# Seismic design of bridges 

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## Bridge types

## Common bridge types

Horizontal slabs or girders supported by abutments and piers.

Common types:

- Slab type
- I-beam type
- Box girder


## Common bridge types

Slab type

- The width $B$ is comparable to the span length $L$
- Applied in case of small spans
- The deck is usually made with voids



## Common bridge types

I- beam type

- Precast (usually) or cast-in-situ beams (rarely)
- Beams are usually prestressed
- Various methods for placing the precast beams at their position (crane, "caro ponte")
- Can be used in difficult site conditions



## Common bridge types

Box girder bridges

- Deck comprises of hollow box of single or multiple cells
- Applied in case of long spans
- The height H might vary along span



## Box girder bridge



## Balanced cantilever bridges

- Built by segmental increment of the two cantilever arms extending from opposite sides of the pier, meeting at the center.
- Usually of box-type with varying height.



## Arch bridges

- Used in cases of long spans
- Difficult construction (usually)
- Several types
- Typical in older times


1900
1920
I. N. Psycharis "Seismic design of bridges"

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## Suspension bridges

- The deck is suspended from cables
- The suspension cables hang from towers and are anchored at each end of the bridge




## Cable-stayed bridges

- Consists of one or more columns (towers or pylons), with cables supporting the bridge deck.
- A type of balanced cantilever bridge. Each part carries its own weight.




## Geometric classification

Normal or skew

- Normal: The axis of each pier is normal to the axis of the bridge.
- Otherwise it is skew



## Geometric classification

Straight or curved


A bridge can be curved and normal

## Structural considerations

## Structural systems

Simply supported spans
Advantages

- Can take differential settlements and tectonic displacements
- Allow prefabrication (precast
 beams)

Disadvantages

- Large moments at the middle of the spans
- Danger of deck fall during earthquakes (require wide sitting areas)
- Not clear seismic response:
- Asynchronous movement of decks
- Danger of impact between adjacent decks


## Structural systems

Continuous deck
Advantages

- Good distribution of moments between supports and spans $\rightarrow$ small deck thickness

- Good seismic behavior:
- The deck acts as a diaphragm $\rightarrow$ all piers move similarly
- Practically, no danger of deck fall

Disadvantages

- Sensitive to differential settlements of piers
- Cannot accommodate tectonic movements


## Structural systems

Decks with Gerber beams
Advantages

- Best balancing of moments between spans and supports


## Disadvantages



- Serious danger of deck fall during earthquakes due to narrow supports
- Special connecting systems required to reduce possibility of fall


## Pier-to-deck connections

Monolithic


Advantages

- Small displacements (stiff structures)

Disadvantages

- Development of seismic moments at the deck
- Thermal variations, shrinkage and creep produce deformation of the piers

Through bearings


Euveris \$opéers

- Flexible systems $\rightarrow$ type of seismic isolation
Disadvantages
- Large seismic displacements (danger of deck fall)
- Piers behave as cantilever $\rightarrow$ large moments at the base


## Pier-to-deck connections

## Connection through bearings

- Types of bearings
- Laminated elastomeric bearings
Allow horizontal displacements
 and rotations
- Pot bearings

Allow only rotations

- Sliding bearings


Can be elastomeric or pot bearings with sliding mechanism in one or in both directions

## Pier-to-deck connections (cont’d)

Seismic stoppers

- Restrict the displacements in order to avoid deck fall
- Typical mechanisms:
- Bumpers
- Cables
- Dowels - sockets (то́ $\mu$ оऽ - عVториіа)

- Usually are activated for large displacements only


## Types of piers

- Wall-type

- Single-column

- Frame (in transverse direction only)

- Hollow cross section



## Types of foundation

－Shallow foundation
－Only on stiff soil
－Large excavations required
－Pile foundation
－Can be applied in all types of soil（except rock）
－Good seismic behaviour
－Extended－pile foundation（ко入んvoпáбба入оı）
－No pile cap，no excavations
－Cannot bear large base moments
－Provides partial fixation at the base of the piers
－Shaft foundation
－Only on stiff and rocky soils


## Damage from earthquakes

## Fall of deck

Caused by large displacements and insufficient length of support at the piers


## Failure of piers

A. Flexural failure


Hanshin Express-way
Kobe Earthquake, Japan, 1995


## Failure of piers

B. Shear failure


## Rupture of crossing faults



## Foundation / soil failure



1 m lateral movement of pier due to soil liquefaction


## Other reasons

Damage at construction joints


## Seismic design

## General principles of bridge design

- In general, bridges are simple structures from the structural point of view. However, they are also sensitive structures.
- Beyond the seismic analysis, the design of bridges must also include:
- Proper detailing for ductile behaviour of the piers even if elastic analysis is performed)
- Check of displacements (bearings, joints, sitting areas)
- Check of ground failure (foundation of piers, infills behind the abutments)
- Check of liquefaction potential or land-sliding in the wider area that might affect the structure


## Seismic design

- Elasto-plastic design is performed (in general).
- Plastic hinges are allowed only in the piers. The bridge deck shall remain within the elastic range.
- Flexural hinges need not necessarily form in all piers. However the optimum behaviour is achieved if plastic hinges develop approximately simultaneously in as many piers as possible.
- As far as possible the location of plastic hinges should be selected at points accessible for inspection and repair.
- Brittle modes of failure (e.g. shear failure) are not allowed.
- Plastic hinges shall not be formed in reinforced concrete sections where the normalized axial force is large.


## Codes

- Eurocode 8: "Design of structures for earthquake resistance" Part 2: Bridges.
- Greek
- "Guidelines for the seismic design of bridges", Circular E39/99 ( $\triangle$ MEOY/o/884/24.12.1999 Y.ПЕ.X ${ }^{(\Delta . E .) .}$
- "Guidelines for the seismic isolation of bridges", Y.ПЕ.XЛ. $\Delta . E$.

Download from:
http://www.iabse.gr/en/en_EngineeringIssues/en_Standards.htm

## Seismic action

Elastic response spectrum ( $\mathrm{S}_{\mathrm{e}}=$ spectral acceleration)
$S_{e}(T)=a_{g} \cdot S \cdot\left[1+\frac{T}{T_{B}} \cdot(n \cdot 2.5-1)\right]$ yıa $0 \leq T \leq T_{B}$
$\mathrm{S}_{\mathrm{e}}(\mathrm{T})=\mathrm{a}_{\mathrm{g}} \cdot \mathrm{S} \cdot \mathrm{n} \cdot 2.5 \quad$ Yıa $\quad \mathrm{T}_{\mathrm{B}} \leq \mathrm{T} \leq \mathrm{T}_{\mathrm{C}}$
$S_{e}(T)=a_{g} \cdot S \cdot \eta \cdot 2.5 \cdot \frac{T_{C}}{T} \quad$ yıa $\quad T_{C} \leq T \leq T_{D}$
$S_{e}(T)=a_{g} \cdot S \cdot n \cdot 2.5 \cdot \frac{T_{C} \cdot T_{D}}{T^{2}} \quad$ yıa $\quad T_{D} \leq T \leq 4 \mathrm{sec}$
where:

$$
\begin{aligned}
& a_{g}=Y_{\mathrm{I}} \cdot a_{g R} \\
& \eta=\sqrt{\frac{10}{\zeta+5}} \geq 0.55 \\
&=\text { damping } \\
& \text { coefficient }(\zeta \text { in } \%)
\end{aligned}
$$

$$
S=\text { soil factor }
$$



## Seismic action

Ground acceleration

| Seismic Hazard Zone | $\mathrm{a}_{\mathrm{gR}}(\mathrm{g})$ |
| :---: | :---: |
| Z 1 | 0.16 |
| $\mathrm{Z2}$ | 0.24 |
| $\mathrm{Z3}$ | 0.36 |

Importance factor (E39/99)

| Bridge importance | $\mathrm{Y}_{\mathrm{I}}$ |
| :---: | :---: |
| Less than average | 0.85 |
| Average | 1.00 |
| Greater than average | 1.30 |

Soil factor and
characteristic periods

| Ground type | $\mathrm{T}_{\mathrm{B}}(\mathrm{sec})$ | $\mathrm{T}_{\mathrm{C}}(\mathrm{sec})$ | $\mathrm{T}_{\mathrm{D}}(\mathrm{sec})$ | S |
| :---: | :---: | :---: | :---: | :---: |
| A | 0.15 | 0.40 | 2.50 | 1.00 |
| B | 0.15 | 0.50 | 2.50 | 1.20 |
| C | 0.20 | 0.60 | 2.50 | 1.15 |
| D | 0.20 | 0.80 | 2.50 | 1.35 |
| E | 0.15 | 0.50 | 2.50 | 1.40 |

## Ground acceleration

$\mathrm{a}_{\mathrm{g}, \mathrm{R}}=$ reference peak ground acceleration on type A ground.

- The reference peak ground acceleration for each seismic zone, corresponds to the reference return period $T_{\text {ncr }}$ of the seismic action for the no-collapse requirement (or equivalently the reference probability of exceedance in $t_{d}=50$ years, $P_{N C R}$ ).

$$
\mathrm{T}_{\mathrm{NCR}}=\frac{1}{1-\left(1-\mathrm{P}_{\mathrm{NCR}}\right)^{1 / t_{\mathrm{d}}}}
$$

The values assigned to each seismic zone correspond to:

$$
P_{N C R}=10 \% \text {, i.e. } T_{N C R}=475 \text { years. }
$$

- An importance factor $Y_{I}=1,0$ is assigned to the reference return period $\mathrm{T}_{\mathrm{NCR}}$.
- For return periods other than the reference, $\mathrm{Y}_{\mathrm{I}} \neq 1,0$ and the design ground acceleration on type $A$ ground, $\mathrm{a}_{\mathrm{g}}$, is equal to:

$$
a_{g}=Y_{I} \cdot a_{g R}
$$

## Ground types

| Ground <br> type | Description of stratigraphic profile | $\mathrm{V}_{\mathrm{s}, 30}$ <br> $(\mathrm{~m} / \mathrm{sec})$ | $\mathrm{N}_{\text {SPT }}$ <br> $(\mathrm{bl./30} \mathrm{~cm})$ | $\mathrm{C}_{\mathrm{u}}$ <br> $(\mathrm{kPa})$ |
| :---: | :--- | :---: | :---: | :---: |
| A | Rock or other rock-like geological <br> formation, including at most 5 m of weaker <br> material at the surface. | $>800$ | - | - |
| B | Deposits of very dense sand, gravel, or <br> very stiff clay, at least several tens of <br> metres in thickness, characterised by a <br> gradual increase of mechanical properties <br> with depth. | $360-800$ | $>50$ | $>250$ |
| C | Deep deposits of dense or medium dense <br> sand, gravel or stiff clay with thickness <br> from several tens to many hundreds of <br> metres. | $180-360$ | $15-50$ | $70-250$ |
| D | Deposits of loose-to-medium cohesionless <br> soil (with or without some soft cohesive <br> layers), or of predominantly soft-to-firm <br> cohesive soil. | $<180$ | $<15$ | $<70$ |
| E | A soil profile consisting of a surface <br> alluvium layer with vs values of type C or D <br> and thickness varying between about 5 m <br> and 20 m, underlain by stiffer material <br> with vs > 800 m/s. |  |  |  |

## Design spectrum



Period, $T$


## Design for ductile behaviour

- Preferred in regions of moderate to high seismicity (economic and safety reasons)
- In bridges of ductile behaviour it is expected that flexural plastic hinges will be formed, normally in the piers, which act as the primary energy dissipating components.
- As far as possible the location of plastic hinges should be selected at points accessible for inspection and repair
- The bridge deck must remain within the elastic range
- Plastic hinges are not allowed in reinforced concrete sections where the normalised axial force $\eta_{k}$ exceeds 0,6
- Flexural hinges need not necessarily form in all piers. However it is desired that plastic hinges develop approximately simultaneously in as many piers as possible
- Piers and abutments connected to the deck through sliding or flexible elastomeric bearings must, in general, remain within the elastic range.


## Limited ductile/essentially elastic behaviour

- Corresponds to a behaviour factor $q \leq 1,5$
- No significant yield appears under the design earthquake
- For bridges where the seismic response may be dominated by higher mode effects (e.g cable-stayed bridges) or where the detailing for ductility of plastic hinges may not be reliable (e.g. due to the presence of high axial force or of a low shear ratio), it is preferable to select an elastic behaviour ( $q=1$ ).


## Behaviour factor

| Type of Ductile Members | q |
| :--- | :---: |
| Reinforced concrete piers: |  |
| $\quad$ Vertical piers in bending $\left(a_{\mathrm{s}} \geq 3,0\right)$ | $3,5 \lambda\left(a_{\mathrm{s}}\right)$ |
| $\quad$ Inclined struts in bending | $2,1 \lambda\left(a_{\mathrm{s}}\right)$ |
| Steel Piers: | 3,5 |
| $\quad$ Vertical piers in bending | 2,0 |
| Inclined struts in bending | 2,5 |
| Piers with normal bracing | 3,5 |
| Piers with eccentric bracing |  |
| Abutments rigidly connected to the deck: | 1,5 |
| $\quad$ In general | 1,0 |
| "Locked-in" structures | 2,0 |
| Arches |  |

$a_{s}=L / h$ is the shear ratio of the pier, where $L$ is the distance from the plastic hinge to the point of zero moment and $h$ is the depth of the cross section in the direction of flexure of the plastic hinge.

$$
\begin{array}{ll}
\text { For } a_{s} \geq 3 & \lambda\left(a_{s}\right)=1,0 \\
\text { For } 3>a_{s} \geq 1,0 & \lambda\left(a_{s}\right)=\left(a_{s} / 3\right)^{1 / 2}
\end{array}
$$

## Behaviour factor

- For $0.3 \leq \eta_{k} \leq 0.6$, a reduced behaviour factor must be used:

$$
q_{r}=q-\frac{\eta_{k}-0.3}{0.3}(q-1)
$$

- The values of the q-factor of the above table may be used only if the locations of all the relevant plastic hinges are accessible for inspection and repair. Otherwise: $q_{r}=0.6 \cdot q \geq 1.0$
- For piles designed for ductile behaviour:
- $q=2,1$ for vertical piles
- $q=1,5$ for inclined piles
- "Locked-in" structures: their mass follows, essentially, the horizontal seismic motion of the ground ( $\mathrm{T} \leq 0,03 \mathrm{sec}$ ): $\mathrm{q}=1$
- Bridges rigidly connected to both abutments, which are laterally encased, at least over $80 \%$ of their area, in stiff natural soil formations with $\mathrm{T} \geq 0,03 \mathrm{sec}: \mathrm{q}=1.5$
- Vertical direction: $q=1$


## Piers with elastic bearings



## Piers with elastic bearings

System ductility:

$$
\mu_{\Delta, \mathrm{f}}=\frac{\Delta_{\text {tot }, \mathrm{u}}}{\Delta_{\text {tot }, \mathrm{y}}}=\frac{\Delta_{\mathrm{c}, \mathrm{u}}+\Delta_{\mathrm{b}}}{\Delta_{\mathrm{c}, \mathrm{y}}+\Delta_{\mathrm{b}}}
$$

Let f be the ratio of the total (deck) displacement over the corresponding column displacement in the elastic region (up to the yield of the pier). At yield:


$$
\mathrm{f}=\frac{\Delta_{\text {tot }, y}}{\Delta_{\mathrm{c}, \mathrm{y}}}=\frac{\Delta_{\mathrm{c}, \mathrm{y}}+\Delta_{\mathrm{b}}}{\Delta_{\mathrm{c}, \mathrm{y}}}=1+\frac{\Delta_{\mathrm{b}}}{\Delta_{\mathrm{c}, \mathrm{y}}} \Rightarrow \frac{\Delta_{\mathrm{b}}}{\Delta_{\mathrm{c}, \mathrm{y}}}=\mathrm{f}-1
$$

Then

$$
\mu_{\Delta, f}=\frac{\mu_{\Delta, c}+\mathrm{f}-1}{\mathrm{f}}
$$

where $\mu_{\Delta, \mathrm{c}}=\frac{\Delta_{\mathrm{c}, \mathrm{u}}}{\Delta_{\mathrm{c}, \mathrm{y}}}$ is the ductility of the column

## Piers with elastic bearings

The above relation can be written as:

$$
\mu_{\Delta, \mathrm{c}}=1+\mathrm{f} \cdot\left(\mu_{\Delta, \mathrm{f}}-1\right)
$$

The value of f depends on the relative stiffness of the bearings w.r.t. the stiffness of the pier: For "soft" bearings, $f$ is large and the ductility of the pier can attain large values.
Example

- $f=5$ and $\mu_{\Delta, f}=3.5$ (design ductility). Then:
$\mu_{\Delta, c}=1+5 \cdot(3.5-1)=13.5$
- $f=5$ and we want $\mu_{\Delta, c}=3.5$. Then, the design must be performed for:

$$
\mu_{\Delta, f}=(3.5+5-1) / 5=1.5
$$

## Piers with elastic bearings

For this reason, EC8 - part 2 (bridges) considers that:
When the main part of the design seismic action is resisted by elastomeric bearings, the flexibility of the bearings imposes a practically elastic behaviour of the system.
The allowed values of $q$ are:
$\mathrm{q} \leq 1.5$ according to EC8
$\mathrm{q}=1.0$ according to E39/99

## Displacement ductility \& Curvature ductility



## Displacement ductility \& Curvature ductility



During elastic response ( $\mathrm{P}<\mathrm{P}_{\mathrm{y}}$ )
$\Delta=\frac{P}{K}$ and $C=\frac{M}{E I}$ where $K=\frac{3 E I}{L^{3}}$ and $M=P \cdot L$
which leads to

$$
\Delta_{y}=\frac{C_{y} \cdot L^{2}}{3}
$$

After yield

$$
\begin{aligned}
& C_{p}=C_{u}-C_{y} \text { and } \Delta_{p}=\Delta_{u}-\Delta_{y} \\
& \theta_{p}=L_{h} \cdot C_{p}=L_{h} \cdot\left(C_{u}-C_{y}\right) \\
& \Delta_{p}=\Delta_{p, 1}+\Delta_{p, 2} \\
& \Delta_{p, 1}=\Delta_{y} \cdot\left(\frac{M_{u}}{M_{y}}-1\right) \text { (hardening) }
\end{aligned}
$$

$$
\Delta_{p, 2}=\theta_{p} \cdot\left(L-0.5 \cdot L_{h}\right)=L_{h} \cdot\left(C_{u}-C_{y}\right) \cdot\left(L-0.5 \cdot L_{h}\right) \text { (plastic rotation) }
$$

Therefore

$$
\Delta_{p}=\Delta_{y} \cdot\left(\frac{M_{u}}{M_{y}}-1\right)+L_{h} \cdot\left(C_{u}-C_{y}\right) \cdot\left(L-0.5 \cdot L_{h}\right)
$$

## Displacement ductility \& Curvature ductility

By definition

$$
\begin{aligned}
& \mu_{\Delta}=\frac{\Delta_{u}}{\Delta_{y}}=1+\frac{\Delta_{p}}{\Delta_{y}} \\
& \mu_{C}=\frac{C_{u}}{C_{y}}=1+\frac{C_{p}}{C_{y}}
\end{aligned}
$$

Finally

$$
\mu_{\Delta}=\frac{M_{u}}{M_{y}}+3 \lambda \cdot\left(\mu_{c}-1\right) \cdot(1-0.5 \cdot \lambda)
$$

$$
\mu_{\mathrm{C}}=1+\frac{\mu_{\Delta}-1}{3 \lambda \cdot(1-0.5 \cdot \lambda)}
$$

where $\lambda=L_{h} / L$
Example: $\lambda=0.1, \mu_{\Delta}=3.0 \Rightarrow \mu_{c}=7.0$


For longitudinal reinforcement of characteristic yield stress $f_{\mathrm{yk}}$ (in MPa) and bar diameter $d_{\mathrm{s}}$, the plastic hinge length $L_{\mathrm{h}}$ may be assumed as follows:

$$
L_{\mathrm{h}}=0,10 L+0,015 f_{\mathrm{yk}} d_{\mathrm{s}}
$$

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## Model of analysis

## Masses

- Permanent masses (with their characteristic value)
- Quasi-permanent values of the masses corresponding to the variable actions: $\Psi_{2,1} Q_{k, 1}$, where $Q_{k, 1}$ is the characteristic value of traffic load and
- $\Psi_{2,1}=0,2$ for road bridges
- $\Psi_{2,1}=0,3$ for railway bridges
- When the piers are immersed in water, an added mass of entrained water acting in the horizontal directions per unit length of the immersed pier shall be considered (see Annex F of EC8-2)


## Model of analysis

## Stiffness of reinforced concrete ductile members

- The effective stiffness of ductile concrete components used in linear seismic analysis should be equal to the secant stiffness at the theoretical yield point.
- In the absence of a more accurate assessment, the approximate methods proposed in Appendix C of EC8-2 may be used.
- E39/99 proposes:
- At piers expected to yield, (EI) efff $=300 \cdot M_{R d, h} \cdot d$, where $M_{R d, h}$ is the moment of resistance and $d$ is the height of the cross section at the place of the plastic hinge.
- At piers expected to respond elastically, the mean value between the above value and the one that corresponds to the uncracked cross section.


## Model of analysis

Stiffness of reinforced concrete ductile members Appendix C of EC8-2

- Method 1

$$
J_{\text {eff }}=0,08 \cdot J_{\mathrm{un}}+J_{\text {cr }}
$$

- $J_{\mathrm{un}}=$ moment of inertia of uncracked section
- $J_{\mathrm{cr}}=M_{\mathrm{y}} /\left(E_{\mathrm{c}} \cdot C_{\mathrm{y}}\right)=$ moment of inertia of cracked section
- Method 2

$$
E_{\lambda_{\text {eff }}}=v \cdot M_{\mathrm{Rd}} / C_{y}
$$

- $v=1,20=$ correction coefficient reflecting the stiffening effect of the uncracked parts of the pier
- $C_{y}=2,1 \varepsilon_{\mathrm{s} /} / d \quad$ for rectangular sections
$C_{y}=2,4 \varepsilon_{s y} / d \quad$ for circular sections
where $d$ is the effective depth of the section


## Model of analysis

Stiffness of elastomeric bearings

$$
K_{b}=\frac{G \cdot A_{b}}{T}
$$

where:
$\mathrm{G}=$ shear modulus
G varies with time, temperature etc. Two cases:
(a) $G_{\text {min }}=G_{b}$; (b) $G_{\text {max }}=1,5 G_{b}$
where $\mathrm{G}_{\mathrm{b}}=1.1 \mathrm{G}_{\mathrm{g}}$ with $\mathrm{G}_{\mathrm{g}}=0.9 \mathrm{~N} / \mathrm{mm}^{2}=900 \mathrm{KPa}$.
$A_{b}=$ effective area of bearing
$A_{b}=\left(b_{x}-d_{E d, x}\right) \cdot\left(b_{y}-d_{E d, y}\right)$ for rectangular bearings of dimensions $b_{x} \times b_{y}$, where $d_{\text {Edx }}, d_{\text {Edy }}$ are the seismic design displacements of the bearing in $x$ - and $y$-direction, respectively.
$\mathrm{T}=$ total thickness of elastomer

## Design displacements

Design seismic displacement

$$
d_{E}=\eta \cdot \mu_{d} \cdot d_{E e}
$$

where
$\mathrm{d}_{\mathrm{Ee}}=$ displacement from the analysis
$\eta$ = damping correction factor
$\mu_{d}= \begin{cases}q & \text { for } T \geq T_{0}=1,25 T_{C} \\ (q-1) \frac{T_{0}}{T}+1 \leq 5 q-4 & \text { for } T<T_{0}\end{cases}$
Total design displacement

$$
d_{E d}=d_{E}+d_{G}+\Psi_{2} \cdot d_{T}
$$

where
$d_{G}=$ displacement due to the permanent and quasi-permanent actions
$d_{\mathrm{T}}=$ displacement due to thermal movements

## Second order effects

Approximate methods may be used for estimating the influence of second order effects on the critical sections (plastic hinges):

$$
\Delta M=\frac{1+q}{2} d_{E d} \cdot N_{E d}
$$

where $N_{E d}$ is the axial force and $d_{E d}$ is the relative transverse displacement of the ends of the member

## Design seismic combination

The design action effects $E_{d}$ in the seismic design situation shall be derived from the following combination of actions:
$G_{k}$ "+" $P_{k}$ "+" $A_{E d}$ "+" $\Psi_{21} Q_{1 k}$ "+" $Q_{2}$
where "+" means "to be combined with" and
$\mathrm{G}_{\mathrm{k}}=$ the permanent loads with their characteristic values
$P_{k}=$ the characteristic value of prestressing after all losses
$A_{\text {ed }}=$ is the most unfavourable combination of the components of the earthquake action
$\mathrm{Q}_{1 \mathrm{k}}=$ the characteristic value of the traffic load
$\Psi_{21}=$ the combination factor
$\mathrm{Q}_{2}=$ the quasi permanent value of actions of long duration (e.g. earth pressure, buoyancy, currents etc.)

## Design seismic combination

- Seismic action effects need not be combined with action effects due to imposed deformations (temperature variation, shrinkage, settlements of supports, ground residual movements due to seismic faulting).
- An exception to the rule stated above is the case of bridges in which the seismic action is resisted by elastomeric laminated bearings. In this case, elastic behaviour of the system shall be assumed and the action effects due to imposed deformations shall be accounted for.
- For wind and snow actions, the value $\psi_{21}=0$ shall be assumed.


## Capacity design

## Purpose

To design structures of ductile behaviour ensuring the hierarchy of strengths of the various structural components necessary for leading to the intended configuration of plastic hinges and for avoiding brittle failure modes.

## Design procedure

Use "capacity design effects":

- For the design of all members intended to remain elastic
- Against all brittle modes of failure.


## Definition

"Capacity design effects" result from equilibrium conditions at the intended plastic mechanism, when all flexural hinges have developed their flexural resistance including overstrength.

## Capacity design

Capacity design is applied:

- For the design of sections that must remain within the elastic range (e.g. deck).
- For the design of all members against non-ductile failure modes (shear of members and shear of joints adjacent to plastic hinges).
- For the design of the foundation (except of pile foundation, where plastic hinges are allowed).
- For the design of seismic stoppers and for bearings, links and holding-down devices used for securing structural integrity.

The capacity design effects need not be taken greater than those resulting from the design seismic combination where the design effects $A_{E d}$ are multiplied by the $q$ factor used.

## Capacity design

Overstrength moment of a section

$$
M_{0}=y_{0} M_{R d}
$$

where
$\mathrm{Y}_{0}=$ overstrength factor

$$
\text { for concrete members: } \mathrm{Y}_{0}=1,35 \text { (EC8 - part 2) }
$$

$$
=1,40(\mathrm{E} 39 / 99)
$$

$M_{R d}=$ design flexural strength of the section, in the selected direction and sense, based on the actual section geometry and reinforcement

Piers with elastomeric bearings
In bridges intended to have ductile behaviour, in the case of piers with elastomeric bearings where no plastic hinges are intended to form, the capacity design effects shall be calculated on the basis of the maximum deformation of the elastomeric bearings and a $30 \%$ increase of the bearing stiffness.

## Calculation of capacity design effects

The following procedure shall be applied for each of the two horizontal directions of the design seismic action.

## Step 1

Calculation of the design flexural strengths $M_{R d}$ and of the overstrength moments $M_{0}$ of the sections of the intended plastic hinges, corresponding to the selected sense and direction of the seismic action $\left(\mathrm{A}_{\mathrm{E}}\right)$.

- The strengths shall be based on the actual dimensions of the cross-sections and the final amount of longitudinal reinforcement.
- The calculation shall consider the interaction with the axial force and eventually with the bending moment in the other direction, both resulting from the combination $G$ " + " $A_{E}$ where G is the sum of the permanent actions (gravity loads and posttensioning) and $A_{E}$ is the design seismic action.


## Calculation of capacity design effects

## Example

Permanent actions G:
top: $\quad M_{x-x, t}=-20 \mathrm{KNm}$
$M_{y-y, t}=-120 \mathrm{KNm}$
$N_{t}=3000 \mathrm{KN}$
bottom: $M_{x-x, b}=30 \mathrm{KNm}$
$M_{y-y, b}=150 \mathrm{KNm}$
$\mathrm{N}_{\mathrm{b}}=3500 \mathrm{KN}$
shear: $V_{x}=27 \mathrm{KN}$


Seismic action $A_{E}$ :

$$
\begin{array}{ll}
\text { top: } & M_{x-x, t}=-300 \mathrm{KNm} \\
& M_{y-y, t}=-1200 \mathrm{KNm} \\
& N_{t}=40 \mathrm{KN} \\
\text { bottom: } & M_{x-x, b}=450 \mathrm{KNm} \\
& M_{y-y, b}=1500 \mathrm{KNm} \\
& N_{b}=40 \mathrm{KN} \\
\text { shear: } & V_{\mathrm{x}}=270 \mathrm{KN}
\end{array}
$$

## Calculation of capacity design effects

## Example (cont'd)

The design flexural strengths ( $\mathrm{M}_{\text {Rd, }, \mathrm{y}-\mathrm{y}}$ ) are calculated considering the interaction with the axial force and the bending moment in the other direction:
top: $\quad N_{t}=3000+40=3040 \mathrm{KN}$

$$
M_{x-x, t}=-20-300=-320 \mathrm{KNm}
$$

bottom: $\mathrm{N}_{\mathrm{b}}=3500+40=3540 \mathrm{KN}$

$$
M_{x-x, b}=30+450=480 \mathrm{KNm}
$$

Let us assume that, for these values and the actual reinforcement, the resulting values are:
top: $\quad M_{R d, t}=1400 \mathrm{KNm}$
bottom: $\mathrm{M}_{\mathrm{Rd}, \mathrm{b}}=1800 \mathrm{KNm}$
The corresponding overstrength moments are (for $\mathrm{Y}_{0}=1,40$ ):
top: $\quad M_{0, t}=1.40 \times 1400=1960 \mathrm{KNm}$
bottom: $M_{0, b}=1.40 \times 1800=2520 \mathrm{KNm}$

## Calculation of capacity design effects

## Step 2

Calculation of the variation of action effects $\Delta A_{C}$ of the plastic mechanism, caused by the increase of the moments of the plastic hinges ( $\Delta \mathrm{M}$ ), from the values due to the permanent actions $\left(M_{G}\right)$ to the moment overstrength of the sections $\left(M_{0}\right)$.

$$
\Delta M=Y_{0} M_{R d}-M_{G}
$$

The effects $\Delta A_{C}$ may in general be estimated from equilibrium conditions.

Example (cont'd):
top: $\quad \Delta M_{t}=1960-120=1840 \mathrm{KNm}$
bottom: $\Delta \mathrm{M}_{\mathrm{b}}=2520-150=2370 \mathrm{KNm}$
The corresponding variation of the shear force is:

$$
\Delta V_{C}=\frac{1840+2370}{10}=421 \mathrm{KN}
$$

## Calculation of capacity design effects

## Step 3

The final capacity design effects $A_{C}$ shall be obtained by superimposing the variation $\Delta A_{C}$ to the permanent action effects $F_{G}$ :

$$
A_{C}=A_{G}+\Delta A_{C}
$$

Example (cont'd):
Capacity design shear: $\mathrm{V}_{\mathrm{C}}=27+421=448 \mathrm{KN}$

## Simplification

When the bending moment due to the permanent actions at the plastic hinge is negligible compared to the moment overstrength of the section ( $M_{G} \ll Y_{0} M_{R d}$ ), the effects $\Delta A_{C}$ can be directly estimated from the effects $A_{E}$ of the design earthquake action. For example, for cantilever piers, the capacity design shear is:

$$
V_{C}=\Delta V_{C}=\frac{V_{0} M_{R d}}{M_{E}} V_{E}
$$

## Earth pressure on abutments and retaining walls

Seismic coefficients

- Horizontal: $k_{h}=a \cdot S / r$
- Vertical: $k_{v}= \pm 0,5 \cdot k_{h}$
where
$\mathrm{a}=\mathrm{a}_{\mathrm{g}} / \mathrm{g}\left(\mathrm{a}_{\mathrm{g}}=\right.$ design ground acceleration on ground type A$)$
$\mathrm{S}=$ soil coefficient
$r=$ coefficient that depends on the amount of permanent displacement which is both acceptable and actually permitted by the adopted structural solution

| Type of retaining structure | r |
| :--- | :---: |
| Free gravity walls that can accept a displacement up to $d_{r}=300 \mathrm{a} \cdot \mathrm{S}(\mathrm{mm})$ | 2,0 |
| Free gravity walls that can accept a displacement up to $d_{r}=200 \mathrm{a} \cdot \mathrm{S}(\mathrm{mm})$ | 1,5 |
| Flexural reinforced concrete walls, anchored or braced walls, reinforced <br> concrete walls founded on vertical piles, restrained basement walls and bridge <br> abutments | 1,0 |

## Earth pressure on abutments and retaining walls

Flexible abutments and walls
Total pressure (static + dynamic):

$$
E_{d}=\frac{1}{2} \cdot \gamma_{s} \cdot\left(1 \pm k_{v}\right) \cdot K \cdot H^{2}
$$

where
H = wall height
K = earth pressure coefficient. It may be computed from the Mononobe - Okabe formula (EC8 - Part 5,
 Appendix E)
$\mathrm{k}_{\mathrm{v}}=$ vertical seismic coefficient
$\gamma_{s}=$ specific weight of the soil
The point of application is considered at height equal to $0,4 \mathrm{H}$.

Earth pressure on abutments and retaining walls
Rigid abutments and walls

## According to Eurocode 8-part 5:

- Neutral static earth pressure

$$
E_{0}=\frac{1}{2} \cdot \gamma_{s} \cdot K_{0} \cdot H^{2}
$$

where $K_{0}=1-\sin \varphi$


- Additional pressure (dynamic):

$$
\Delta \mathrm{E}_{\mathrm{d}}=\mathrm{a} \cdot \mathrm{~S} \cdot \mathrm{Y}_{\mathrm{s}} \cdot \mathrm{H}^{2}
$$

The point of application may be taken at mid-height


Earth pressure on abutments and retaining walls

Rigid abutments and walls

## According to E39/99:

- Neutral static earth pressure $\mathrm{E}_{0}$
- Two cases for the additional (dynamic) pressure:
- Limited flexible walls:

$p_{d}=0,75 \cdot a \cdot \gamma_{s} \cdot H$
Uniform distribution of pressure:

$$
p_{d}=0,75 \cdot a \cdot \gamma_{s} \cdot H
$$

- Non-deformable walls:
- Top: $\quad p_{d}=1,50 \cdot a \cdot \gamma_{s} \cdot H$
- Bottom: $\mathrm{p}_{\mathrm{d}}=0,50 \cdot a \cdot \mathrm{r}_{\mathrm{s}} \cdot \mathrm{H}$



## Abutments flexibly connected to the deck

The following actions, assumed to act in phase, should be taken into account:

- Earth pressures (flexible abutments)

When the earth pressures are determined on the basis of an acceptable displacement of the abutment ( $r>1$ ), it should be ensured that this displacement can actually take place before a potential failure of the abutment itself occurs. For this reason, the body of the abutment is designed using the seismic part of the earth pressure increased by $30 \%$.

- Inertia forces acting on the mass of the abutment and on the mass of earthfill lying over its foundation determined using the design ground acceleration $\mathrm{a}_{\mathrm{g}}$.
- Actions from the bearings determined:
- From capacity design effects if a ductile behaviour has been assumed for the bridge.
- From the reaction on the bearings resulting from the seismic analysis if the bridge is designed for $\mathrm{q}=1,0$.


## Abutments rigidly connected to the deck

The following actions should be taken into account:

- Inertia forces acting on the mass of the structure which may be estimated using the Fundamental Mode Method. A behaviour factor $q=1,5$ shall be used.
Abutments buried in strong soils for more than $80 \%$ of their height can be considered as fully locked-in. In that case, q $=1$ shall be used and the inertia forces are determined on the basis of the design ground acceleration $\mathrm{a}_{\mathrm{g}}$.
- Static earth pressures acting on both abutments $\left(\mathrm{E}_{0}\right)$.
- Additional seismic earth pressures $\Delta \mathrm{E}_{\mathrm{d}}=\mathrm{E}_{\mathrm{d}}-\mathrm{E}_{0}$. The pressures $\Delta \mathrm{E}_{d}$ are assumed to act in the same direction on both abutments.
Reactions on the passive side may be taken into account, estimated on the basis of horizontal soil moduli corresponding to the specific geotechnical conditions.


## Resistance verification of concrete sections

In general, verifications of shear resistance shall be carried out in accordance with par. 6.2 of EC 2 with some additional rules.

For the flexural resistance of sections, the following conditions shall be satisfied

Structures of limited ductile behaviour ( $q \leq 1,5$ )

$$
\begin{aligned}
& \quad E_{d} \leq R_{d} \\
& E_{d}=\text { the design action effect under the seismic load } \\
& \text { combination including second order effects } \\
& \mathrm{R}_{\mathrm{d}}=\text { the design flexural resistance of the section. }
\end{aligned}
$$

## Resistance verification of concrete sections

## Structures of ductile behaviour

- Flexural resistance of sections of plastic hinges:

$$
M_{\mathrm{Ed}} \leq M_{\mathrm{Rd}}
$$

$M_{E d}=$ the design moment under the seismic load combination, including second order effects
$M_{R d}=$ the design flexural resistance of the section.

- Flexural resistance of sections outside the region of plastic hinges:

$$
M_{C} \leq M_{R d}
$$

$M_{C}=$ the capacity design moment
$M_{R d}=$ the design resistance of the section, taking into account the interaction of the corresponding design effects (axial force and when applicable the bending moment in the other direction).

## Minimum overlap length

At supports, where relative displacement between supported and supporting members is intended under seismic conditions, a minimum overlap length, $\mathrm{L}_{\text {ov }}$, shall be provided, which may be estimated as:

$$
L_{\mathrm{ov}}=L_{\mathrm{m}}+d_{\mathrm{eg}}+d_{\mathrm{es}}
$$

where:
$L_{m}=$ the minimum support length securing the safe transmission of the vertical reaction with $L_{m} \geq 40 \mathrm{~cm}$.
$d_{\text {eg }}=$ the effective displacement of the two parts due to differential seismic ground displacement, which can be estimated from the procedure given in the following.
$d_{\text {es }}=$ the effective seismic displacement of the support due to the deformation of the structure, which can be estimated from the procedure given in the following.

## Minimum overlap length

Calculation of $\mathrm{d}_{\text {eg }}$

$$
\mathrm{d}_{\mathrm{eg}}=\mathrm{L}_{\text {eff }} \cdot \mathrm{v}_{\mathrm{g}} / \mathrm{c}_{\mathrm{a}} \leq 2 \cdot \mathrm{~d}_{\mathrm{g}}
$$

where:
$\mathrm{L}_{\text {eff }}=$ the effective length of deck, taken as the distance from the deck joint in question to the nearest full connection of the deck to the substructure.
"full connection" means a connection of the deck to a substructure member, either monolithically or through fixed bearings or seismic links.
$\mathrm{v}_{\mathrm{g}}=$ peak ground velocity, estimated from the design ground acceleration $a_{g}$ using the relation: $\mathrm{v}_{\mathrm{g}}=0,16 \cdot \mathrm{~S} \cdot \mathrm{~T}_{\mathrm{C}} \cdot \mathrm{a}_{\mathrm{g}}$.
$c_{a}=$ apparent phase velocity of the seismic waves in the ground.
$\mathrm{d}_{\mathrm{g}}=$ design value of the peak ground displacement.

## Minimum overlap length

## Calculation of $\mathrm{d}_{\text {es }}$

- For decks connected to piers either monolithically or through fixed bearings, acting as full seismic links: $d_{e s}=d_{E d}$, where $d_{E d}$ is the total longitudinal design seismic displacement.
- For decks connected to piers or to an abutment through seismic links with slack equal to $s: d_{e s}=d_{E d}+s$.

In the case of an intermediate separation joint between two sections of the deck, $L_{\text {ov }}$ should be estimated by taking the square root of the sum of the squares of the values calculated for each of the two sections of the deck. In the case of an end support of a deck section on an intermediate pier, $\mathrm{L}_{\mathrm{ov}}$ should be estimated as above and increased by the maximum seismic displacement of the top of the pier $\mathrm{d}_{\mathrm{E}}$.

## Seismic isolation of bridges

## Concept

Application of special isolating system, aiming to reduce the response due to the horizontal seismic action.

The isolating units are arranged over the isolation interface, usually located under the deck and over the top of the piers/abutments.

## Methods

- Lengthening of the fundamental period of the structure
- reduces forces
- increases displacements
- Increasing the damping
- reduces displacements
- may reduce forces
- Combination of the two effects.



## Basic requirements

## Isolators

Each isolator unit must provide a single or a combination of the following functions:

- vertical-load carrying capability combined with increased lateral flexibility and high vertical rigidity
- energy dissipation (hysteretic, viscous, frictional)
- re-centring capability
- horizontal restraint (sufficient elastic rigidity) under nonseismic service horizontal loads

Increased reliability is required for the strength and integrity of the isolating system, due to the critical role of its displacement capability for the safety of the structure.

## System

The seismic response of the superstructure and substructures shall remain essentially elastic.

## Isolators with hysteretic behaviour

$$
\left.\begin{array}{rl}
\mathrm{d}_{\mathrm{y}}= & \text { yield displacement } \\
\mathrm{d}_{\mathrm{bd}}= & \text { design displacement } \\
& \text { of the isolator that } \\
& \text { corresponds to the } \\
& \text { design displacement } \\
& \mathrm{d}_{\mathrm{cd}} \text { of the system } \\
\mathrm{F}_{\mathrm{y}}= & \text { yield force at } \\
& \text { monotonic loading } \\
\mathrm{F}_{\mathrm{max}}= & \text { force at maximum } \\
& \text { displacement } \mathrm{d}_{\mathrm{bd}}
\end{array}\right\}
$$



$d_{y}=d_{L y}=$ yield displacement of lead core
$F_{y}=F_{L y^{\prime}}\left(1+K_{R} / K_{L}\right)$ where $F_{L y}=$ yield force of lead core
$\mathrm{K}_{\mathrm{e}}=\mathrm{K}_{\mathrm{R}}+\mathrm{K}_{\mathrm{L}}=$ elastic stiffness
$\mathrm{K}_{\mathrm{p}}=\mathrm{K}_{\mathrm{R}}=$ post-elastic stiffness

## Isolators with viscus behaviour

The force is zero at the maximum displacement, therefore viscous isolators do not contribute to the effective stiffness of the isolating system
$\mathrm{F}=\mathrm{C} \cdot \mathrm{v}^{\mathrm{a}}$
For sinusoidal motion:
$\mathrm{d}(\mathrm{t})=\mathrm{d}_{\mathrm{bd}} \cdot \sin (\omega \mathrm{t})$
$F(t)=F_{\text {max }} \cdot[\cos (\omega t)]^{a}$
$\mathrm{F}_{\text {max }}=\mathrm{C} \cdot\left(\mathrm{d}_{\mathrm{bd}} \cdot \omega\right)^{\mathrm{a}}$


Dissipated energy per cycle:

$$
\mathrm{E}_{\mathrm{D}}=\lambda(\mathrm{a}) \cdot \mathrm{F}_{\max } \cdot \mathrm{d}_{\mathrm{bd}}
$$

$\lambda(a)=2^{(2+a)} \cdot \Gamma(1+0,5 a) / \Gamma(2+a)$
$\Gamma()$ is the gamma function

## Isolators with friction behaviour

A. Flat sliding surface
$\mathrm{F}_{\text {max }}=\mu_{\mathrm{d}} \cdot \mathrm{N}_{\mathrm{Sd}}$
where:
$\mu_{d}=$ dynamic friction coefficient
$\mathrm{N}_{\mathrm{Sd}}=$ normal force through the device

Such devices can result in substantial permanent offset displacements.
Therefore, they should be used in combination with devices providing adequate
 restoring force.

## Isolators with friction behaviour


B. Spherical sliding surface of radius $R_{b}$
$F_{0}=\mu_{\mathrm{d}} \cdot \mathrm{N}_{\mathrm{Sd}}$
$F_{\text {max }}=\mu_{d} \cdot N_{S d}+K_{p} \cdot d_{b d}$
$\mathrm{K}_{\mathrm{p}}=\mathrm{N}_{\mathrm{Sd}} / \mathrm{R}_{\mathrm{b}}$
Dynamically, the device behaves as an inverted pendulum with period

$$
T=2 \pi \sqrt{\frac{R_{b}}{g}}
$$

Dissipated energy per cycle:

$$
\mathrm{E}_{\mathrm{D}}=4 \cdot \mathrm{~F}_{0} \cdot \mathrm{~d}_{\mathrm{bd}}
$$

Friction Pendulum System (FPS)

## Fundamental mode spectrum analysis

- The deck is assumed rigid
- The shear force transferred through the isolating interface shall be estimated considering the superstructure to behave as a single degree of freedom system using:
- the effective stiffness of the isolation system, $\mathrm{K}_{\text {eff }}$
- the effective damping of the isolation system, $\zeta_{\text {eff }}$
- the mass of the superstructure, $\mathrm{m}_{\mathrm{d}}=\mathrm{W}_{\mathrm{d}} / \mathrm{g}$
- the spectral acceleration $\mathrm{S}_{\mathrm{e}}\left(\mathrm{T}_{\text {eff }}, \zeta_{\text {eff }}\right)$ that corresponds to the effective period $\mathrm{T}_{\text {eff }}$ and the effective damping $\zeta_{\text {eff }}$


## Fundamental mode spectrum analysis

- Effective stiffness:

$$
K_{\text {eff }}=\sum K_{\text {eff }, i}
$$

where $\mathrm{K}_{\text {eff, }}$ is the composite stiffness of the isolator unit and the corresponding substructure (pier) i.

In the calculation of the composite stiffness of each pier, the flexibility of the foundation must also be considered:

$$
\frac{1}{\mathrm{~K}_{\text {eff,i }}}=\frac{1}{\mathrm{~K}_{\mathrm{c}, \mathrm{i}}}+\frac{1}{\mathrm{~K}_{\mathrm{b}, \mathrm{eff}, \mathrm{i}}}+\frac{1}{\mathrm{~K}_{\mathrm{t}, \mathrm{i}}}+\frac{\mathrm{H}_{\mathrm{i}}^{2}}{\mathrm{~K}_{\mathrm{r}, \mathrm{i}}}
$$

where
$\mathrm{K}_{\mathrm{b}, \text { eff }}=$ the effective stiffness of the isolators of the pier
$\mathrm{K}_{\mathrm{c}, \mathrm{i}}=$ stiffness of the column of the pier
$\mathrm{K}_{\mathrm{t}, \mathrm{i}}=$ translational stiffness of the foundation in the horizontal direction
$\mathrm{K}_{\mathrm{r}, \mathrm{i}}=$ rotational stiffness of the foundation
$H_{i}=$ height of the pier measured from the level of the foundation

Fundamental mode spectrum analysis

- Total effective damping of the system:

$$
\zeta_{\text {eff }}=\frac{1}{2 \pi} \cdot \frac{\sum E_{D, i}}{K_{\text {eff }} \cdot d_{c d}^{2}}
$$

where
$\sum E_{D, i}$ is the sum of the dissipated energies of all isolators $i$ in a full deformation cycle at the design displacement $\mathrm{d}_{\mathrm{cd}}$.

Then, the damping correction factor is:

$$
\eta_{\text {eff }}=\sqrt{\frac{0,10}{0,05+\zeta_{\text {eff }}}}
$$

- Effective period of the system:

$$
\mathrm{T}_{\text {eff }}=2 \pi \sqrt{\frac{\mathrm{~m}_{\mathrm{d}}}{\mathrm{~K}_{\text {eff }}}}
$$

Fundamental mode spectrum analysis

- Spectral acceleration and design displacement (for $T_{\text {eff }}>T_{C}$ )

| $T_{\text {eff }}$ | $S_{e}$ | $d_{c d}$ |
| :---: | :---: | :---: |
| $T_{C} \leq T_{\text {eff }}<T_{D}$ | $a_{g} \cdot S \cdot \eta_{\text {eff }} \cdot 2.5 \cdot \frac{T_{C}}{T_{\text {eff }}}$ | $\frac{T_{\text {eff }}}{T_{C}} \cdot d_{C}$ |
| $T_{D} \leq T_{\text {eff }}$ | $a_{g} \cdot S \cdot \eta_{\text {eff }} \cdot 2.5 \cdot \frac{T_{C} \cdot T_{D}}{T_{\text {eff }}^{2}}$ | $\frac{T_{D}}{T_{C}} \cdot d_{C}$ |

where

$$
d_{C}=\frac{0,625}{\pi^{2}} \cdot a_{g} \cdot S \cdot \eta_{\text {eff }} \cdot T_{C}^{2}
$$

## Multi-mode spectrum analysis

- The effective damping $\zeta_{\text {eff }}$ is applied only to modes having periods higher than $0,8 \mathrm{~T}_{\text {eff. }}$. For all other modes, the damping ratio corresponding to the structure without seismic isolation should be used.

The effective damping $\zeta_{\text {eff }}$ and the effective period $T_{\text {eff }}$ are calculated as in the fundamental mode spectrum analysis.

- The design displacement, $\mathrm{d}_{\mathrm{d}, \mathrm{m},}$ and the shear force, $\mathrm{V}_{\mathrm{d}, \mathrm{m},}$ that are transferred through the isolation interface, calculated from the multi-mode spectrum analysis, are subject to lower bounds equal to $80 \%$ of the relevant effects $\mathrm{d}_{\mathrm{d}, \mathrm{f}}$ and $\mathrm{V}_{\mathrm{d}, \mathrm{f}}$ calculated in accordance with the fundamental mode spectrum analysis. In case that this condition is not met, the effects of the multimode spectrum analysis will be multiplied by $0,80 \cdot \mathrm{~d}_{\mathrm{d}, \mathrm{f}} / \mathrm{d}_{\mathrm{d}, \mathrm{m}}$ and $0,80 \cdot \mathrm{~V}_{\mathrm{d}, \mathrm{f}} / \mathrm{V}_{\mathrm{d}, \mathrm{m}}$ respectively. If the bridge cannot be approximated (even crudely) by a single degree of freedom model, the effects $d_{d, f}$ and $V_{d, f}$ can be obtained from the fundamental mode.


## Verifications

## Isolating system

- In order to meet the required increased reliability, the isolating system shall be designed for increased displacements:

$$
d_{d, i n}=Y_{I S} d_{d}
$$

where $\mathrm{Y}_{\text {IS }}=1,50$ is an amplification factor applicable only to the design displacements of the isolation system.

- All components of the isolating system shall be capable of functioning at the total maximum displacement:

$$
d_{\text {max }}=d_{d, \text { in }}+d_{G}+1 / 2 d_{T}
$$

where $d_{G}$ is the displacement due to the permanent and quasipermanent actions and $d_{\mathrm{T}}$ is the displacement due to thermal movements.

- No lift-off of isolators carrying vertical force is allowed under the design seismic combination.


## Verifications

## Substructures and superstructure

- Derive the internal seismic forces $\mathrm{E}_{\mathrm{E}, \mathrm{A}}$ from an analysis with the seismic action for $\mathrm{q}=1$.
- Calculate the design seismic forces $\mathrm{E}_{\mathrm{E}}$, due to seismic action alone, that correspond to limited ductile / essentially elastic behaviour, from the forces $E_{E, A}$ :

$$
\mathrm{E}_{\mathrm{E}}=\mathrm{E}_{\mathrm{E}, \mathrm{~A}} / \mathrm{q}
$$

with $\mathrm{q} \leq 1,5$.

- All structure members should be verified for:
- Forces $\mathrm{E}_{\mathrm{E}}$ in bending with axial force
- Forces $E_{E, A}$ in shear
- The foundation will be verified for forces $E_{E, A}$.

