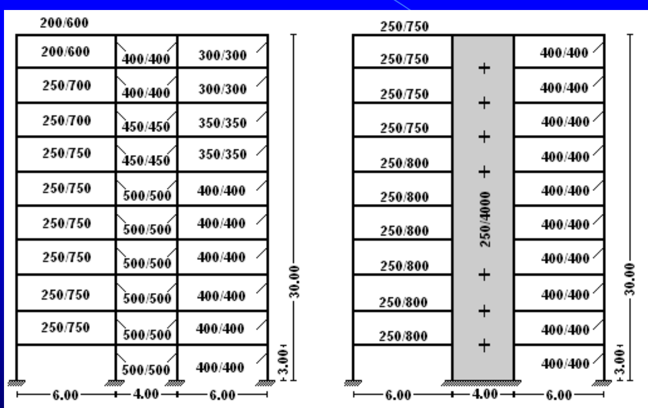
Displacement-based assessment (Priestley 1997)

Examples of nonlinear assessment of buildings

Seismic performance of multistorey R/C buildings designed to the new Eurocode 8 (Kappos et al. 2003)

- Trial application of the new provisions for **DC H** to two typical multi-storey buildings
  - one with a reinforced concrete (R/C) **frame** system
  - one with a **dual** (frame+wall) system
- Same buildings previously designed (Kappos & Athanas-siadou, 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**

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



q=5.85 **q=5.40**

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

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: $\theta_p = k_V (\varphi_u - \varphi_y)(k_m l_{po})$

  (ii) Shear force exceeding the corresponding capacity of the member at the maximum ductility level

- ***Global*** (storey failure): Dual criterion based on

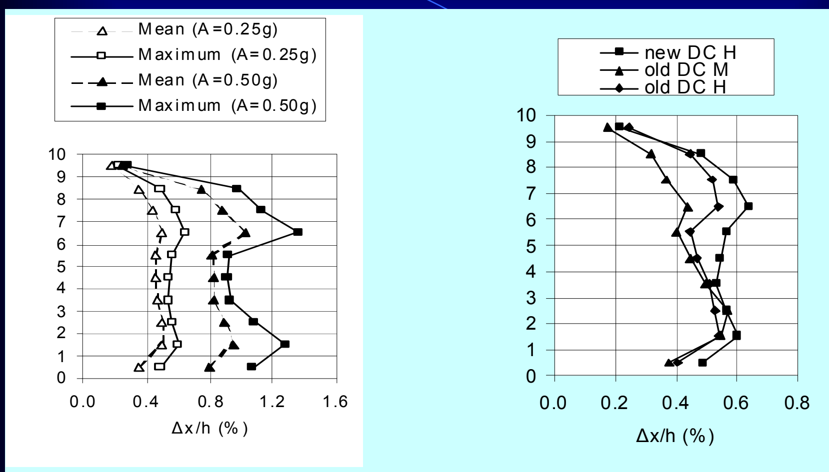
  (i) limiting interstorey drift of 2% and

  (ii) simultaneous development of a sidesway collapse mechanism

Input motions: 6 records from Greece (from 3 earthquakes)

→ scaled to modified spectrum intensity (SI_m)

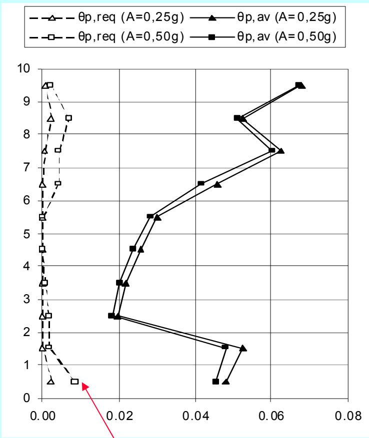
Interstorey drift ratios for frame structures



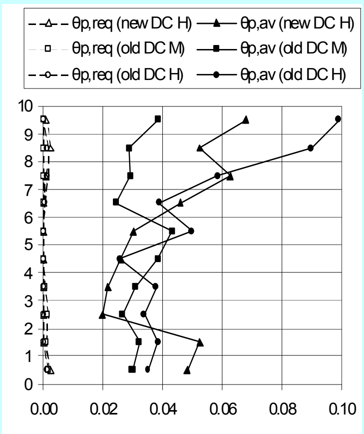
mean and max drifts for new DC H frame

comparison with the old EC8 (A=0.25g)

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



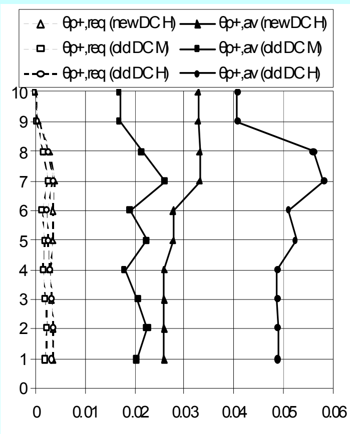
performance of the new DC H frame



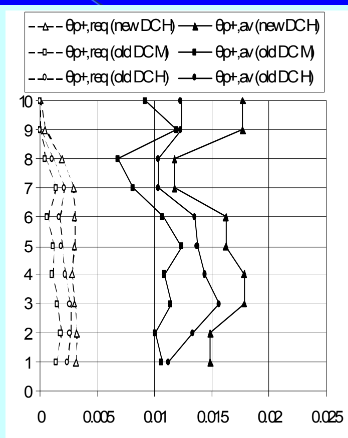
comparison with the 'old' EC8 (A=0.25g)

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

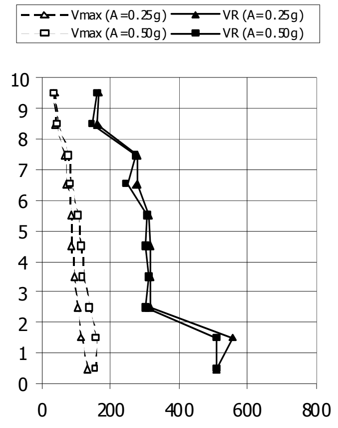


positive values

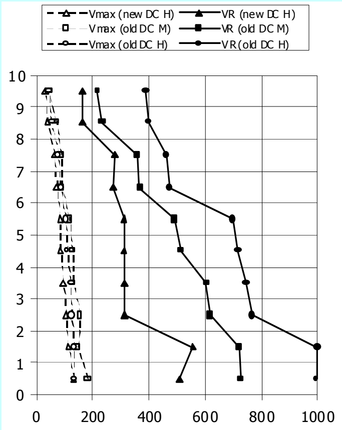


negative values

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

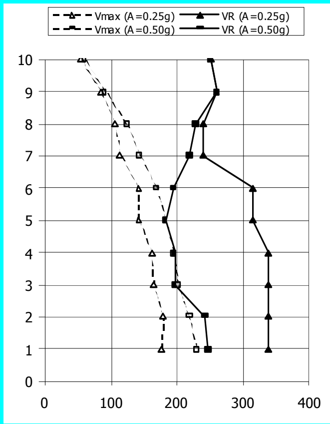


performance of the new DC H frame

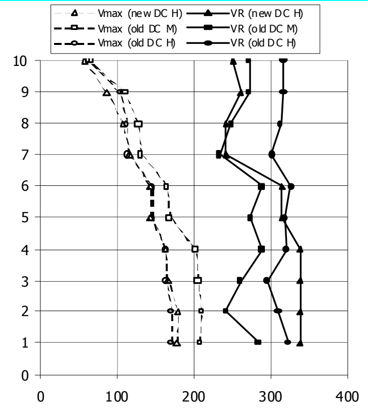


comparison with the old EC8 (A=0.25g)

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

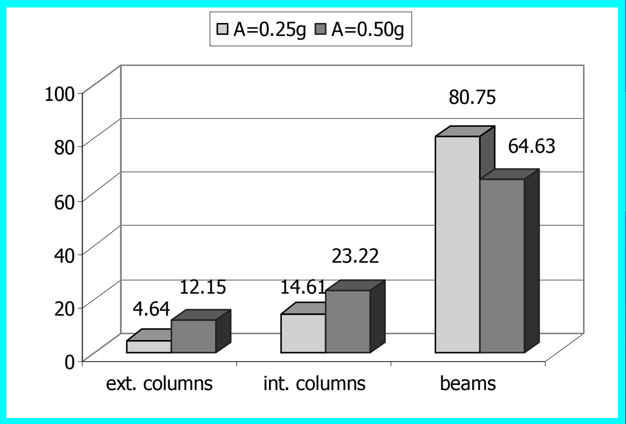


performance of the new DC H frame

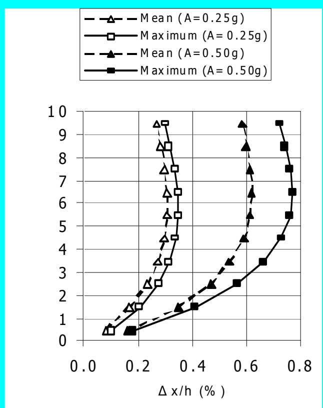


comparison with the old EC8 (A=0.25g)

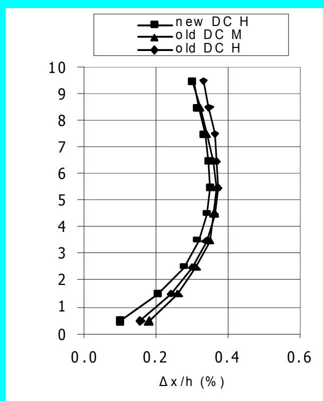
Percentage of the dissipated energy in the structural members of the frame structure



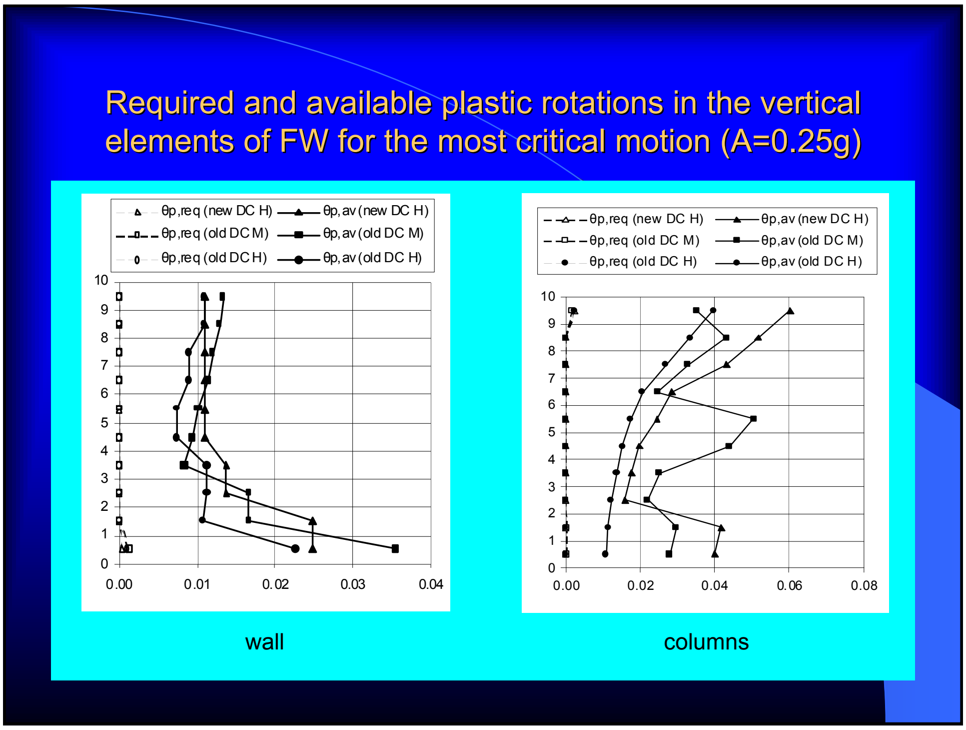
Interstorey drift ratios for **dual** structures

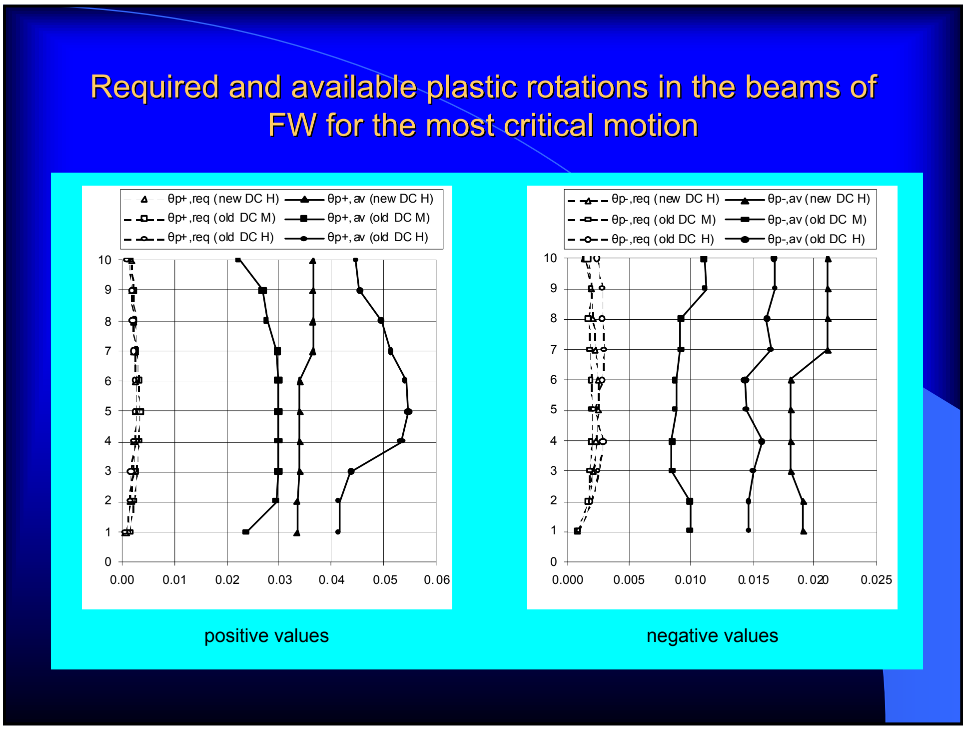


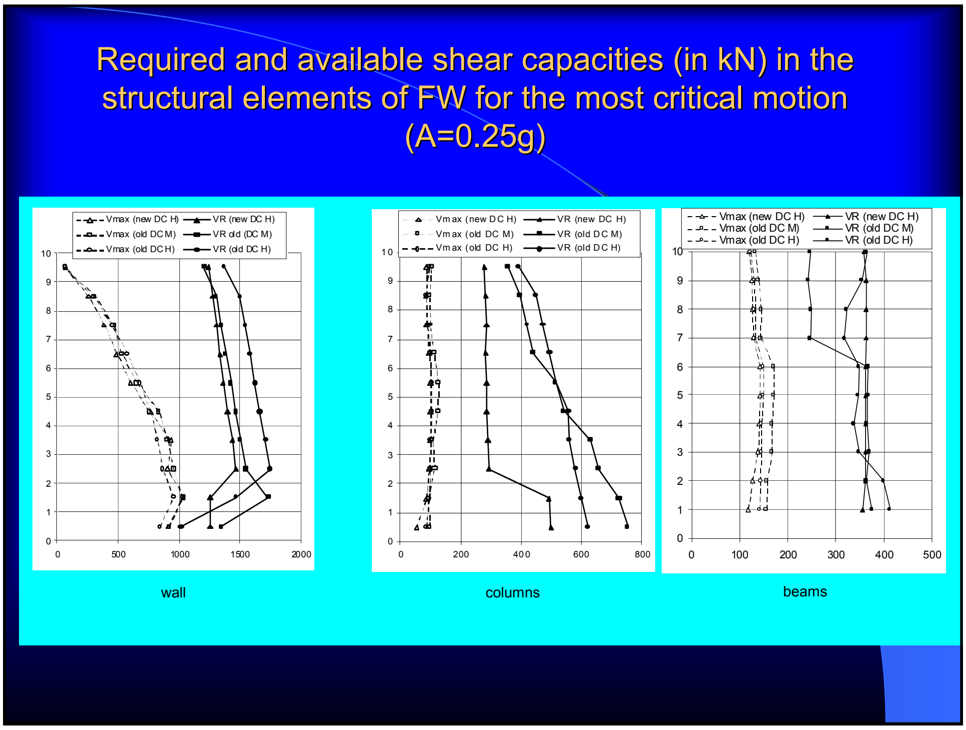
mean and maximum drifts for the new DC H frame

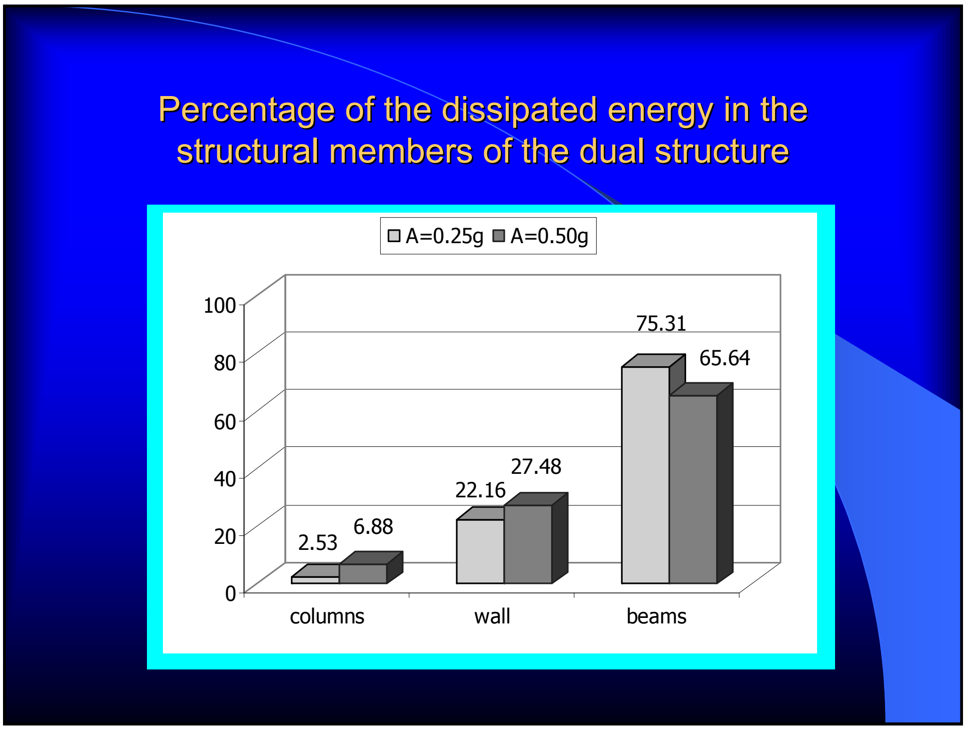


comparison with the old EC8 (A=0.25g)


Required and available plastic rotations in the vertical elements of FW for the most critical motion (A=0.25g)
θp,req (new DC H)
θp,av (new DC H)
θp,req (old DC M)
θp,av (old DC M)
θp,req (old DC H)
θp,av (old DC H)
wall
columns


Required and available plastic rotations in the beams of FW for the most critical motion
θp+,req (new DC H)
θp+,av (new DC H)
θp+,req (old DC M)
θp+,av (old DC M)
θp+,req (old DC H)
θp+,av (old DC H)
θp-,req (new DC H)
θp-,av (new DC H)
θp-,req (old DC M)
θp-,av (old DC M)
θp-,req (old DC H)
θp-,av (old DC H)
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)
Vmax (old DC M)
Vmax (old DC H)
VR (new DC H)
VR old (DC M)
VR (old DC H)
wall
Vmax (new DC H)
Vmax (old DC M)
Vmax (old DC H)
VR (new DC H)
VR (old DC M)
VR (old DC H)
columns
Vmax (new DC H)
Vmax (old DC M)
Vmax (old DC H)
VR (new DC H)
VR (old DC M)
VR (old DC H)
beams


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