Equaljoints Plus

Valorisation of knowledge for European pre-QUALified steel JOINTS

Czech Technical University in Prague

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Seismic analysis methods

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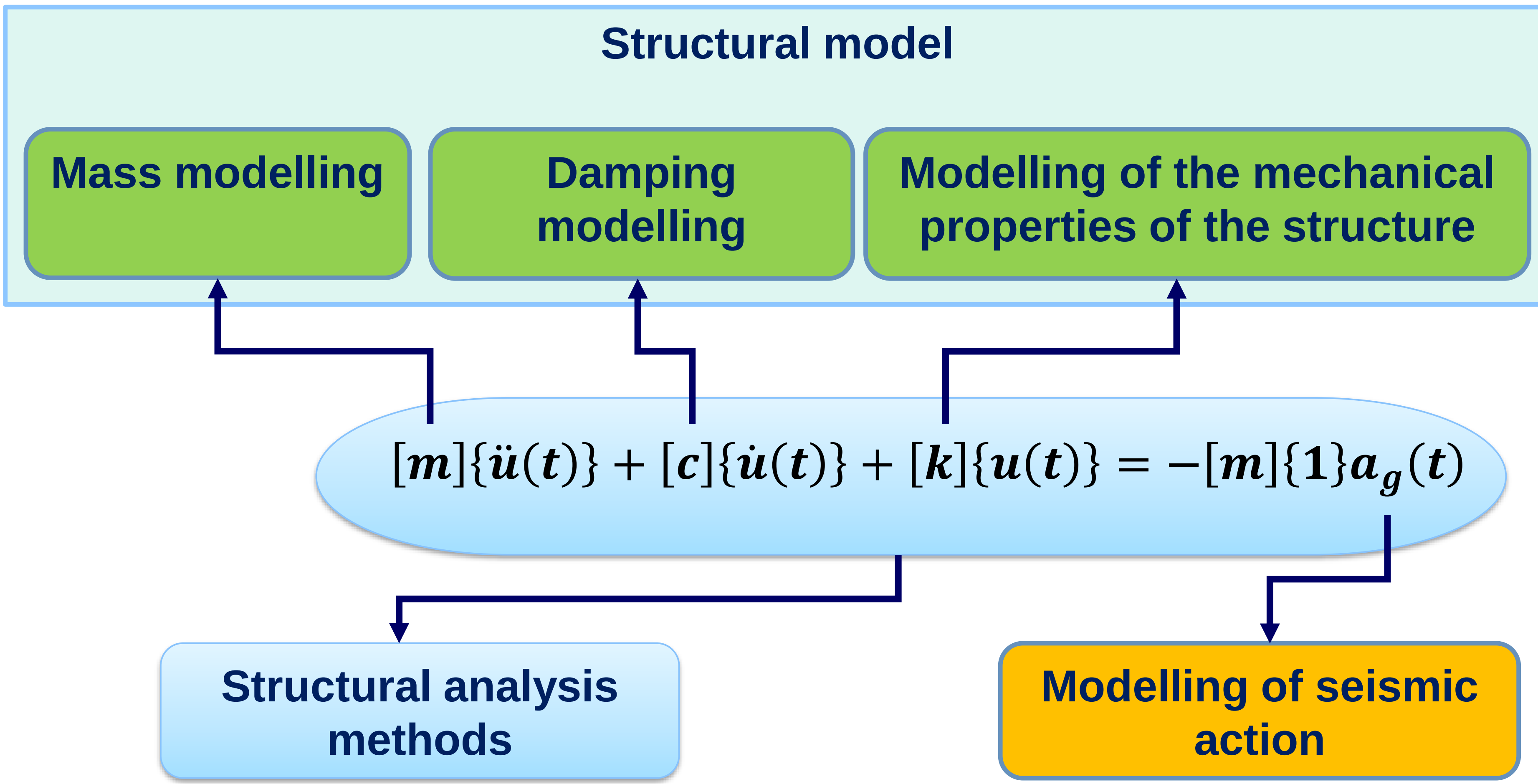
Structural analysis for seismic action



Structural analysis methods in EC8

- **Lateral force method (LFM)**
- **<u>Modal response spectrum analysis (MRS)</u>. *Default method in EC8***

Elastic analysis / Conventional design

- **Nonlinear static analysis (pushover)**
- **Nonlinear time-history analysis (NLTH)**

Plastic analysis / Advanced design

- **Linear time-history analysis (LTH)**

Elastic analysis / Not in EC8

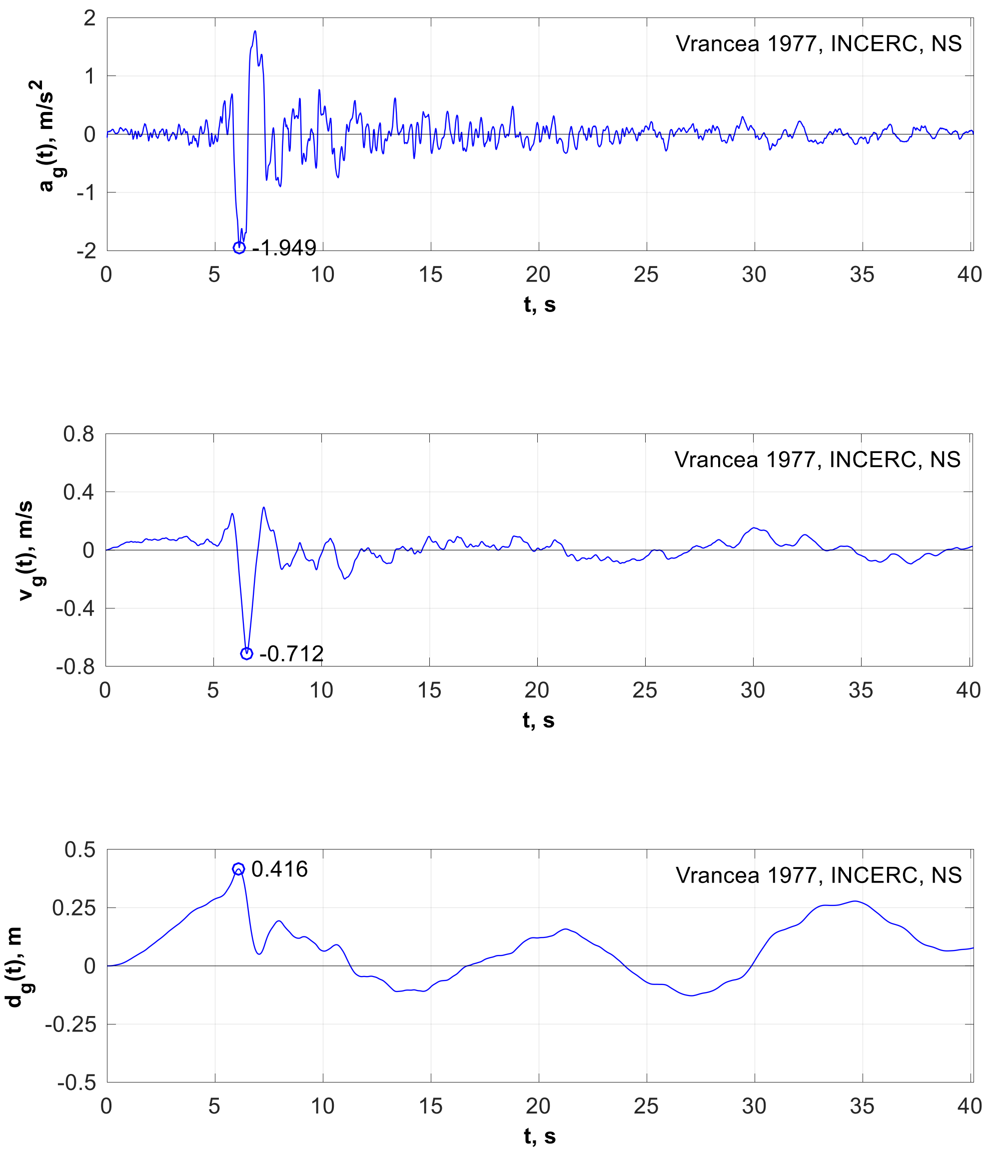
		Structural model	
		Elastic	Plastic
Time-history response	NO	LFM or MRS	Pushover
	YES	LTH	NLTH

Seismic action

$$[\boldsymbol{m}]\{\ddot{\boldsymbol{u}}(\boldsymbol{t})\} + [\boldsymbol{c}]\{\dot{\boldsymbol{u}}(\boldsymbol{t})\} + [\boldsymbol{k}]\{\boldsymbol{u}(\boldsymbol{t})\} = -[\boldsymbol{m}]\{\mathbf{1}\}\boxed{\boldsymbol{a}_{\boldsymbol{g}}(\boldsymbol{t})}$$

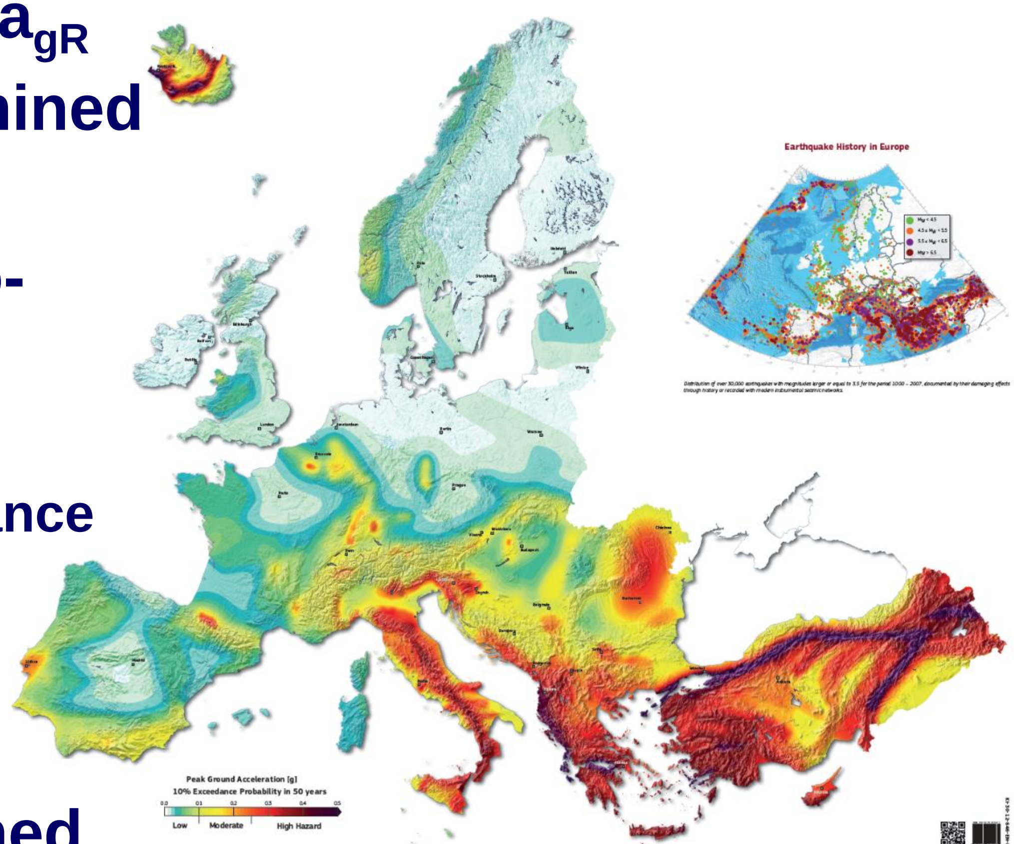
Seismic action

- **Ground shaking – the earthquake effect most relevant for the seismic design of buildings**
- **Quantified through ground acceleration, velocity and displacement time histories**

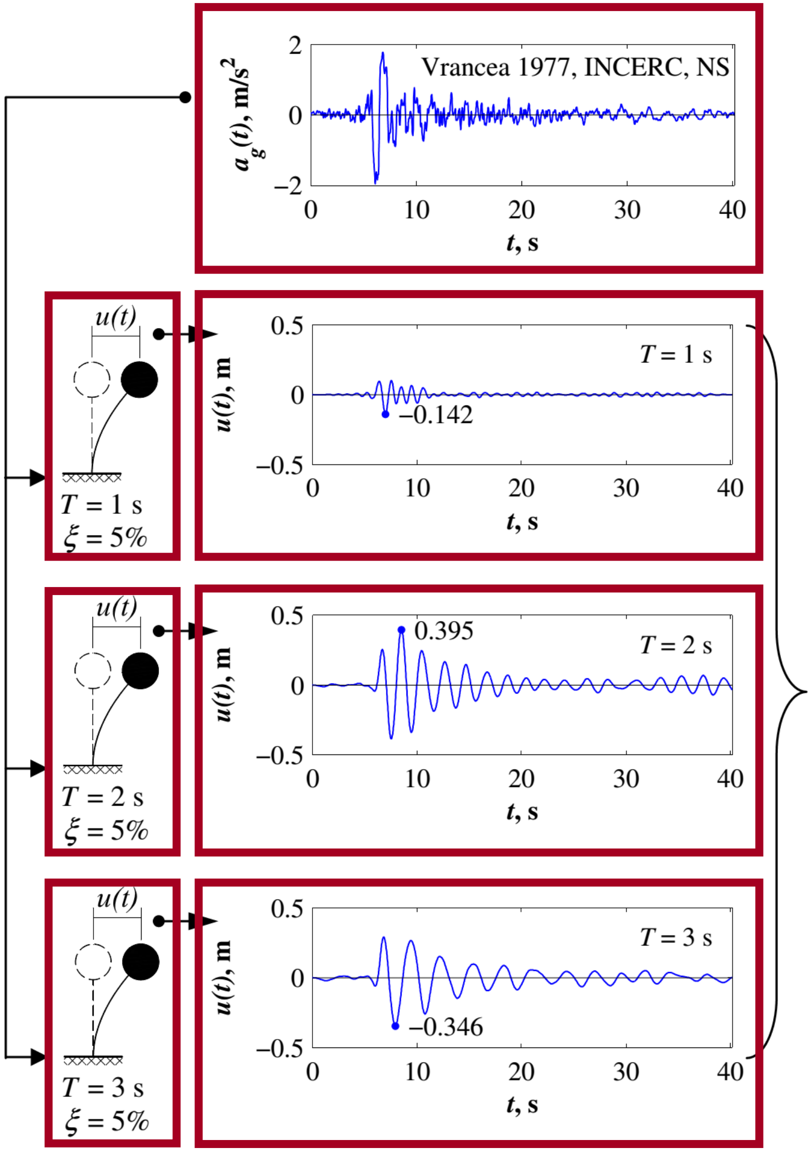


Seismic hazard

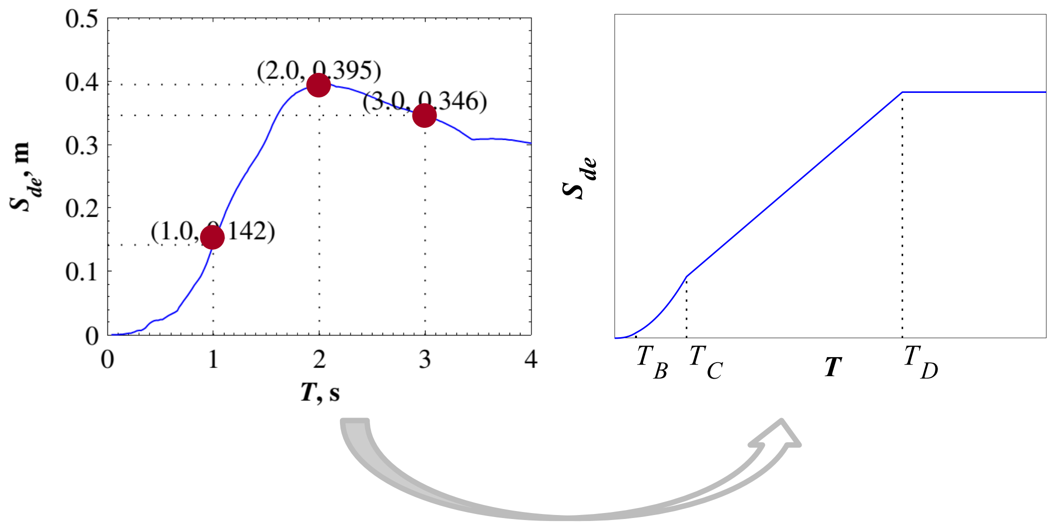
- **Seismic hazard described in EC8 in terms of the reference peak ground acceleration (PGA) on type A ground, a_{gR}**
- **a_{gR} is a Nationally Determined Parameter (NDP)**
- **Corresponds to ULS ("no-collapse" requirement)**
  - **475 years return period**
  - **10% probability of exceedance in 50 years**
- **Corresponds to an importance factor γ_I=1.0**
- **The design PGA is obtained as $a_g = \gamma_I \times a_{gR}$**



Elastic displacement response spectrum



Response spectrum: representation of peak values of seismic response (displacement, velocity, acceleration) of a SDOF system versus natural period of vibration, for a given critical damping ratio

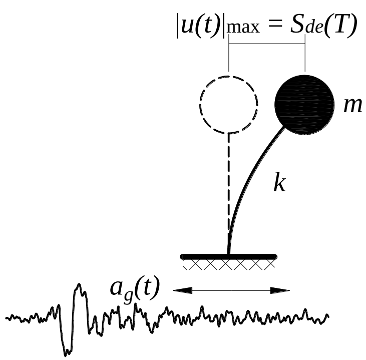


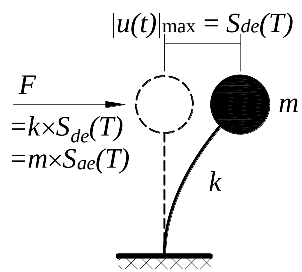
Idealised displacement resp. spectrum

Pseudo-velocity and pseudo-acceleration

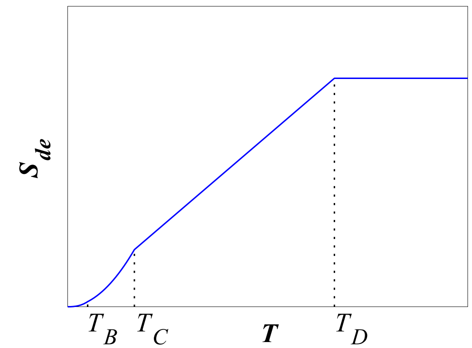
- **Equivalent static force *F* corresponding to displacement S_{de}:**

$$F = kS_{de} = m\omega_n^2 S_{de} = mS_{ae}$$

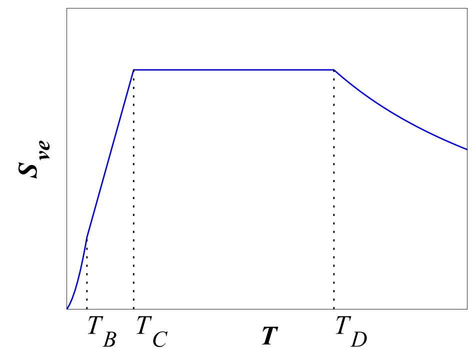




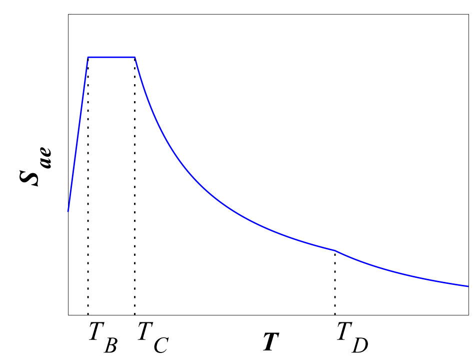
- **Intuitive:**
  Force = mass × acceleration
- **Pseudo-acceleration response spectrum used in EC8-1 for characterisation of seismic action**



$$S_{de}$$



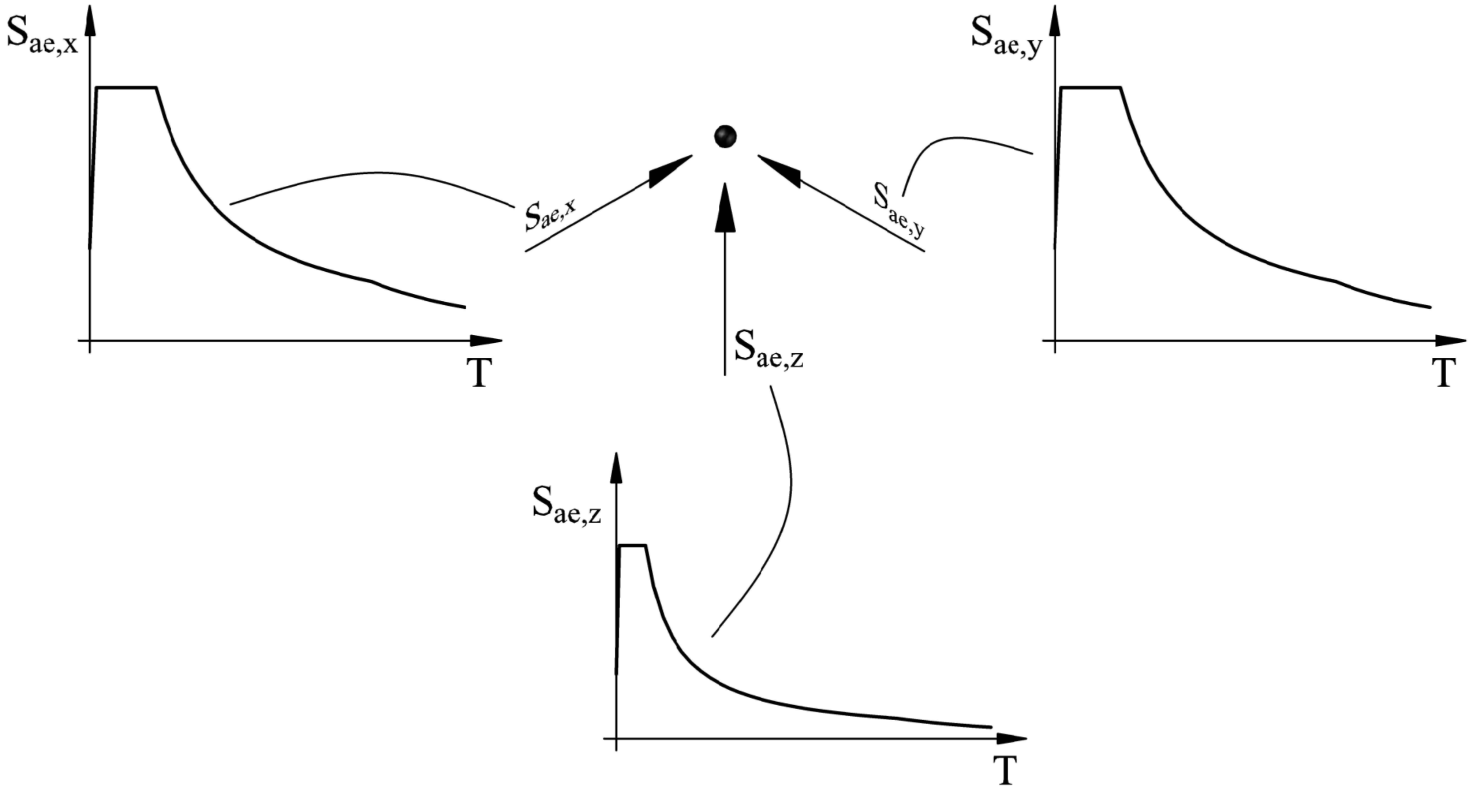
$$S_{ve} = \omega_n S_{de}$$



$$S_{ae} = \omega_n^2 S_{de}$$

Basic representation of the seismic action

- **EC8 describes the seismic action using elastic (pseudo-acceleration) response spectra**
  - Two (identical) horizontal components
  - One vertical component



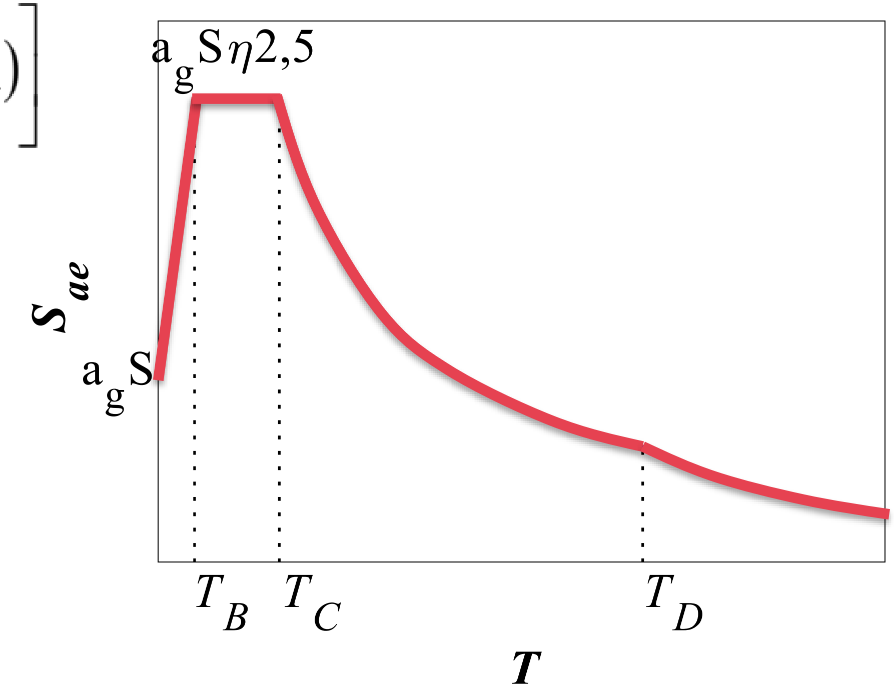
Horizontal elastic response spectrum

$$0 \le T \le T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1)\right]$$

$$T_B \le T \le T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5$$

$$T_C \le T \le T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5\left[\frac{T_C}{T}\right]$$

$$T_D \le T \le 4\text{s} : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5\left[\frac{T_C T_D}{T^2}\right]$$



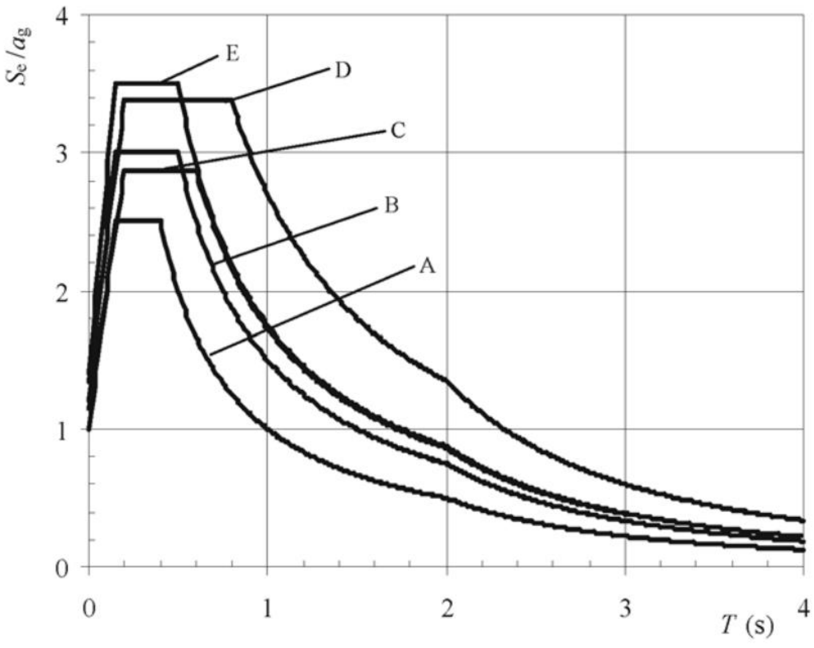
a_g – design PGA

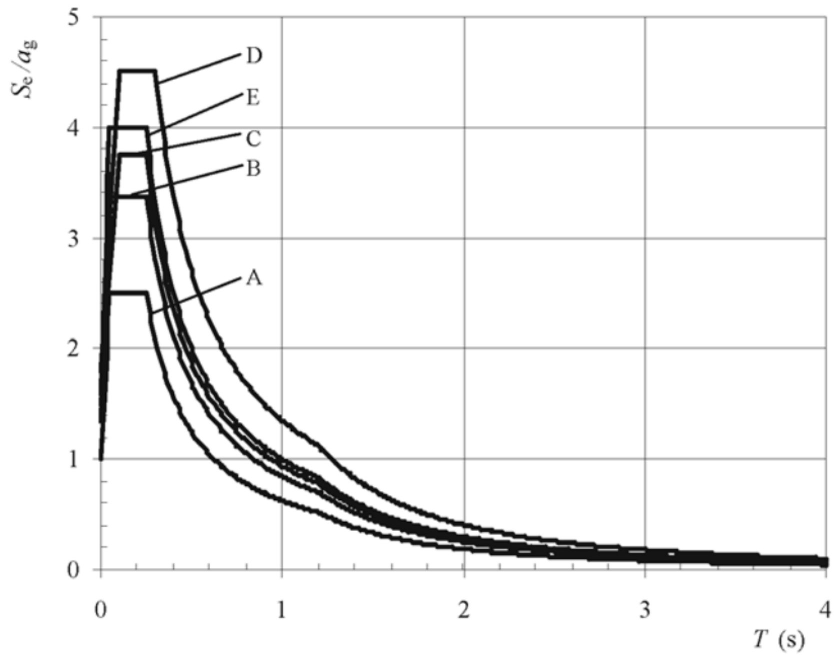
T_B, T_C, T_D – control periods

S – soil factor

η - damping correction factor. Reference value of damping is 5%, for which η=1.0

Horizontal elastic response spectrum





Type 1 spectrum

Recommended for surface-wave magnitude, $M_s > 5{,}5$

Type 2 spectrum

Recommended for surface-wave magnitude, $M_s \leq 5{,}5$

Design spectrum for elastic analysis

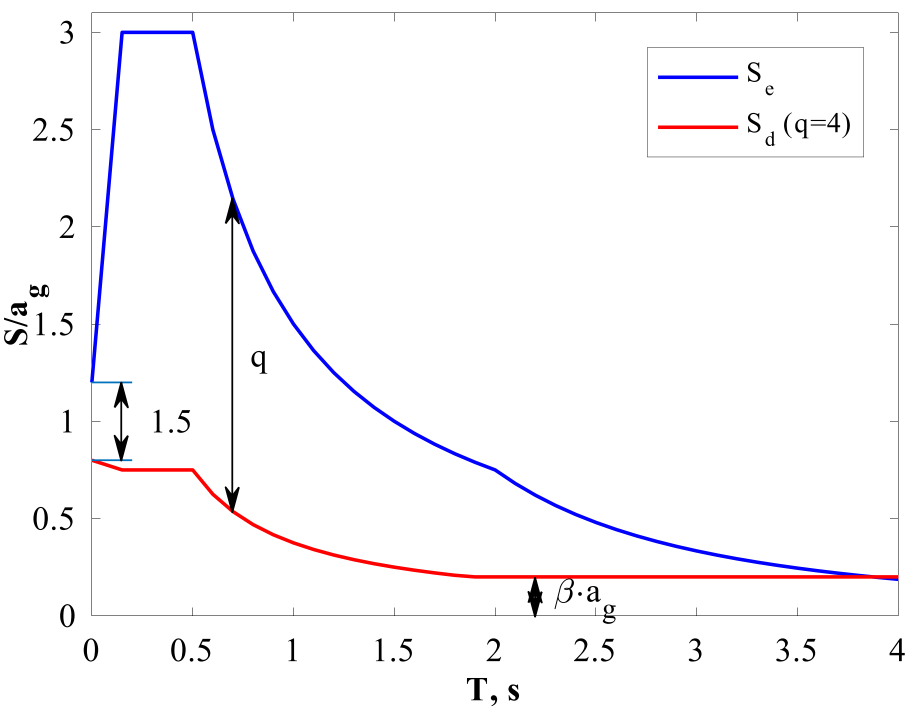
- **In an elastic analysis, the capacity of the structure to dissipate energy is taken into account by performing the analysis based on a response spectrum reduced with respect to the elastic one, called a "design spectrum".**

$$0 \le T \le T_{\mathrm{B}} : \; S_{\mathrm{d}}(T) = a_{\mathrm{g}} \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_{\mathrm{B}}} \cdot \left(\frac{2{,}5}{q} - \frac{2}{3} \right) \right]$$

$$T_{\mathrm{B}} \le T \le T_{\mathrm{C}} : \; S_{\mathrm{d}}(T) = a_{\mathrm{g}} \cdot S \cdot \frac{2{,}5}{q}$$

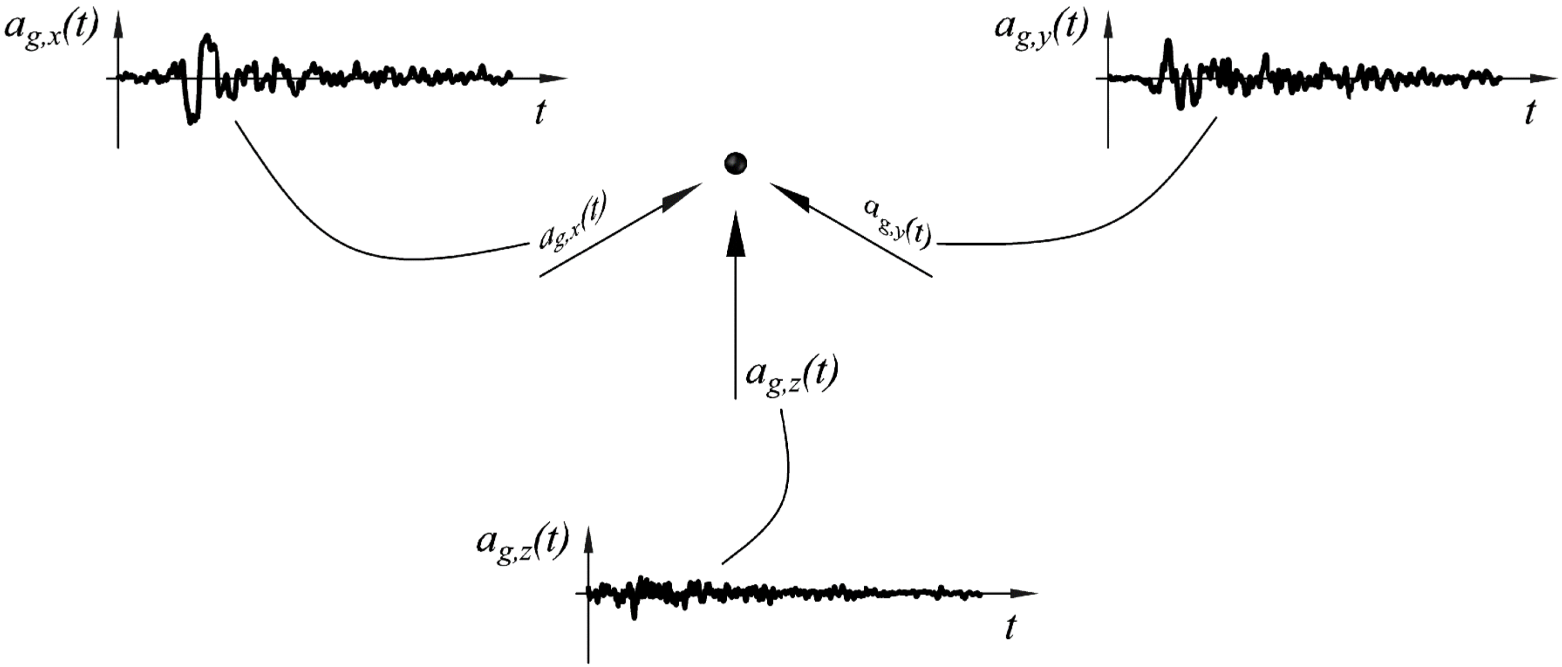
$$T_{\mathrm{C}} \le T \le T_{\mathrm{D}} : \; S_{\mathrm{d}}(T) \begin{cases} = a_{\mathrm{g}} \cdot S \cdot \dfrac{2{,}5}{q} \cdot \left[\dfrac{T_{\mathrm{C}}}{T} \right] \\ \ge \beta \cdot a_{\mathrm{g}} \end{cases}$$

$$T_{\mathrm{D}} \le T : \quad S_{\mathrm{d}}(T) \begin{cases} = a_{\mathrm{g}} \cdot S \cdot \dfrac{2{,}5}{q} \cdot \left[\dfrac{T_{\mathrm{C}} T_{\mathrm{D}}}{T^2} \right] \\ \ge \beta \cdot a_{\mathrm{g}} \end{cases}$$



"Alternative" representation of seismic action in EC8

- The seismic motion may also be represented in terms of ground acceleration time-histories (or velocity and displacement)
- When a spatial model of the structure is required, the seismic motion shall consist of three simultaneously acting accelerograms. The same accelerogram may not be used simultaneously along both horizontal directions.



Accelerograms: selection

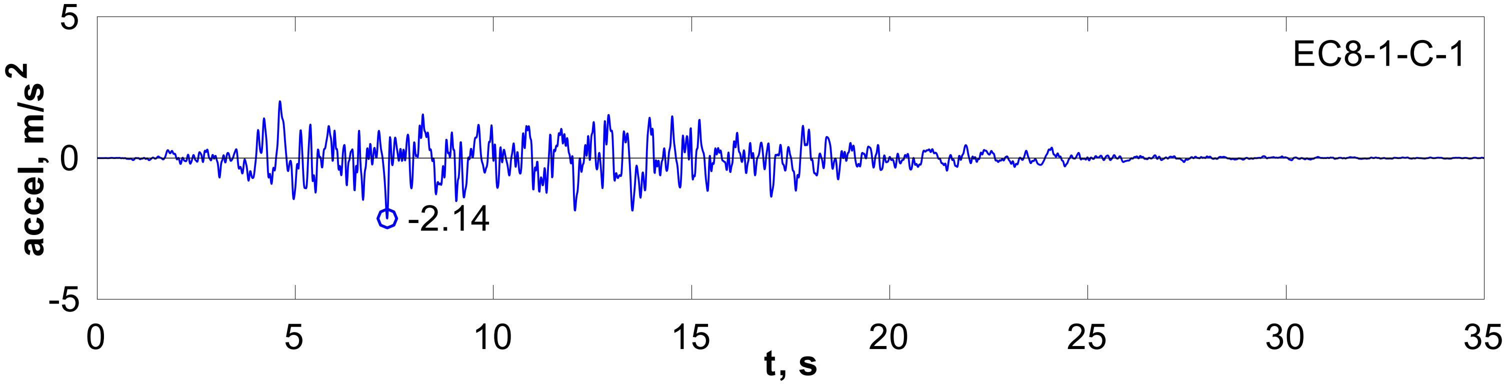
- **<u>Artificial accelerograms</u>, matching the code elastic response spectra. The duration of accelerograms should be consistent with the magnitude and other relevant features of the seismic event.**

- **<u>Recorded accelerograms</u>, provided the samples are qualified to the seismogenetic features of the source and to the soil conditions at the site.**

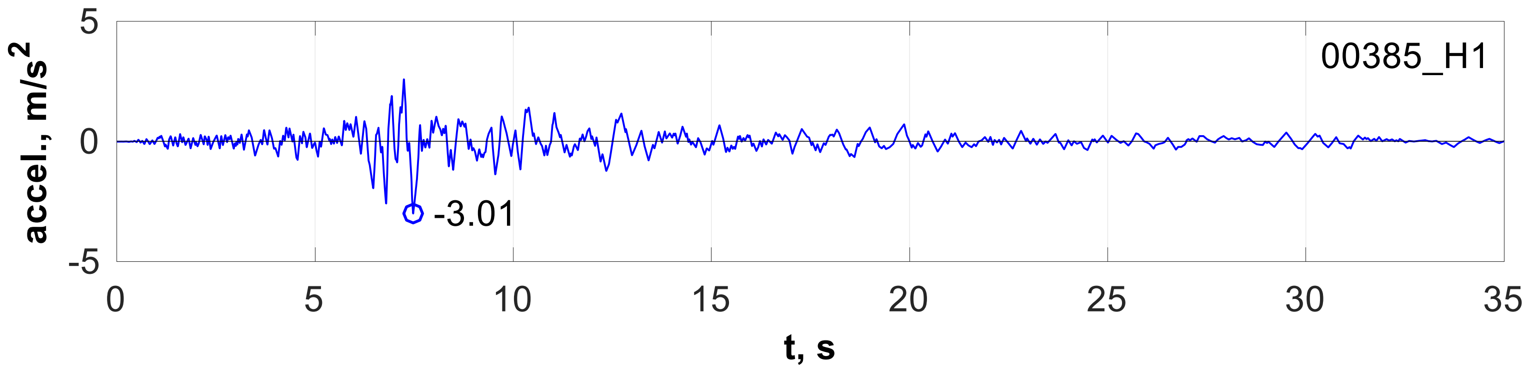
- **<u>Simulated accelerograms</u>, generated through a physical simulation of source and travel path mechanisms, complying with the requirements for recorded accelerograms.**

Accelerograms: selection

- **Artificial accelerogram**

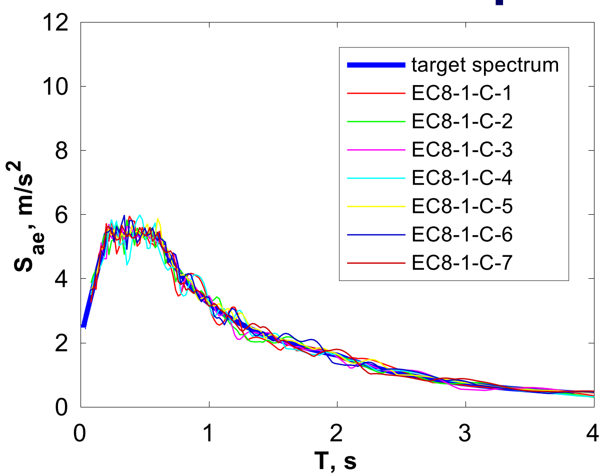


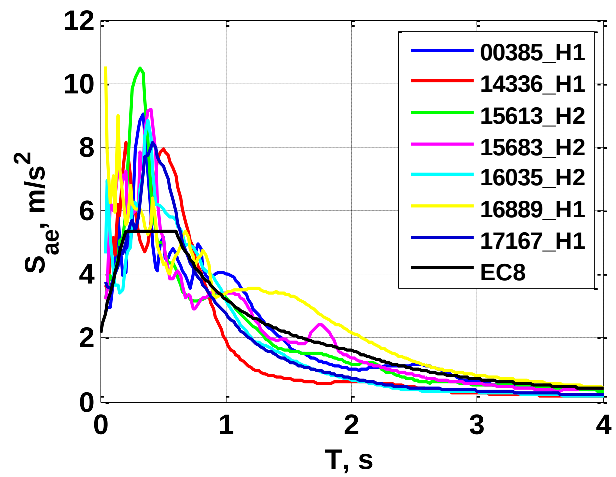
- **Recorded accelerogram**



Accelerograms: scaling

- **Eurocode 8 – for any selection procedure, the following should be observed:**
  - PGA of individual time-histories should not be smaller than the codified PGA atop of soil layers ($a_g \cdot S$)
  - In the range of periods $0.2T_1$-$2T_1$ no value of the mean spectrum, calculated from all time histories, should be less than 90% of the corresponding value of the code elastic response spectrum (lower limit ($0.2T_1$) accounts for higher modes of vibration, while upper limit (1.5-$2.0T_1$) accounts for "softening" of the structure due to inelastic response)





artificial accelerograms **recorded accelerograms**

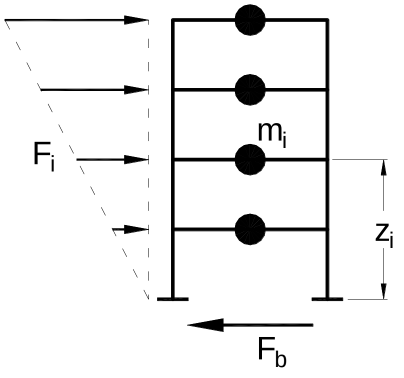
Accelerograms: number of records

- **Due to uncertainties related to characterisation of seismic motion, a large enough number of accelerograms should be used in a dynamic analysis**

- **At least <u>three</u> accelerograms $\Rightarrow$ seismic evaluation based on <u>peak</u> values of response**

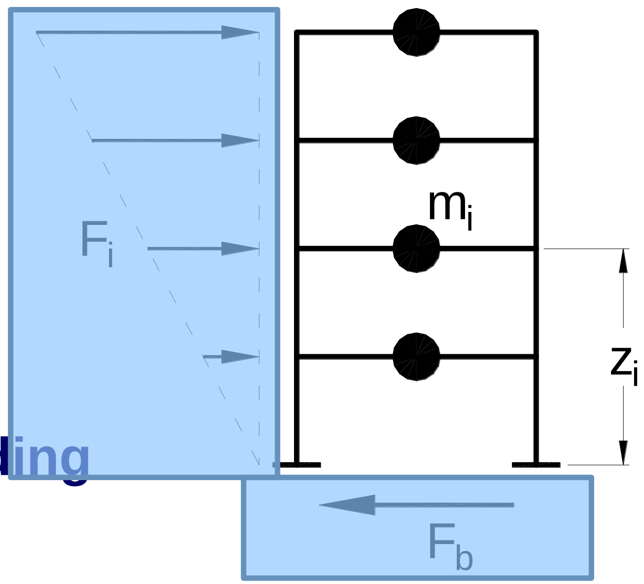
- **At least <u>seven</u> accelerograms $\Rightarrow$ seismic evaluation based on <u>mean</u> values of response**

Lateral force method



Lateral force method

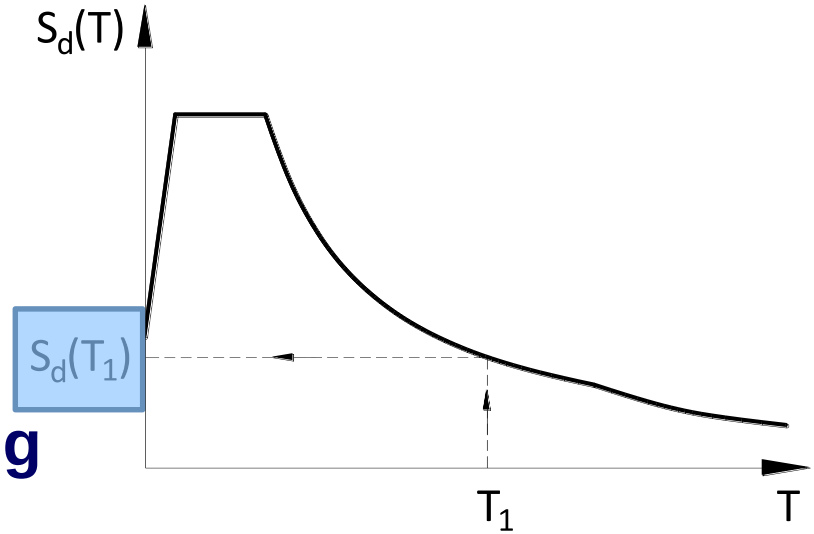
- Elastic static analysis under a set of lateral forces applied at the level of building storeys (masses)
- A simplified modal response spectrum analysis, that considers the contribution of the fundamental mode only
- May be used for structures whose seismic response is not influenced significantly by higher modes of vibration
- EN 1998-1 criteria for fulfilling this requirement:
  - fundamental period of vibration $T_1 \leq 2.0$ sec and $T_1 \leq 4T_C$
  - structure regular in elevation.
- Procedure:
  - Compute the base shear force F_b
  - Determine lateral forces F_i by distributing the base shear over the height of the building



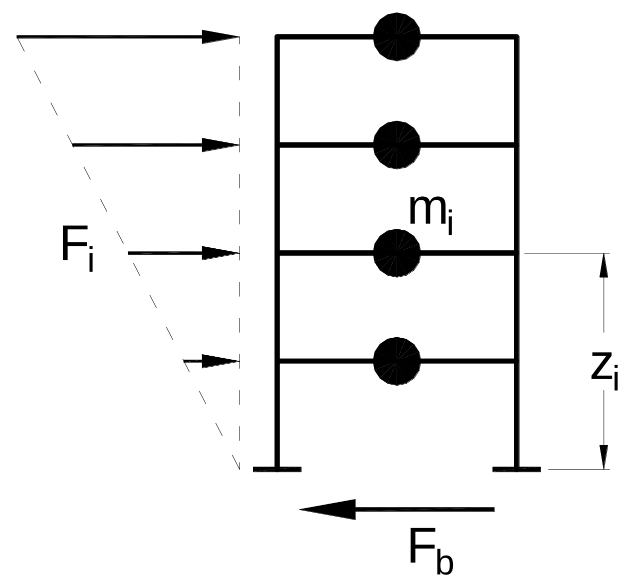
Lateral force method

- **Base shear force (EN 1998-1):**

$$F_b = S_d(T_1)\, m\, \lambda$$



- **$S_d(T_1)$ - ordinate of the design response spectrum corresponding to the fundamental period T_1**
- **$m = \sum m_i$ - total mass of the structure**
- **λ - correction factor (contribution of the fundamental mode of vibration using the concept of effective modal mass):**
  $\lambda = 0.85$ if $T_1 \le T_C$ and the structure is higher than two storeys, and
  $\lambda = 1.0$ in all other cases



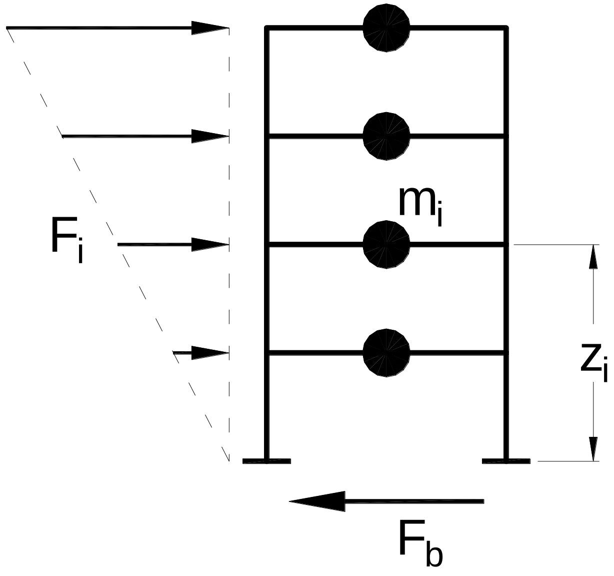
Lateral force method

- **Lateral force at storey *i* (EN 1998-1):**
  - **F_b - base shear force in the fundamental mode of vibration**
  - **s_i - displacement of the mass *i* in the fundamental mode shape. Have to be obtained from an eigenvalue analysis.**
  - **n - number of storeys in the structure**
  - **m_i - storey mass**

$$F_i = F_b \frac{m_i s_i}{\sum_{i=1}^{N} m_i s_i}$$

- **Fundamental mode shape may be approximated by horizontal displacements increasing linearly with height (avoiding eigenvalue analysis)**

$$F_i = F_b \frac{m_i z_i}{\sum_{i=1}^{N} m_i z_i}$$



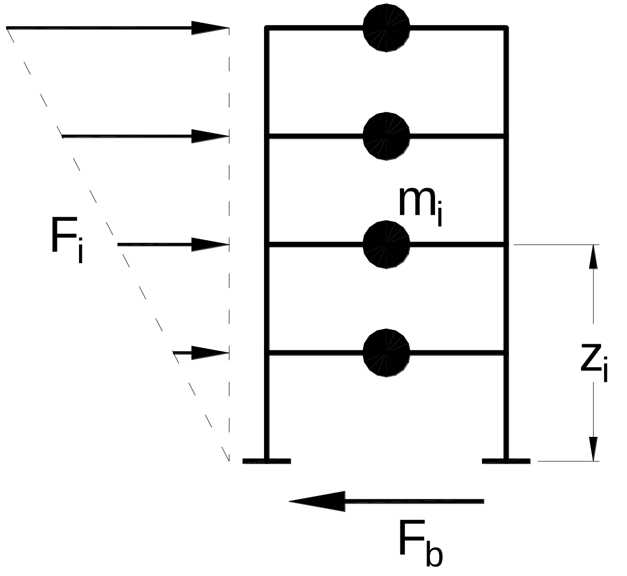
Lateral force method

- **For structures with height <40m the fundamental mode shape may be approximated using empirical formula:**

$$T_1 = C_t H^{3/4}$$

  - **C_t = 0.085 moment-resisting steel frames,**
  - **C_t = 0.075 moment resisting reinforced concrete frames or steel eccentrically braced frames,**
  - **C_t = 0.05 all other structures.**

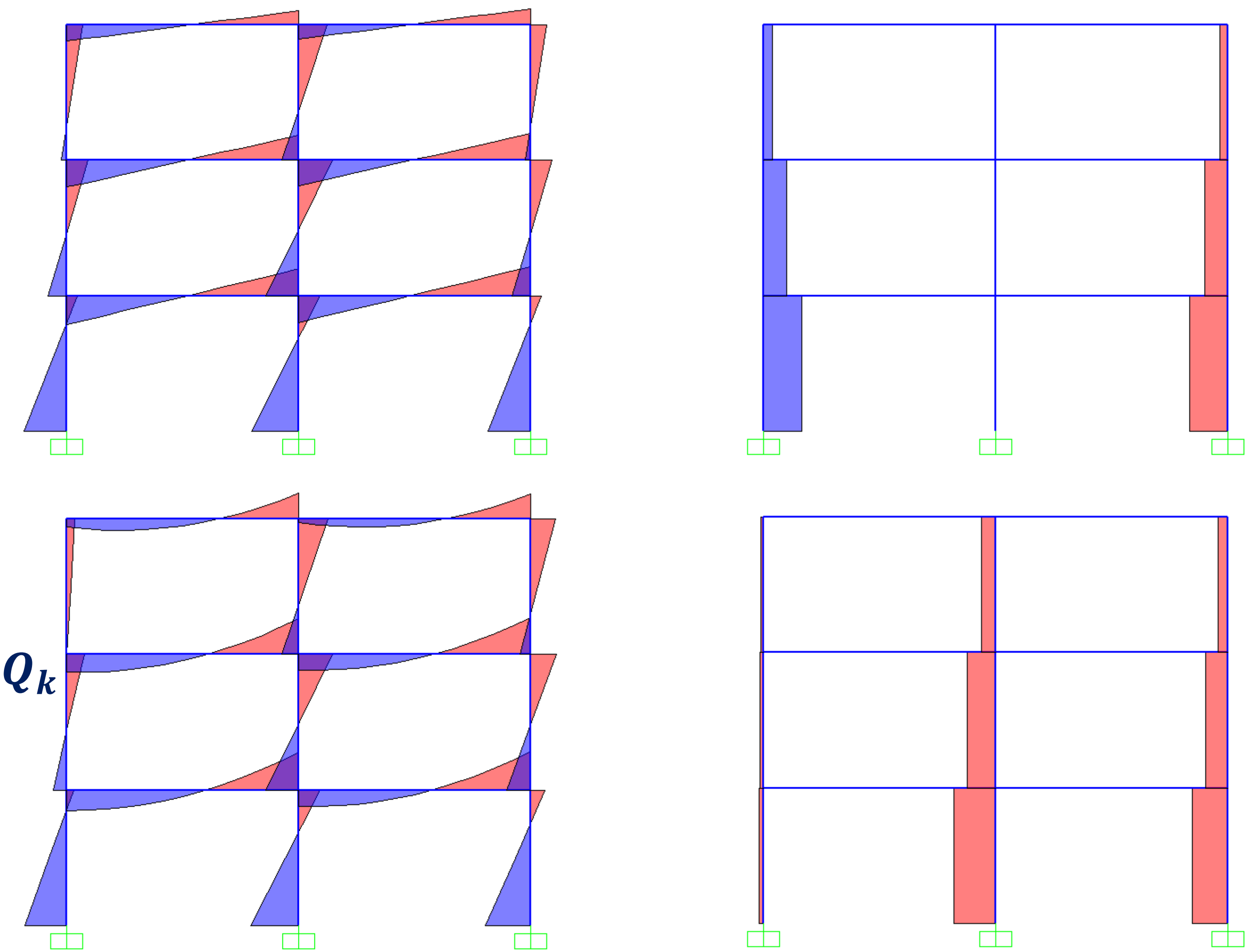
- **Empirical formulation of T_1 generally conservative (smaller values than those obtained using eigenvalue analysis)**



Lateral force method

A_{ed}

$$\sum G_k + \sum \psi_2 Q_k + A_{Ed}$$



M **N**

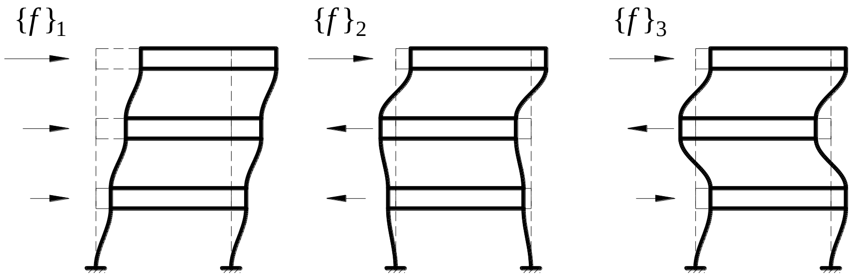
Lateral force method

- The lateral force method provides peak values of response (forces, moments), which is convenient for design but
  - Forces and moments are conventional (corresponding to design response spectrum, reduced using behaviour factor q)
  - Lateral deformations obtained from analysis are incorrect, needing further calculation

- May be applied on 2D and 3D models of the structure

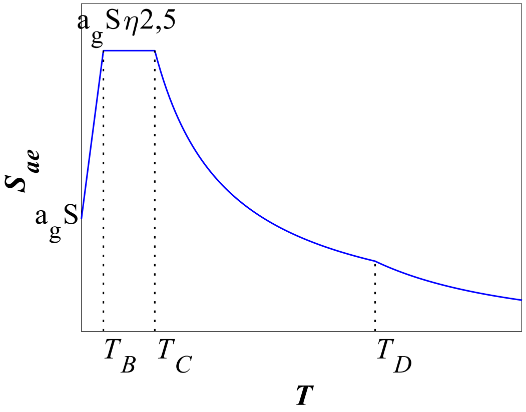
- Shall not be used when vertical component of the seismic action shall be accounted for

Modal response spectrum analysis



Modal response spectrum (MRS) analysis

- The equation of motion is solved by decoupling the system of N differential equations into N independent equations using modal superposition

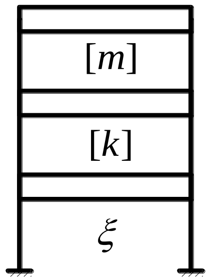


- Seismic action is modelled using response spectra

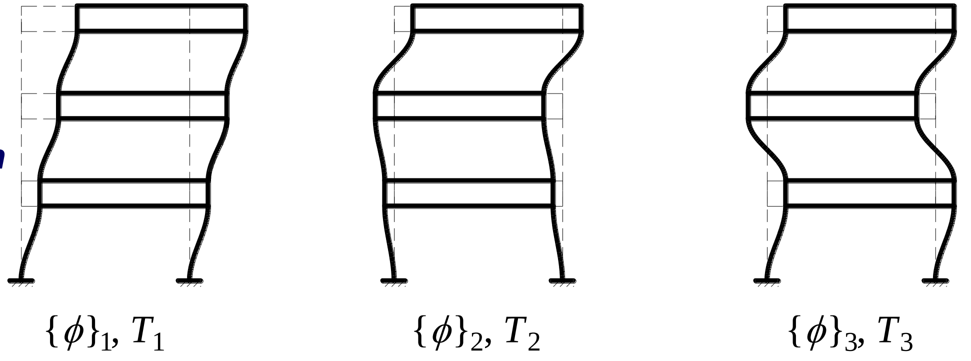
- Modal response spectrum analysis:
  - Is the default analysis method in EC8
  - May be used in al cases
  - Is compulsory for structures that cannot be analysed using the lateral force method

MRS analysis: procedure

1. **Define structural properties**
   - mass [*m*] and stiffness [*k*] matrices
   - critical damping ratio ξ_n
   (full damping matrix is NOT necessary)

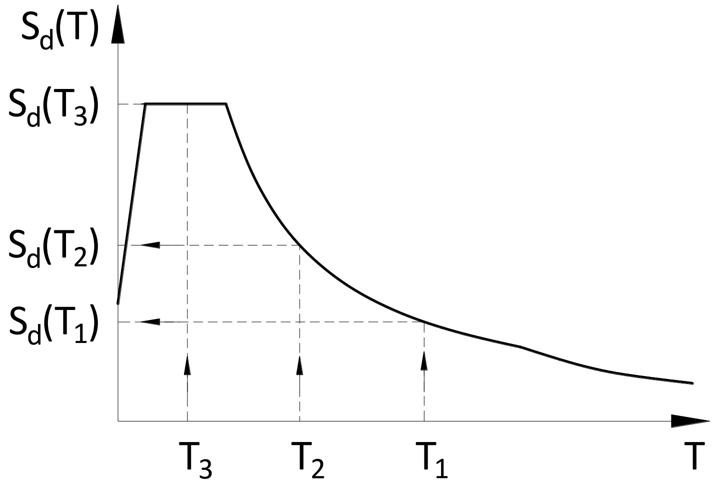


2. **Determine modal properties of the structure using eigenanalysis:**
   – **Natural modes of vibration $\{\phi\}_n$**
   – **Natural periods and corresponding circular frequencies $T_n = 2\pi/\omega_n$**

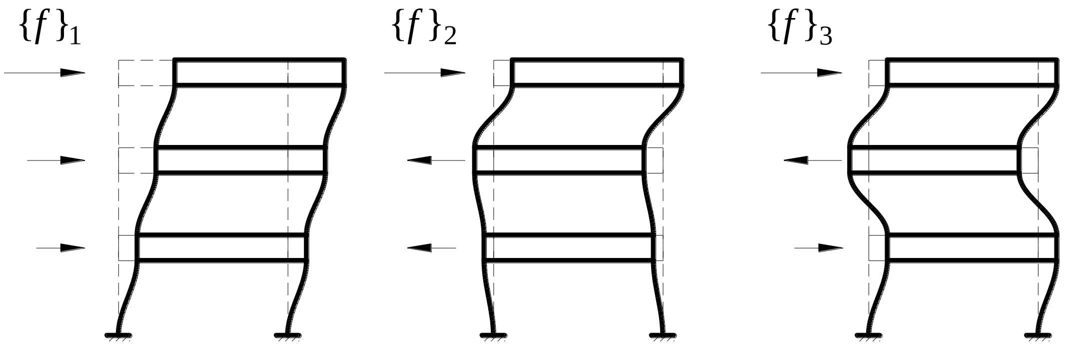


MRS analysis: procedure

3. **For each mode of vibration *n*:**
   - **Obtain spectral accelerations $S_d(T_n)$ corresponding to periods T_n**

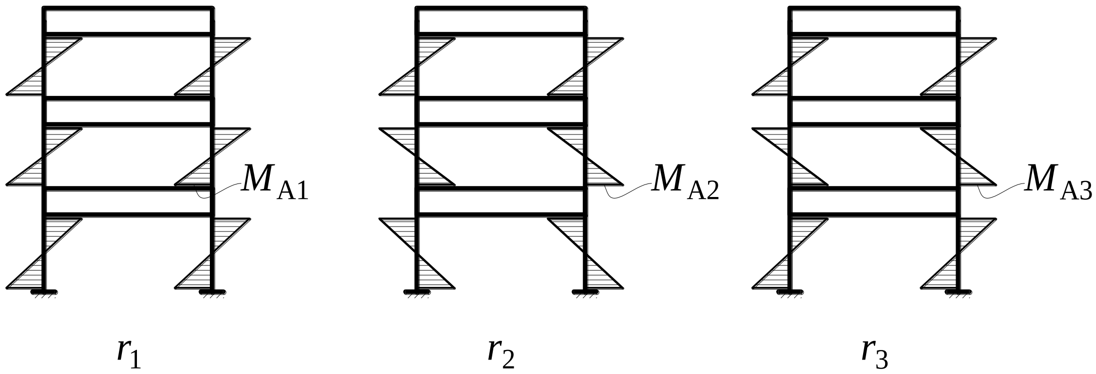


   - **Compute the equivalent static forces $\{f\}_n$**

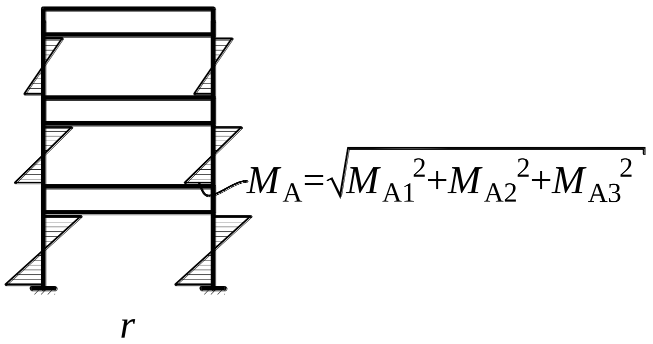


MRS analysis: procedure

– **compute response quantity r_n from forces $\{f\}_n$, for each response quantity**



4. **Combine modal contributions r_n to obtain total response using ABS, SRSS or CQC combination methods**

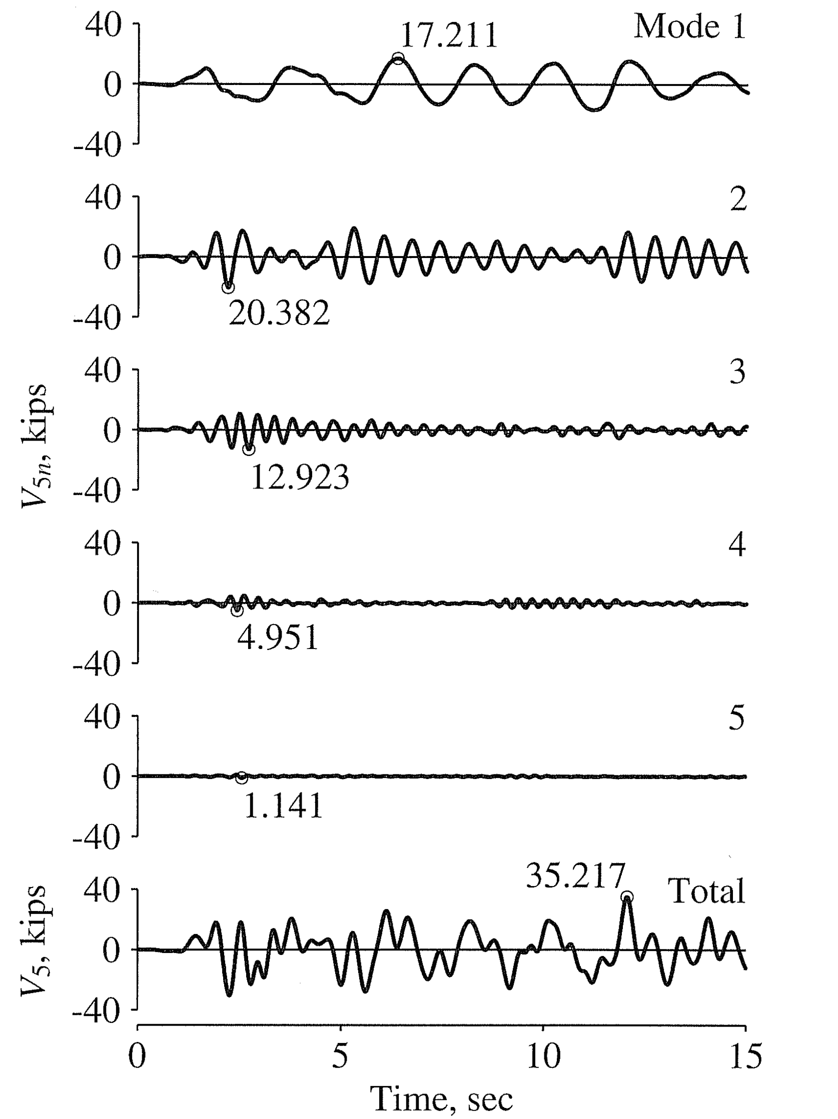


MRS analysis: modal combination rules

- **Generally time-history response in different modes of vibrations is different**

- **Peak response occurs at different times in different modes of vibrations. However, in MRS analysis the time information is unknown, due to response spectra.**

- **Total response need to be estimated using statistical modal combination rules**



MRS analysis: modal combination rules

- **Sum of absolute values (ABS): if consecutive modes are not independent ($T_k \cong T_{k+1}$)**

$$r = \sum_{n=1}^{N} |r_n|$$

- **Square root of sum of squares (SRSS): if consecutive modes are independent ($T_k \neq T_{k+1}$)**

*** Response in two modes *k* and *k*+1 may be considered independent if $T_{k+1} \leq 0.9T_k$**

$$r = \sqrt{\sum_{n=1}^{N} r_n^2}$$

- **Complete quadratic combination (CQC): may be used always. Recommended when structural analysis software is used.**

$$r = \sqrt{\sum_{i=1}^{N} \sum_{n=1}^{N} \rho_{in} r_i r_n}$$

MRS analysis: modal mass

- **Effective modal mass quantifies the contribution of the n-th mode of vibration to the total response (in terms of base shear)**

  $$M_n^* = \frac{\left(\sum_{j=1}^{n} m_j \phi_{jn}\right)^2}{\sum_{j=1}^{n} m_j \phi_{jn}^2}$$

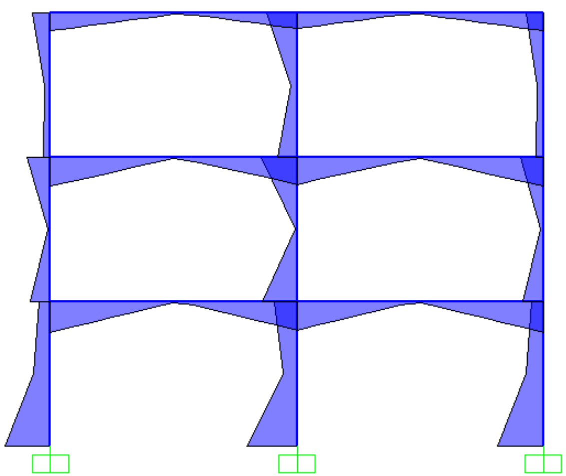
  The sum of effective modal masses over all *N* modes is equal to the total mass of the structure

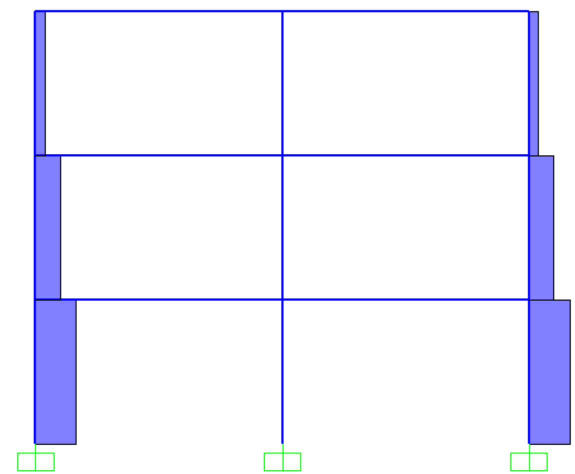
  $$\sum_{n=1}^{N} M_n^* = \sum_{j=1}^{N} m_j$$

- **For a structure with many dynamic degrees of freedom (DOFs) it is not feasible considering ALL modes of vibration**
- **Number of modes that need to be considered in analysis:**
  - **the sum of effective modal masses for the considered modes should amount to at least 90% of the total mass of the structure,**
  - **all modes with effective modal mass larger than 5% of the total mass of the structure were considered in analysis**
- **The 90% rule for effective modal mass shall be verified for each relevant direction of the structure**

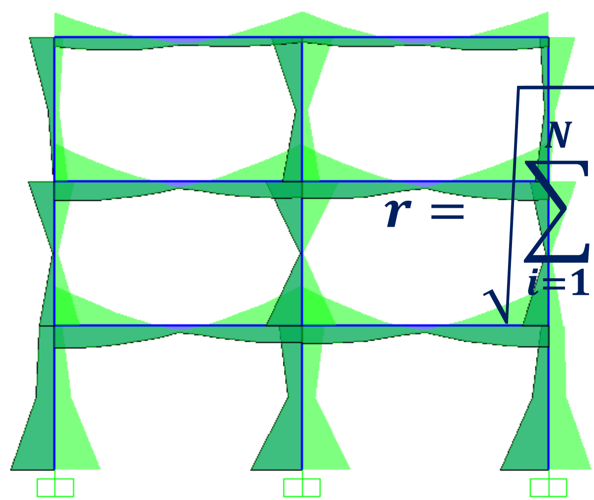
Modal response spectrum analysis

A_{ed}

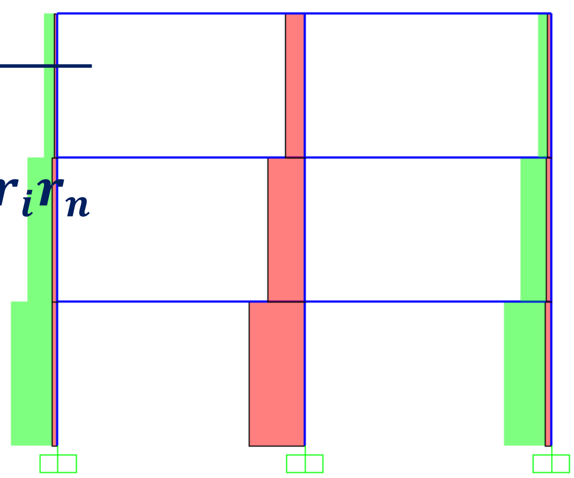




$$\sum G_k + \sum \psi_2 Q_k + A_{Ed}$$



$$r = \sqrt{\sum_{i=1}^{N} \sum_{n=1}^{N} \rho_{in} r_i r_n}$$

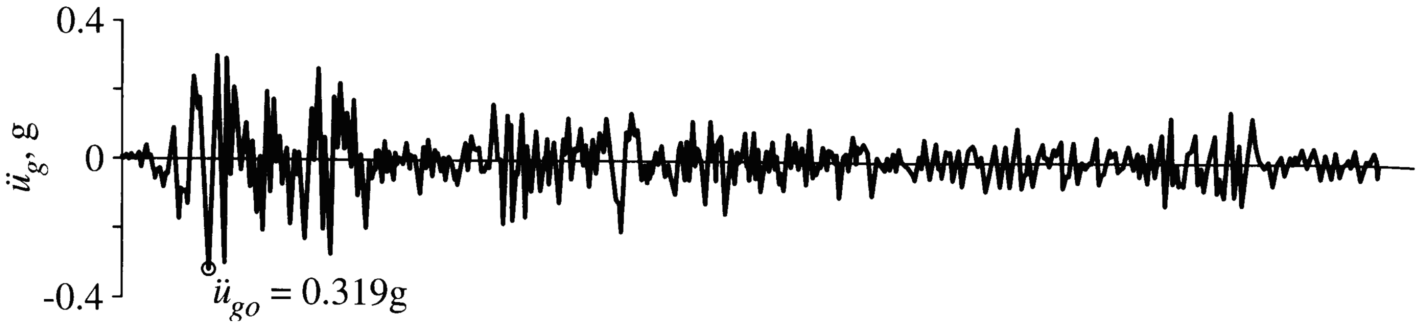


M **N**

MRS analysis

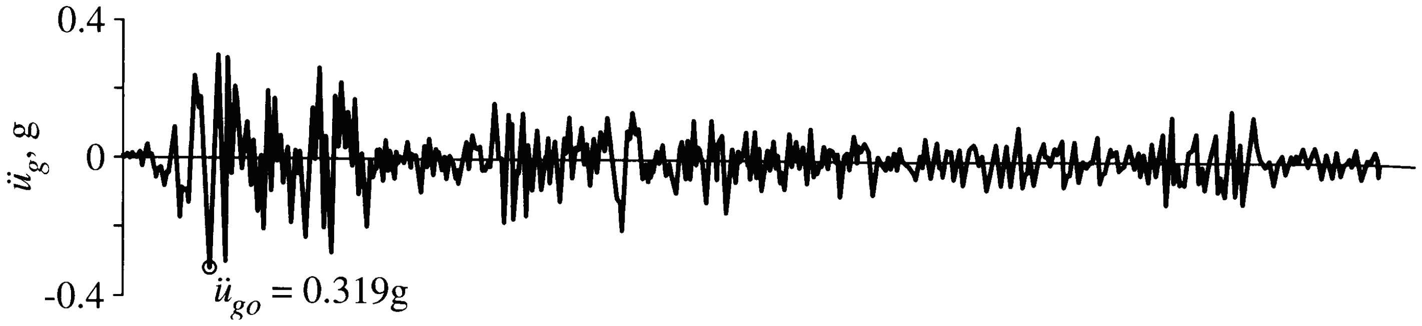
- **MRS analysis provides peak values of response (forces, moments), which is convenient for design but**
  - **Forces and moments are conventional (corresponding to design response spectrum, reduced using behaviour factor q)**
  - **Lateral deformations obtained from analysis are incorrect, needing further calculation**
- **May be applied on 2D and 3D models of the structure**
- **May include the vertical component of the seismic action**
- **Major drawbacks:**
  - **Response quantities "loose" their signs, due to modal combination rules.**
  - **Correlation of response quantities is unknown (e.g. M-N).**
- **Whenever possible, compute local response quantities (e.g. lateral interstorey drifts, stresses) directly using the MRS analysis, and not by using other results of the analysis**

Linear time-history analysis



Linear time-history (LTH) analysis

- **Modelling of seismic action: accelerograms digitized at time steps of 0.005 – 0.02 sec**



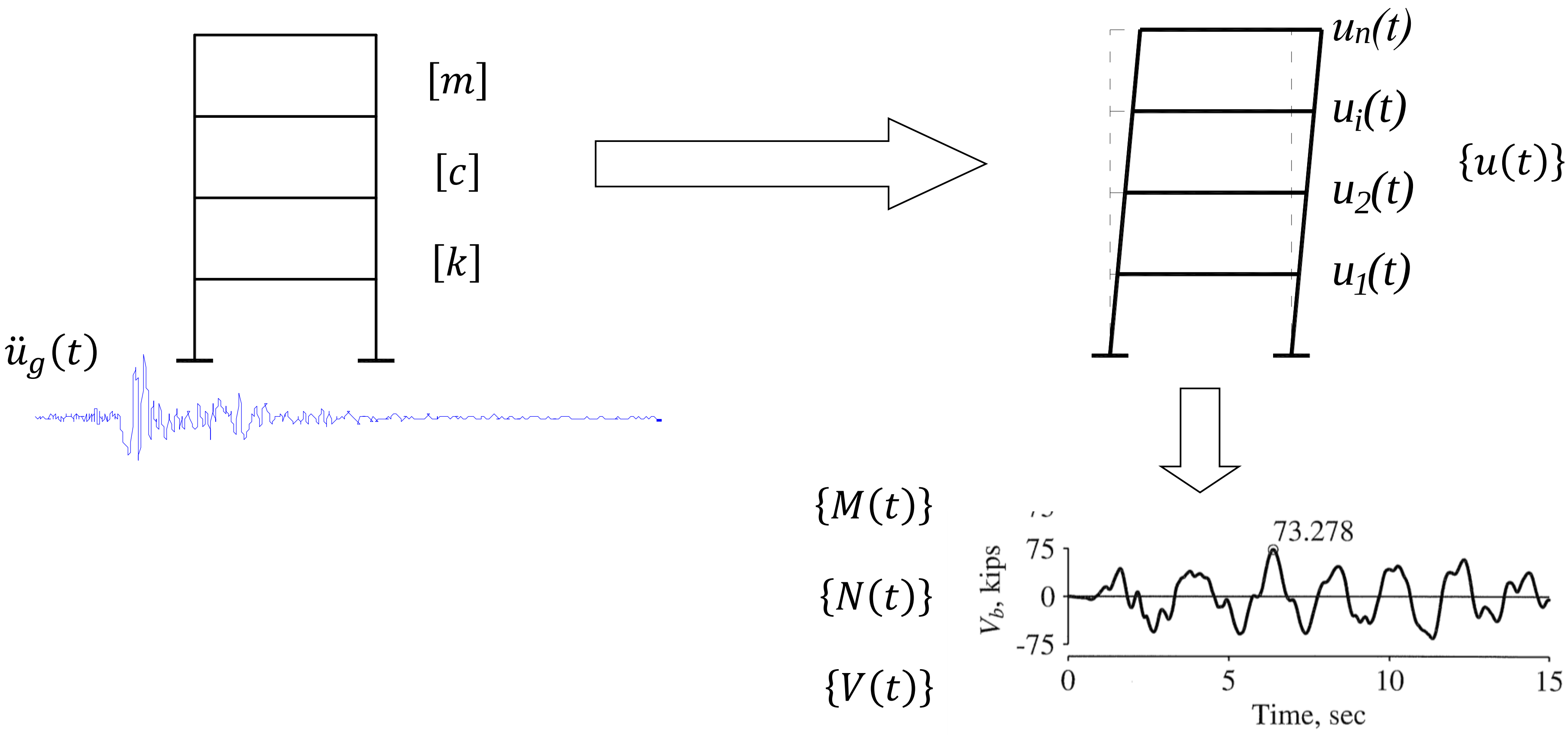
- **Elastic response of structural components**

- **Time history response is obtained through direct integration (numerical methods) of the equation of motion**

Linear time-history (LTH) analysis

- **For a system with *N* degrees of freedom, there are *N* coupled differential equations to be solved numerically**

$$[\boldsymbol{m}]\{\ddot{\boldsymbol{u}}\} + [\boldsymbol{c}]\{\dot{\boldsymbol{u}}\} + [\boldsymbol{k}]\{\boldsymbol{u}\} = -[\boldsymbol{m}]\{\boldsymbol{1}\}\ddot{\boldsymbol{u}}_{\boldsymbol{g}}(\boldsymbol{t})$$

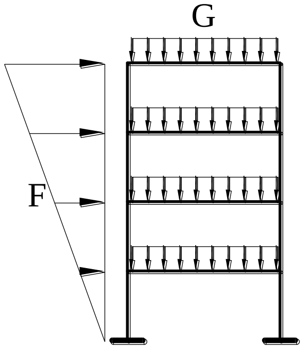


Linear time-history (LTH) analysis

- **Advantages of linear dynamic analysis over modal response spectrum analysis:**
  - **it is more accurate mathematically,**
  - **signs of response quantities (such as tension or compression in a brace) are not lost as a result of the combination of modal responses, and**
  - **story drifts are computed more accurately.**

- **The main disadvantages of linear dynamic analysis are:**
  - **the need to select and scale an appropriate suite of ground motions, and**
  - **analysis is resource-intensive**
  - **large amount of results $\Rightarrow$ a time-consuming post-processing of results.**

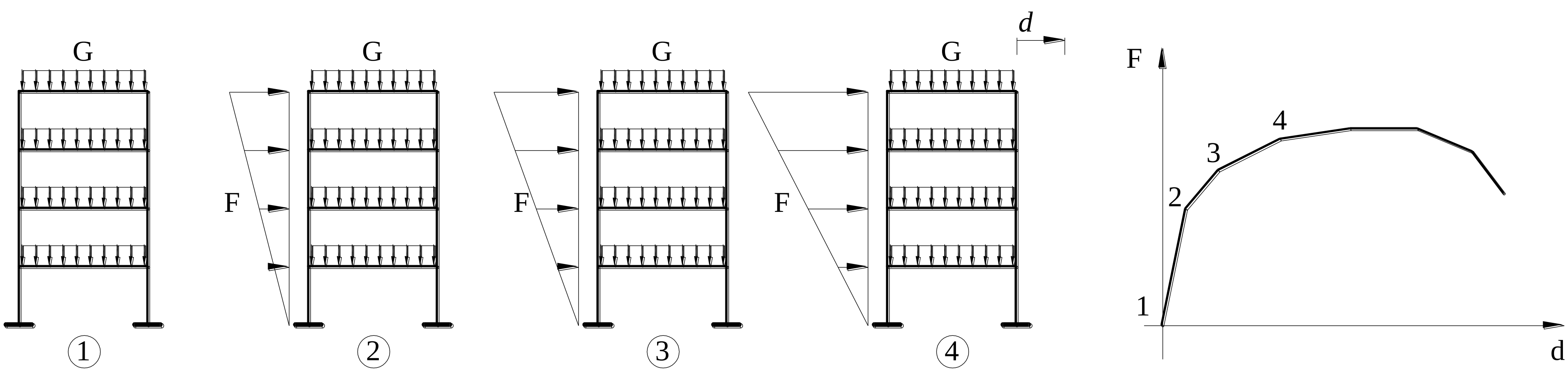
Nonlinear static analysis



Nonlinear static analysis (pushover)

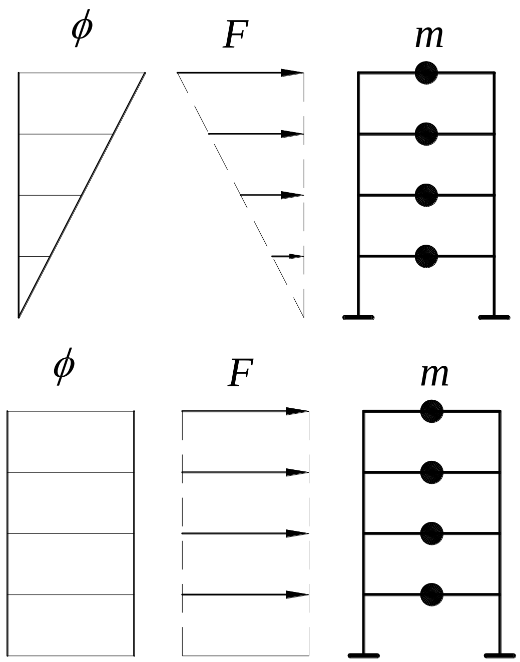
- **Nonlinear static analysis under constant gravity loading and monotonically increasing lateral forces (whose distribution represents the inertia forces expected during ground shaking)**
- **Application of loading:**
  - **Gravity loading: force control**
  - **Lateral forces: displacement control**
- **Control elements:**
  - **Base shear force**
  - **Control displacement (top displacement)**

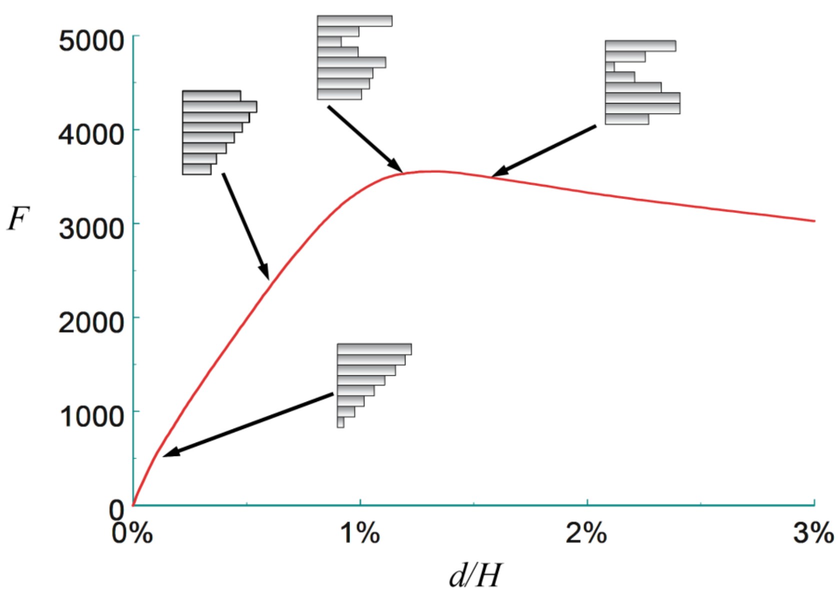
pushover (capacity) curve



Nonlinear static analysis (pushover)

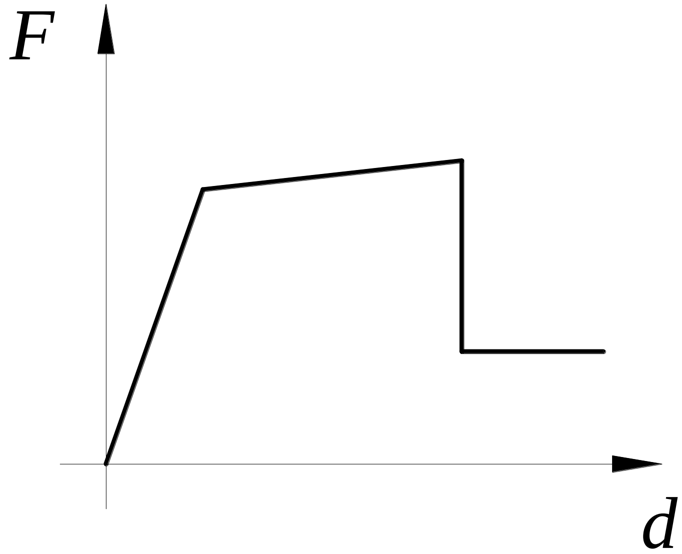
- **Assumes that response is governed by a single mode of vibration, and that it is constant during the analysis**
- **Distribution of lateral forces (applied at storey masses):**
  - **modal (usually first mode – inverted triangle)**
  - **uniform: lateral forces proportional to storey masses**
  - **"adaptive" distributions possible, but less common, requiring specialised software**





Nonlinear static analysis (pushover)

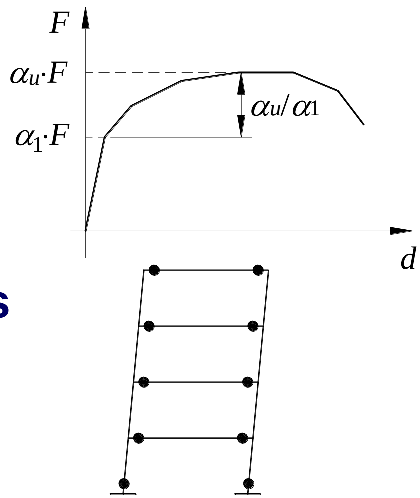
- **Modelling of structural components: inelastic monotonic force-deformation obtained from envelopes of cyclic response**



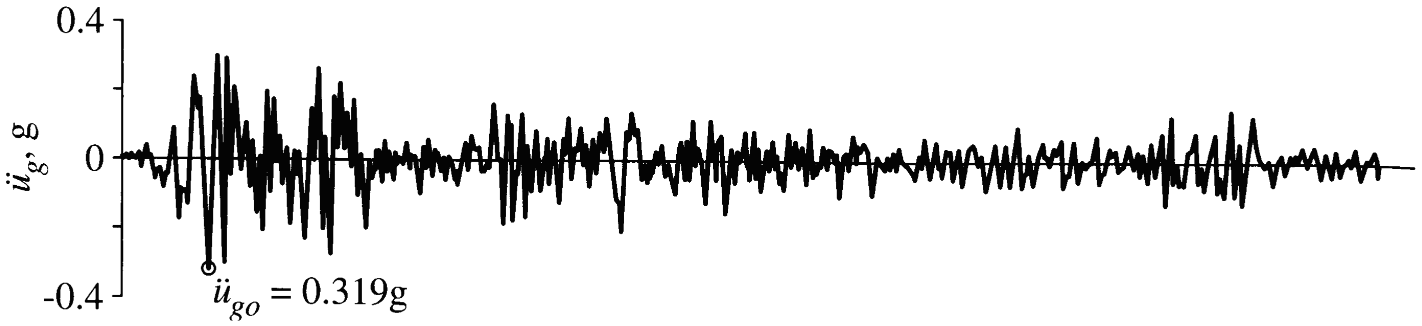
- **Applicable to low-rise regular buildings, where the response is dominated by the fundamental mode of vibration.**
- **Provides the capacity of the structure and does not give directly the demands associated with a level of seismic action. *Demands may be obtained using the N2 method.***

Nonlinear static analysis (pushover)

- Represents a direct evaluation of overall structural response, though failure not straightforward to model and assess
  - Failure to resist further gravity loading
  - Failure of the first vertical element essential for stability of the structure
- Allows evaluation of plastic deformations – the most relevant response quantity in the case of plastic response
- Allows evaluation of the plastic mechanism and redundancy of the structure (α_u/α_1 ratio)
- Design (performance assessment) criteria:
  - Plastic deformation demands in dissipative components
  - Strength demands in non-dissipative components
- Incomplete guidance in EC8-1 on nonlinear modelling of structural components. Additional information on performance criteria available in EC8-3 (Evaluation of existing structures).

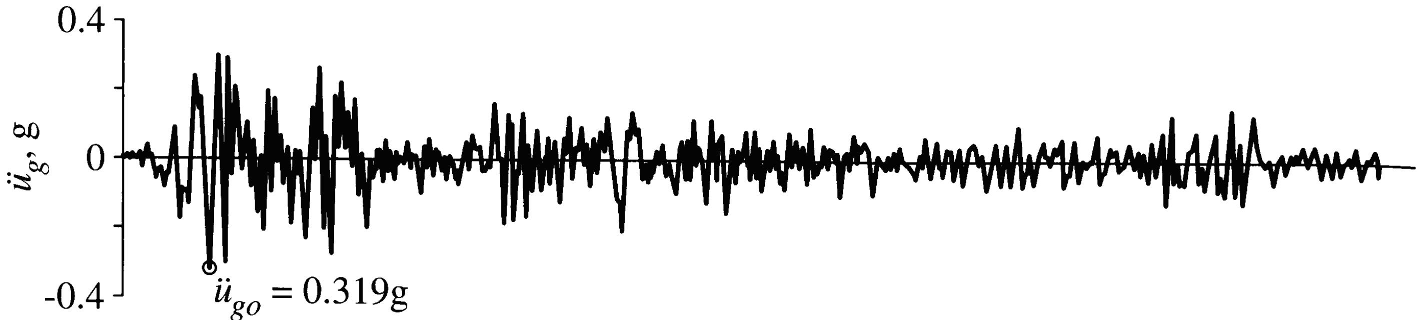


Nonlinear time-history (NLTH) analysis

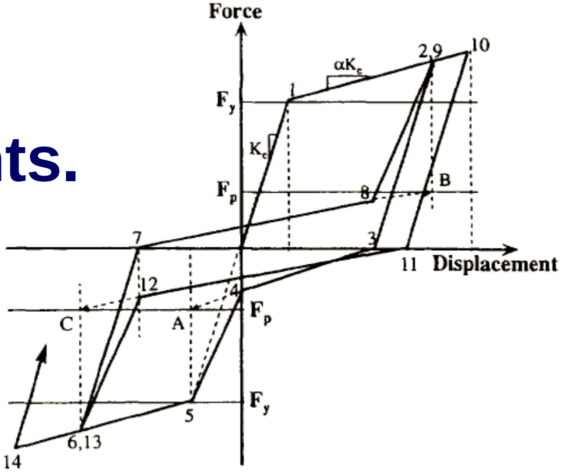


Nonlinear time-history (NLTH) analysis

- **Modelling of seismic action: accelerograms digitized at time steps of 0.005 – 0.02 sec**



- **Plastic response of structural components. Structural model should include cyclic response of members, and, eventually, strength and stiffness degradation**

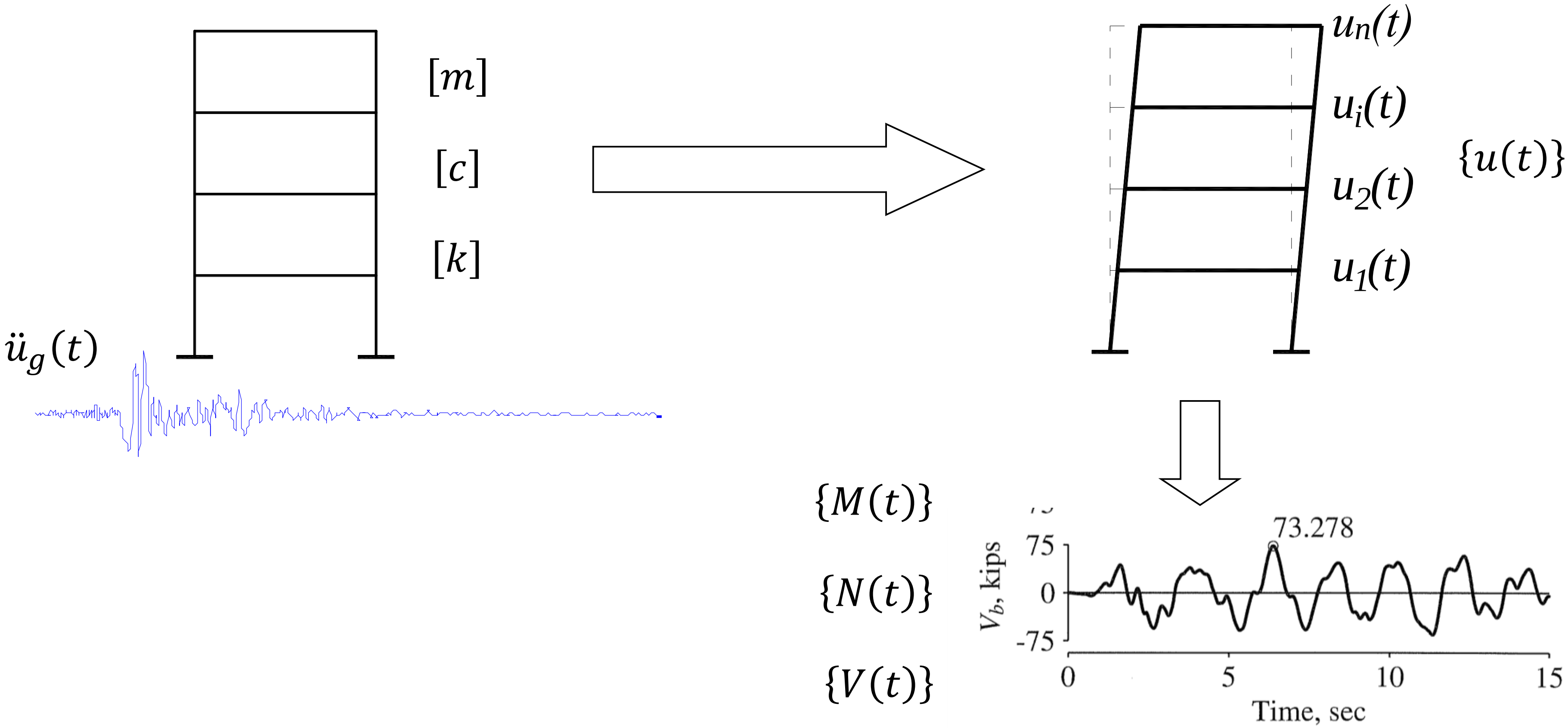


- **Time history response is obtained through direct integration (numerical methods) of the equation of motion**

Nonlinear time-history (NLTH) analysis

- **For a system with *N* degrees of freedom, there are *N* coupled differential equations to be solved numerically**

$$[\boldsymbol{m}]\{\ddot{\boldsymbol{u}}\} + [\boldsymbol{c}]\{\dot{\boldsymbol{u}}\} + \{\boldsymbol{f}_s(\mathbf{u})\} = -[\boldsymbol{m}]\{\mathbf{1}\}\ddot{\boldsymbol{u}}_g(\boldsymbol{t})$$



Nonlinear time-history (NLTH) analysis

- NLTH analysis is not used as part of the normal design process for typical structures. In some cases, however, it is recommended, and in certain cases required, to obtain a more realistic assessment of structural response and verify the results of simpler methods of analysis. Such is the case for systems with highly irregular force-deformation relationships.
- The principal aim of NLTH analysis is to determine if the computed deformations of the structure are within appropriate limits. Strength requirements for the dissipative components do not apply because element strengths are established prior to the analysis. These initial strengths typically are determined from a preliminary design using linear analysis.
- Design (performance assessment) criteria:
  - Plastic deformation demands in dissipative components
  - Strength demands in non-dissipative components

Nonlinear time-history (NLTH) analysis

- **Advantages:**
  - The most "realistic" modelling of a structure under seismic action
  - Direct assessment of seismic performance at member and structure levels
  - Ability to model a wide variety of nonlinear material behaviors, geometric nonlinearities (including large displacement effects), gap opening and contact behavior, and nonclassical damping, and to identify the likely spatial and temporal distributions of inelasticity

- **Disadvantages:**
  - Increased effort to develop the analytical model
  - Modelling of the structure requires specialised knowledge
  - Analysis is resource-intensives
  - Large amount of results $\Rightarrow$ a time-consuming post-processing of results
  - Sensitivity of computed response to system parameters

Combination with other actions and mass modelling

$$\boxed{[\boldsymbol{m}]}\{\ddot{\boldsymbol{u}}(\boldsymbol{t})\} + [\boldsymbol{c}]\{\dot{\boldsymbol{u}}(\boldsymbol{t})\} + [\boldsymbol{k}]\{\boldsymbol{u}(\boldsymbol{t})\} = -[\boldsymbol{m}]\{\boldsymbol{1}\}\boldsymbol{a}_{\boldsymbol{g}}(\boldsymbol{t})$$

Combination with other actions

- **Seismic load combination according to EN 1990**

$$\sum_{j\geq 1} G_{k,j} + \sum_{i\geq 1} \psi_{2,i} Q_{k,i} + A_{Ed}$$

  - $G_{k,j}$ **- characteristic permanent action** j
  - $Q_{k,i}$ **- characteristic variable action** i
  - A_{ed} **- design seismic action** $A_{ed} = \gamma_I A_{ek}$
  - $\psi_{2,i}$ **- coefficient for determination of quasipermanent value of the variable action** Q_i
  - γ_I **- importance coefficient**

Type of variable action	$\psi_{2,i}$
Imposed loads: residential and offices	0,3
Imposed loads: commercial facilities	0,6
Imposed loads: storage facilities	0,8
Snow loads	0,2
Wind and variation of temperature	0

Seismic mass

- **The inertial effects of the design seismic action shall be evaluated by considering the presence of the masses associated with all gravity loads in the seismic design situation**

$$\sum_{j\geq 1} G_{k,j} + \sum_{i\geq 1} \psi_{\mathrm{E},i} Q_{k,i} \qquad \psi_{E,i} = \varphi \cdot \psi_{2,i}$$

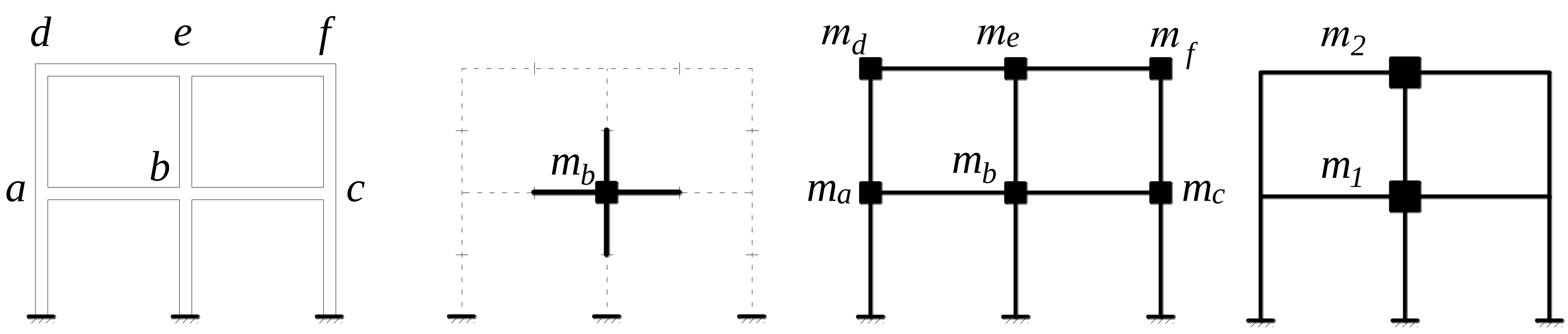
ψ_{Ei} - combination coefficient for variable action *i*

- **take into account the likelihood of the loads $Q_{k,i}$ not being present over the entire structure during the earthquake**
- **may also account for a reduced participation of masses in the motion of the structure due to the non-rigid connection between them**

Type of variable Action	Storey	φ
Categories A-C	Roof	1,0
	Storeys with correlated occupancies	0,8
	Independently occupied storeys	0,5
Categories D-F and Archives		1,0

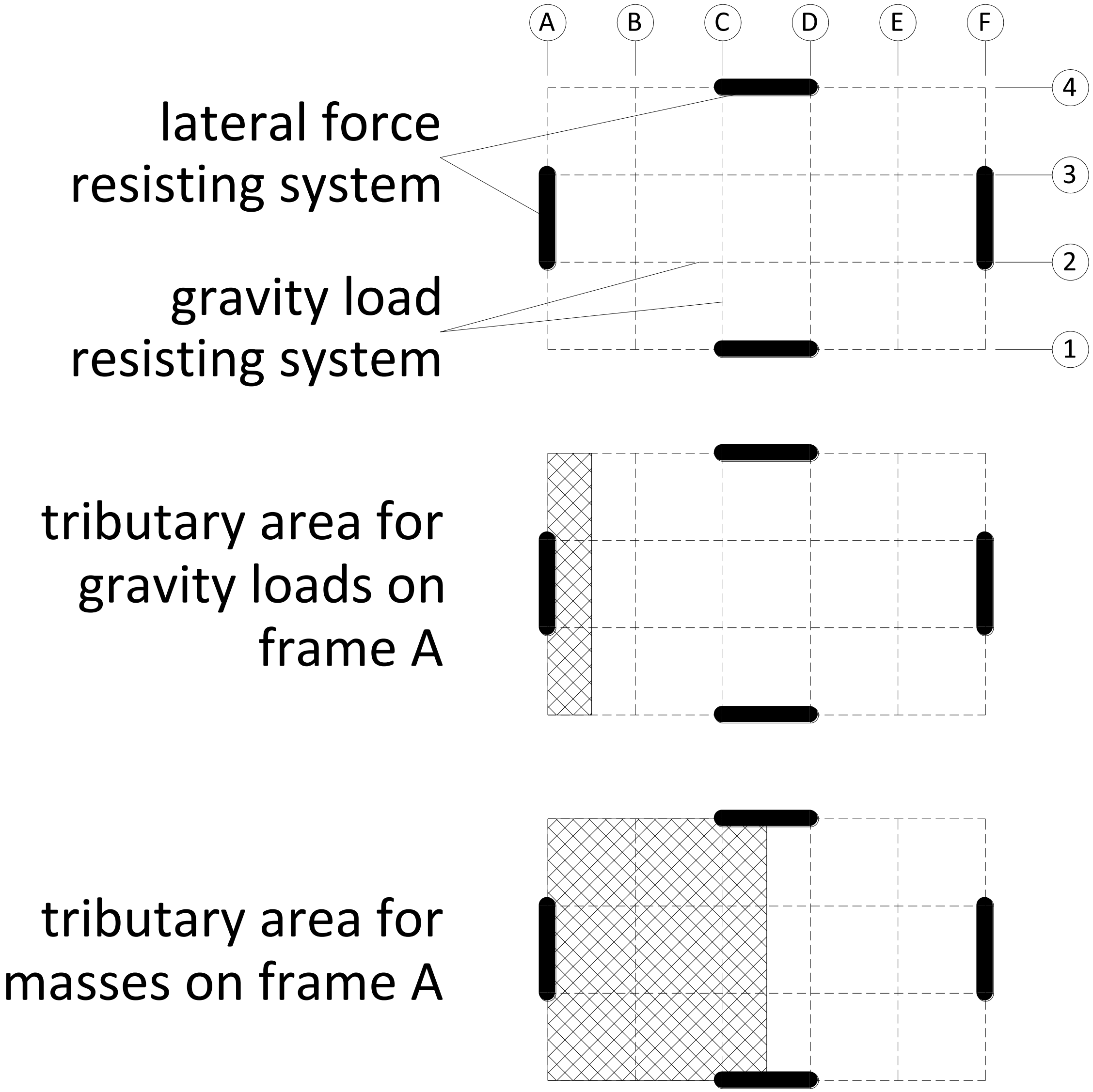
Modelling of mass: 2D

- Generally mass is distributed through the structure
- Mass lumped in nodes to reduce the number of dynamic degrees of freedom
  - 2 translational DOFs
  - 1 rotational DOFs (generally neglected)
- For rigid floors and if vertical component of the seismic action neglected, masses can be lumped at the floor levels (1 DOF per storey)



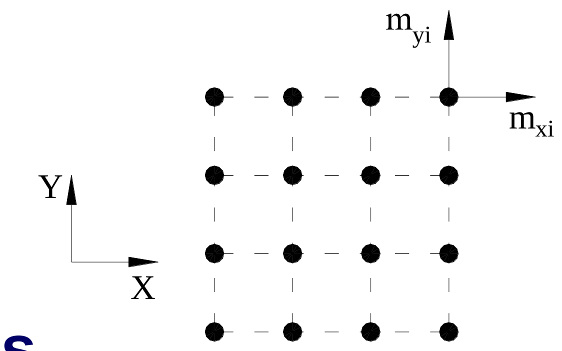
Modelling of mass: 2D

Care should be taken in modelling masses for 2D analysis, as tributary area for gravity loads will generally be different from the tributary area for masses

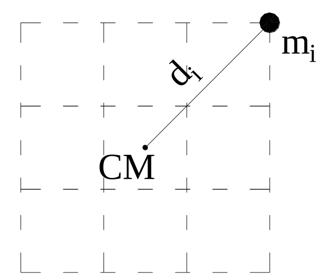


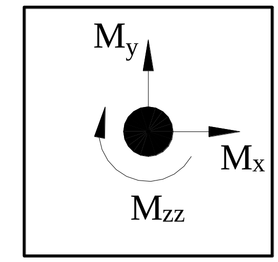
Modelling of mass: 3D

- **Mass lumped in nodes to reduce the number of dynamic degrees of freedom**
  - 3 translational DOFs
  - 3 rotational DOFs (generally neglected)
- **For structures with flexible diaphragms, lumped masses in nodes should approximate the real distribution of mass**



- **For structures with rigid diaphragms, the mass may be lumped in the centre of mass (CM) of the storey**
  - Two translational components $$M_x = M_y = \sum m_i$$
  - Mass moment of inertia (rotational component) $$M_{zz} = \sum m_i d_i^2$$





*** Note: mass is automatically lumped in the CM when rigid diaphragm constraints are enforced in structural analysis software**

Modelling of $\boxed{\text{damping}}$

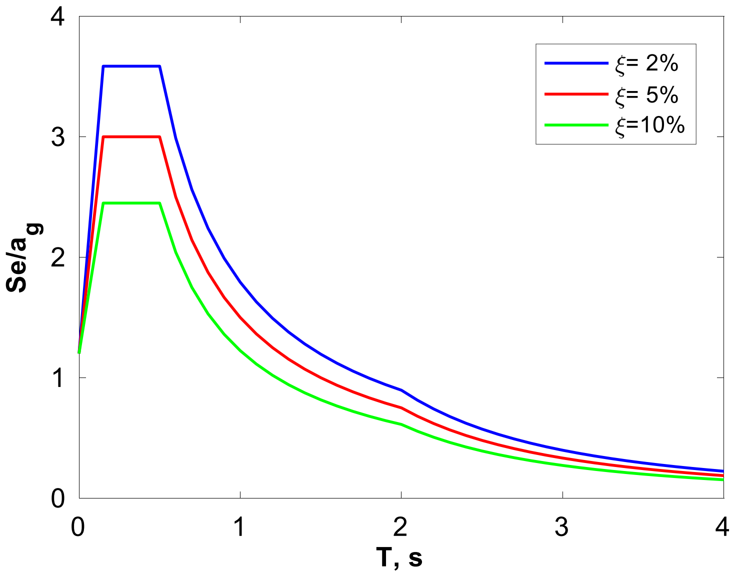
$$[\boldsymbol{m}]\{\ddot{\boldsymbol{u}}(\boldsymbol{t})\} + \boxed{[\boldsymbol{c}]}\{\dot{\boldsymbol{u}}(\boldsymbol{t})\} + [\boldsymbol{k}]\{\boldsymbol{u}(\boldsymbol{t})\} = -[\boldsymbol{m}]\{\mathbf{1}\}\boldsymbol{a}_{\boldsymbol{g}}(\boldsymbol{t})$$

Modelling of damping for LFM, MRS and pushover analyses

- **Response spectrum used for characterisation of seismic action in**
  - **Lateral force method (LFM)**
  - **Modal response spectrum (MRS) analysis**
  - **Nonlinear static analysis (pushover) + N2**
- **In these cases structural damping may be simply modelled by adjusting the elastic response spectrum through the damping correction factor**

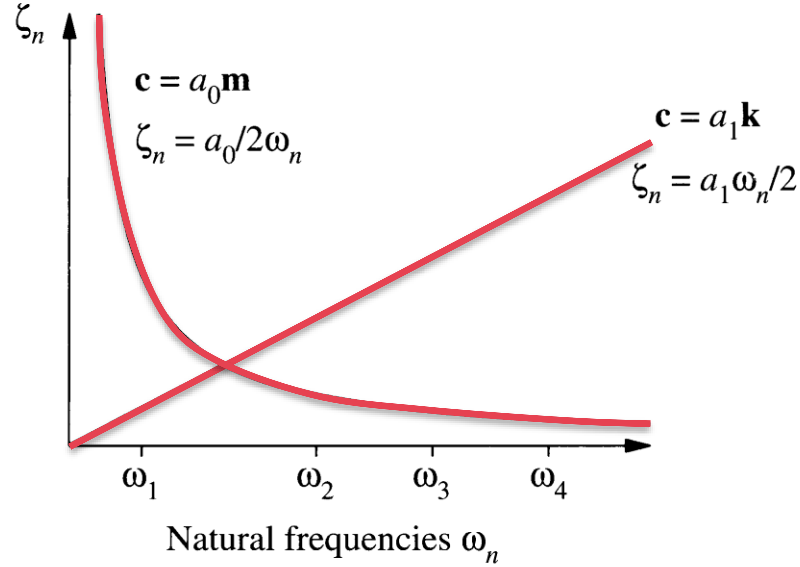
$$\eta = \sqrt{10/(5+\xi)} \geq 0,55$$

ξ is the damping ratio of the structure, expressed as percentage



Modelling of damping for time-history analysis

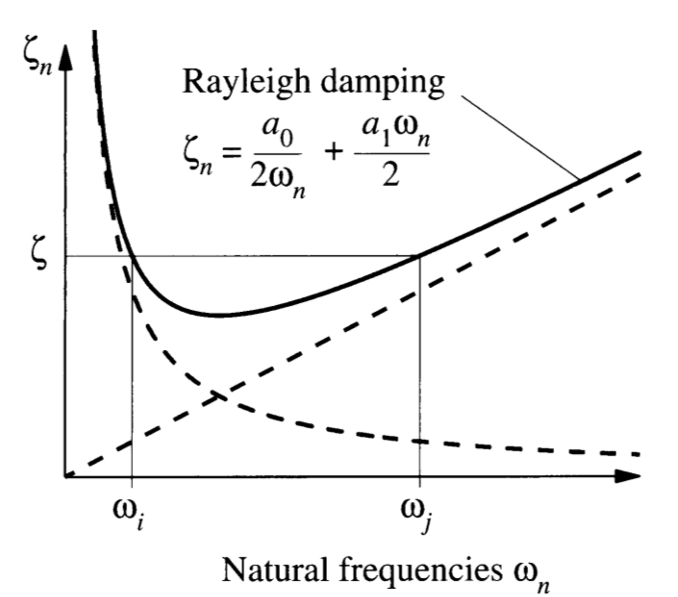
- **Rayleigh model:** $[c] = a_0[m] + a_1[k]$
  - mass proportional $[c] = a_0[m]$
  - stiffness proportional $[c] = a_1[k]$



- **Coefficients a_0 and a_1 may be obtained as:**

$$a_0 = \xi \frac{2\omega_i\omega_j}{\omega_i + \omega_j} \qquad a_1 = \xi \frac{2}{\omega_i + \omega_j}$$

- **The specified damping (ξ) enforced only for modes *i* and *j*. Other modes will be**
  - overdamped ($\omega<\omega_i$ or $\omega>\omega_j$) or
  - underdamped ($\omega_i<\omega<\omega_j$)



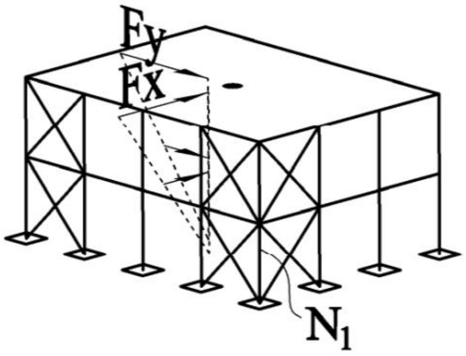
Modelling of damping for time-history analysis

- It is suggested to specify equivalent viscous damping in the range of 1% to 5% of critical damping over the range of elastic periods from 0.2T to 1.5T (where T is the fundamental period of vibration).

- If the damping matrix is based on the initial stiffness of the system, artificial damping may be generated by system yielding. In some cases, the artificial damping can completely skew the computed response.

- One method to counter this occurrence is to base the damping matrix on the mass and the instantaneous tangent stiffness.

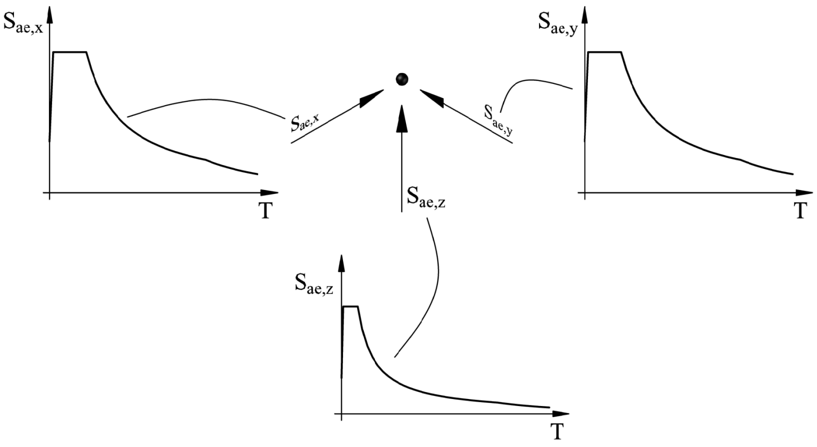
Combination of the effects of the components of seismic action



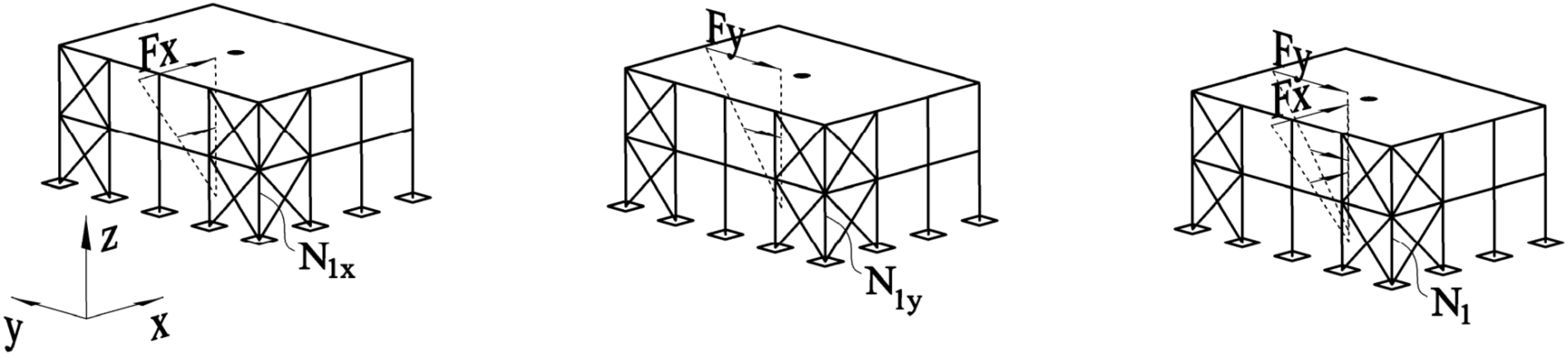
Combination of effects of the components of seismic action

- **Seismic action has components along three orthogonal axes:**
  - **2 horizontal components**
  - **1 vertical components**

  acting simultaneously.

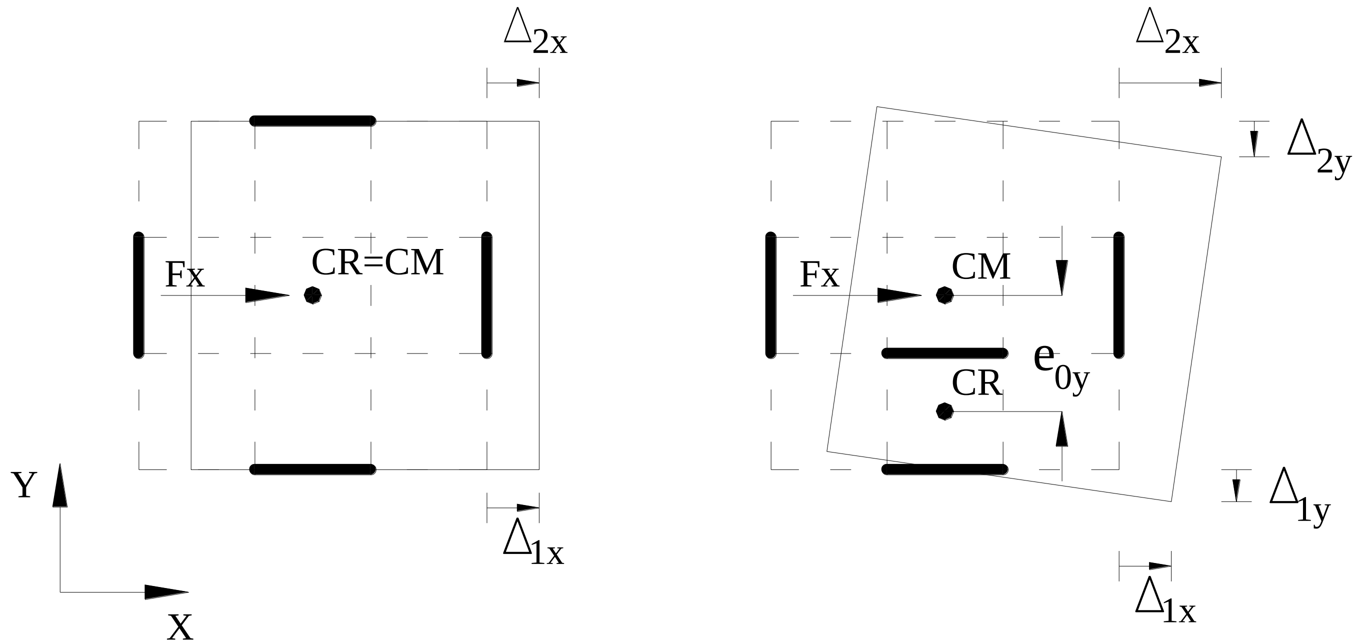


- **Some response quantities are influenced by two or three components of the seismic action**



Combination of effects of components of the seismic action

- **Some response quantities are influenced by two or three components of the seismic action, especially when the structure is irregular in plan**



Combination of effects of components of the seismic action: LTH or NLTH analyses

3D model of the structure

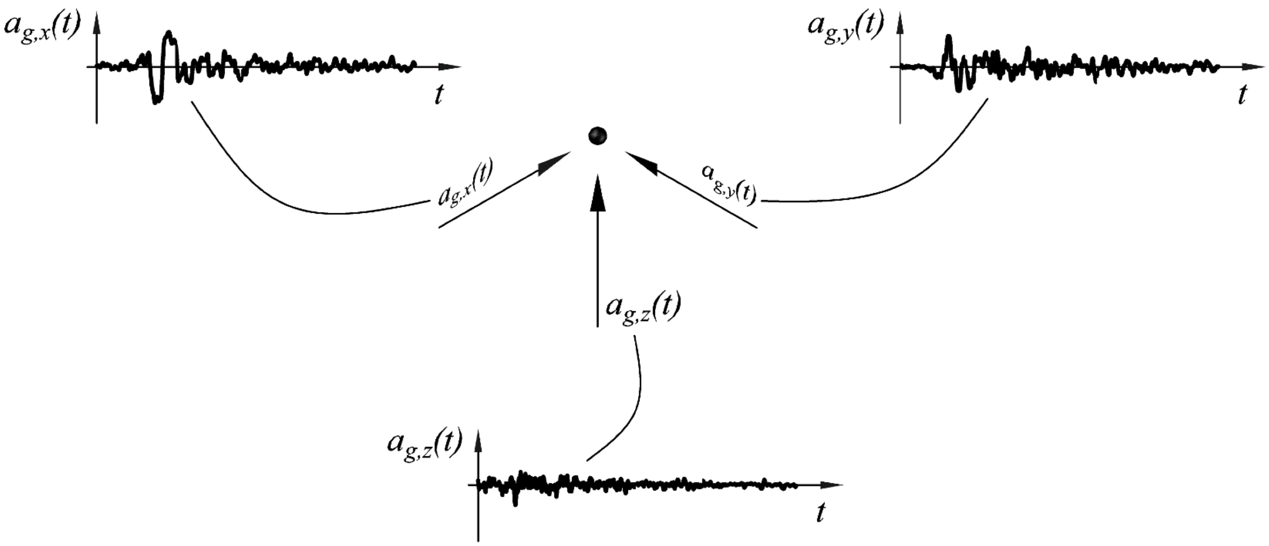
+

Nonlinear time-history analysis

+

Simultaneous application of *non-identical* accelerograms along the main directions of the structure

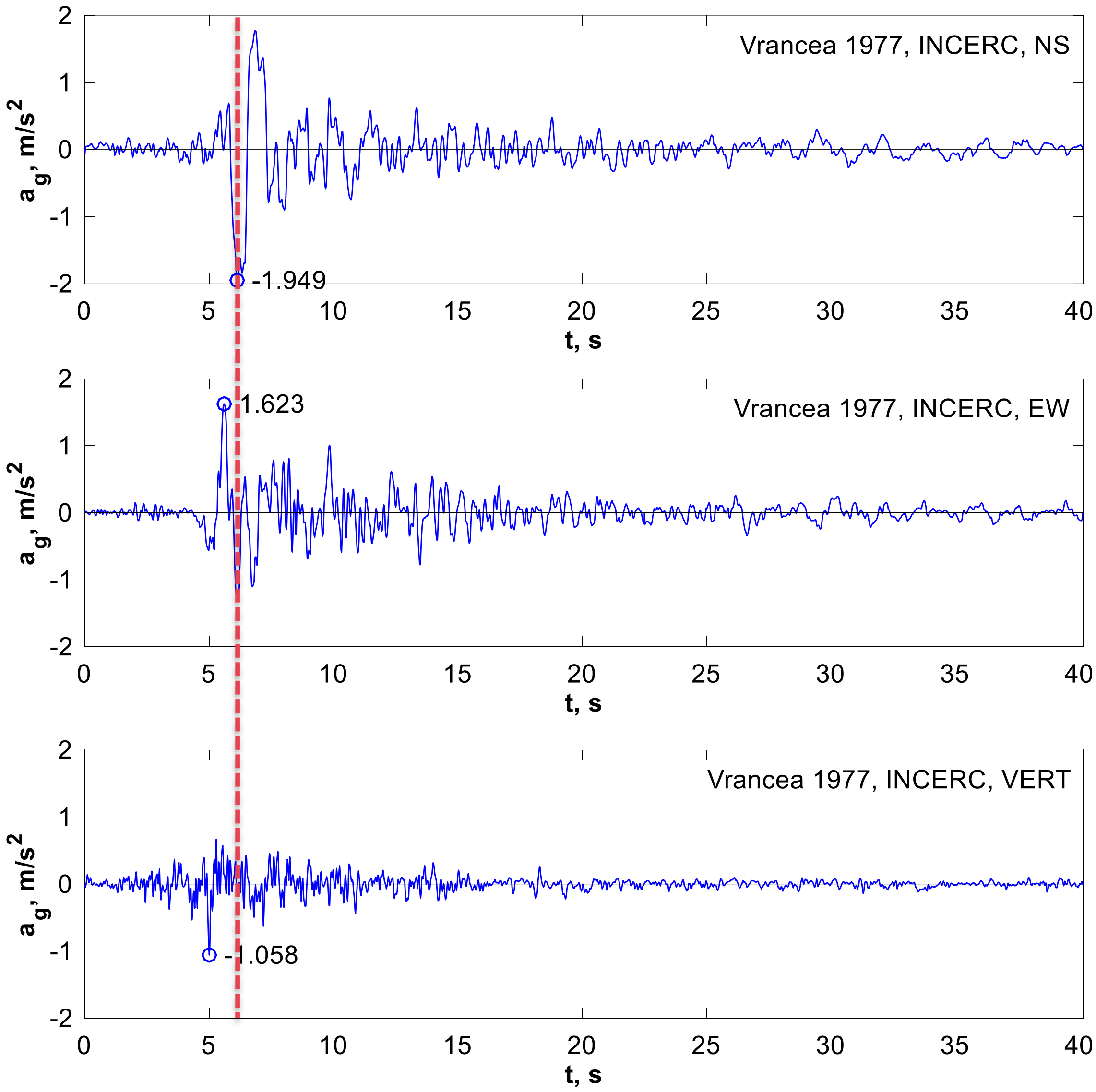
Spatial character of the seismic action accounted for directly



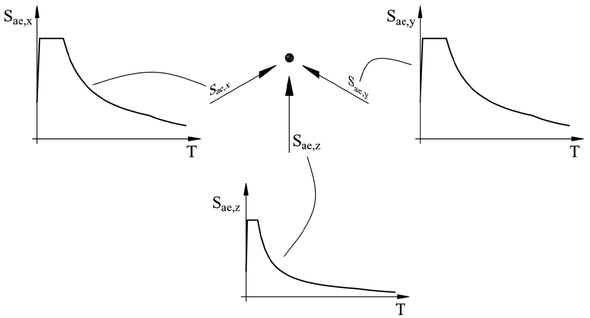
Combination of effects of components of the seismic action

- **Peak values of a_g for horizontal motion do NOT occur at the same time instant**

- **Peak values of response do NOT occur at the same time instant**



Combination of effects of components of the seismic action: LFM and MRS analyses



- **The combination of the horizontal components of the seismic action may be accounted using the SRSS rule:**
  - **Seismic response is evaluated separately for each direction of seismic action**
  - **Peak value of response from the simultaneous action of two horizontal components is obtained by the SRSS combination of directional response**
  - **Provides safe estimates, independent of the coordinate system,**
  - **Has the drawback of lost correlation of different response quantities**

$$E_{Ed} = \sqrt{E_{Edx}^2 + E_{Edy}^2 + E_{Edz}^2}$$

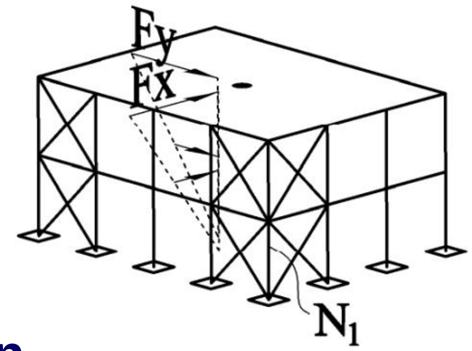
- **Alternative method for combination of components of seismic actions (100+30% rule)**

$$0.3E_{Edx}"+"0.3E_{Edy}"+"E_{Edz}$$

$$E_{Edx}"+"0.3E_{Edy}"+"0.3E_{Edz}$$

$$0.3E_{Edx}"+"E_{Edy}"+"0.3E_{Edz}$$

Combination of effects of components of the seismic action: pushover analysis



- When using non-linear static (pushover) analysis and applying a spatial model, the SRSS or 100+30% combination rules should be applied, considering
  - the forces and deformations due to the application of the target displacement in the x direction as E_{Edx} and
  - the forces and deformations due to the application of the target displacement in the y direction as E_{Edy}.
- The internal forces resulting from the combination should not exceed the corresponding capacities.

$$E_{Ed} = \sqrt{E_{Edx}^2 + E_{Edy}^2}$$

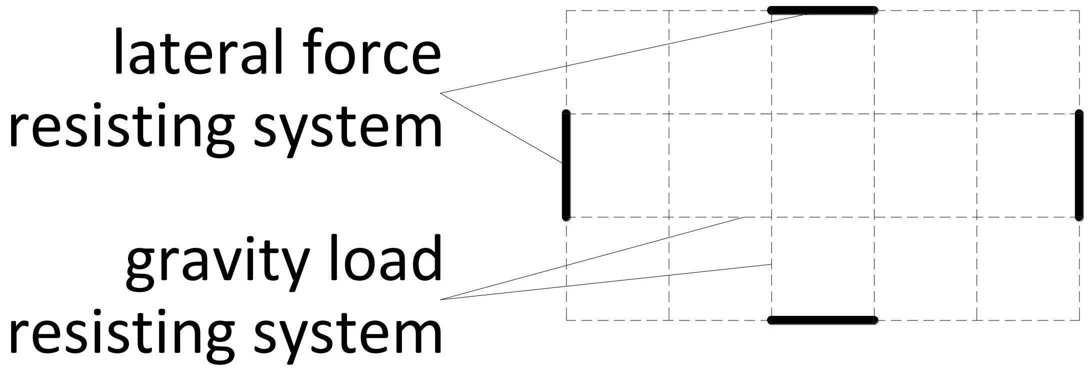
$$E_{Edx}\text{"} + \text{"}0.3E_{Edy}\text{"}$$

$$0.3E_{Edx}\text{"} + \text{"}E_{Edy}\text{"}$$

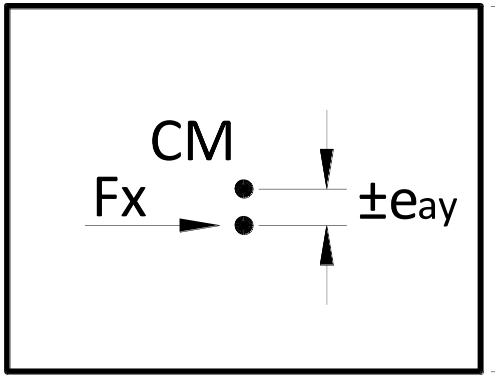
* The rule is not physically sound for pushover analysis, as it uses superposition of effects, which is not correct for plastic analysis

Combination of effects of components of the seismic action

- **For buildings satisfying the regularity criteria in plan and in which walls or *independent bracing systems* in the two main horizontal directions are the only primary seismic elements, the seismic action may be assumed to *act separately* along the two main orthogonal horizontal axes of the structure.**



Accidental torsion

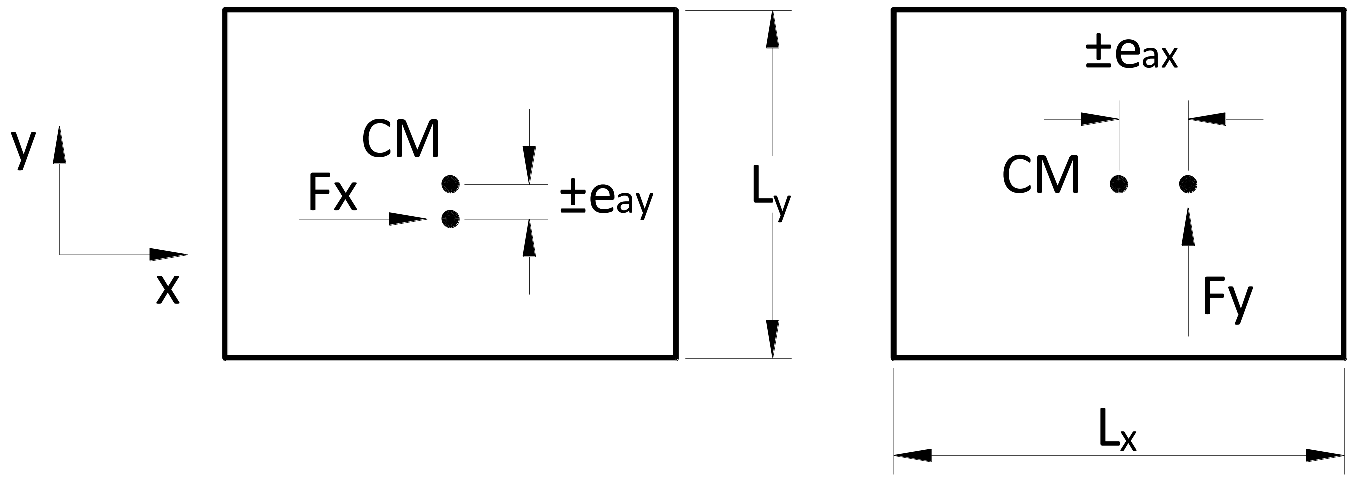


Accidental torsion: general

- **In order to account for uncertainties in the location of masses and in the spatial variation of the seismic motion, the calculated centre of mass (CM) at each floor *i* shall be considered as being displaced from its nominal location in each direction by an accidental eccentricity:**

$$e_{ai} = \pm 0.05\ L_i$$

  - **e_{ai} is the accidental eccentricity of storey mass *i* from its nominal location, applied in the same direction at all floors;**
  - **L_i is the floor-dimension perpendicular to the direction of the seismic action.**



Accidental torsion: 2D models

- **For LFM, MRS, LTH analyses on 2D structural models the accidental torsional effects may be accounted for by multiplying the action effects in the individual load resisting elements resulting from the application of lateral forces by a factor δ**

$$\delta = 1 + 1.2 \frac{x}{L_e}$$

  - x **is the distance of the element under consideration from the centre of mass of the building in plan, measured perpendicularly to the direction of the seismic action considered;**
  - L_e **is the distance between the two outermost lateral load resisting elements, measured perpendicularly to the direction of the seismic action considered.**
- **For pushover analysis the accidental torsional effects may be accounted for by amplifying the target displacement resulting from analysis by a factor δ.**
- **EC8-1 is silent about the NLTH analysis using 2D models. It may be assumed that scaling of accelerograms by δ could be appropriate.**

Accidental torsion: 3D models / LFM

- **If the lateral stiffness and mass are symmetrically distributed in plan and unless the accidental eccentricity is taken into account by a more exact method, the accidental torsional effects may be accounted for by multiplying the action effects in the individual load resisting elements resulting from the application of lateral forces by a factor δ**

$$\delta = 1 + 0.6\frac{x}{L_e}$$

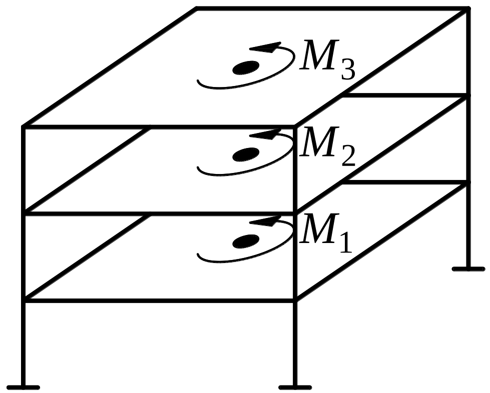
  – x **is the distance of the element under consideration from the centre of mass of the building in plan, measured perpendicularly to the direction of the seismic action considered;**

  – L_e **is the distance between the two outermost lateral load resisting elements, measured perpendicularly to the direction of the seismic action considered.**

Accidental torsion: 3D models / MRS analysis

- **For MRS analysis of 3D models the accidental torsional effects may be determined as the envelope of the effects resulting from the application of static loadings, consisting of sets of torsional moments M_{ai} about the vertical axis of each storey i:**

$$M_{ai} = e_{ai} F_i$$



  – M_{ai} **is the torsional moment applied at storey i about its vertical axis;**

  – e_{ai} **is the accidental eccentricity of storey mass i for all relevant directions;**

  – F_i **is the horizontal force acting on storey i, as derived using the lateral force method.**

Accidental torsion:
3D model / pushover analysis

- **Pushover analysis may significantly underestimate deformations at the stiff/strong side of a torsionally flexible structure, i.e. a structure with a predominantly torsional first mode of vibration. The same applies for the stiff/strong side deformations in one direction of a structure with a predominately torsional second mode of vibration.**
- **For such structures, displacements at the stiff/strong side shall be increased, compared to those in the corresponding torsionally balanced structure.**
- **Buildings <u>irregular in plan</u> shall be analysed using a <u>3D model</u>. Two independent analyses with lateral loads applied in one direction only may be performed.**
- **Torsional effects may be accounted for by amplifying the displacements of the stiff/strong side based on the results of an elastic modal analysis of the spatial model.**

Accidental torsion:
3D models / LTH and NLTH analyses

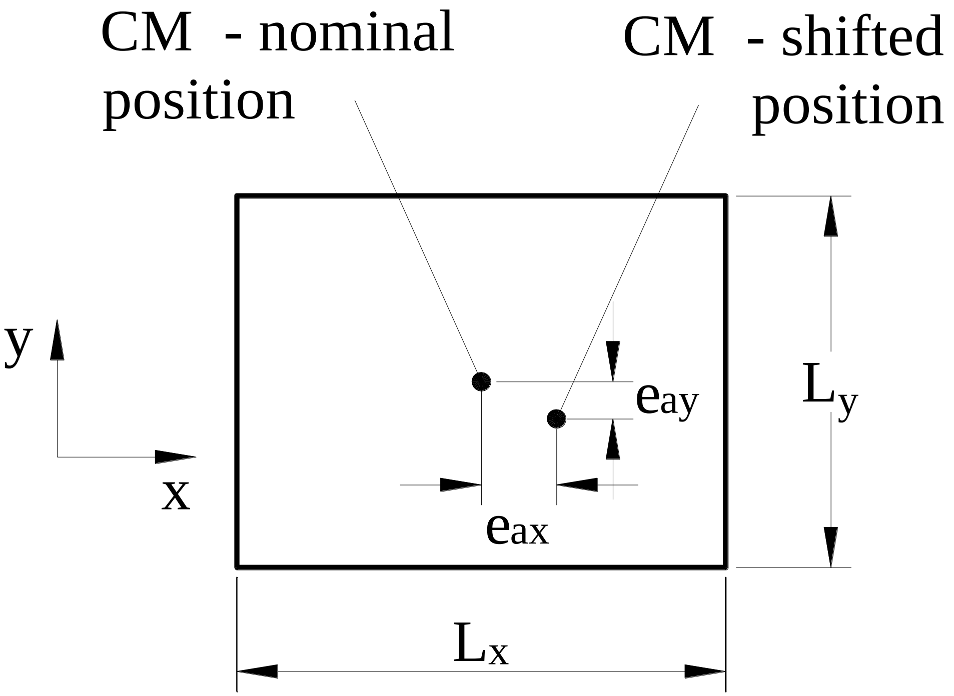
- **For spatial models (3D): the accidental torsional effects accounted for by shifting the centre of mass from its nominal location with the value of the eccentricity in each of the two horizontal directions**
- **Accidental eccentricity $e_{ai} = \pm 0.05\, L_i$ (EN 1998-1)**
- **This requires four cases to be analysed:**

  $e_{ax}+e_{ay}$

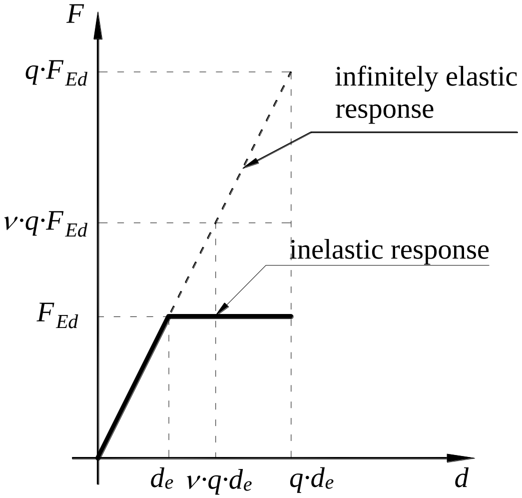
  $e_{ax}-e_{ay}$

  $-e_{ax}+e_{ay}$

  $-e_{ax}-e_{ay}$



Displacement analysis

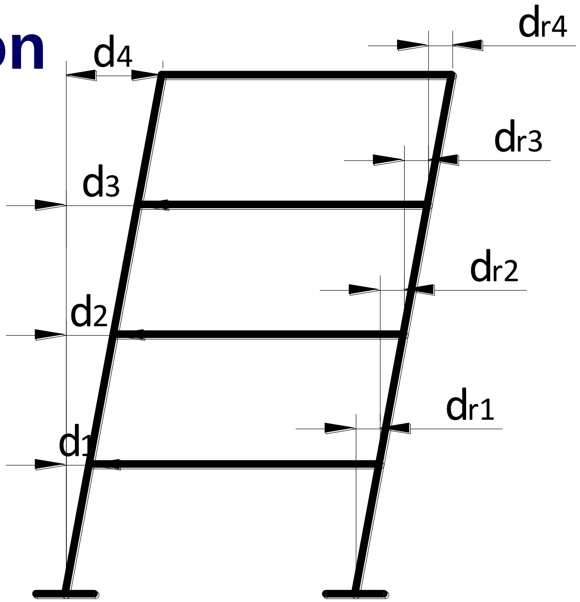


Displacements in seismic design

- **Information about structural displacements are necessary primarily for:**
  - **checking the requirements of the damage limitation state (DLS)**
  - **checking second-order effects (at the ultimate limit state - ULS)**
  - **seismic joint condition (at ULS)**
- **Damage limitation state is checked by limiting the interstorey drifts, evaluated as the difference of the average lateral displacements at the top and bottom of the storey under consideration**

$$d_s \leq d_{r,a}^{DL}$$

  - **brittle non-structural elements** $d_{r,a}^{DL} = 0.005h$
  - **ductile non-structural elements** $d_{r,a}^{DL} = 0.0075h$
  - **without non-structural elements** $d_{r,a}^{DL} = 0.01h$

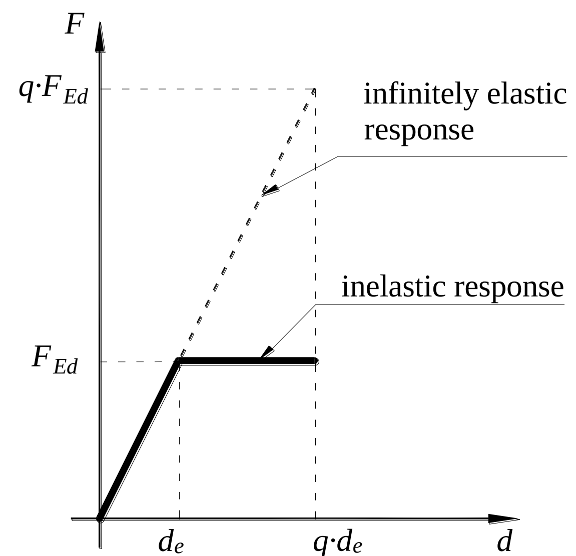


Displacements at ULS

- **For plastic analysis methods (pushover and NLTH analysis), displacements are obtained directly from the output of the analysis**
- **For elastic analysis methods (LFM, MRS and LTH analysis) displacements obtained from structural analysis are unrealistic, due to the fact that the design seismic action is reduced by the behaviour factor q. Displacements at the ULS are estimated based on the "equal-displacement rule":**

$$d = q d_e$$

  - ***d* lateral displacement at ULS**
  - **d_e lateral displacements determined from design earthquake action**

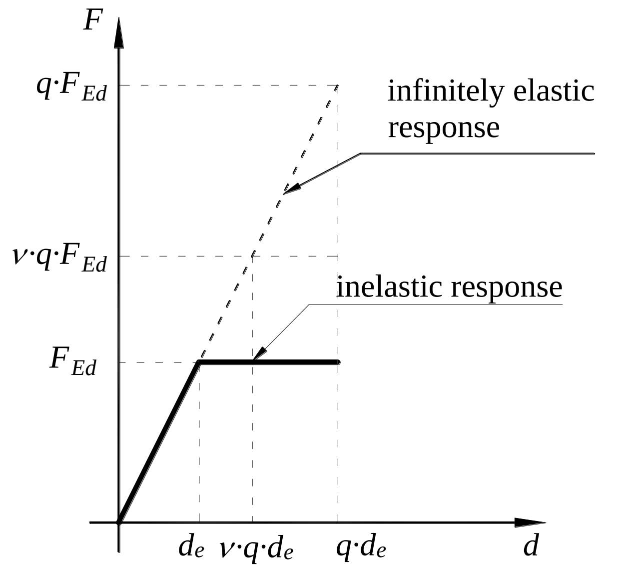


Displacements at DLS

- **For plastic analysis methods (pushover and NLTH analysis), displacements are obtained directly from the output of the analysis**
- **For elastic analysis methods (LFM, MRS and LTH analysis) displacements obtained from structural analysis are unrealistic, due to the fact that the design seismic action is reduced by the behaviour factor q. Displacements at the DLS are estimated based on the "equal-displacement rule":**

$$d_s = \nu q d_e$$

  - **d_s lateral displacement at DLS**
  - **d_e lateral displacements determined from design earthquake action**
  - **ν reduction factor to account for a lower mean return period of DLS earthquake (ν=0.4-0.5)**



Second-order effects

$$\theta = \frac{P_{tot} d_r}{V_{tot} h}$$

Second-order effects: plastic analysis

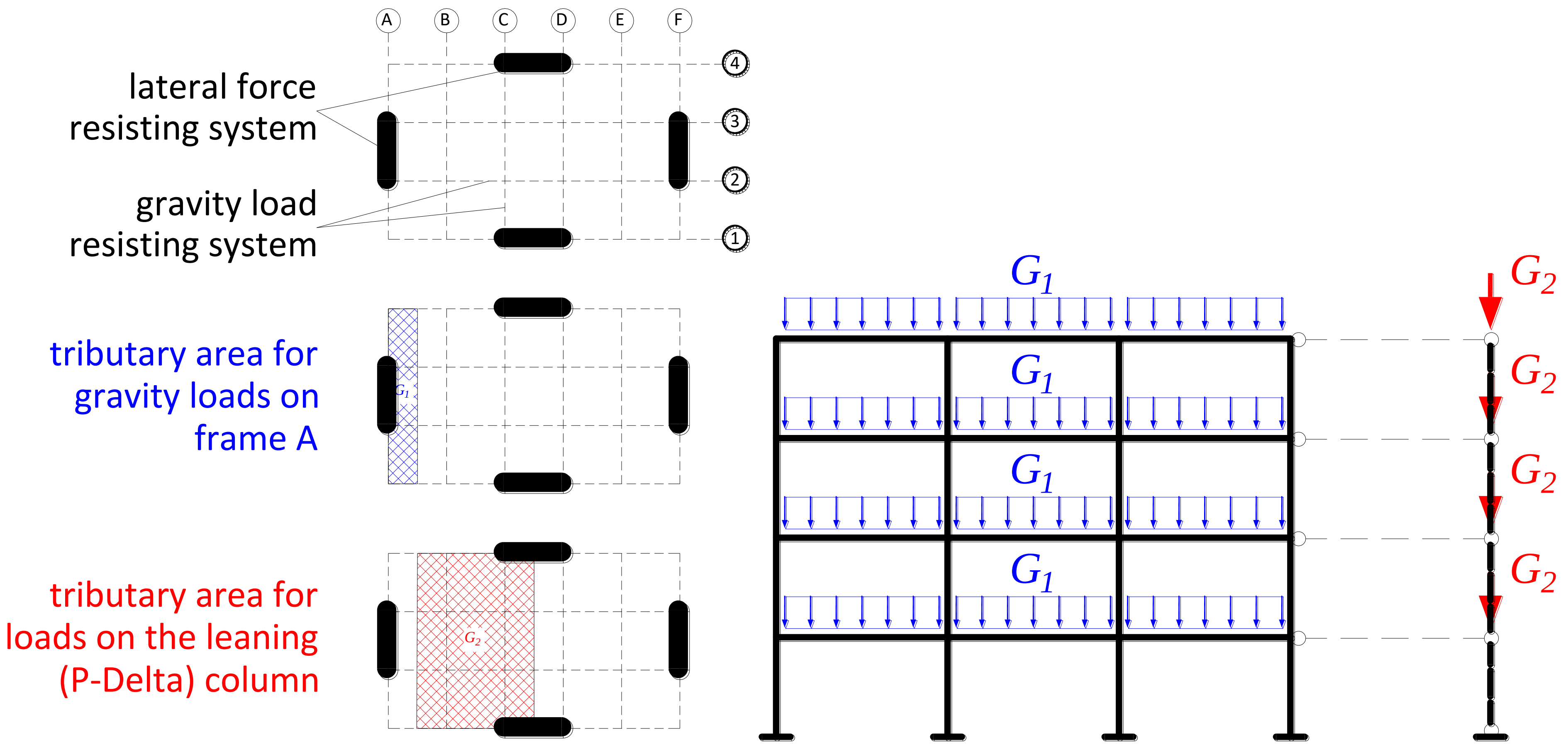
- In case of pushover and nonlinear time-history analysis, second order (P-delta) effects can be taken into account directly in the structural analysis software

- The procedure is straightforward for 3D models

- For 2D models, second order effects produced by gravity frames should be applied to the lateral force resisting ones. Gravity actions applied on gravity frames produce second order effects which are resisted by lateral force resisting frames.

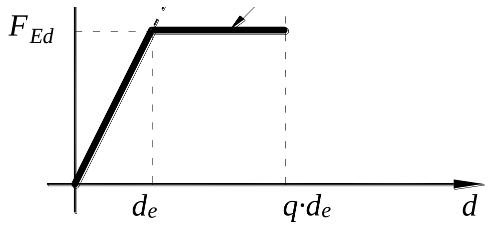
Second-order effects: plastic analysis (2D models)

- **Example of considering gravity forces on gravity frames ("leaning column – *P-Delta*") in second-order analysis of lateral force resisting frames**



Second-order effects: elastic analysis

- In the case of elastic analysis using design response spectrum (LFM, MRS or LTH), second-order effects are assessed using the θ coefficient:



$$\theta = \frac{P_{tot} d_r}{V_{tot} h} = \frac{P_{tot} q\, d_{re}}{V_{tot} h}$$

  - θ is the interstorey drift sensitivity coefficient;
  - P_{tot} is the total gravity load at and above the storey considered in the seismic design situation;
  - d_r is the design interstorey drift at the ULS;
  - d_{re} interstorey drift determined from design earthquake action
  - V_{tot} is the total seismic storey shear; and
  - h is the interstorey height.
- If $0{,}1 < \theta \leq 0{,}2$, the second-order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1 - \theta)$. The value of the coefficient θ shall not exceed 0,3

Choice of analysis method and structural model

Choice of analysis method and structural model

- **MRS analysis on 3D model may be used always**
- **Allowed simplifications:**

Regularity		Allowed simplification		Behaviour factor (q)
Plan	Elevation	Model	Linear-elastic analysis	
YES	YES	2D	* Lateral force	Reference value
YES	NO	2D	MRS	Reduced value (by 20%)
NO	YES	3D	* Lateral force	Reference value
NO	NO	3D	MRS	Reduced value (by 20%)

*Only if $T_1 < \min(4T_C; 2.0\ \text{s})$

- **Plan irregularity: large torsional effects ⇒ 3D models**
- **Vertical irregularities: significant contribution of higher modes of vibration ⇒**
  - modal response spectrum analysis
  - reduced values of behaviour factor

Choice of analysis method and structural model

- **Lateral force method:**
  - Simple to apply to 2D models
  - Similar results with modal response spectrum analysis for regular low-rise buildings
  - Conservative results for taller structures
  - Conservative results for 3D models (due to simplified method of accounting for accidental torsion)

- **Modal response spectrum analysis:**
  - Convenient, especially for 3D models
  - Major draw-back – uncorrelated response quantities

- **Linear time-history analysis (not mentioned explicitly in EC8)**
  - Alternative to modal response spectrum analysis when correlation of response quantities is important

Choice of analysis method and structural model

- **Pushover analysis:**
  - Relatively simple to apply for 2D models
  - Limited guidance in EC8 on modelling of structural components
  - Not suited for direct design, but very useful for assessment of structural performance (plastic mechanism, redundancy, etc.)

- **Non-linear time-history analysis:**
  - Most "accurate", but requires specialised knowledge
  - Limited guidance in EC8 on modelling of structural components
  - Time-consuming and resource-intensive
  - Not suited for direct design, but very useful for assessment of structural performance, especially for unconventional systems

Děkuji vám!