Introduction to Earthquake Engineering

Seismic Design of Conventional Structures

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Basic Design Requirements

No Collapse Requirement:

EC 8 - 2

The structure shall be designed and constructed to withstand the seismic action defined in Section 3 without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events[5]. The reference seismic action is associated with a reference probability of exceedance in 50 years and a reference return period.

ultimate limit state

⇩

10 % in 50 years ⇒ return Period: 475 years

Damage Limitation Requirement:

The structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence than the seismic action used for the verification of the "no collapse requirement", without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself. The reference seismic action to be taken into account for the "damage limitation requirement" has a low probability of exceedance in 10 years. In the absence of more precise information, the reduction factor presented in 4.4.3.2 may be used to obtain the seismic action for the verification of the "damage limitation requirement".

serviceability limit state

⇩

10 % in 10 years ⇒ return Period: 95 years

Basic Design Requirements

EC 8 - 2

Importance Categories:

(3)P Reliability differentiation is implemented by classifying structures into different importance categories. To each importance category an importance factor γ_I is assigned. Wherever feasible this factor should be derived so as to correspond to a higher or lower value of the return period of the seismic event (with regard to the reference return period), as appropriate for the design of the specific category of structures (see 3.2.1(3)).

Importance category	Buildings	g_1
IV	Buildings whose integrity during earthquake is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.	1,4
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions, etc.	1,2
II	Ordinary buildings, not belonging to the other categories	1,0
I	Buildings of minor importance for public safety, e.g. agricultural building, etc.	0,8

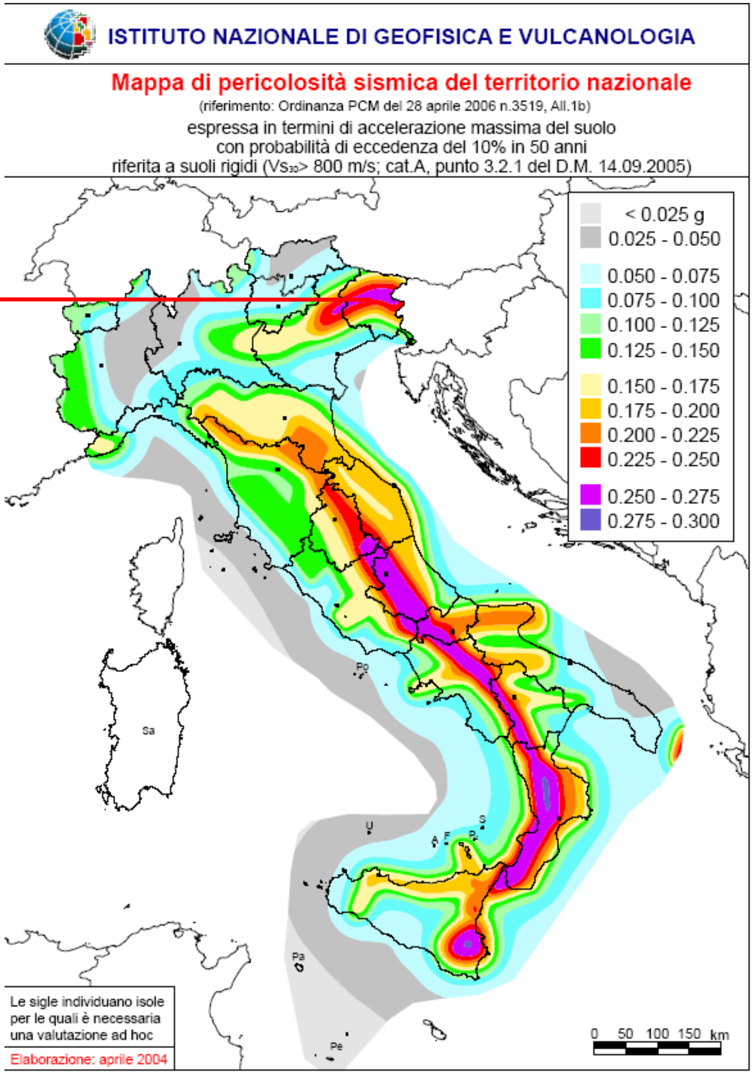
Recommended importance factors

Representation of Seismic Action

Seismic Zones

Reference peak-ground acceleration:

Udine



$$a_{gR} = 0.275 \text{ g}$$

$$a_g = 0.275 \times 1.2 = 0.33 \text{ g}$$

Importance factor building category III

Representation of Seismic Action

Classification of Subsoil Classes

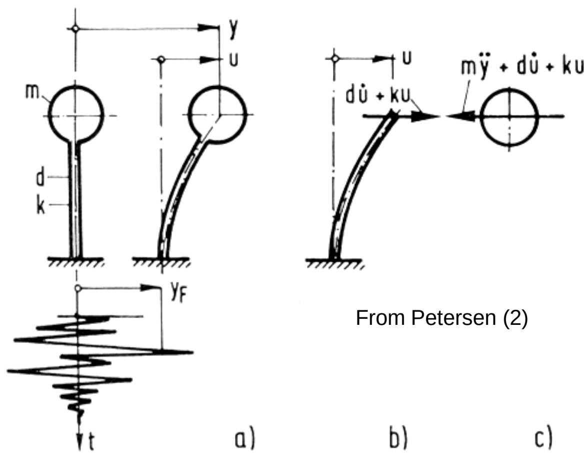
EC 8 - 3

Subsoil class	Description of stratigraphic profile	Parameters		
		$V_{s,30}$ (m/s)	N_{SPT} (bl/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface	> 800	–	–
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of m in thickness, characterised by a gradual increase of mechanical properties with depth	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of m	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with $V_{s,30}$ values of class C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with $V_{s,30}$ > 800 m/s			
S_1	Deposits consisting – or containing a layer at least 10 m thick – of soft clays/silts with high plasticity index (PI > 40) and high water content	< 100 (indicative)	–	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in classes A –E or S_1			

Representation of Seismic Action in Codes

Elastic Response Spectra

viscously damped SDOF oscillator



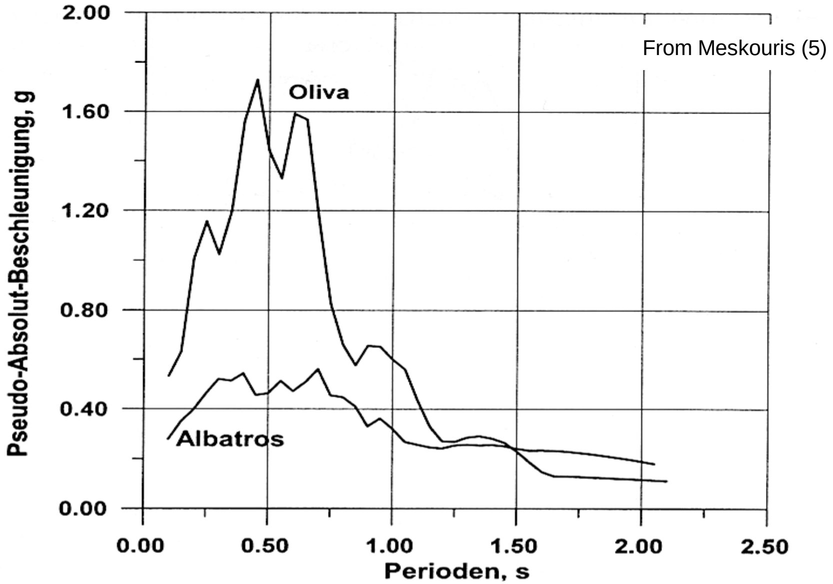
From Petersen (2)

$$m \cdot \ddot{u} + d \cdot \dot{u} + k \cdot u = -m \cdot \ddot{y}_F(t) \qquad \Big| \cdot \frac{1}{m}$$

$$\ddot{u} + 2 \cdot \omega \cdot \varsigma \cdot \dot{u} + \omega^2 \cdot u = -\ddot{y}_F(t)$$

where: Eigenfrequency: $\omega = \sqrt{\frac{k}{m}}$

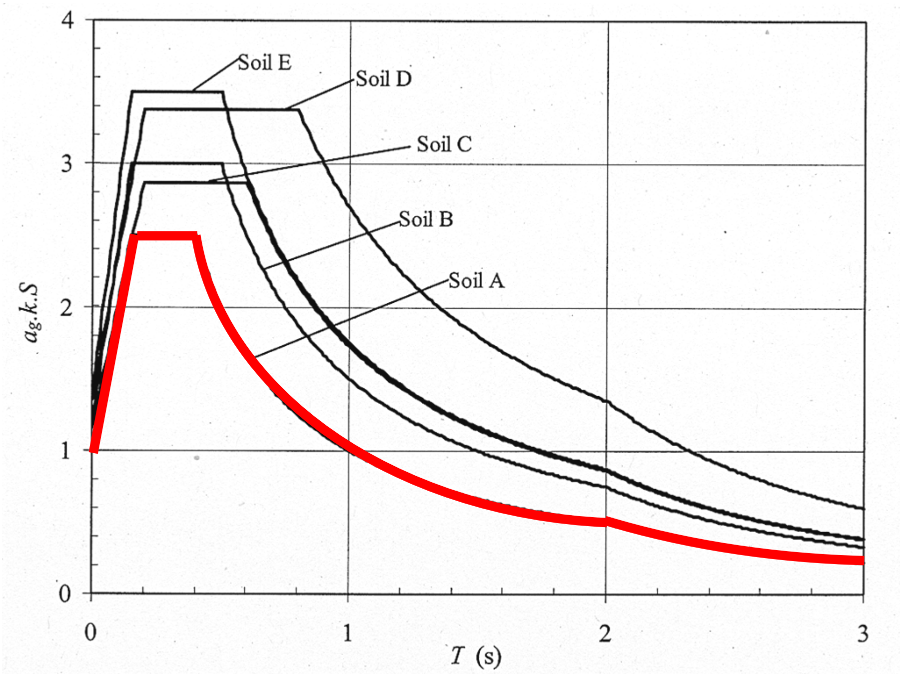
Damping ratio: $\varsigma = \frac{d}{2 \cdot m \cdot \omega}$



- solving this equation for various ω and ζ, but only for one specific accelerogram
- the maximum absolute acceleration of this solution gives us the abscissa for the following diagram

Representation of Seismic Action in Codes

Horizontal Elastic Design Spectra



Subsoil Class	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

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

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

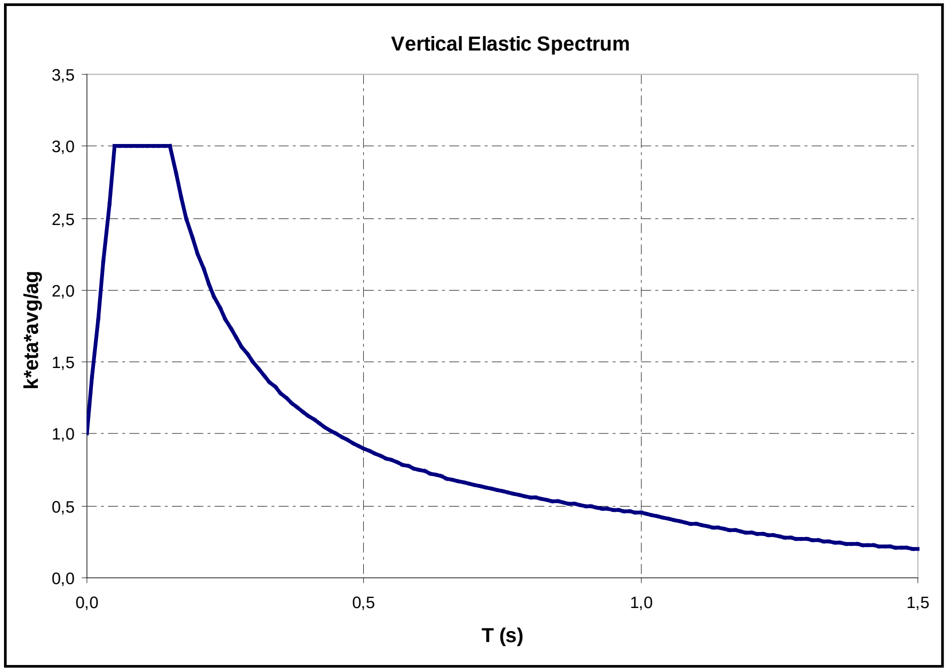
$$T_c \le T \le T_D: \quad S_e(T) = a_g \cdot k \cdot S \cdot \eta \cdot 2,5 \left[\frac{T_C}{T}\right]$$

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

EC 8 - 3

Representation of Seismic Action in Codes

Vertical Elastic Design Spectra



EC 8 - 3

Spectrum	a_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Type 1	0,90	0,05	0,15	1,0

$$0 \le T \le T_B: \quad S_{ve}(T) = a_{v,g} \cdot k\left[1 + \frac{T}{T_B} \cdot (\eta \cdot 3{,}0 - 1)\right]$$

$$T_B \le T \le T_C: \quad S_{ve}(T) = a_{v,g} \cdot k \cdot \eta \cdot 3{,}0$$

$$T_c \le T \le T_D: \quad S_{ve}(T) = a_{v,g} \cdot k \cdot \eta \cdot 3{,}0 \cdot \left[\frac{T_C}{T}\right]$$

$$T_D \le T \le 4\,\mathrm{sec}: S_{ve}(T) = a_{v,g} \cdot k \cdot \eta \cdot 3{,}0 \cdot \left[\frac{T_C \cdot T_D}{T^2}\right]$$

Representation of Seismic Action in Codes

Time-History Representation

EC 8 - 3

General:

- representation of seismic motion in terms of ground acceleration time-history is allowed
- for spatial models the same accelerogram should not be used simultaneously along both horizontal directions
- the description of seismic action may be made by using artificial accelerograms

Artificial accelerograms:

- Artfificial accelerograms shall be generated to match the elastic response spectra given by EC 8
- the duration of the generated accelerogram shall be consistent with the relevant features of the seismic event underlying the establishment of a_g
- the number of accelerograms to be used shall be such as to give a stable statistic measure of the response quantities of interest

Recorded or simulated accelerograms:

- The use of recorded or simulated accelerograms is allowed if the used samples are adequately qualified with regard to the seismogenic features of the sources and to the soil conditions for the site of question

Representation of Seismic Action in Codes

Inertia Efects

EC 8 - 3

(1)P The design value E_d of the effects of actions in the seismic design situation shall be determined according to 6.4.3.4 of EN1990

(2)P The inertial effects of the seismic action shall be evaluated by taking into account the presence of the masses associated to all gravity loads appearing in the following combination of actions:

$$\Sigma G_{kj} \text{"+"} \Sigma \psi_{Ei} \cdot Q_{ki} \qquad (3.16)$$

where

ψ_{Ei} combination coefficient for variable action i.

(3) The combination coefficients ψ_{Ei} take into account the likelihood of the loads $\psi_{2i} \cdot Q_{ki}$ being not present over the entire structure during the occurrence of the earthquake. These coefficients may also account for a reduced participation of masses in the motion of the structure due to the non-rigid connection between them.

(4) Values of ψ_{2i} are given in EN 1990 and values of ψ_{Ei} for buildings or other types of structures are given in the relevant Parts of EN 1998.

Combination With Other Actions

Combination Coefficients for Variable Actions

EC 8 - 4

(1)P The combination coefficients ψ_{2i} for the design of buildings (see 3.2.4(1)P) are given in Annex A1 of EN 1990.

(2)P The combination coefficients ψ_{Ei} introduced in 3.2.4(2)P for the calculation of the effects of the seismic actions shall be computed from the following expression:

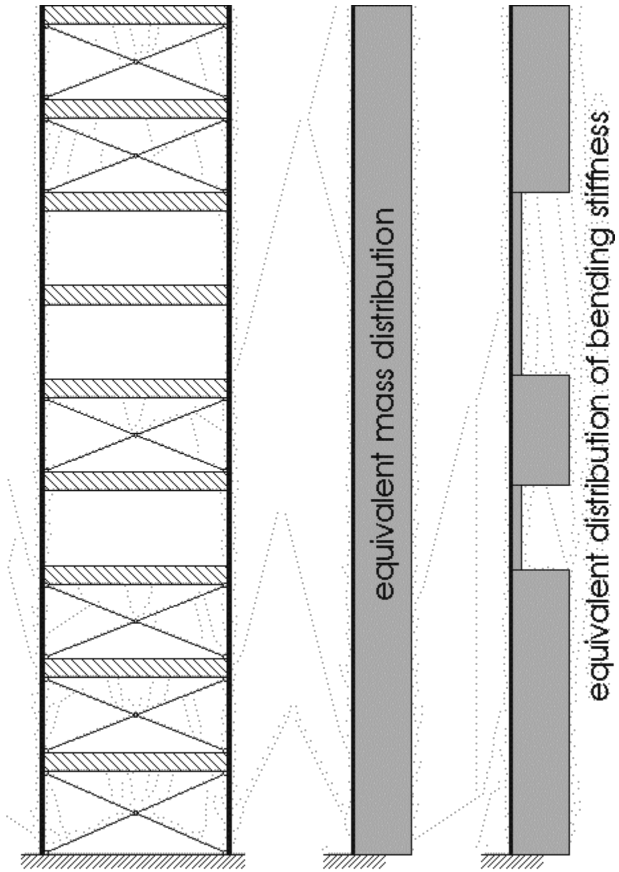
$$\psi_{Ei} = \varphi \cdot \psi_{2i} \tag{4.2}$$

where the values of φ shall be obtained from Table 4.2.

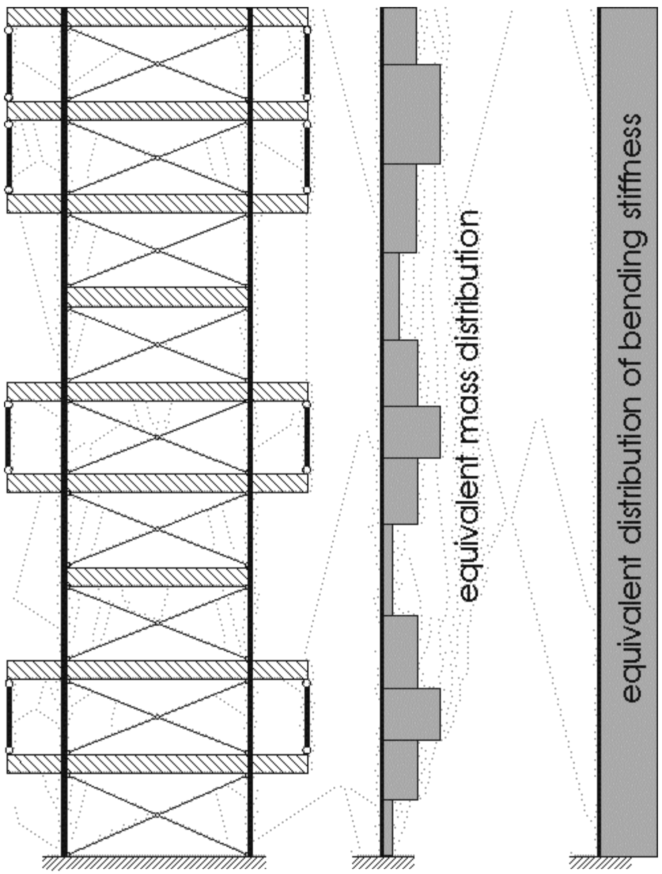
Type of variable Action	Occupation of storeys	Storey	φ
Categories A-C*	storeys independently occupied	roof other storeys	1,0 0,5
Categories A-C*	some storeys having correlated occupancies	roof storeys with correlated occupancies other storeys	1,0 0,8 0,5
Categories D-F* and Archives			1,0

Structural Requirements

Regularity



- masses regular
- stiffness irregular



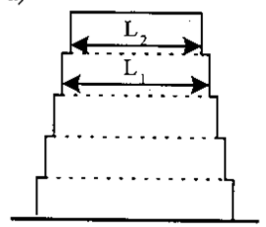
- stiffness regular
- masses irregular

Structural Requirements

Regularity

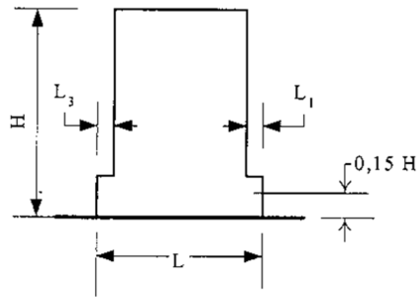
EC 8 - 4

a)



$$\frac{L_1 - L_2}{L_1} \leq 0{,}20$$

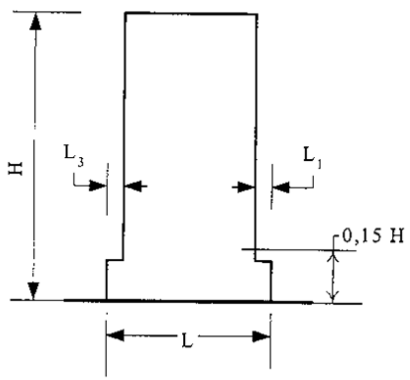
b)



$$\frac{L_3 + L_1}{L} \leq 0{,}20$$

(setback occurs above 0,15H)

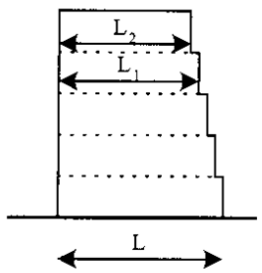
c)



$$\frac{L_3 + L_1}{L} \leq 0{,}50$$

(setback occurs below 0,15H)

d)

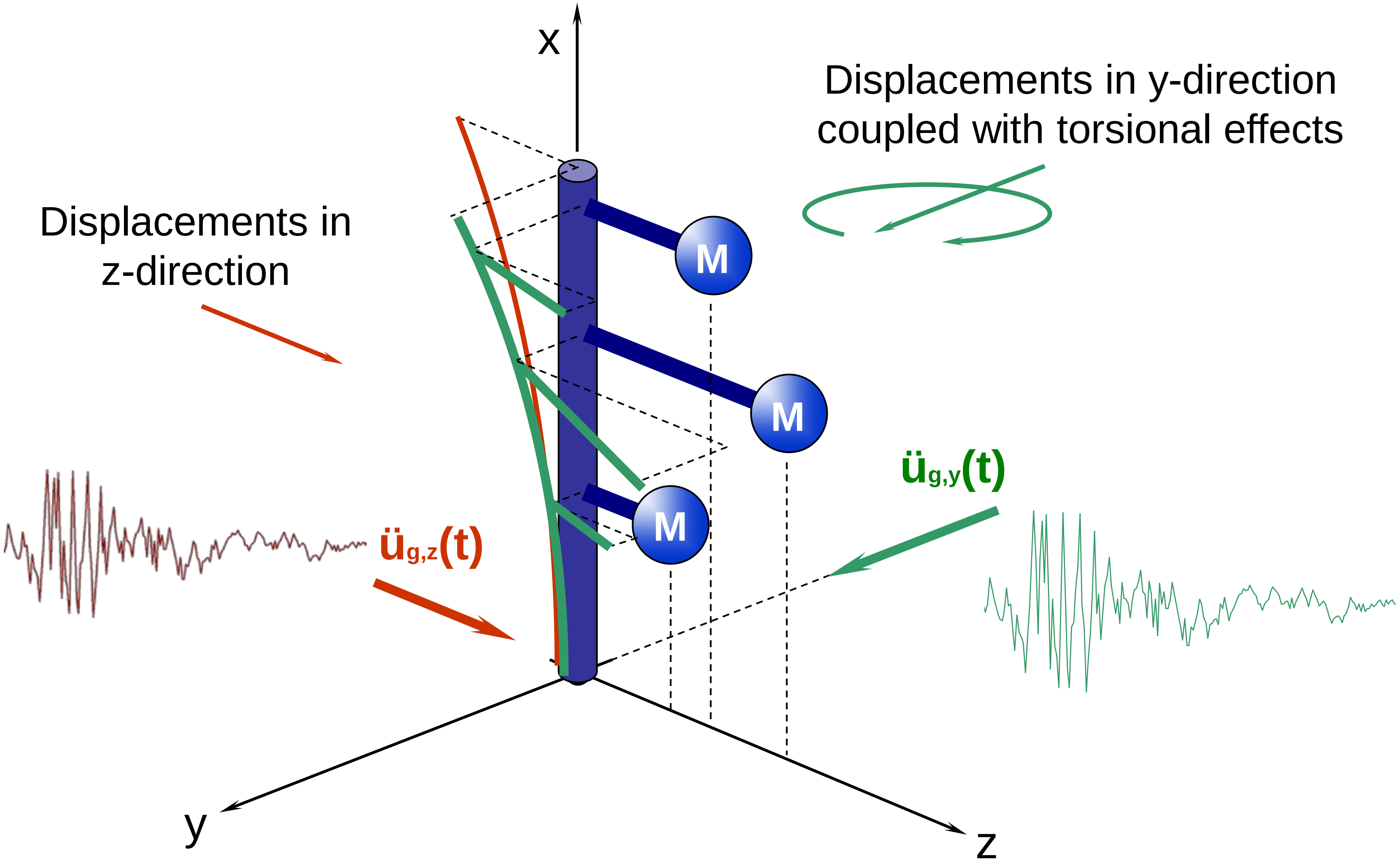


$$\frac{L - L_2}{L} \leq 0{,}30$$

$$\frac{L_1 - L_2}{L_1} \leq 0{,}10$$

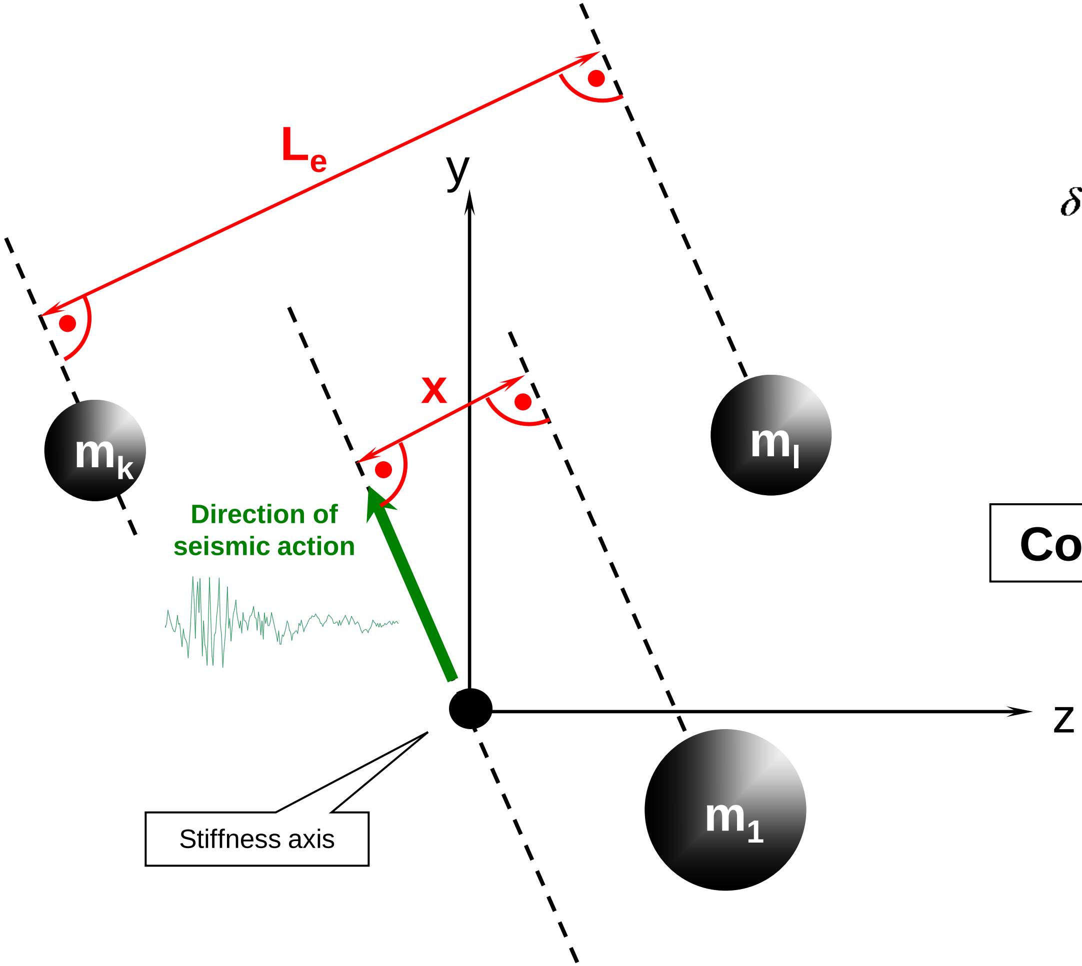
Structural Requirements

Torsional Effects



Structural Requirements

- The accidental torsional effects may be accounted for by multiplying the action effects resulting in the individual load resisting elements from above with the following factor:



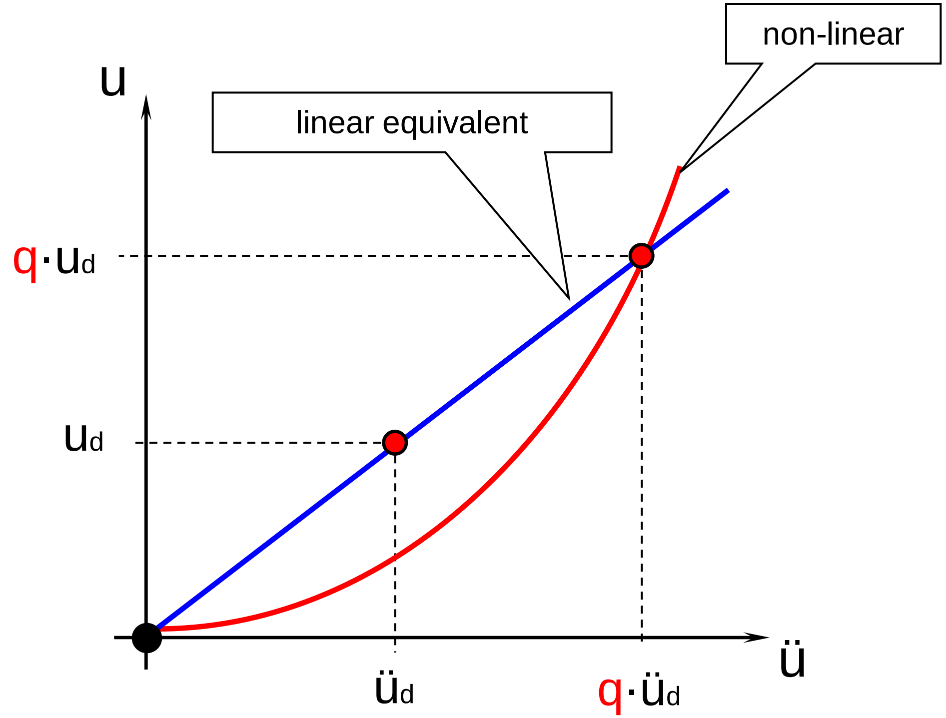
$$\delta = 1 + 0,6 \cdot \frac{x}{L_e}$$

Considering mass m_1

Linear Methods of Analysis

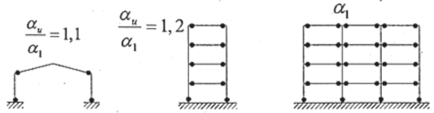
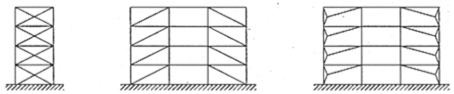
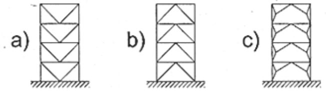
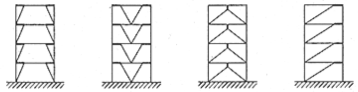
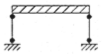
Definition of the q – Faktor

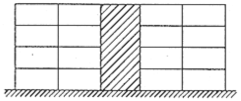
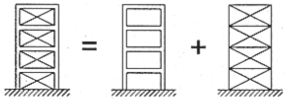
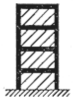
to account for non-linear behavior



Linear Methods of Analysis

q-Factors According to EC-8

	Ductility Class	
	H	M
a) Moment resisting frame. $\frac{\alpha_u}{\alpha_1}=1,1$ $\frac{\alpha_u}{\alpha_1}=1,2$ $\frac{\alpha_u}{\alpha_1}=1,3$  • Dissipative zones in the beams and bottom of columns	$5\frac{\alpha_u}{\alpha_1}$	4
b) Frame with concentric bracings.		
Diagonal bracings.  Dissipative zones -tension diagonals only-.	4	4
V - bracings.  Dissipative zones (tension & compression diagonals).	2,5	2
c) Frame with eccentric bracings. $\frac{\alpha_u}{\alpha_1}=1,2$  – Dissipative zones (bending or shear links).	$5\frac{\alpha_u}{\alpha_1}$	4
d) Inverted pendulum. $\frac{\alpha_u}{\alpha_1}=1$  - Dissipative zones at the column base.  $\frac{\alpha_u}{\alpha_1}=1,1$ - Dissipative zones in columns $N_{Sd}/N_{Pl.Rd}>0,3$	$2\frac{\alpha_u}{\alpha_1}$	2

	Ductility Class	
	H	M
e) Structures with concrete cores or concrete walls. 	See section 5.	
f) Moment resisting frame with concentric bracing.  $\frac{\alpha_u}{\alpha_1}=1,2$ Dissipative zones: in moment frame and in tension diagonals.	$4\frac{\alpha_u}{\alpha_1}$	4
g) Moment resisting frames with infills. 		
Unconnected concrete or masonry infills, in contact with the frame.	2	2
Connected reinforced concrete infills.	See section 7.	
Infills isolated from moment frame: see moment frames.	$5\frac{\alpha_u}{\alpha_1}$	4

Linear Methods of Analysis

Non-Linear Design Spectrum

EC 8 - 3

$$0 \le T \le T_B : S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2,5}{q} - \frac{2}{3} \right) \right]$$

$$T_B \le T \le T_C : S_d(T) = a_g \cdot S \cdot \frac{2,5}{q}$$

$$T_C \le T \le T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C}{T} \right] \\ \ge \beta \cdot a_g \end{cases}$$

$$T_D \le T : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C \cdot T_D}{T^2} \right] \\ \ge \beta \cdot a_g \end{cases}$$

$S_d(T)$:	*ordinate of the design spectrum*
q:	*behaviour factor*
β:	*lower bound factor for the spectrum*
	recommended value: $\beta = 0,2$

Linear Methods of Analysis

Lateral Force Method

General:
- structures can be analysed by a planar model for both horizontal directions
- higher modes do not have a significant influence on the structural response.

➢ These requirements deemed to be satisfied in buildings which fulfil **regularity requiremetns** and both of the following conditions

$$T_1 \le \begin{cases} 4 \cdot T_c & \textit{where } T_1 \textit{ is the highest natural Period} \\ 2{,}0 \text{ sec} & \textit{for one of the both main direction} \end{cases}$$

Subsoil Class	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Base shear force:

➢ The seismic base shear force F_b for each main direction is determined as follows

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

$S_d(T_1)$ — *ordinate of the chosen design spectrum at period* T_1

T_1 — *highest natural Period of the structure in the direction considered*

m — *total mass of the building in regard to other actions*

λ — *correction factor*

$\lambda = 0{,}85$ *if* $T_1 \le 2 \cdot T_C$, *or* $\lambda = 1{,}0$ *otherwise*

Linear Methods of Analysis

Lateral Force Method

Simplified analysis of 1st mode period T_1:

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

For structures lower than 40 m:

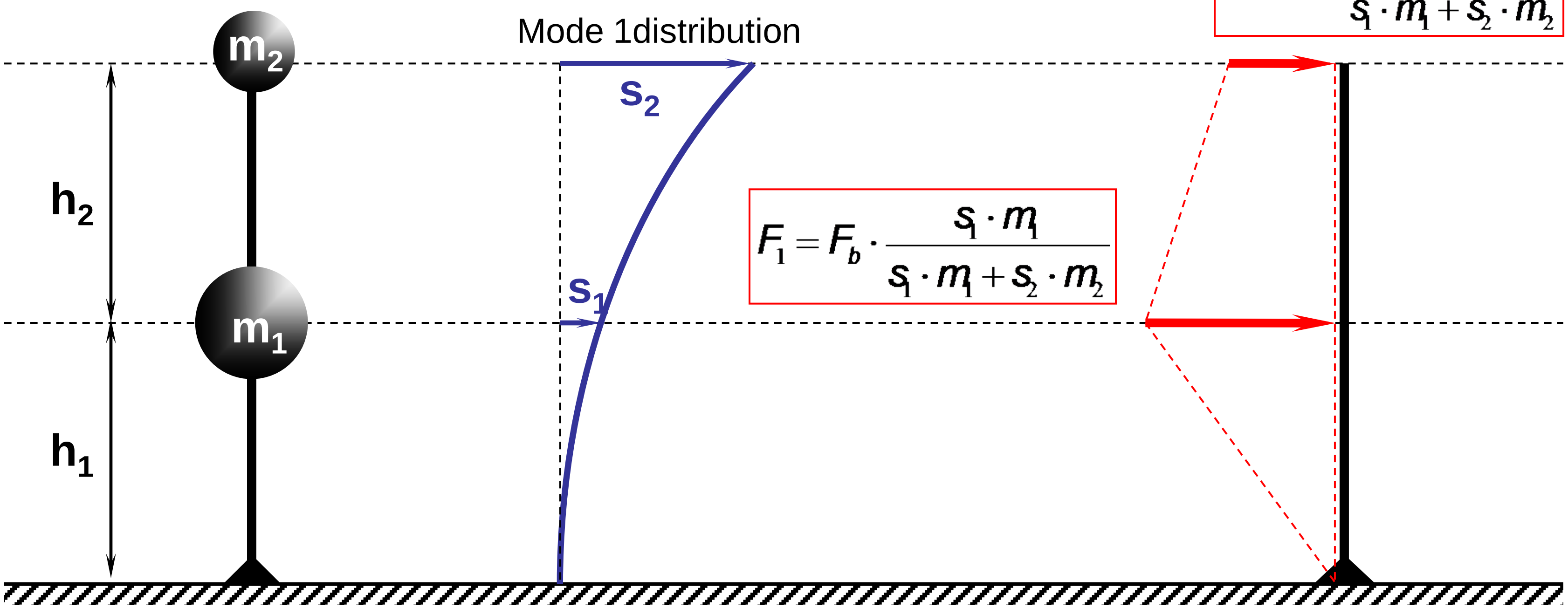
- C_t = 0,085 for steel frames
- C_t = 0,075 for rc- frames and excentricaly braced steel frames
- C_t = 0,05 for all other structures

Distribution of the horizontal seismic forces:

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_j \cdot m_j}$$

- The seismic action effects shall be determined by applying horizontal forces F_i to all storey masses m_i

$$F_2 = F_b \cdot \frac{s_2 \cdot m_2}{s_1 \cdot m_1 + s_2 \cdot m_2}$$



$$F_1 = F_b \cdot \frac{s_1 \cdot m_1}{s_1 \cdot m_1 + s_2 \cdot m_2}$$

Linear Methods of Analysis

Response Spectrum Analysis

General:

- the structures have to comply with the criteria for **regularity**
- in some cases the structure shall be analysed using a spatial model
- the response of all modes of vibration contributing significantly to the global response shall be taken into account
  - demonstrating that the sum of the effektive modal masses for the modes taken into account amounts to at **least 90% of total mass** of the structure
  - demonstrating that all modes with effective modal masses **greater than 5% of the total mass** are considered

Combination of modal responses:

- the response in two vibration modes i and j may be considered as independent of each other, if their Periods T_i and T_j satisfy the following condition: $T_i \leq 0,9 \cdot T_j$
- whenever all relevant modal responses may be regarded as independent of each other, the maximum value of the global response may be taken as:

$$S = \sqrt{\sum_{j=1}^{n} S_j^2}$$

Non-Linear Methods of Analysis

Static Push-Over Analysis

General:

- A nonlinear static analysis under constant gravity loads and monotonically increasing horizontal loads
- It is possible to analyse the structure with two perpendicular planar models

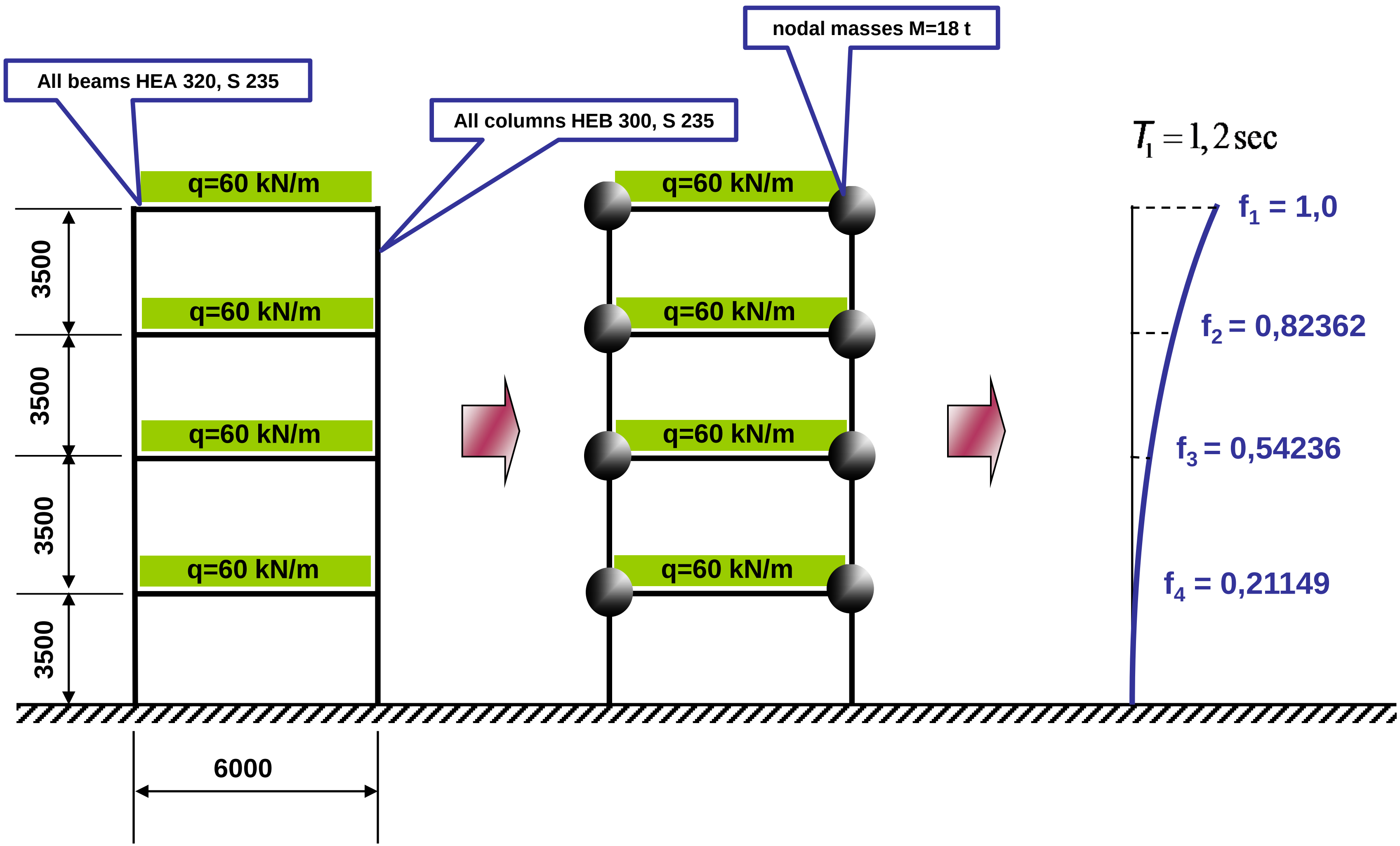
Lateral loads:

- EC 8 requires the analysis of two vertical distributions of lateral forces:

  1. a ‚uniform' pattern with lateral forces that are proportional to the masses
  2. a ‚modal' pattern proportional to the lateral force distribution determined from linear analysis (see "Linear Methods of Analysis")

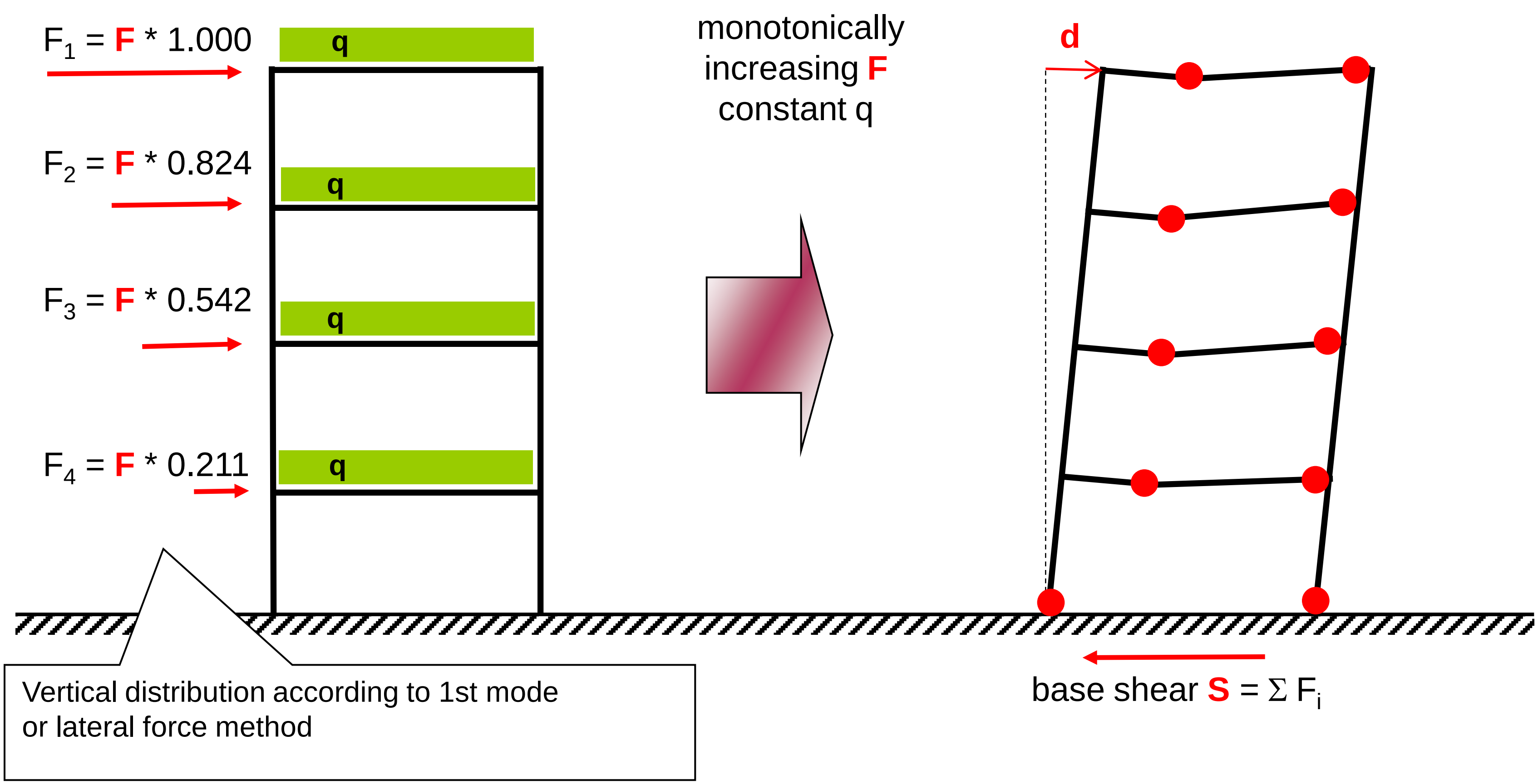
Non-Linear Methods of Analysis

Static Push-Over Analysis - Example

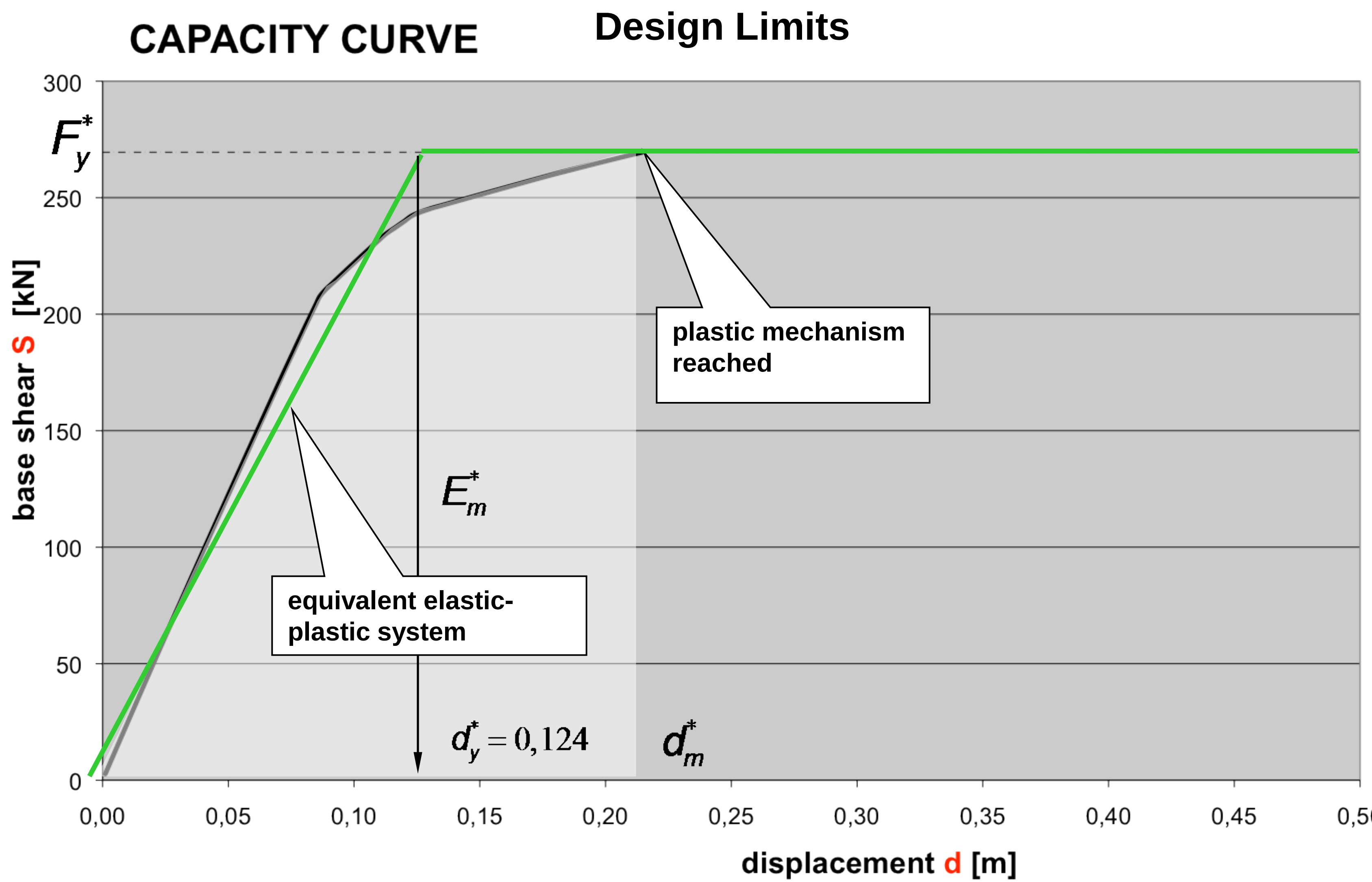


Non-Linear Methods of Analysis

Static Push-Over Analysis - Example



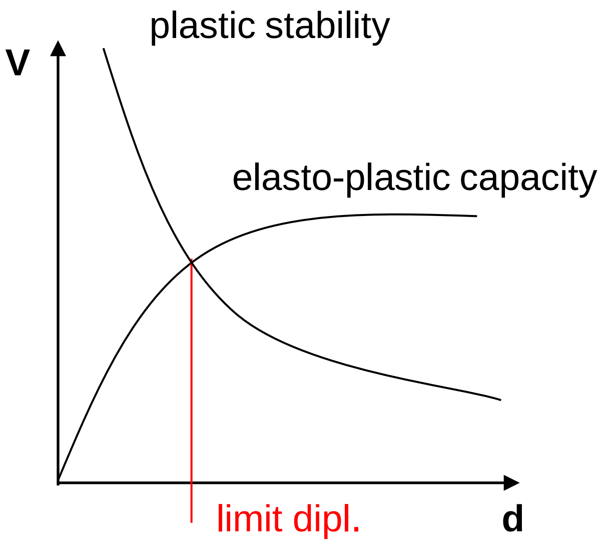
Non-Linear Methods of Analysis

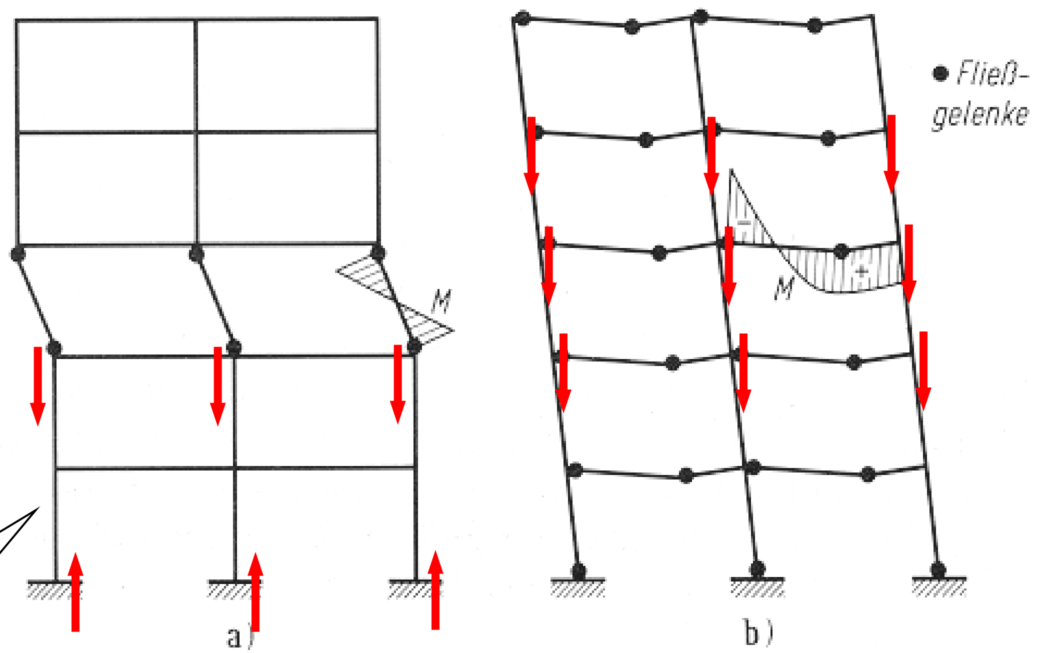


Non-Linear Methods of Analysis

Design Limits

non-linear cyclic behavior of frames





Failure in soft storey

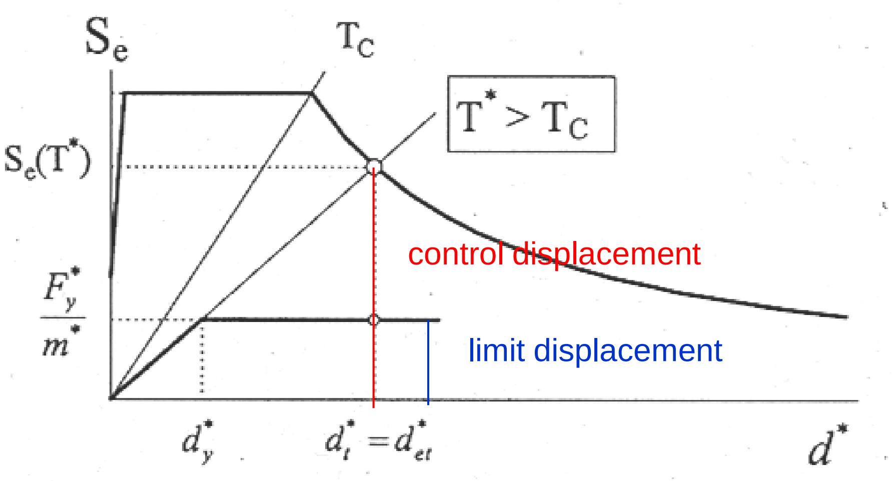
From Müller, Keintzel (2)

Non-Linear Methods of Analysis

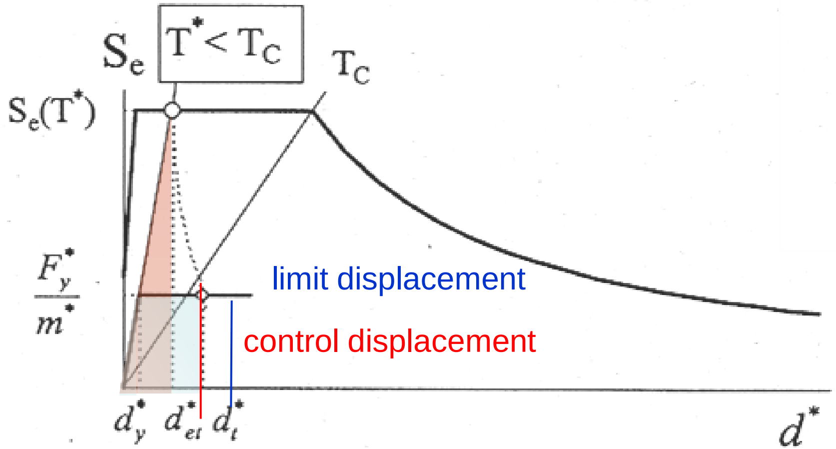
Static Push-Over Analysis - Verification

Assuming an elastic system, the relationship between accelerations and displacements is given by:

$$u = \left(\frac{T}{2 \cdot \pi}\right)^2 \cdot \ddot{u}$$



elastic displ. ~ plastic displ.



elastic energy ~ plastic energy

Non-Linear Methods of Analysis

Time History

- Non-linear structural models under a number of (generated) earthquakes
- Using less than 7, peak non-linear displacements must be used for design
- With 7 or more, average displacements can be used

Design limits may be:

- Ultimate deformation capacity of a member (e.g. rotational capacity of a plastic hinge

- Overall plastic instability due to vertical loads

References

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