COST C26: Urban Habitat Constructions under Catastrophic Events
WG2: Earthquake resistance

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TYPOLOGY OF SEISMIC MOTION AND SEISMIC ENGINEERING DESIGN

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Introduction

Earthquake forces on structures → a characteristic case where an action can be exceptional and therefore lead to catastrophic events.

It is admitted that there exists a high probability that the value of the seismic forces will at some time exceed the value prescribed in the design.

- Inherent uncertaint nature of the seismic action
- Incomplete or inadequate knowledge of the structural behavior.

Seismological aspects

It has been demonstrated that the characteristics of the ground motions vary between recording stations.
Two main regions with different types of ground motions are considered (Gioncu et al 2000).

- The far-source region.
- The near-source region.
  - The vertical component could be greater than the horizontal ones.
  - The significance of higher vibration modes increases
  - Due to the pulse characteristics of the actions, the ductility demands could be very high.

After Kobe earthquake it has been verified that earthquake loading conditions in the near-source region subject buildings to more severe conditions than previously assumed.

Influence of the ground conditions

The following parameters influence the amplification or attenuation of the seismic action on the structures

- the thickness of the soft and stiff soil layers,
- the shear wave velocities of the rock and soil layers,
- the soil/rock impedance ratio,
- the layering properties of the soil layers etc.

Attention to

- landsliding,
- liquefaction
- surface fault rupture

Structure related items

- Magnification of the seismic action on **short period structures**
- **Connections** in steel structures have been identified as crucial for the structural response after the Kobe and Northridge earthquakes.
- Concrete structures suffer from **micro-cracks** induced by relatively moderate earthquakes that influence the structural response under design-level earthquakes.
- The case of **old existing structures** has to be identified as one where the seismic events may be catastrophic due to the fact they were designed (if so) with old codes, proved to be inadequate.

Overview of the presentation

First part (ground motion – uncertainty)

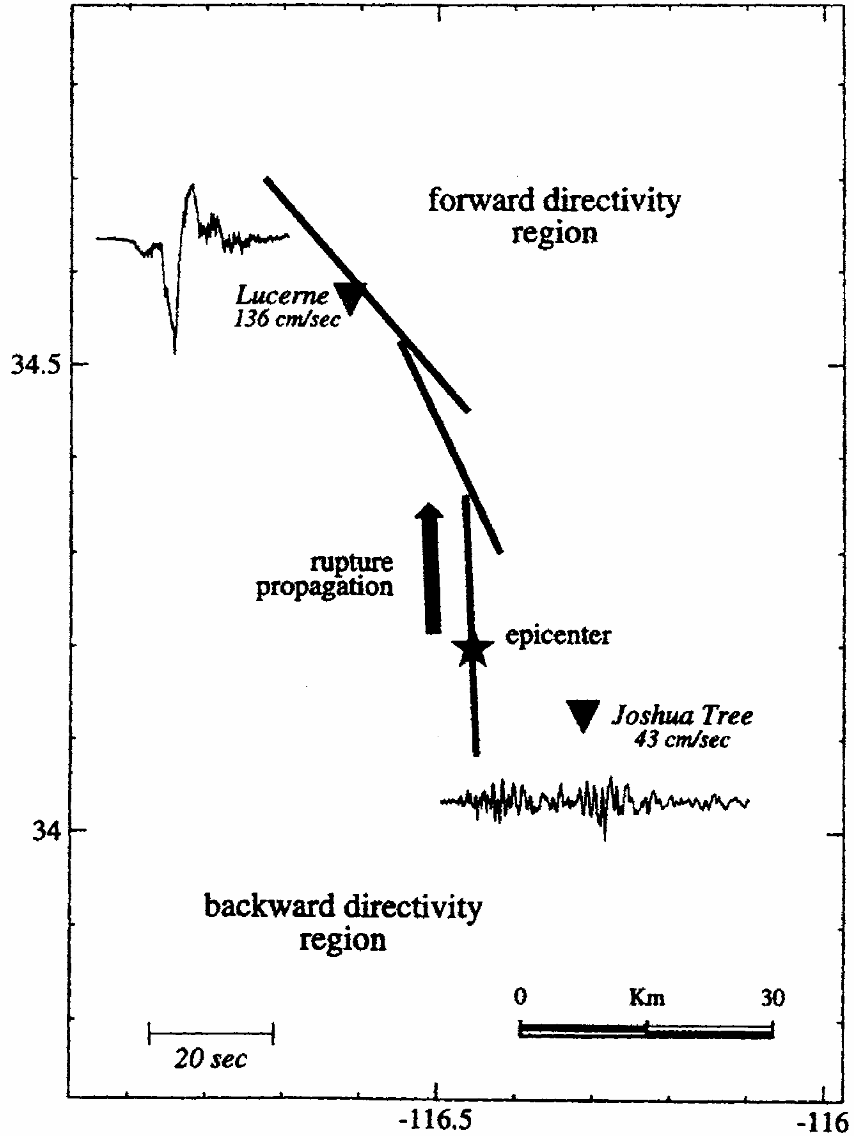
- Seismic motions with specific characteristics that lead to exceptional actions on structures.
- Near-fault ground motions and the local site parameters are examined and the latest developments in the field are presented.
- Modeling of the ground motion specifically for the needs of the seismic analysis of structures.
- Behaviour of structures in the short period range and the corresponding magnification of the seismic action that has been observed.
- Uncertainty in structural analysis (uncertainty in the seismic motion parameters and uncertainty in the model parameters).

Second part (structural behaviour)

- Performance based design as a tool for the analysis of the structural behaviour under extreme seismic events.
- Influence of the connection behaviour in steel structures
- Capacity design methodology for the design and evaluation of the seismic resistance of reinforced concrete structures.
- Direct displacement-based design approach for the design of reinforced concrete structures.

Near-fault ground motions

- Near-field (distance to fault < 20-60 km): site position with respect to the focus is important
- Forward directivity - rupture propagates toward a site: large period, high-amplitude velocity pulse of short duration
- Backward directivity - rupture propagates away from a site: short period, low-amplitude motion, long duration



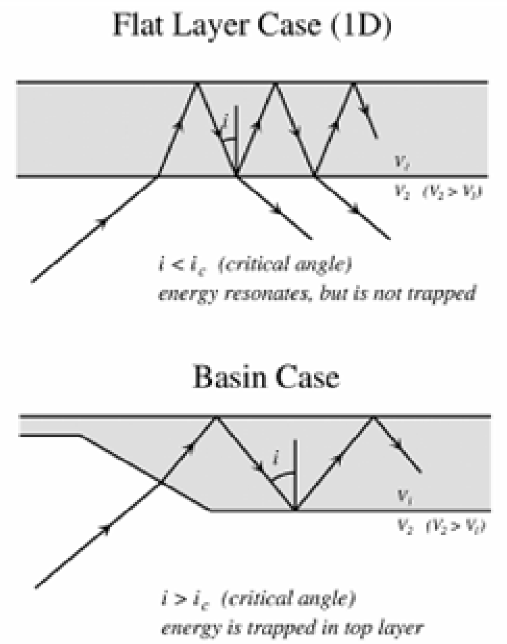
Near-fault ground motions

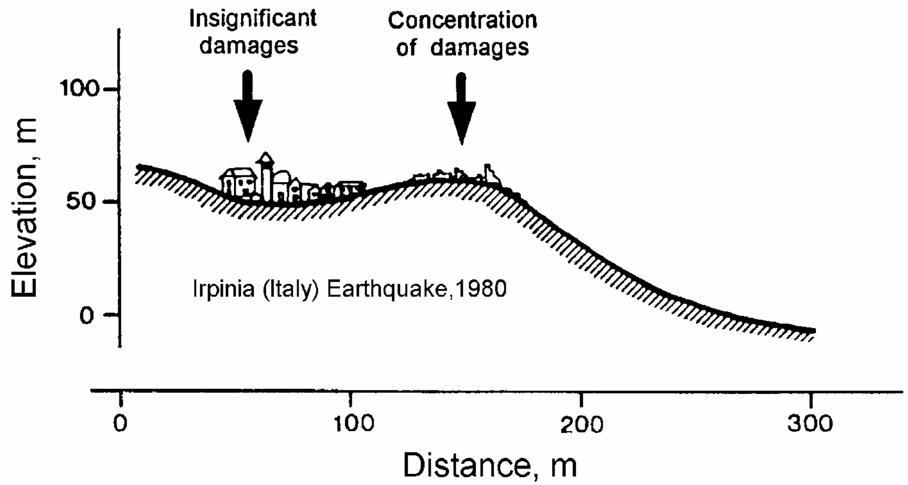
- Near-fault regions:
  - large period, high-amplitude velocity pulse
  - large vertical component
- Near-fault in design codes:
  - scarcely represented
  - when considered (by an amplification coefficient - UBC97), does not account for change in frequency content of the ground motion

Local site conditions

Important parameters affecting ground motion characteristics.

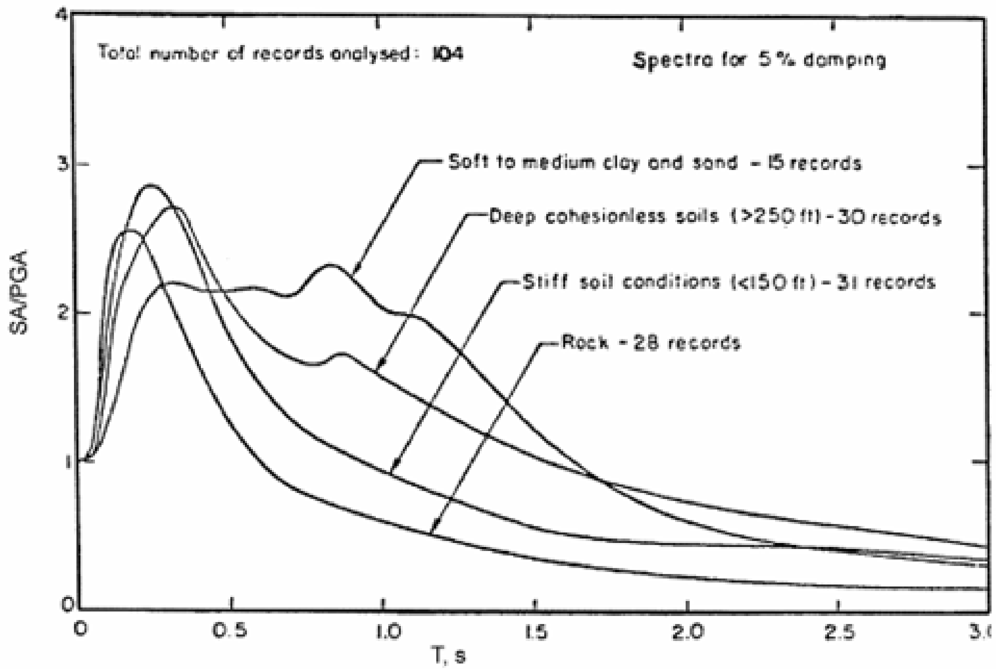
- Basin effects
  - seismic wave may be "trapped" inside the basin
  - amplification and increase of duration of the seismic motions
- Surface topography: amplification of seismic motion for irregular topographies
  - crest
  - canyon
  - slope





Local site conditions

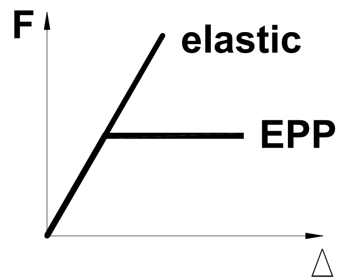
- Frequency content of the ground motion
  - Stiff soil: amplification of spectral accelerations in the short-period range
  - Soft soil: amplification of spectral accelerations in the long-period range



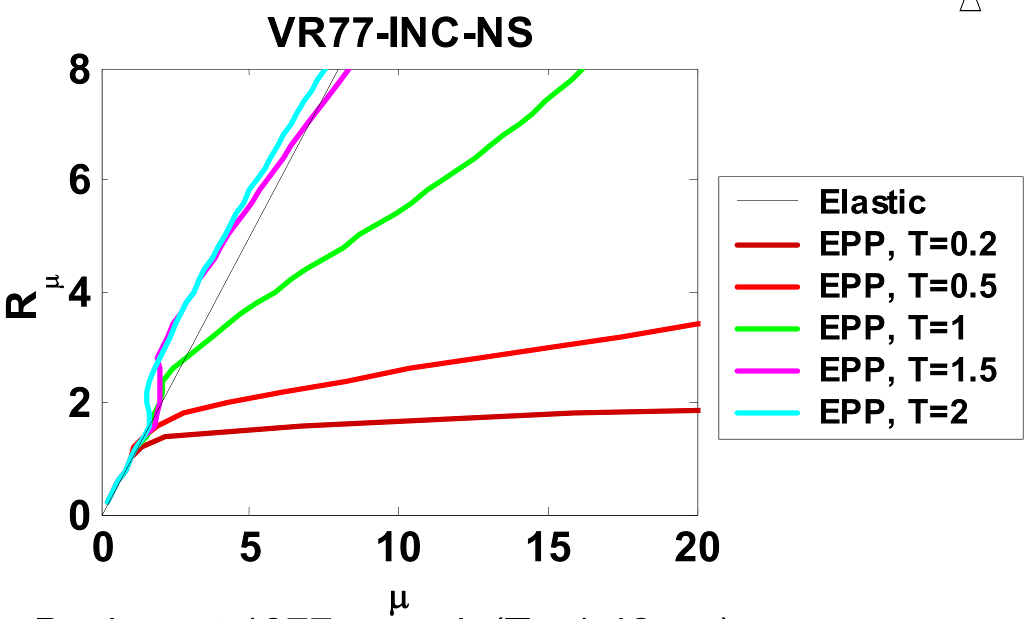
Seed et al, 1976 (NEHRP 2000)

Influence of the frequency content on the inelastic structural response

- Structures designed for seismic forces lower than the ones corresponding to an elastic response
- Components of the force reduction factors $R=R_\mu R_S$:
  - ductility-related R_μ (major contribution)
  - overstrength R_S
- Frequency content strongly influences inelastic structural response:
  - $T<T_C$: "equal displacements"
  - $T>T_C$: "equal energy"



$$R_\mu = \begin{cases} (\mu-1)\dfrac{T}{T_C}+1 & for \quad T \le T_C \\ \mu & for \quad T > T_C \end{cases}$$



Bucharest 1977 record (T_c=1.42sec)

Ground motions with control period larger than the system's period impose large ductility demands

Seismic motions with high T_C values

- Soft soils
- Directivity effect in case of near-field earthquakes
- Check: (Stratan 2003)
  - 496 European records
  - $6.5<M_w<7.8$
  - PGA>0.9 m/s^2

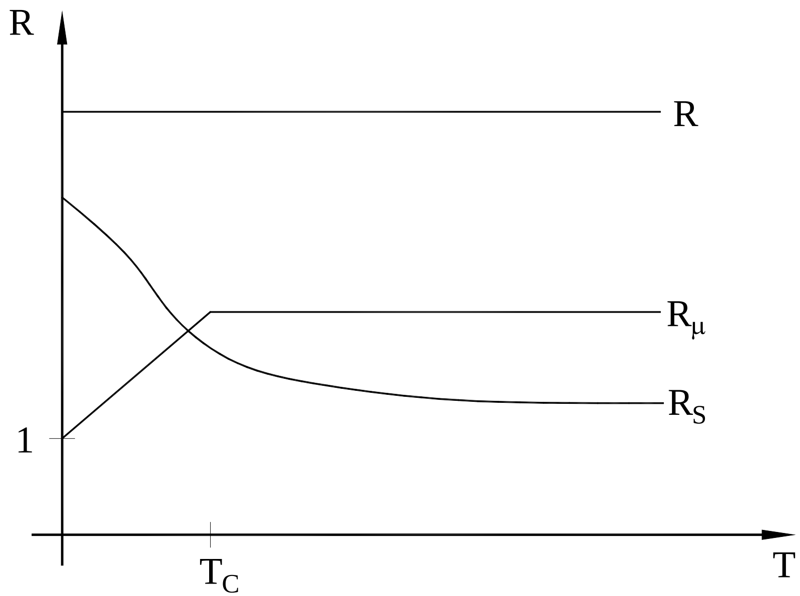
Values of T_C computed and records divided into two groups:

$0.3<T_C<0.4$
All motions were recorded on rock or firm soil

$1.1<T_C<1.7$
Motions recorded on soft soils or were near-field records (<35 km)

Seismic force reduction factors

- In spite of the strong relationship between R_μ and the frequency content of the ground motion, code force reduction factors are constant:
  - empirically based on structural performance in past earthquakes
  - larger overstrength of low-period structures
  - smaller ductility-related force reduction factor for low-period structures



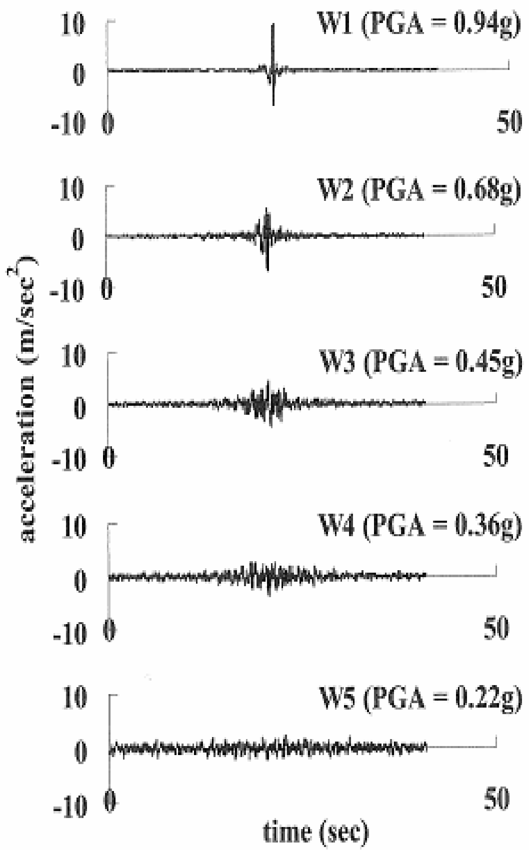
- This simplification may not be correct for ground motions with large values of control period T_C:
  - soft soil conditions
  - directivity effects in near-fault ground motions

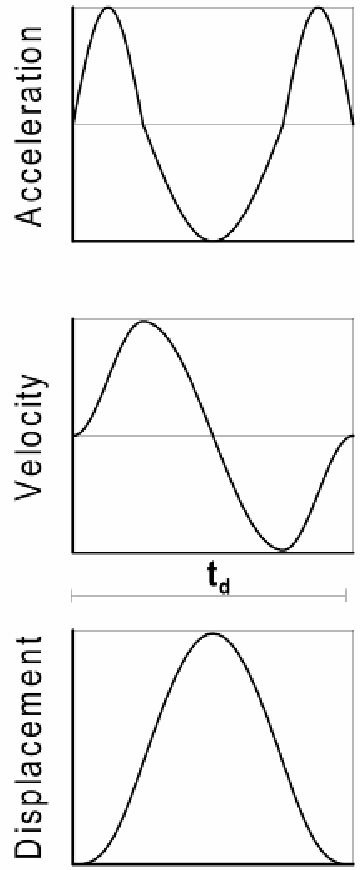
MODELLING OF GROUND-MOTION AND SEISMIC ANALYSIS OF STRUCTURES

Time history representation of ground motion

For many years the structural design was performed with elastic analysis and reduced seismic forces.
The recently introduced nonlinear methods (pushover, time-history nonlinear analysis) require the accurate representation of the ground motion.

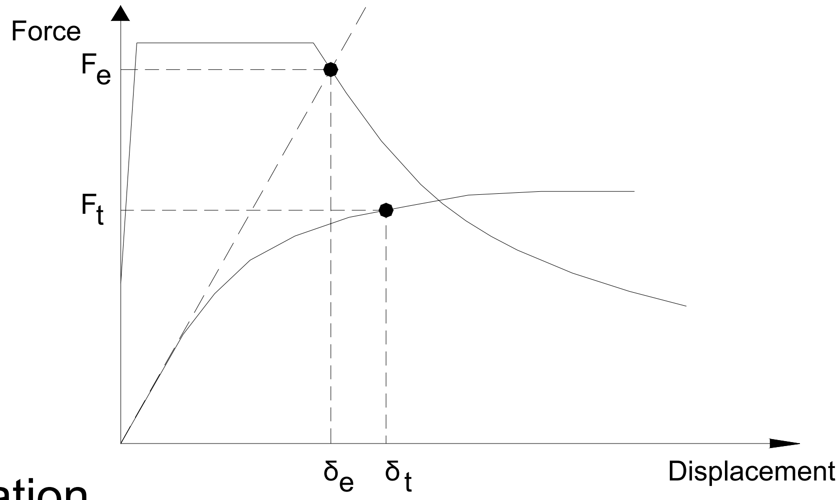
- Recorded accelerograms: should be adequately qualified with regard to the seismogenetic features of the sources and to the soil conditions appropriate to the site.
- Simulated accelerogram: generated through physical simulation of seismic source, travel path, and local site conditions.
- Artificial accelerograms: generated so as to match the code elastic spectrum.
- Simple pulses can be used to model ground motion (especially useful in case of near-fault ground motions).





Magnification of seismic action on short period structures

Reliability of the Displacement Coefficient Method



The DCM is based on the statistical analysis of the results obtained by the time history analysis of SDOF oscillators of various types.

The target displacement is given by the equation

$$\delta_t = C_0\ C_1\ C_2\ C_3\ S_a\ g\ (T/2\pi)^2$$

C_0 : relates the spectral displacement with the displacement of the upper level of the building.

C_1 :takes in to account the magnification of the maximum displacement due to inelastic behaviour.

$C_1 = 1/R + (1-1/R)T_g/T$, με $C_1 < 2$ γιά $T < 0.1 sec$, $C_1 = 1$ για $T > T_C$

R : elastic strength ratio

T_g : characteristic period (dependent on the soil)

C_2 : takes into account the quality of the hysteretic response

C_3 : takes into account the increased displacements when the second-order effects become significant.

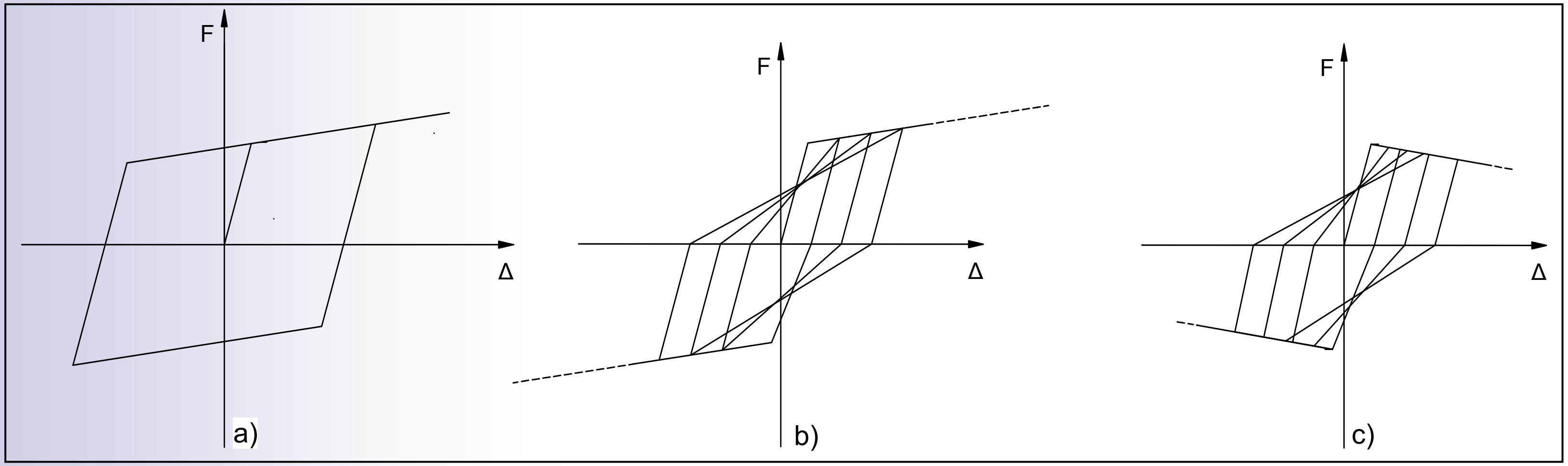
RECORDS USED IN THE ANALYSIS

Selection from a database of 220 records with earthquakes between 1980 and 1994 according to the following criteria:

- Peak ground acceleration (PGA) >0.1g
- Magnitude M_L >4.4 in the Richter scale

Αρ	Code	Date	Magnit.	Station	Comp	PGA (g)	Char.period(sec)
1	ARGO183-1	17/01/1983	M_L=6.5	Argostoli	L	0.171	0.35
2	ATHENS-2	07/09/1999	M_L=5.9	Halandri	T	0.159	0.33
3	ATHENS-3	07/09/1999	M_L=5.9	ΚΕΔΕ	T	0.302	0.5
4	ATHENS-4	07/09/1999	M_L=5.9	ΓΥΣ	L	0.121	0.45
5	ARGO183-7	23/03/1983	M_L=5.7	Argostoli	T	0.192	0.55
6	ZAK188-4	16/10/1988	M_L=5.5	Zante	T	0.170	0.375
7	KAL186-1	13/09/1986	M_L=5.5	Kalamata	T	0.273	0.3
8	EDE190-1	21/12/1990	M_L=5.4	Edessa	L	0.101	0.4
9	ARGO183-8	24/03/1983	M_L=5.1	Argostoli	T	0.305	0.4
10	PAT393-2	14/07/1993	M_L=5.1	Patras	T	0.401	0.35
11	LEF194-1	25/02/1993	M_L=5.1	Lefkas	T	0.136	0.4
12	KYP187-1	10/06/1987	M_L=5.0	Kyparissia	T	0.127	0.25
13	ARGO192-1	23/01/1992	M_L=5.0	Argostoli	L	0.204	0.35
14	PYR193-8	26/03/1993	M_L=5.0	Pyrgos	L	0.165	0.5
15	KAL286-2	15/09/1986	M_L=4.8	Kalamata	T	0.263	0.5
16	LEF188-2	24/04/1988	M_L=4.5	Lefkas	T	0.245	0.3
17	IER183-3	26/08/1983	M_L=4.4	Ierissos	T	0.178	0.5

Models used in the analysis



Type A
Elastoplastic model with 5% hardening.
Corresponds to perfect response and is used here as a reference model.

Type B
Elastoplastic model with 5% hardening but with reduced stiffness.
Characteristic for wall systems with dominant the bending response.
It’s typical for new buildings designed according to newer theories.

Type C
Elastoplastic model with softening behaviour (-10%) and with reduced stiffness.
It’s typical for masonry systems where, for increased displacements, the strength is reduced.

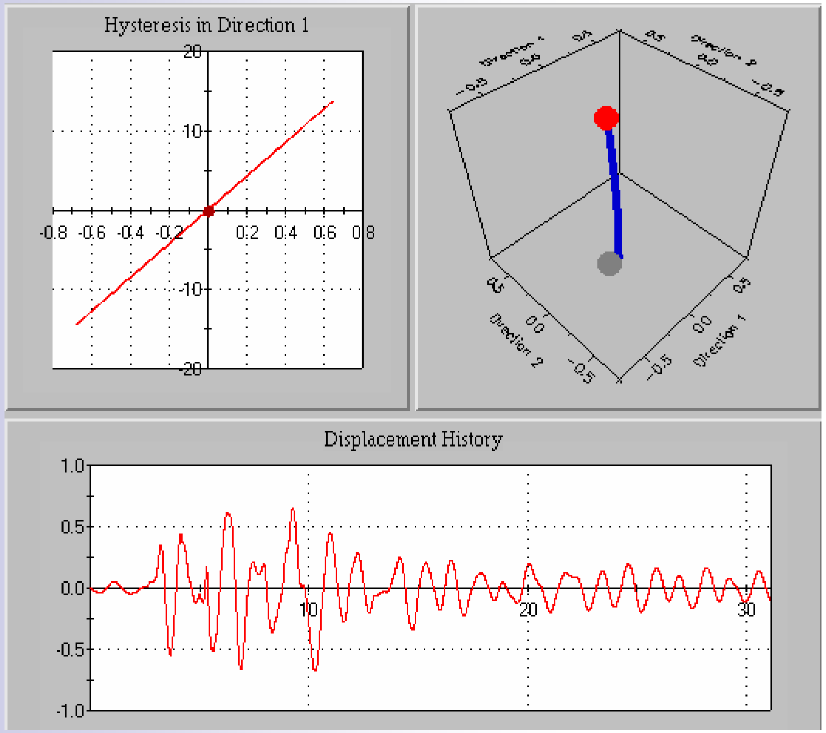
Two types of dynamic analyses were performed

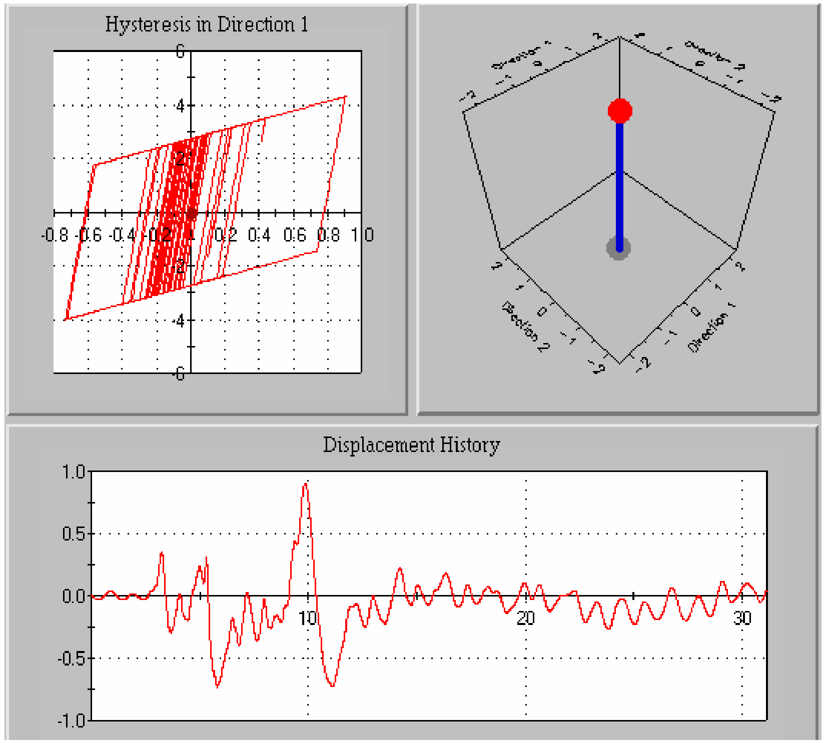
- ➢In the first type the displacement ductility *μ* was considered as constant (*μ*=2,4,6,8)
- ➢In the second type the strength reduction factor *R* was considered as constant (*R*=2,3,4,5,6).

The ratio d_n/d_e is monitored, where

d_n : maximum displacement of the non-linear oscillator

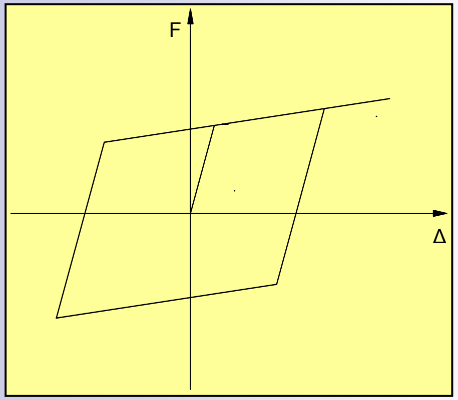
d_e : maximum displacement of the elastic oscillator having the same stiffness with the initial stiffness of the non-linear oscillator.

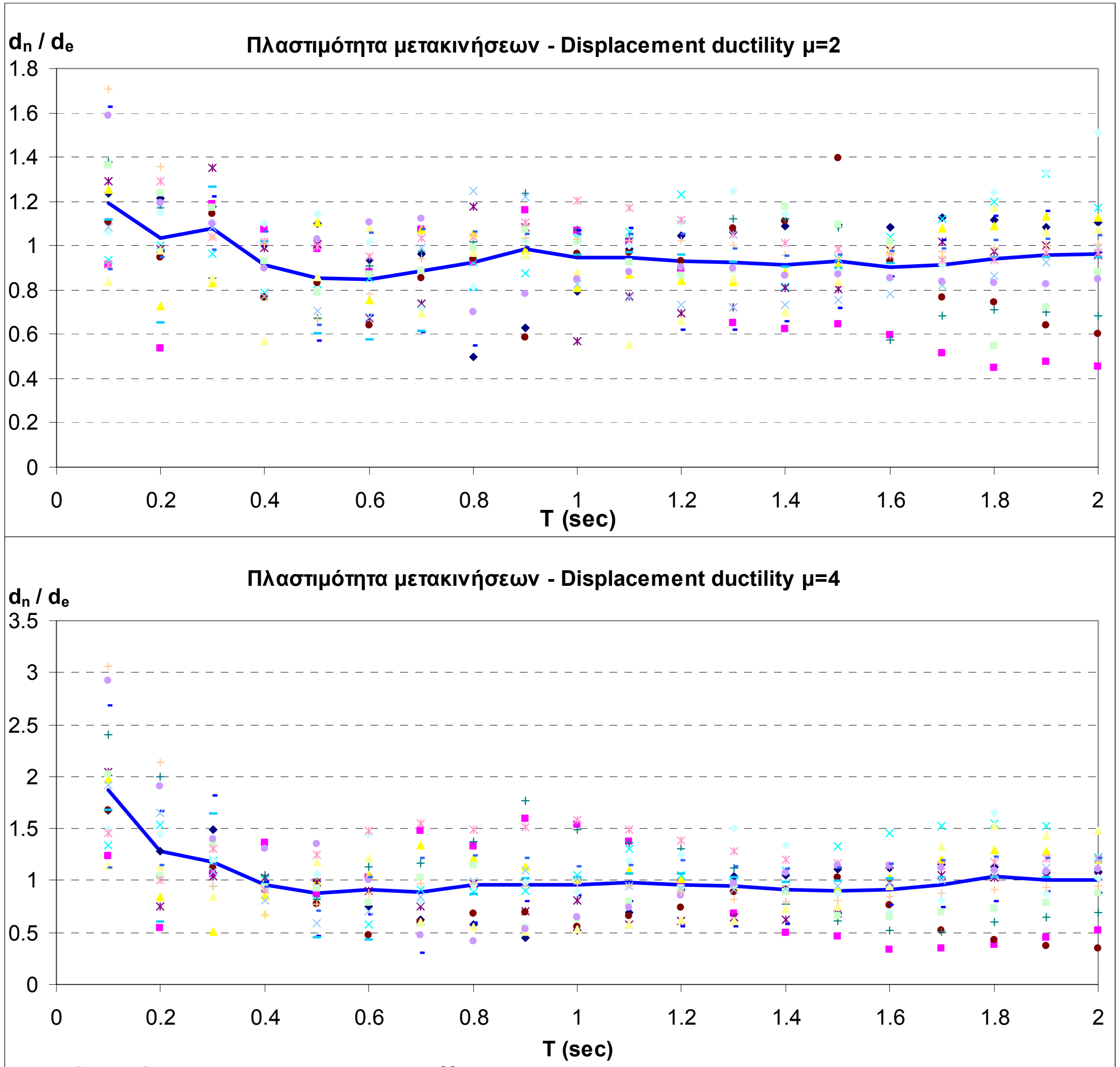




Type A model

Results for constant μ

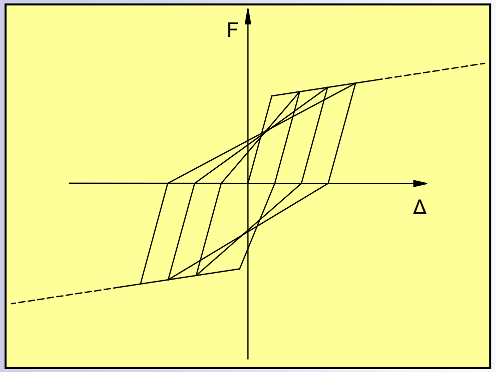


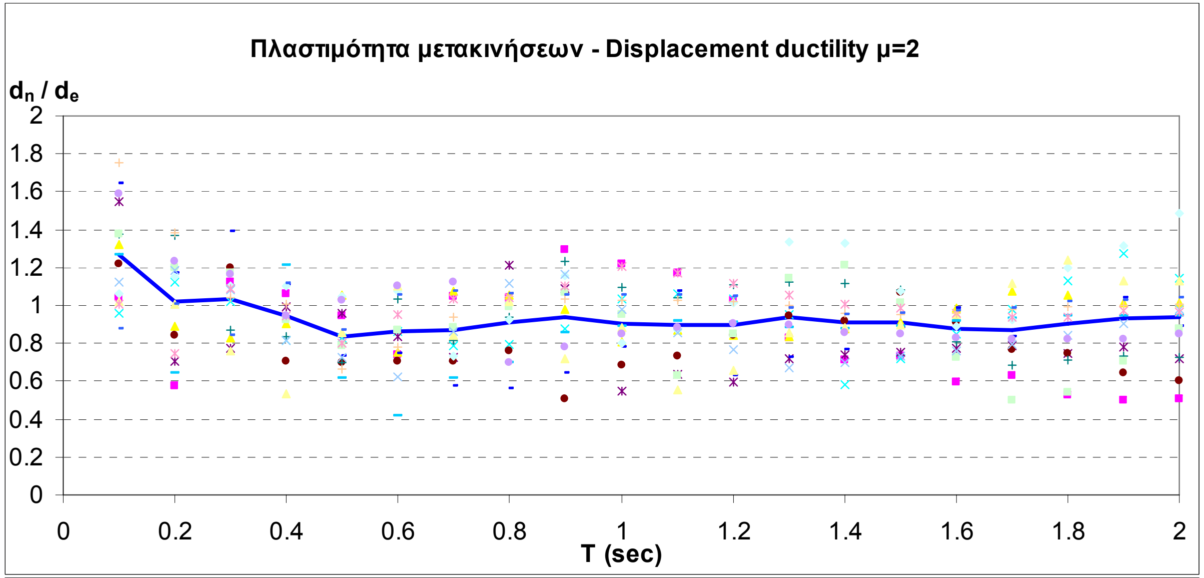


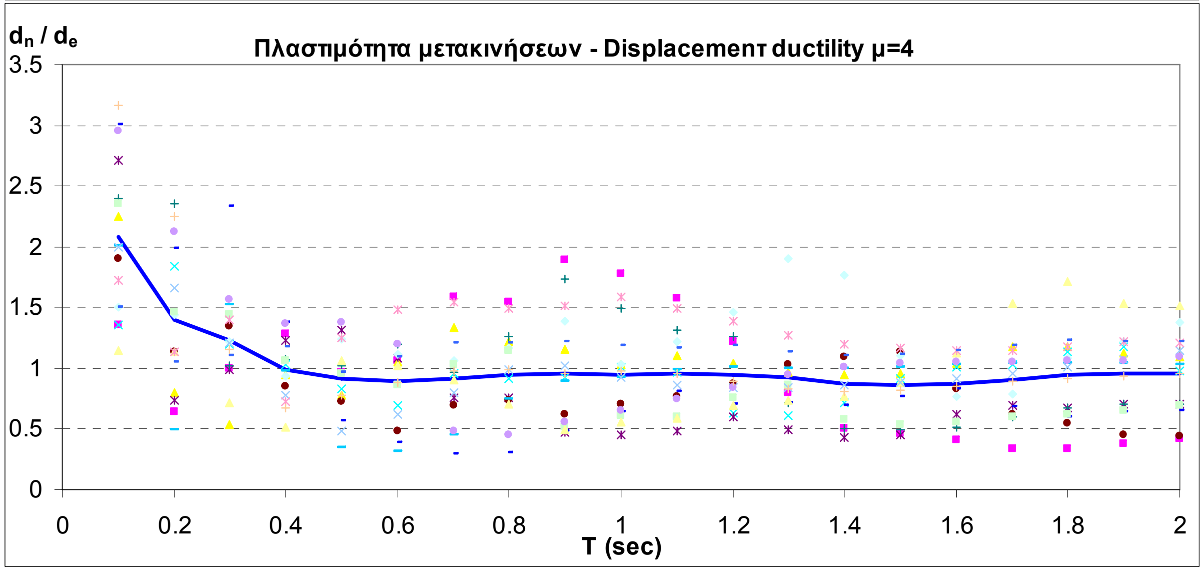
- ➢As the ductility increases, the scattering increases as well.
- ➢For T > 0.4-0.6 sec, the mean values are very close to 1.
- ➢The scattering increases for lower values of the period.
- ➢For T <0.4-0.6sec, the increase of the ductility leads to larger mean values

Type B model

Results for constant μ

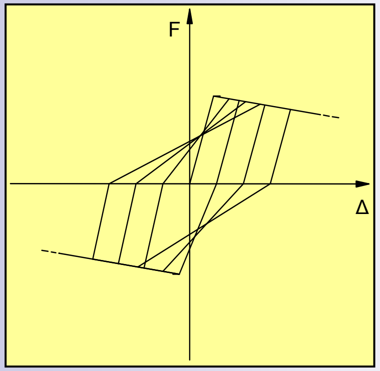


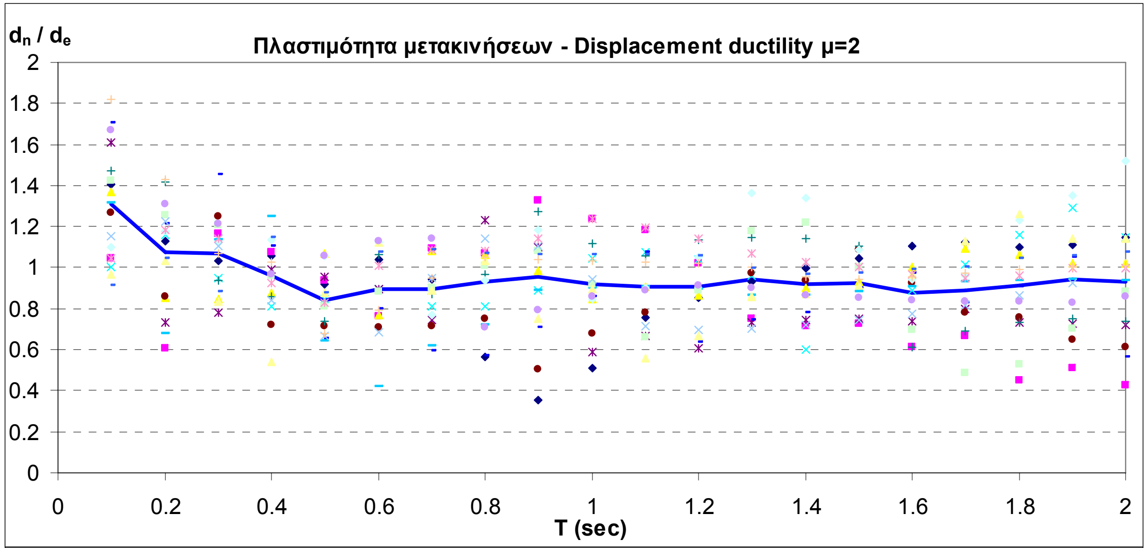


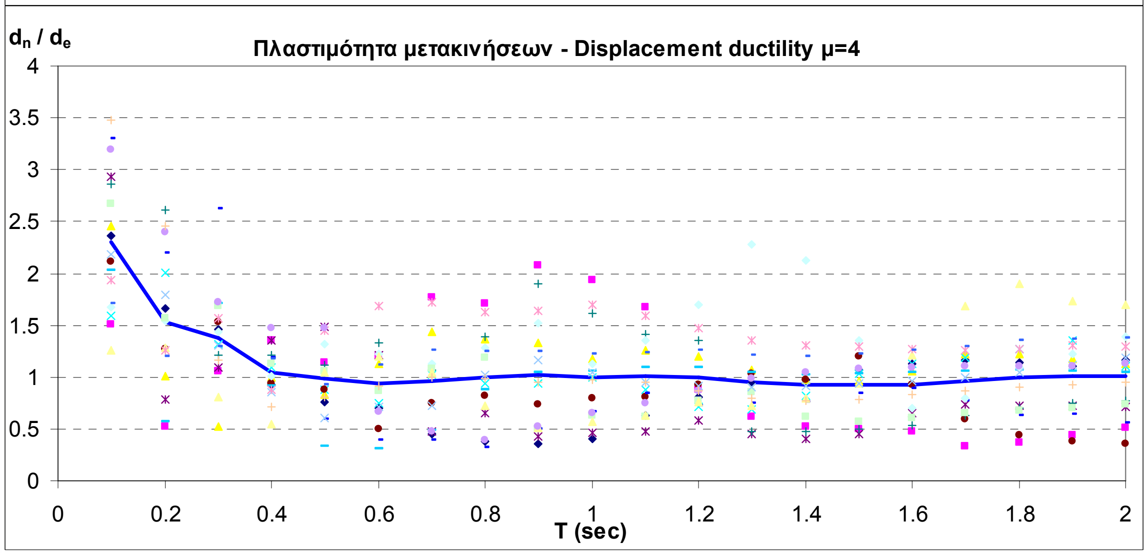


- The mean values for periods between 0.1-0.4 sec tend to be greater than the corresponding ones for the type A model

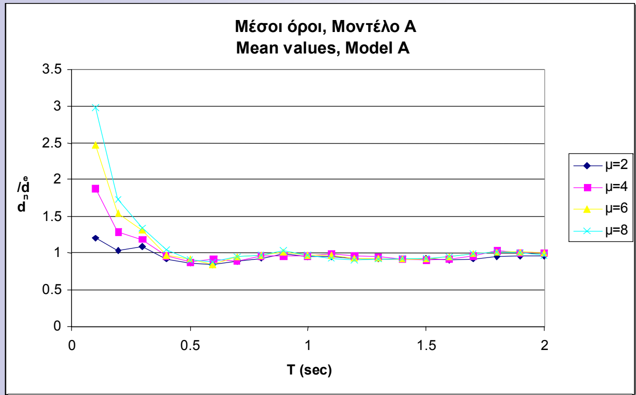
Type C model
Results for constant μ

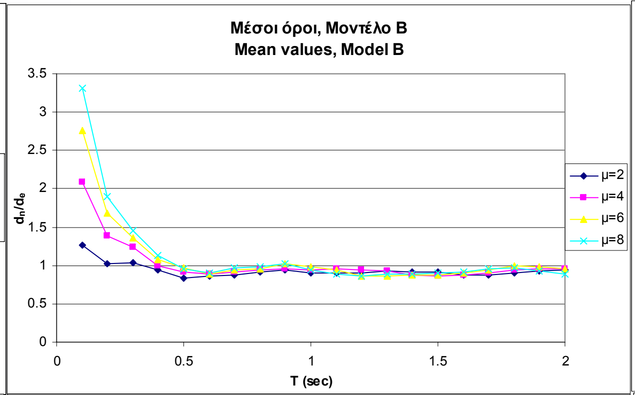


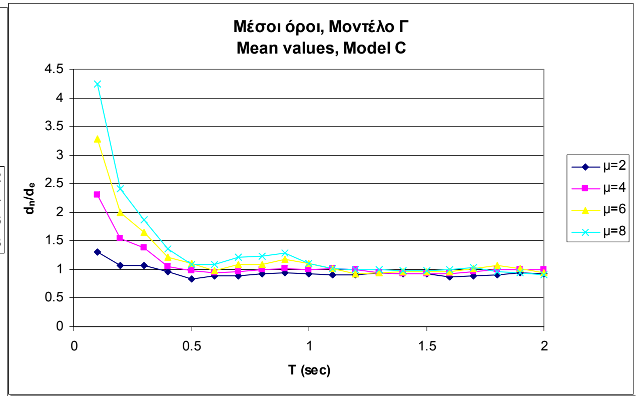


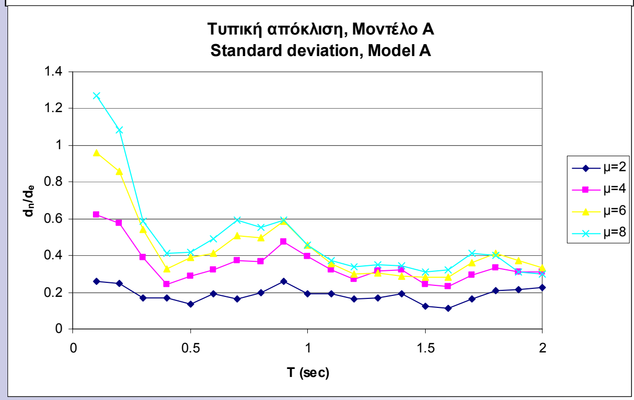


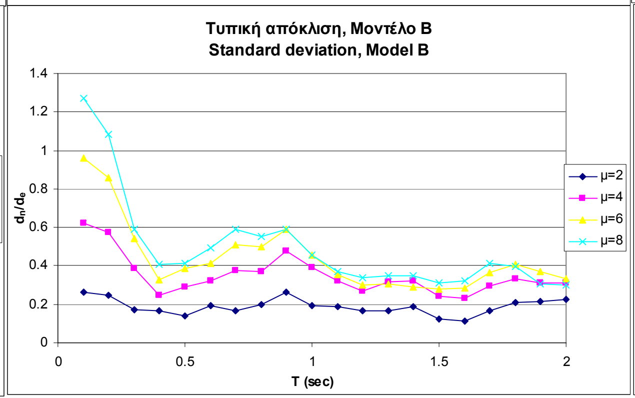
- The scattering increases (values between 0.4 and 6.5)
- The mean values for T between 0.1-0.4 sec are significantly higher than those of the type A model.

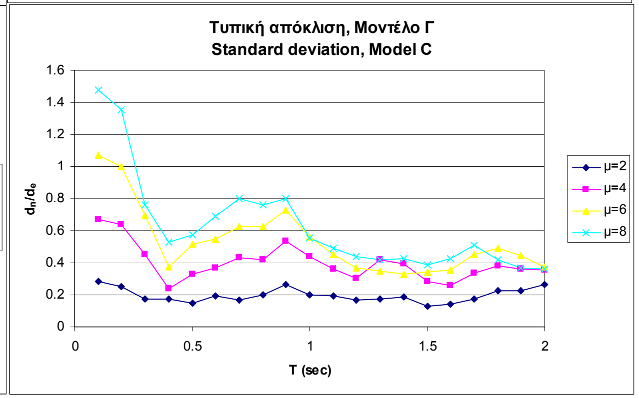








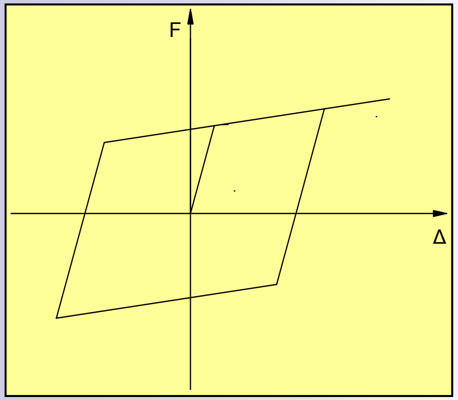


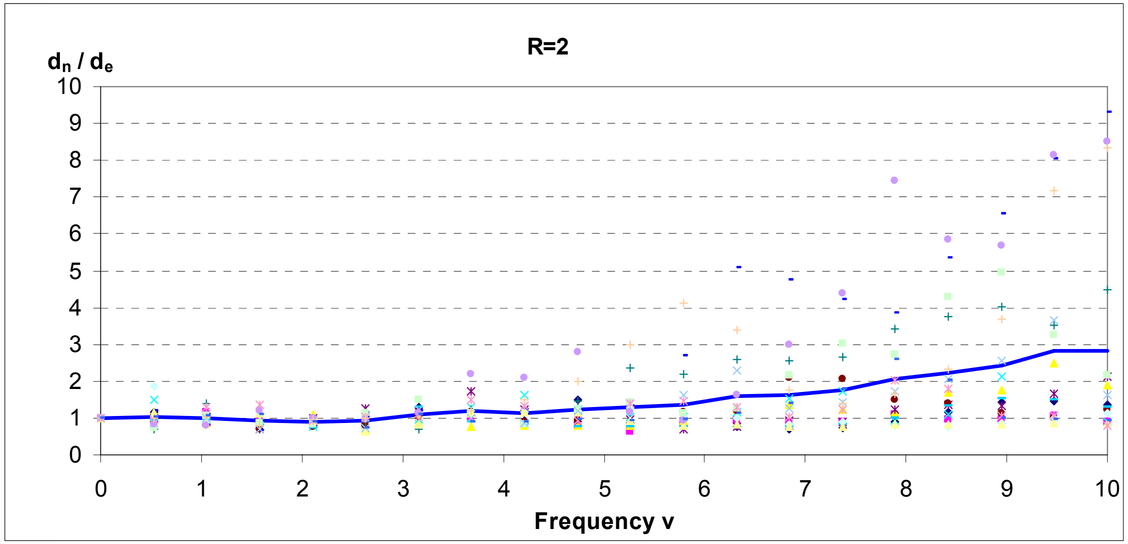


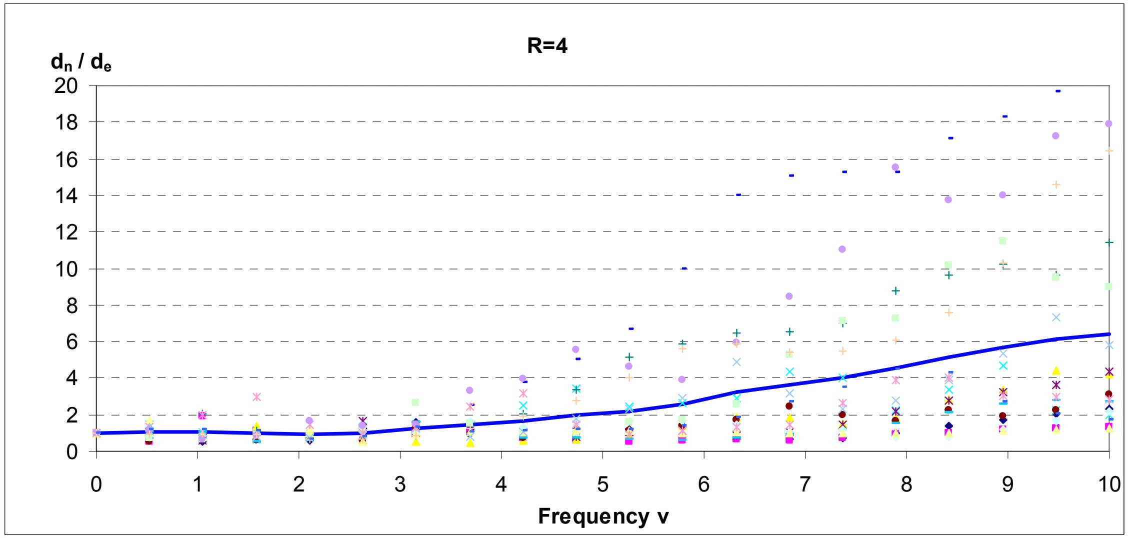
- The mean values are smaller than 1 for large periods (the linear systems overestimate the displacements of the non-linear systems).
- In the short period range, the results differ according to the ductility level.
- The mean values are greater than 1 even for high periods (especially for large values of the displacement ductility) in the results of Model C
- The values of the standard deviation in Model C are greater than the corresponding values of the type A and B models.

Type A model

Results for constant R



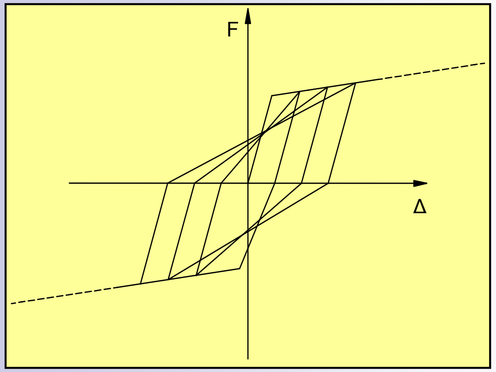


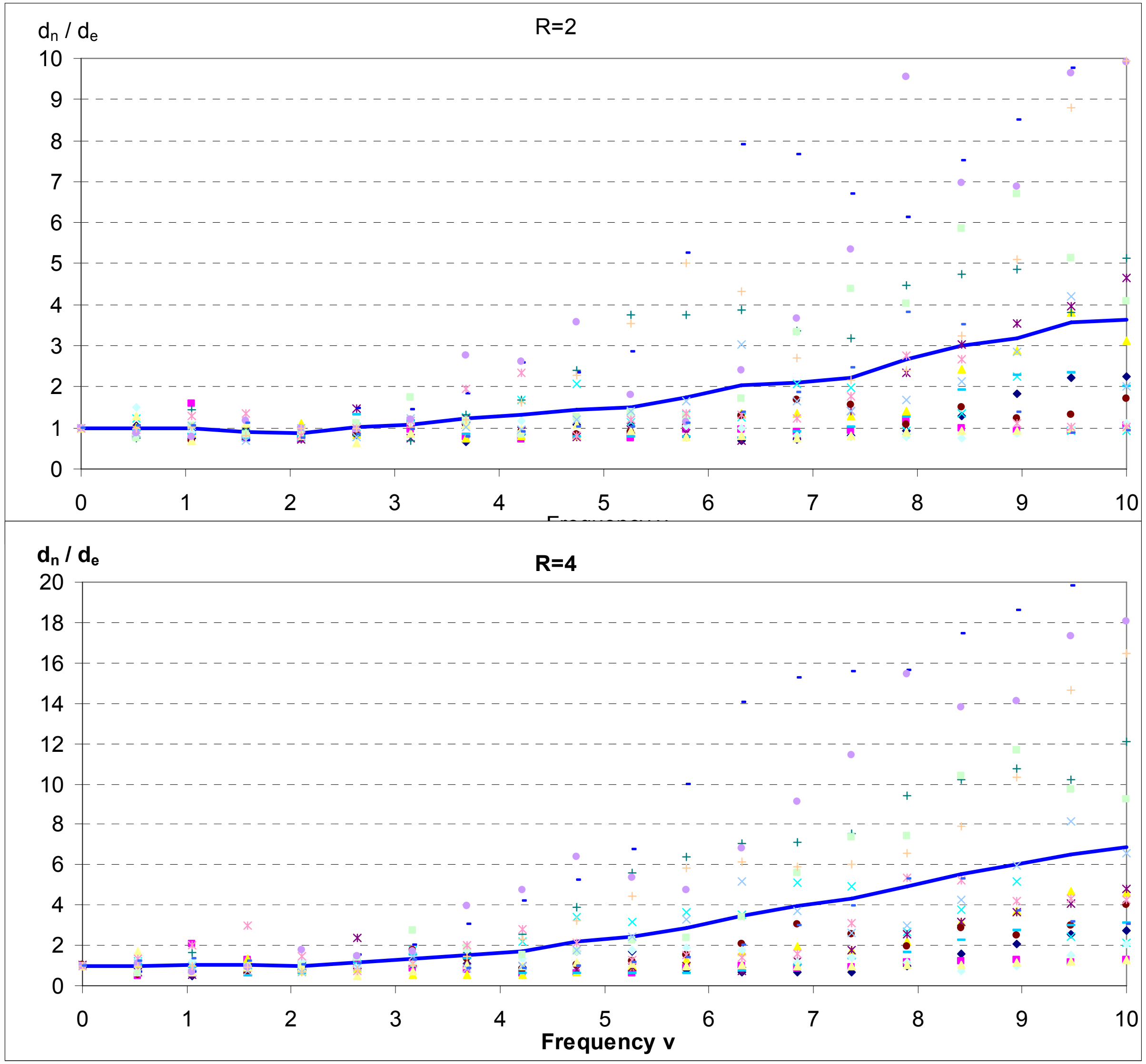


- The results are close to 1 until a frequency of about 2.5.
- After that value, a great scattering occurs.

Type B model

Results for constant R

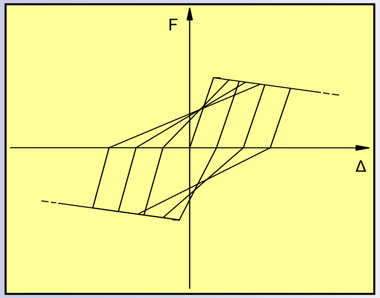


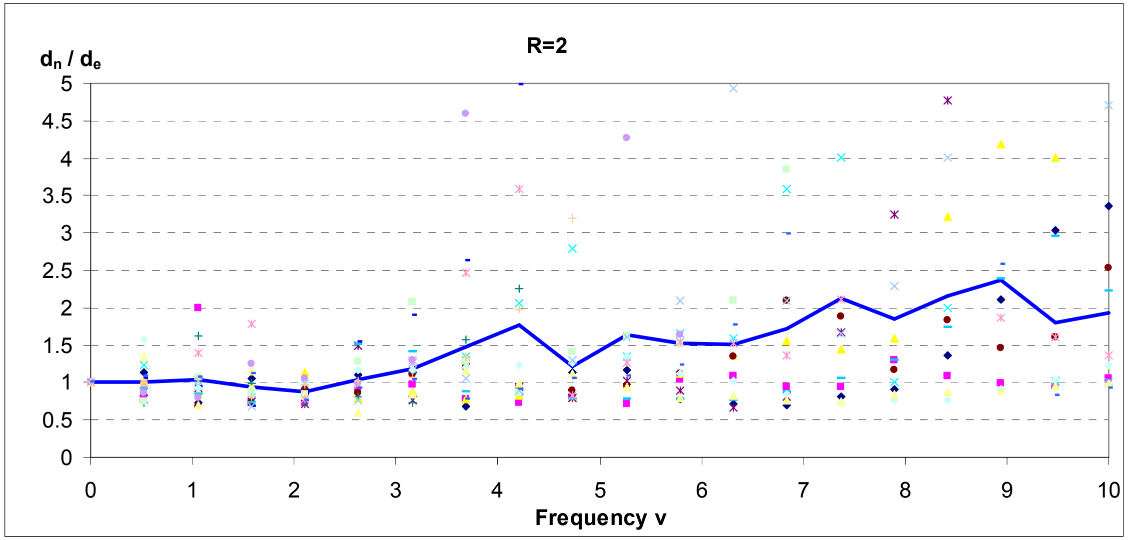


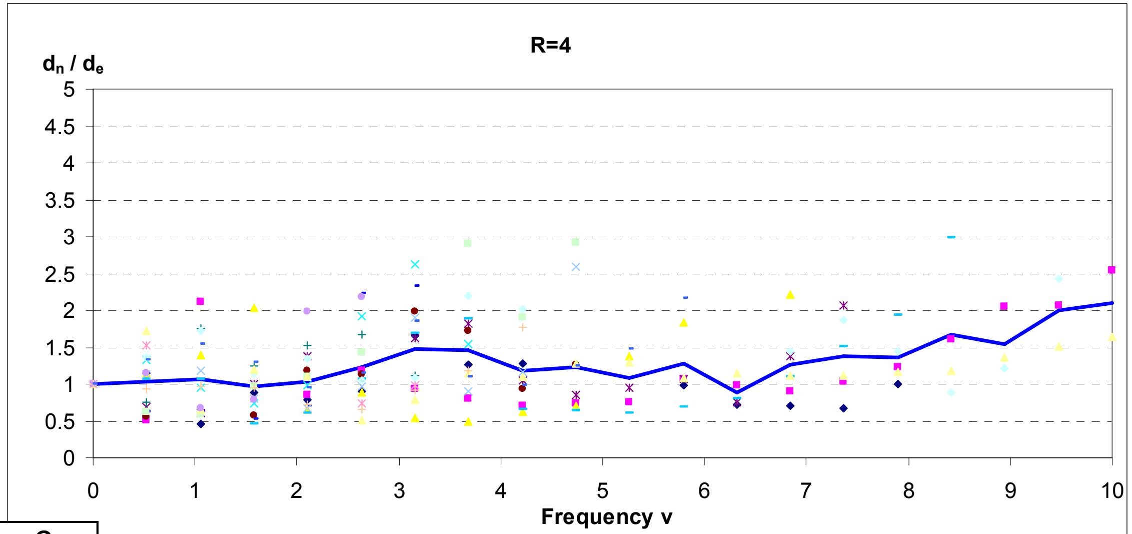
➢The mean values are increased with respect to the values that correspond to model A.

Type C model

Results for constant R



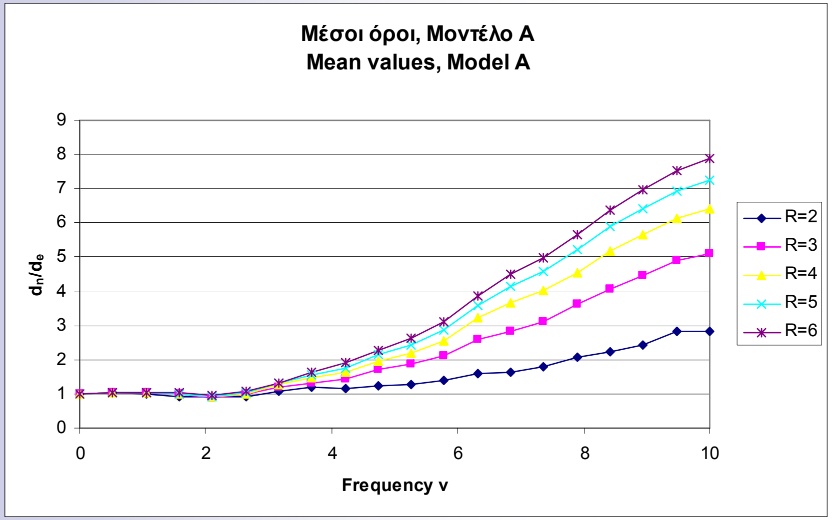


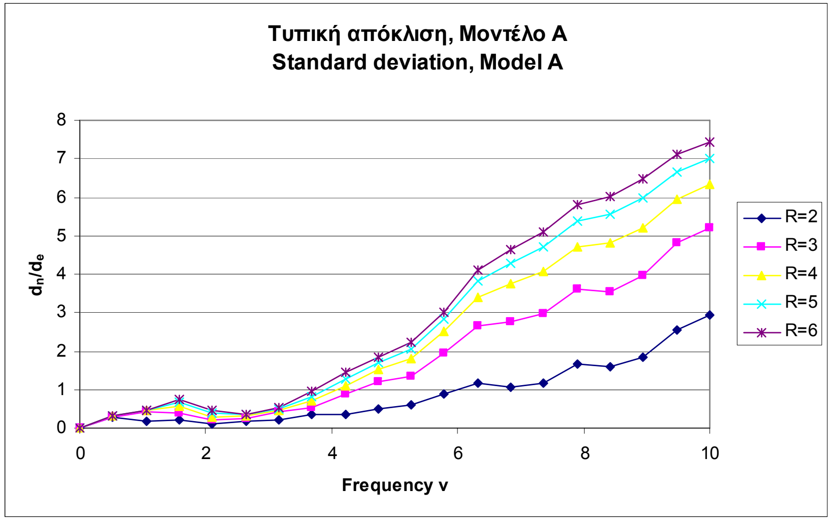


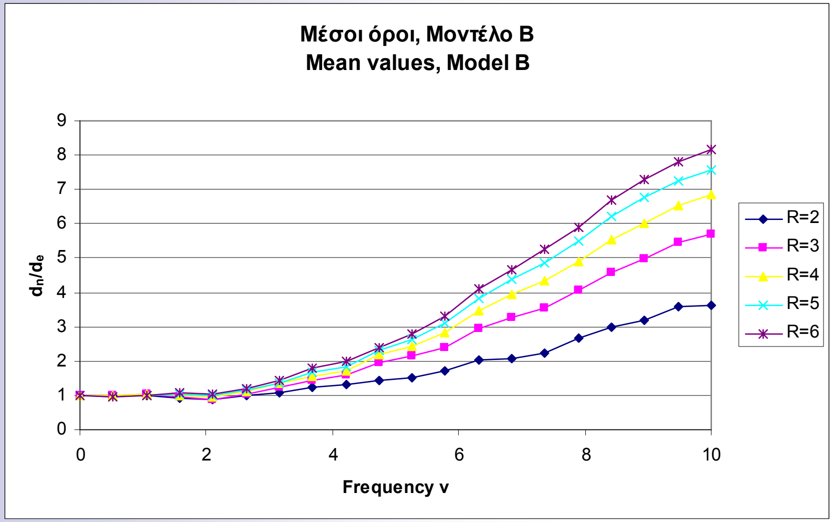
- These models are vulnerable to collapse
- From the 1700 examined oscillators (17 ground motions x 20 frequency values x 5 levels for R), 692 oscillators failed.

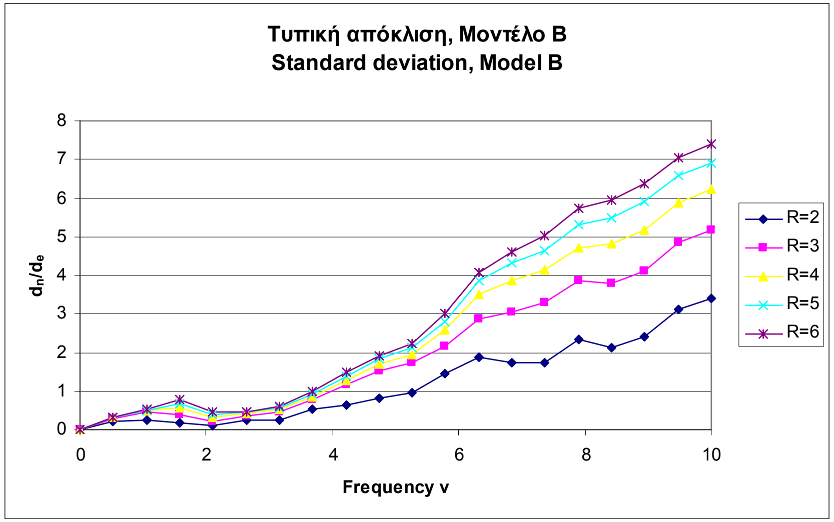
R	2	3	4	5	6
Failed oscillators	53	113	145	176	205
Percentage %	15.5	33.2	42.7	51.8	60.0

Structures exhibiting negative hardening (softening) should remain elastic in order to avoid collapse

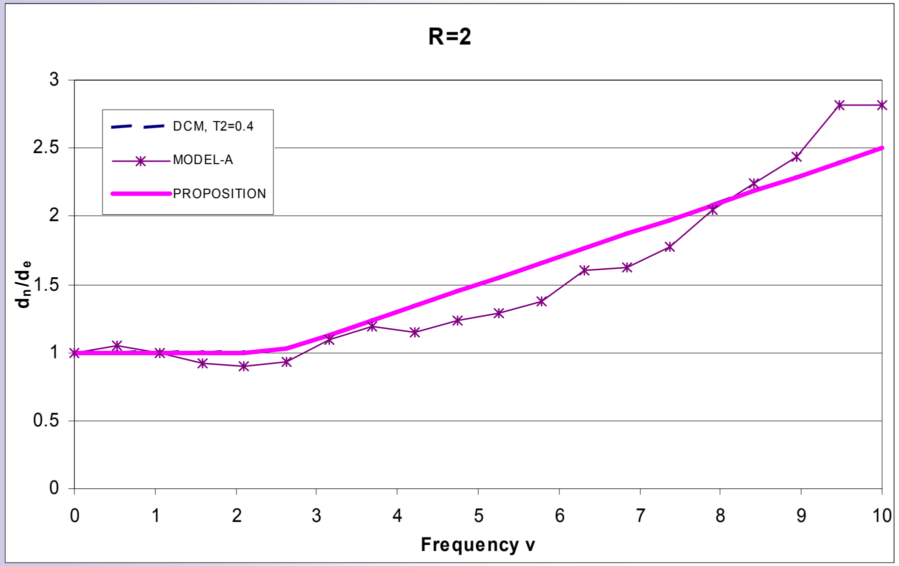


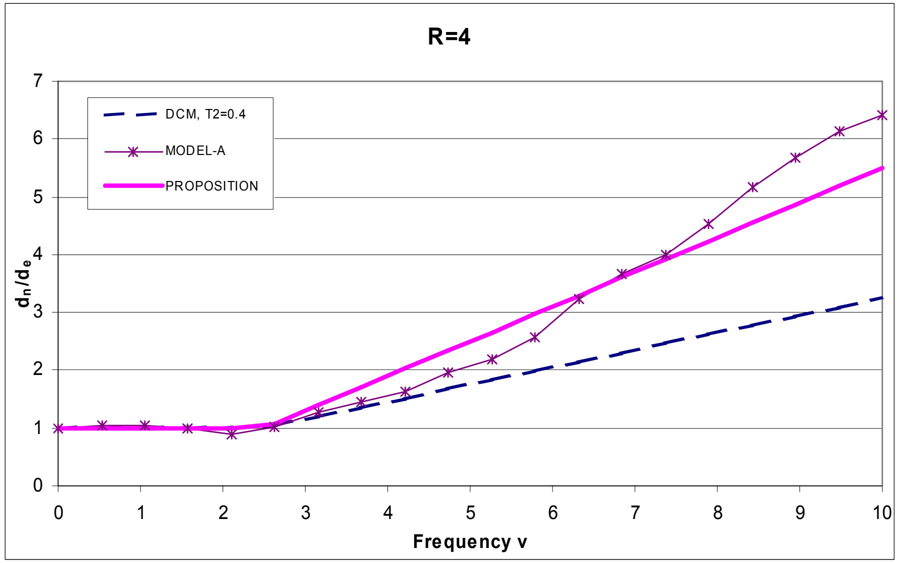


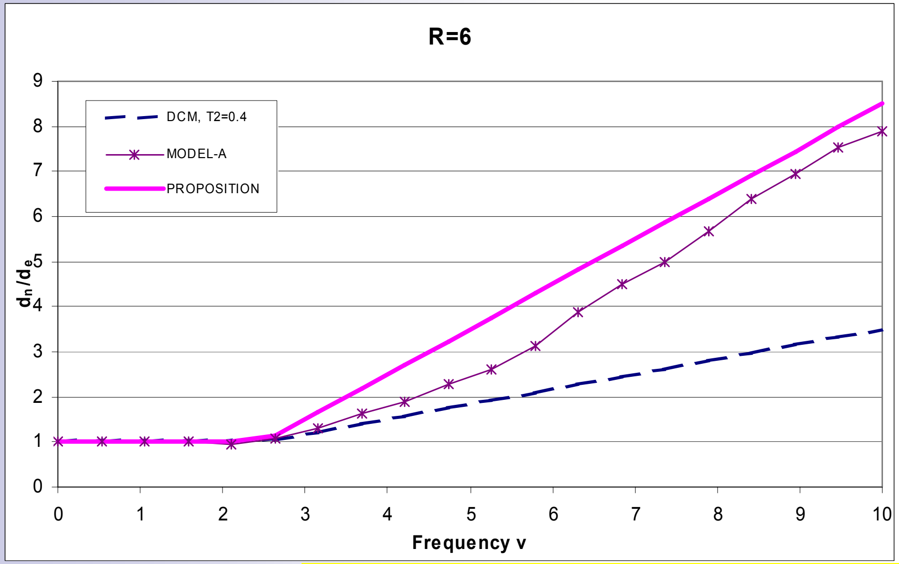




- The mean values are close to 1 for v<2.5, for all *R* levels.
- For v>2.5, the mean values increase, depending on *R*.
- Large values of the standard deviation appear in the high-frequency range.







- For (v<2.5) the values of C_1 **are reliable**.
- **For high frequencies, there is a strong dependance on *R***
- *C_1 cannot describe effectively the response of inelastic systems with large R values in the high frequency range.*

Proposal $C_1 = 1 + (R-1)(T_g/T - 1)/2$ για $T_g < T < 0.1sec$

Uncertainty in structural earthquake assessment and simulation

Uncertainty in engineering analysis

In general data and models are uncertain. This fact has a significant influence for the results of the analysis. Uncertainty has to be described with suitable models and considered within the analysis.

Classification of uncertainty

Fuzziness

- **Stochastic uncertainty**
  A random result (e.g. of an experiment under identical boundary conditions) are observed almost indefinitely
- **Informal uncertainty**
  The system overview is incomplete or if only a small number of observations are available.
- **Lexical uncertainty**
  The uncertainty is quantified by linguistic variables, transformed onto a numerical scale.

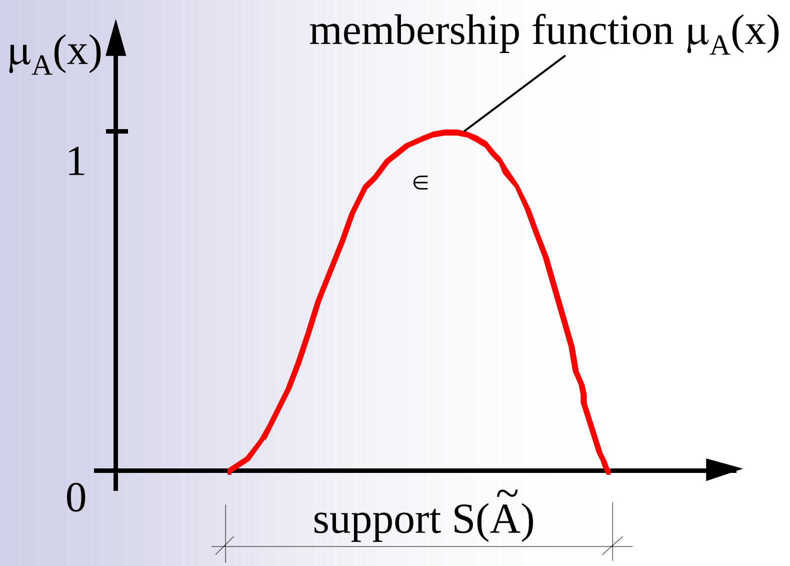
Examples of fuzziness

- earthquake loading
- storm loading
- impact loads
- aging processes
- damage
- material parameters

Modelling of uncertainty with fuzzy variables

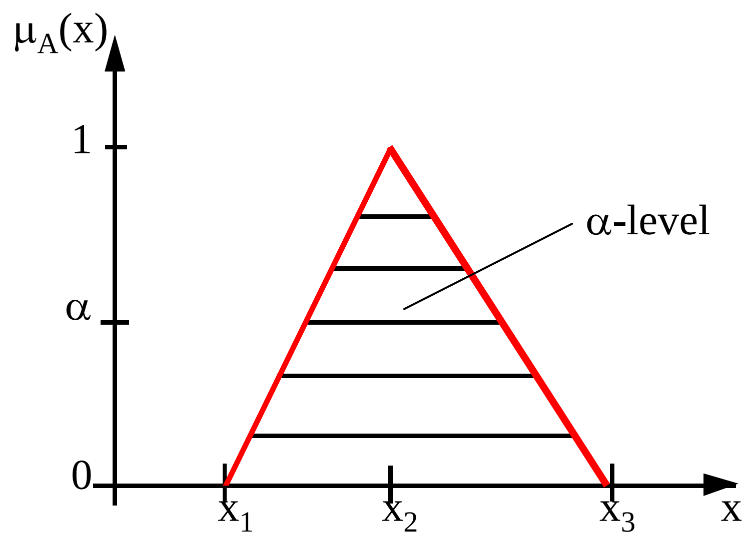
fuzzy set

$$\tilde{A} = \left\{ \left(\mathbf{x};\, \mu_A(\mathbf{x}) \right) \middle|\ \mathbf{x} \in \mathbf{X} \right\}$$



Expresses the „certainty“ with which a quantity is known

triangular fuzzy number $\tilde{A}$ and α-levels:



From the fuzzy quantity crisp sets

$$A_{\alpha_k} = \{ x \in \mathbf{X} \mid \mu(\mathbf{x}) \geq \alpha_k \}$$

may be extracted for real numbers a_k (0, 1].

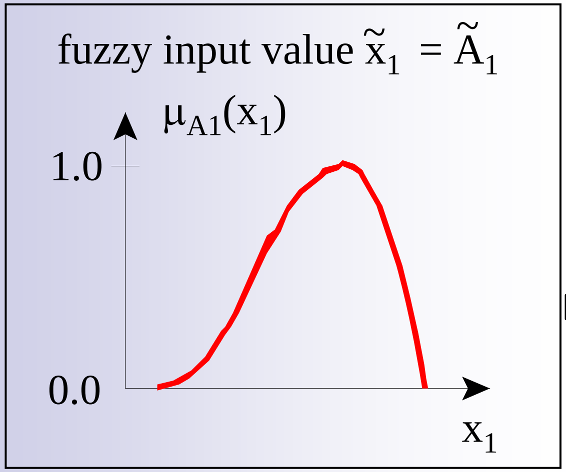
These crisp sets are called *a-level sets*.

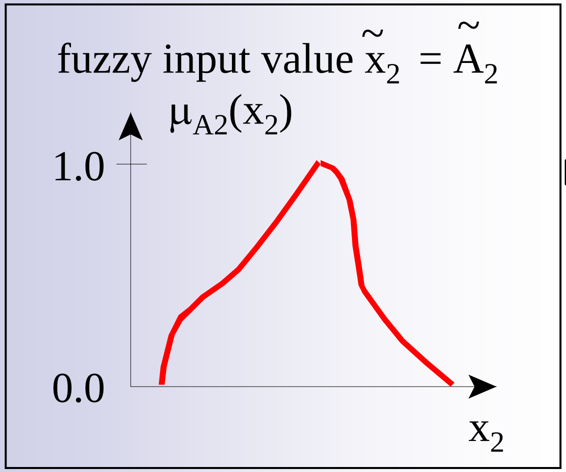
Solution technique

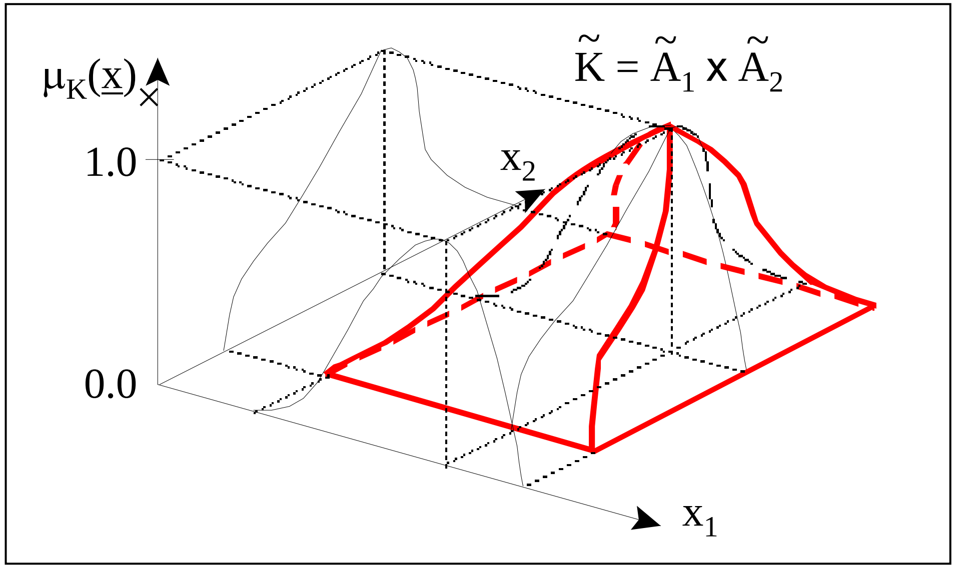
cartesian product

$\tilde{K} = \tilde{A}_1 \times \tilde{A}_2 \times \ldots \times \tilde{A}_n$ (interaction of fuzzy quantities)

$$\tilde{K} = \left\{ \begin{array}{l} \left[\underline{x} = (x_1; x_2; \ldots; x_n); \mu_K(\underline{x}) = \mu_K(x_1; x_2; \ldots; x_n)\right] \\ \quad \mid x_i \in X_i; \mu_K(\underline{x}) = \min\left[\mu_{Ai}(x_i)\right]; i = 1; \ldots; n \end{array} \right\}$$

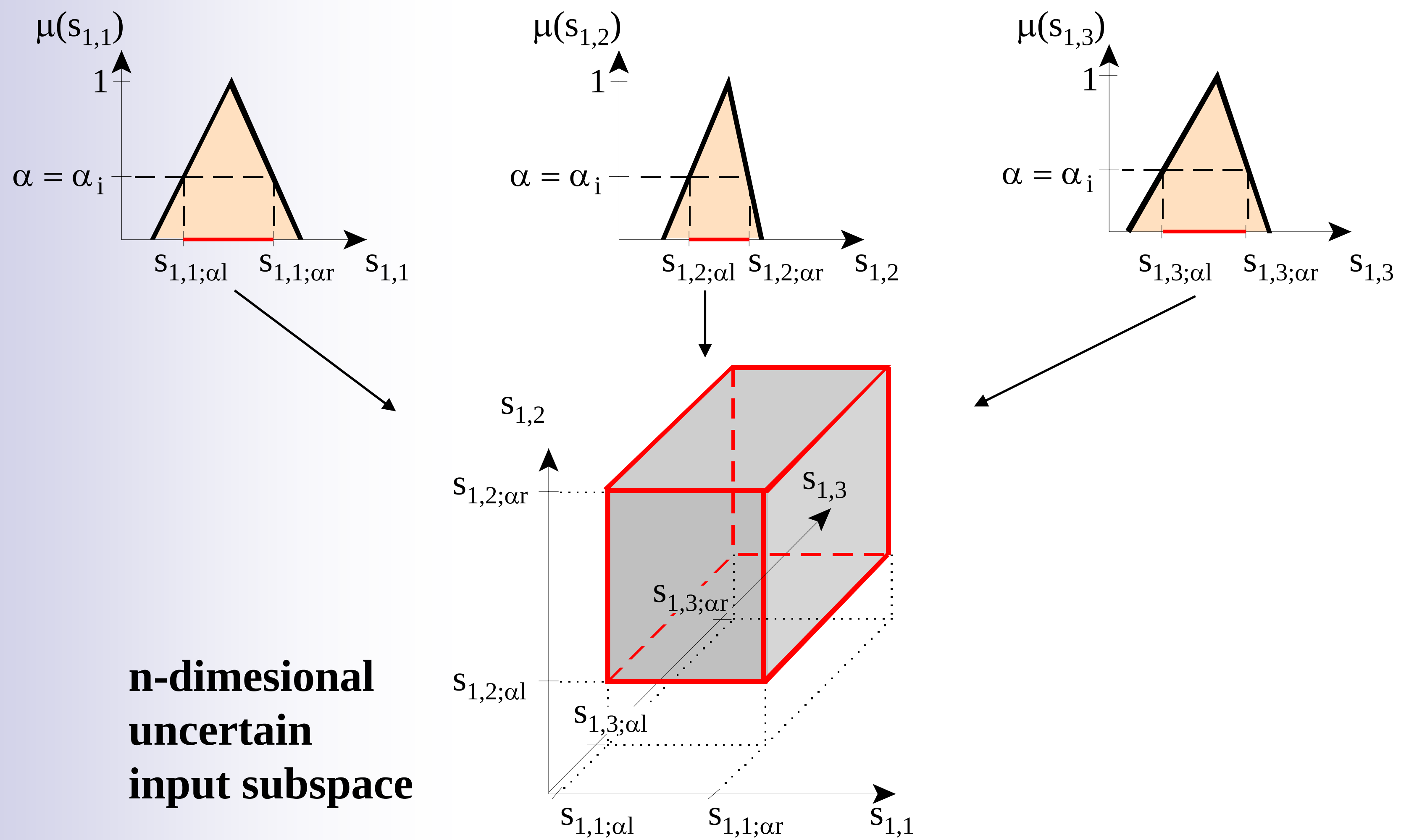






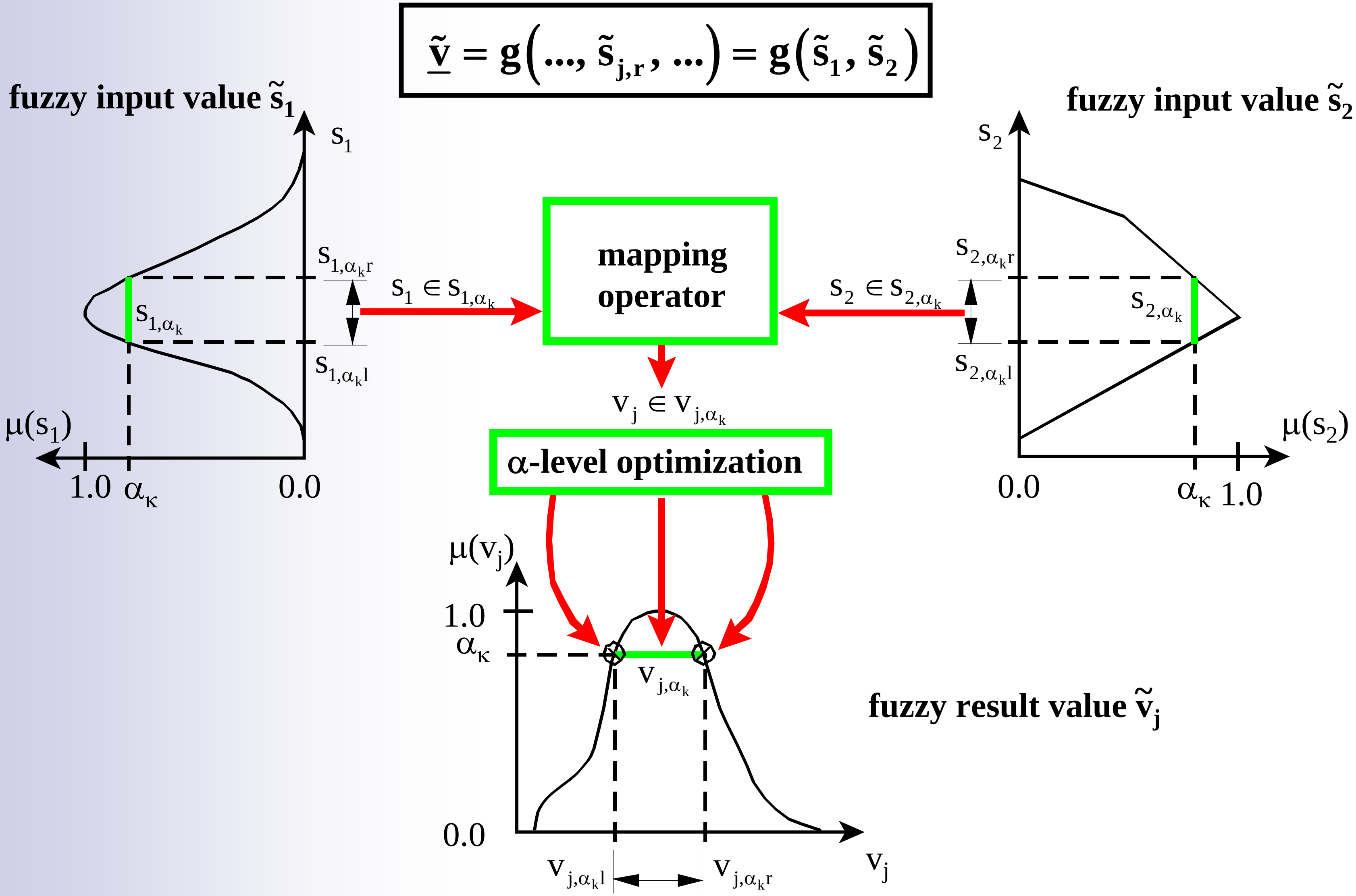
Solution technique

generation of an uncertain input subspace (bunch parameters $\tilde{s}_{1,1}$, $\tilde{s}_{1,2}$, $\tilde{s}_{1,3}$)

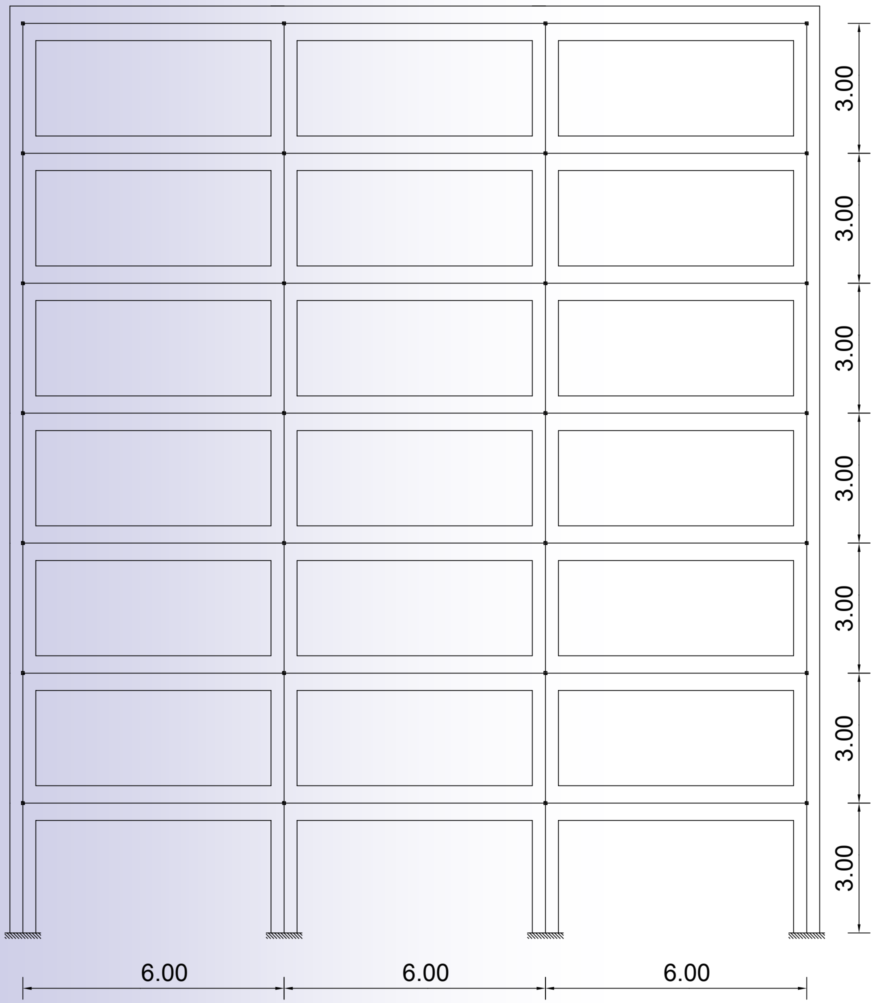


n-dimesional uncertain input subspace

Solution technique



Illustrative example



Loads

- Dead load 30.0 kN/m
- Live load 15.0 kN/m

Seismic papameters

- Spectral acceleration: 0.16g,
- Soil type: B,
- Effective damping: 5%
- Importance factor 1.00
- Foundation factor: 1.00

Beams

upper reinforcement 8.0cm^2
Lower reinforcement 4.0cm^2

Columns

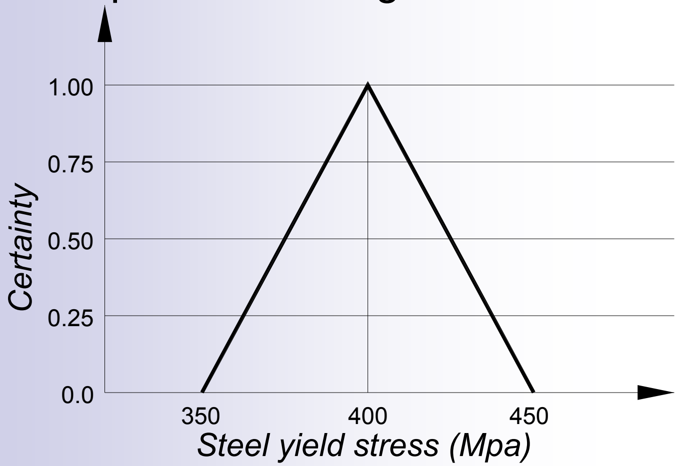
Total amount of the longitudinal reinforcement 20.24cm^2 (uniformly distributed along the perimeter of the column).

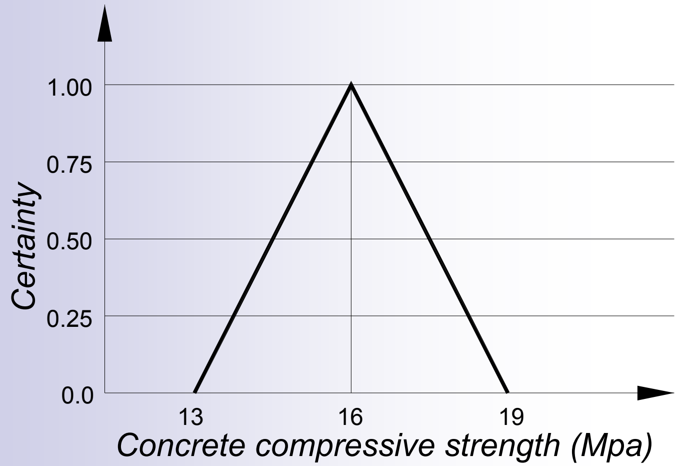
Analysis

Elastoplastic (pushover)
Determination of target displacement (ATC40)

Input quantities

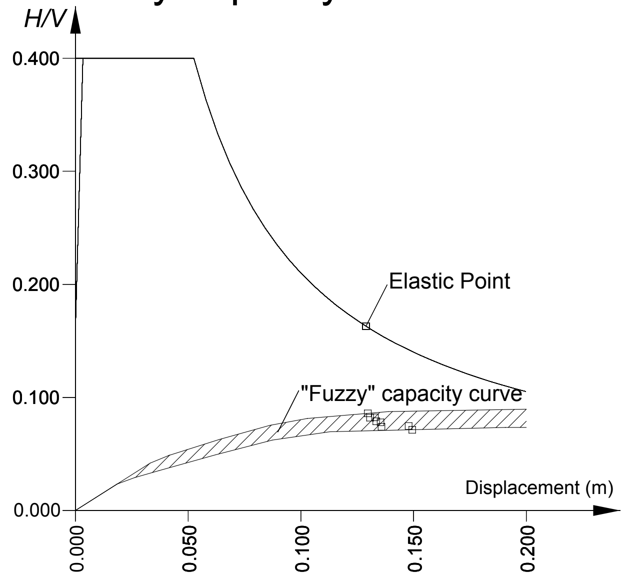
Fuzzy numbers representing the steel yield stress and the concrete compressive strength



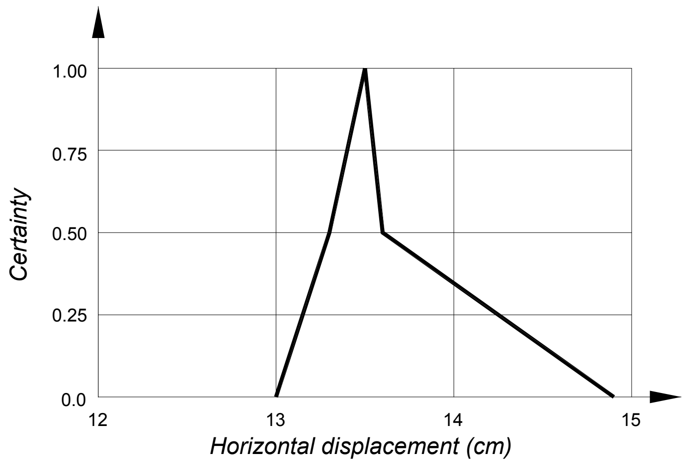


Response quantities

Fuzzy capacity curve



Fuzzy top-horizontal displacement



Capacity Design Methodology for Design and Evaluation of Seismic Resistance of RC Building Structures

RESIST-INELA Methodology

[Author: Golubka Necevska-Cvetanovska, IZIIS, 1991]

MAIN PURPOSE:

- Define strength and deformability capacity of the structure
- Define nonlinear behaviour of the structure for a given earthquake effect
- Evaluation of seismic resistance

STEP 1 Definition of the structural system of the building and determination of the quantity and quality of the built-in material.

STEP 2 Determination of the Q- Δ diagram for each element and the storey Q- Δ diagrams (RESIST computer program).

STEP 3 Definition of the seismic parameters and the design criteria.

STEP 4 Nonlinear dynamic analysis of the structural system for a given earthquake effect (INELA computer program).

STEP 5 Selection of an optimal system in newly designed structures and evaluation of the seismic resistance for the existing structures

Application of RESIST-INELA Methodology

Settlement Kapistec

Application of RESIST-INELA Methodology

Settlement Jane Sandanski

Application of RESIST-INELA Methodology

Settlement Novo Lisice

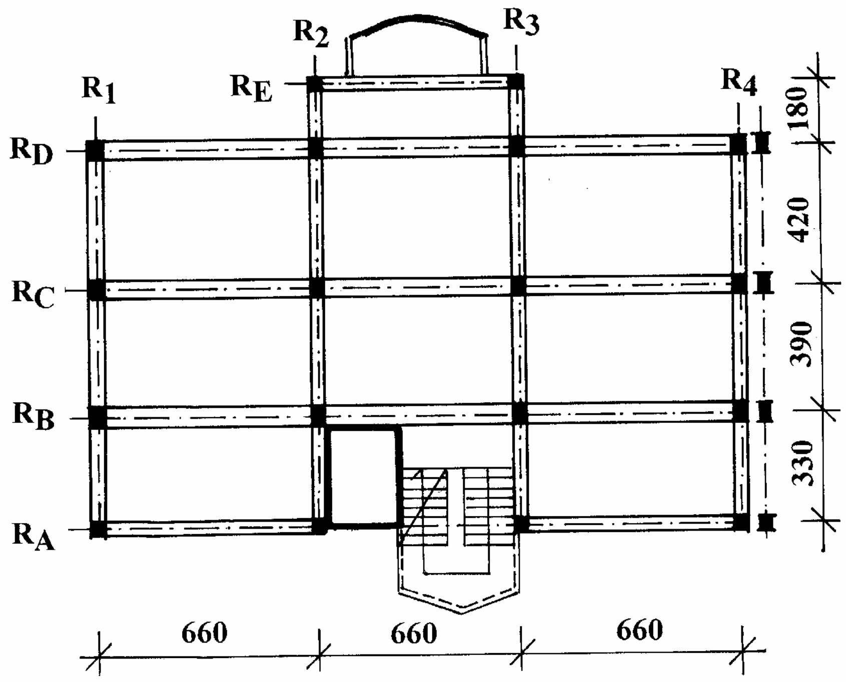
Application of RESIST-INELA Methodology

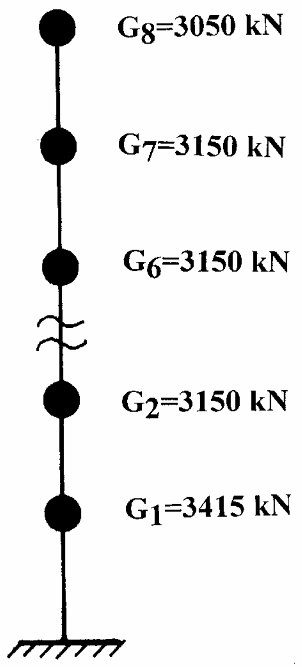
Settlement John Kennedy

Correlation with Modern Design Codes

To conceive particular aspects of EC8 for frame structural systems and compare them with the requirements and criteria prescribed with our national existing seismic regulations (SP-81), and methodology developed at Institute of Earthquake Engineering and Engineering Seismology – Skopje, several structures were analyzed.

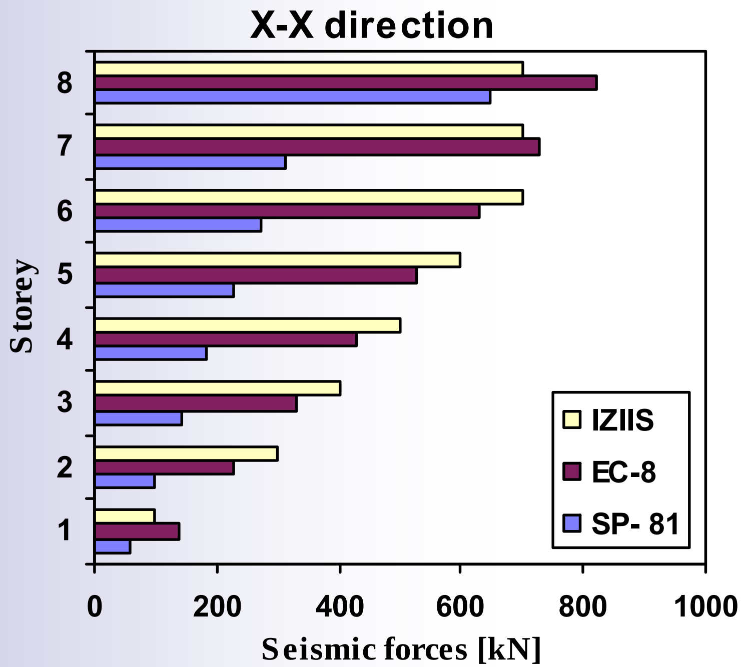
Presented here are the results from the analysis of structure B-2, Unit 4, "Vardar" settlement – Skopje, (fYRepublic of Macedonia).

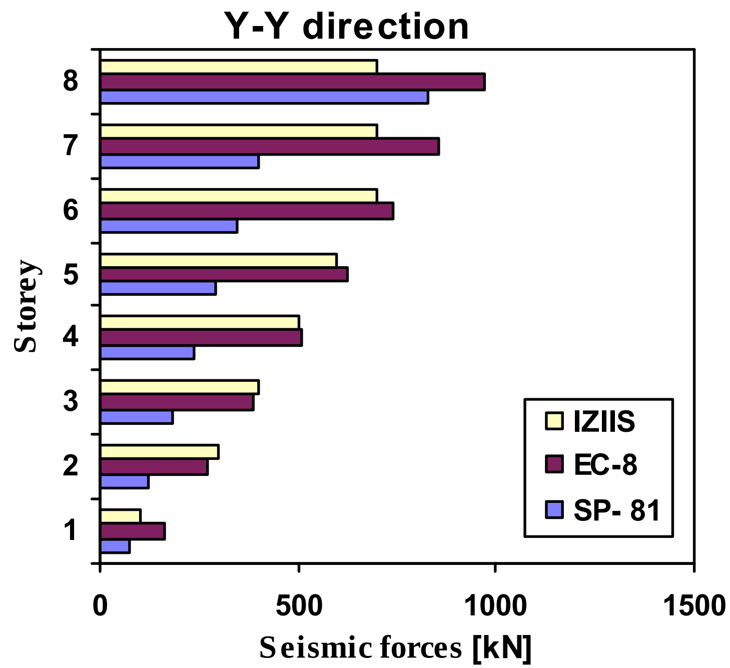




Characteristic plan and weights of building storeys

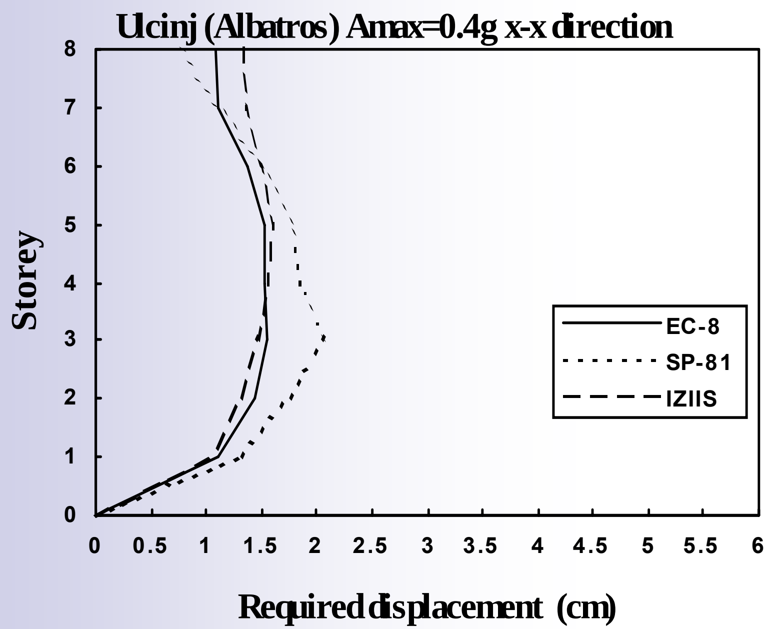
Correlation with Modern Design Codes

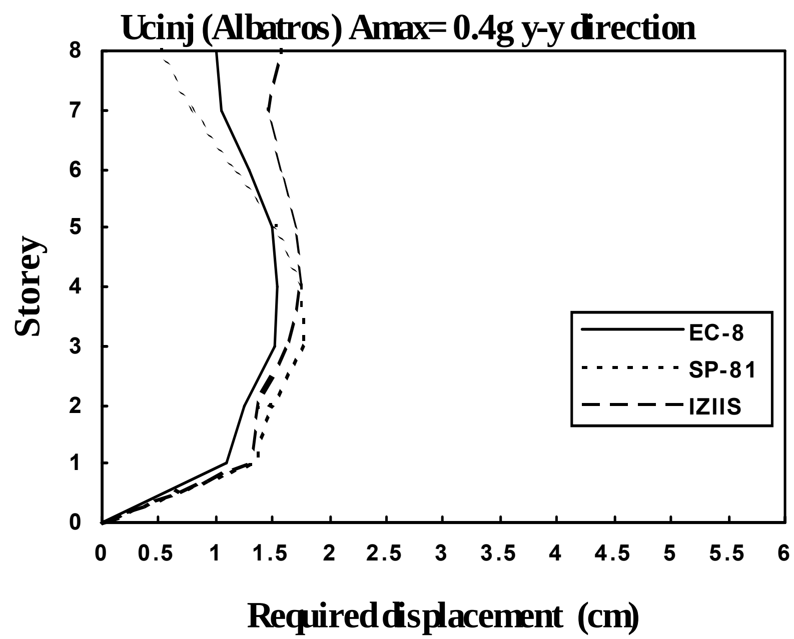




Seismic forces obtained according to SP-81, EC8 and IZIIS Methodology

Correlation with Modern Design Codes





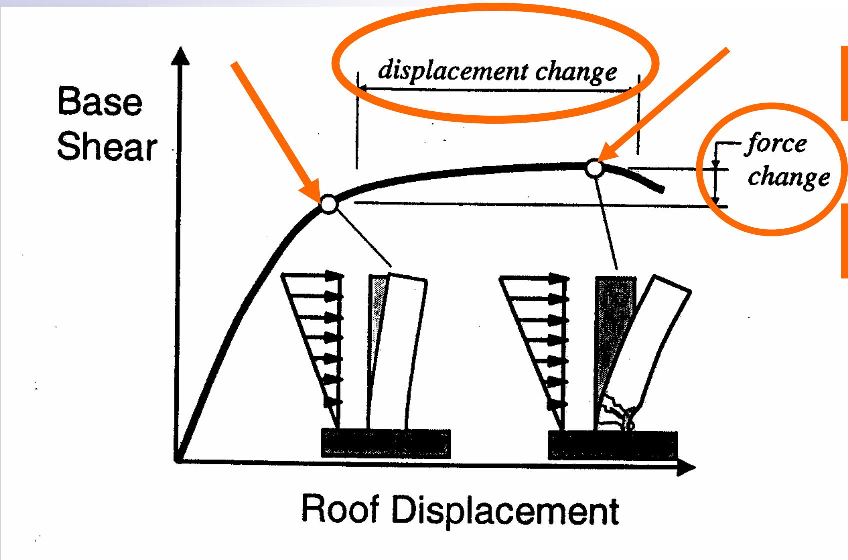
Required displacements for Ulcinj (Albatros) Earthquake

Direct Displacement Based Design Approach for Design of RC Frame Building Structures

Displacement-Based Seismic Design, DBD

Displacement-Based Design vs. Forced -Based Design

Displacement-based seismic design is defined broadly as any seismic design method in which displacement-related quantities are used directly to judge performance acceptability.



Concrete cracking and reinforcement yielding

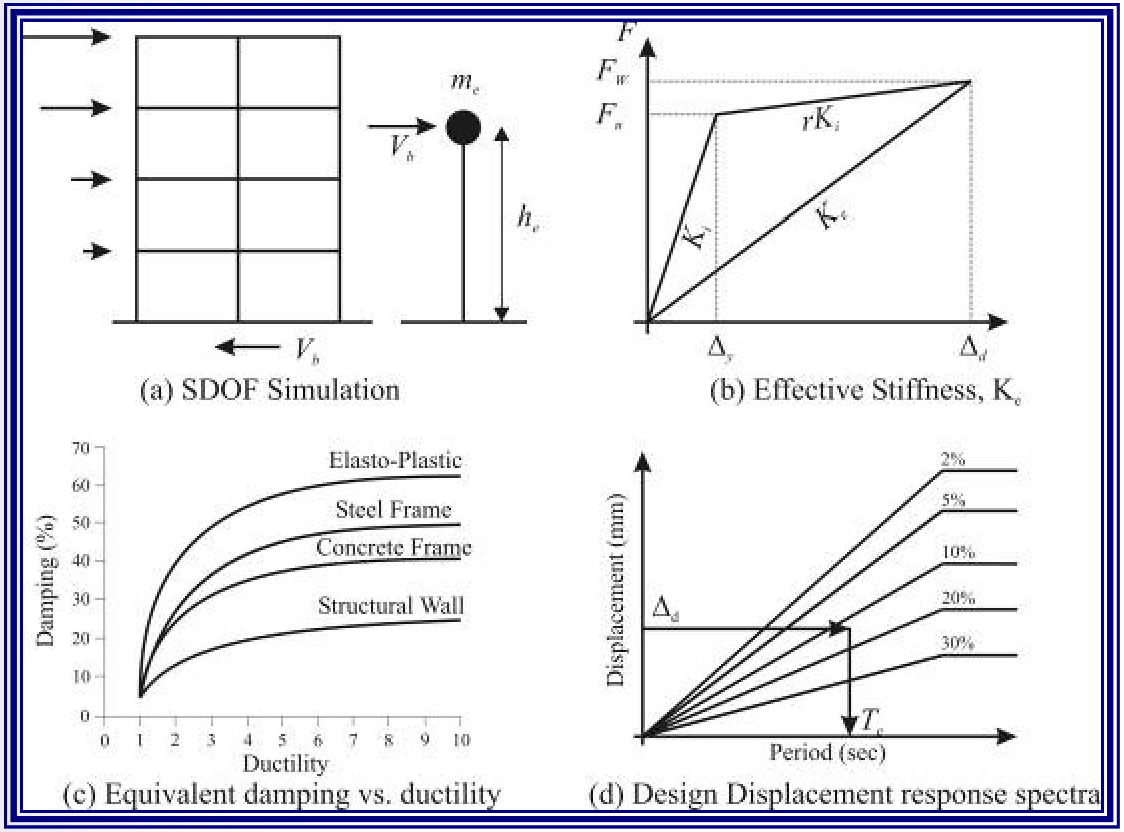
Formation of flexural plastic hinge

The extent of damage is related to the amount of deformation (turn to displacement) in plastic hinge

Insignificant change of forces despite change of wall behaviour from elastic to deeply nonlinear

Direct Displacement-Based Seismic Design, DDBD

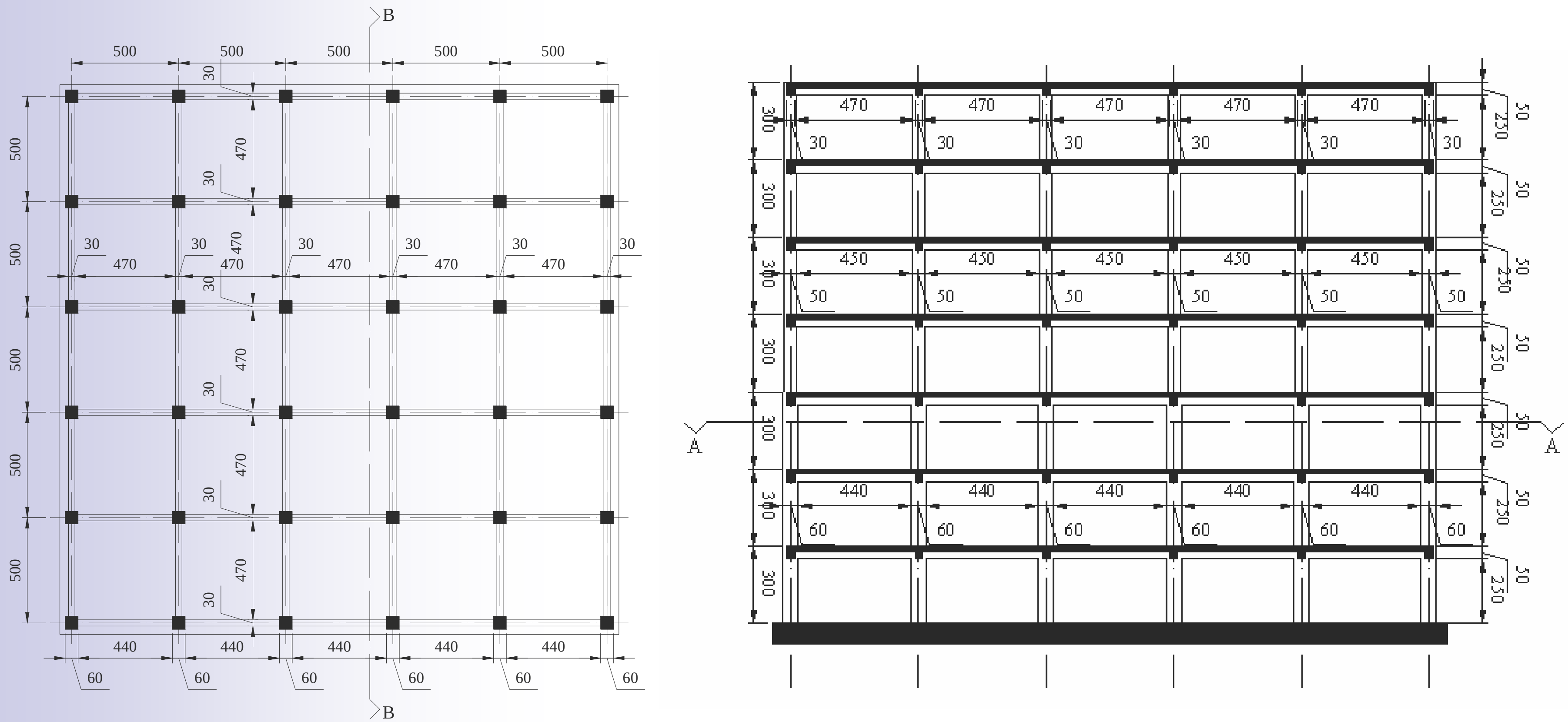
The most developed and the most important method in the field of direct displacement-based design of RC building structures, both with frame and with shear wall lateral bearing system is the Priestley's method, (Priestley, 2000, 2002).



"Substitute structure approach", (Shibata and Sozen, 1975)

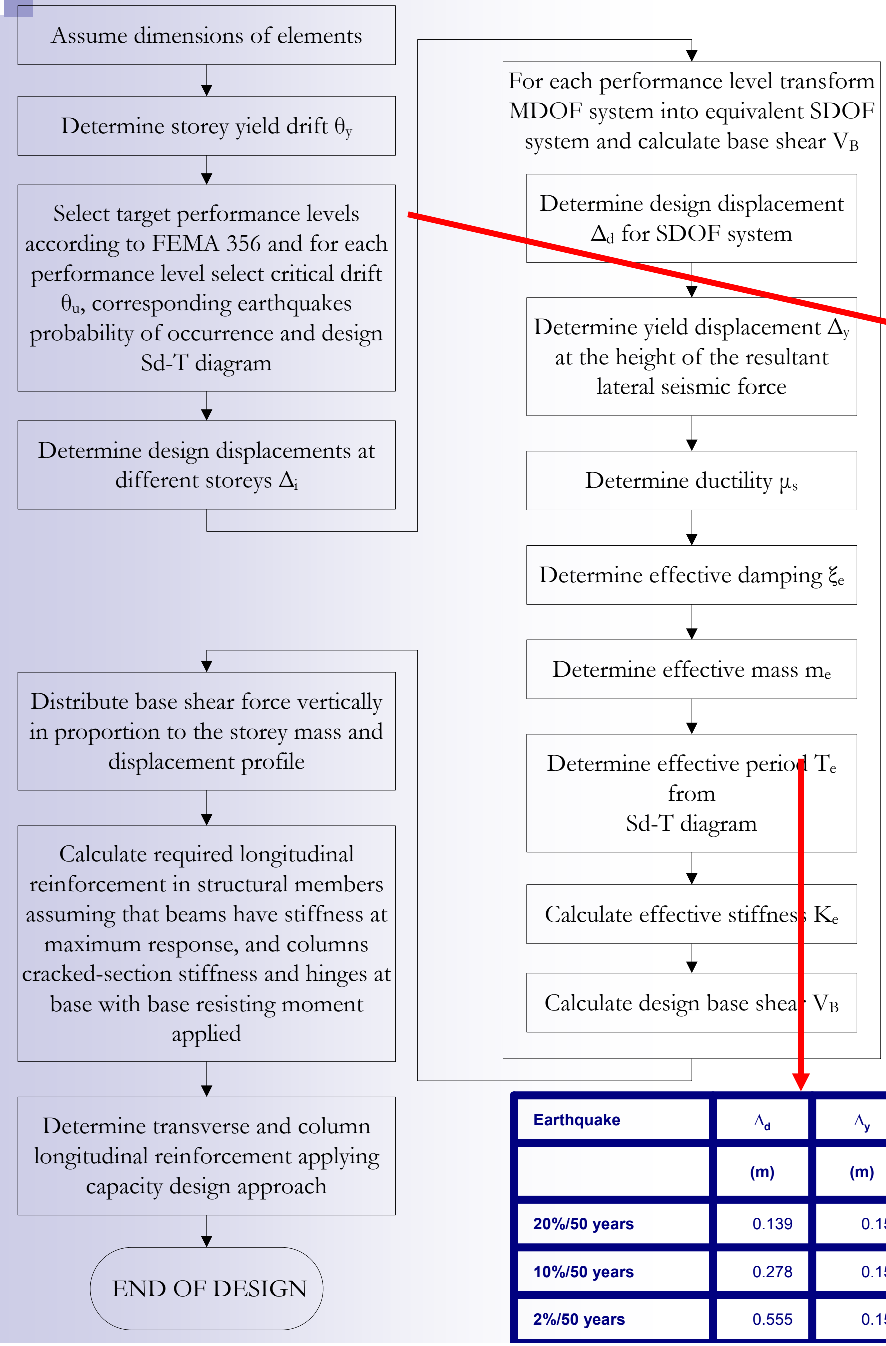
Design Example – RC Frame Building

In order to determine the effort needed for direct displacement-based design of a RC frame building, an example structure is designed using the Priestley's method (Priestley, 2000, 2002) with minor changes that do not affect the essence of the original. Later, an example structure is analyzed using a nonlinear static procedure, (Terzic, 2006).



Layout of typical storey and cross-section of the structure

Design Example – Applied Method



Performance Level	Probability of earthquake occurrence	Return Period (yrs)	Critical Drift θ_u (%)
IO	20%/50 year	225	1
LS	10%/50 year	475	2
CP	2%/50 year	2500	4

Earthquake	Δ_d	Δ_y	m_e	μ	ξ	T_e	K_e	V_B
	(m)	(m)	(kg)			(s)	(kN/m)	(kN)
20%/50 years	0.139	0.15	3704984	0.92	3.81	2.65	20828.32	2890.59
10%/50 years	0.278	0.15	3704984	1.85	12.94	6.01	4049.46	1123.98
2%/50 years	0.555	0.15	3704984	3.70	19.40	6.50	3461.94	1921.81

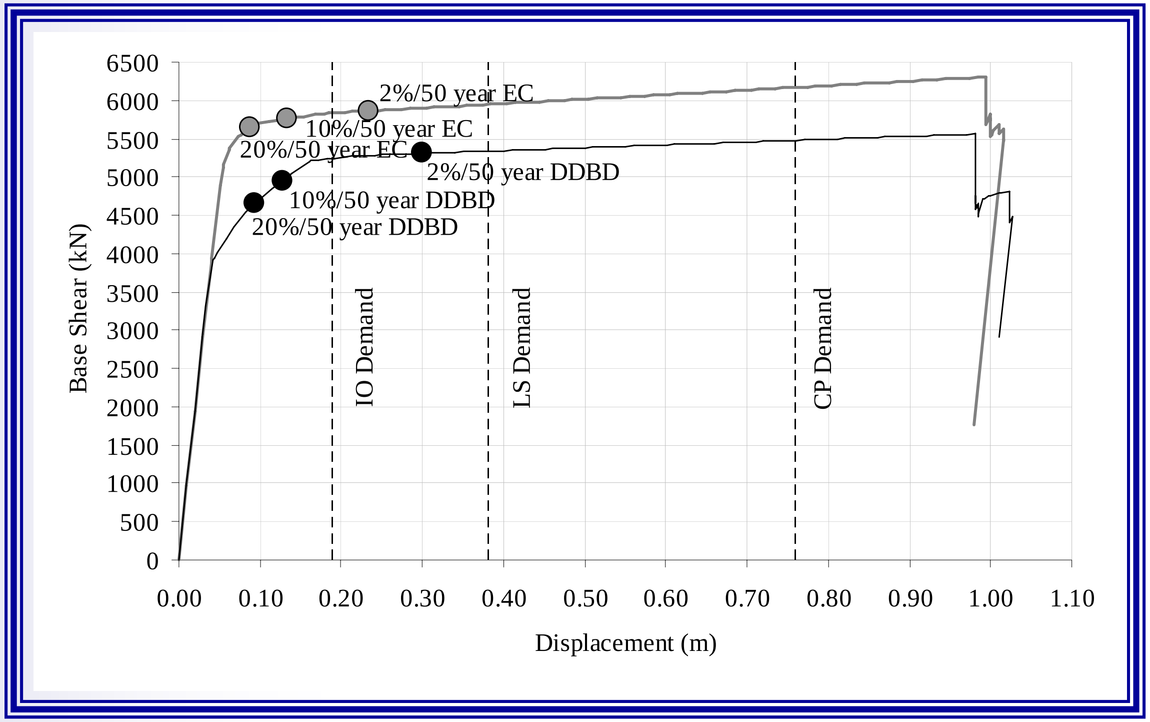
Analysis Results for Target Drifts

Target drifts-DDBD

Earthquake	Drift (%)	Percentage of Members Achieving State		
		IO	LS	CP
20%/50 year	0.42	39	0	0
10%/50 year	0.61	66	0	0
2%/50 year	1.43	100	0	0

Target drifts-EC8

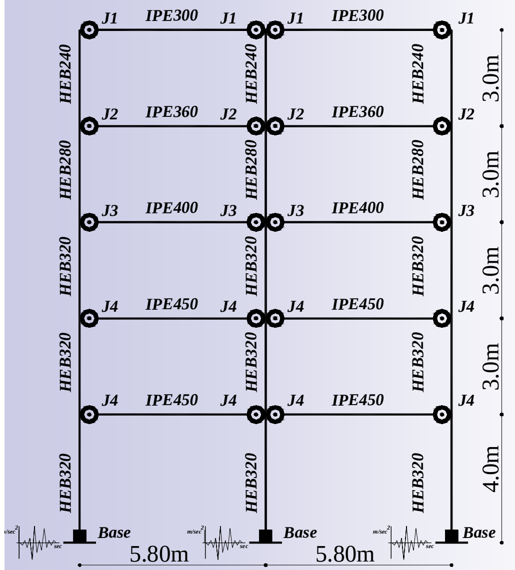
Earthquake	Drift (%)	Percentage of Members Achieving State		
		IO	LS	CP
20%/50 year	0.39	2	0	0
10%/50 year	0.66	61	0	0
2%/50 year	1.25	100	0	0

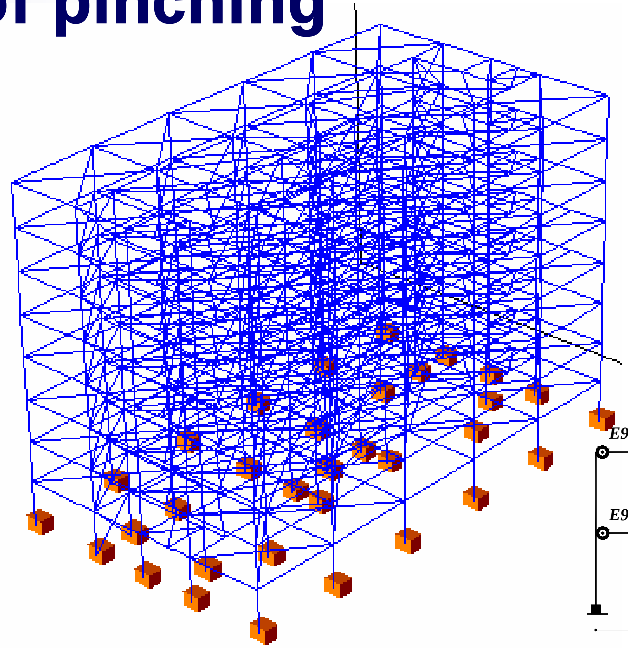


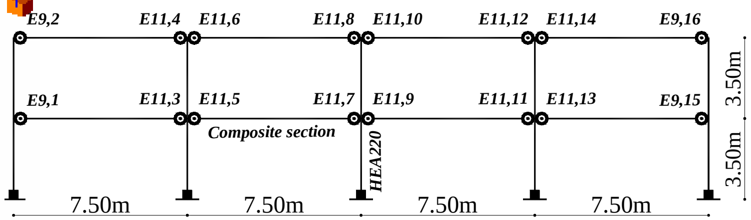
Capacity curve for building designed according to DDBD and EC8

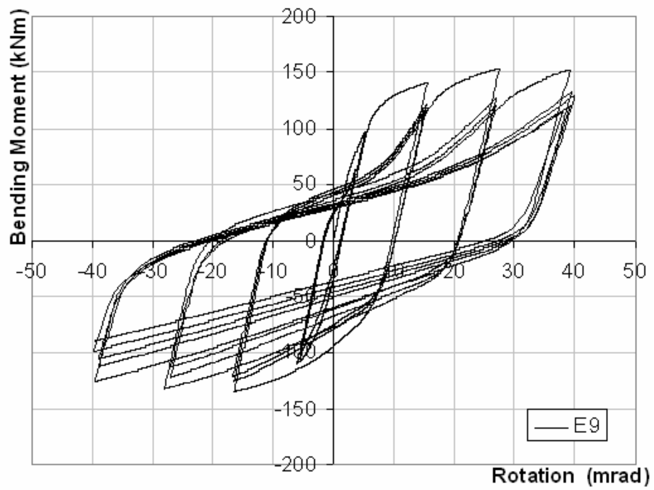
Influence of connection behaviour on the seismic response of structures

1. Influence of pinching

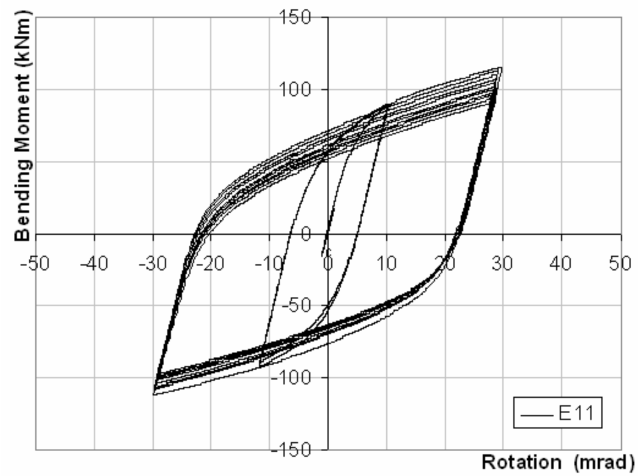








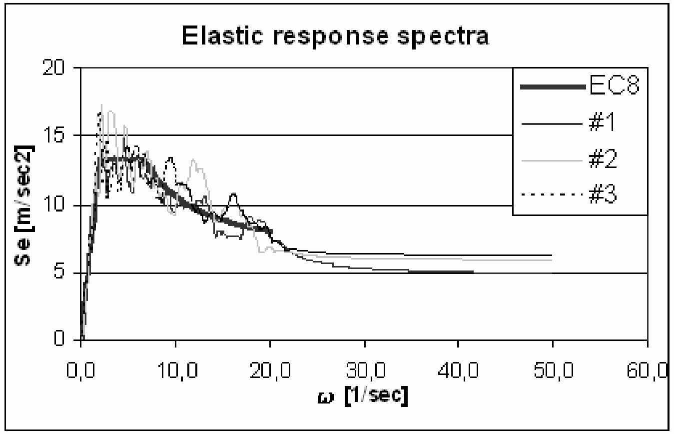
Hysteretic curve for external E9 connection

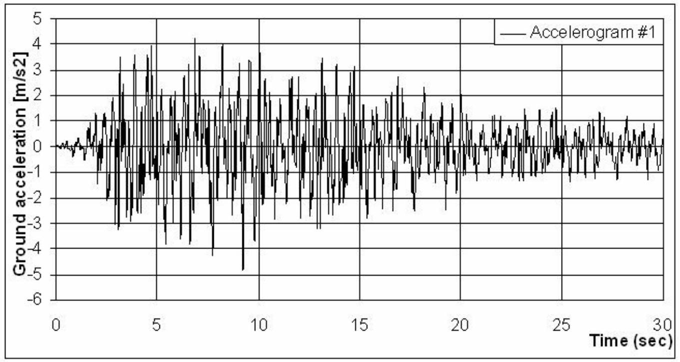


Hysteretic curve for internal E11 connection

Performed analyses

- Dynamic
- N2 (uniform)
- N2 (modal)





Analysis results

Connections	rot. (mrad) dynamic	rot. (mrad) N2 (uniform)	rot. (mrad) N2 (modal)
E 9,15	48.0	27.6	30.2
E11,7	46.3	30.4	33.3
J231E/J4	15.7	13.9	15.8
J53EF/J-X610	8.8	16.9	20.3

Conclusion

For study of the behaviour of the connections, the monotonic methods have some limitations, because they give insufficient hysteretic information.

2. Comparative effect of extreme event

Influence of the connections simulated

- with semi-rigid behaviour and partial strength
- with rigid behaviour and full strength