Seismic design of bridges

Lecture 1

Ioannis N. Psycharlis
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

![Slab type diagram](image)
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
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

Example of a skew bridge
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 → small deck thickness

● Good seismic behavior:
  ♦ The deck acts as a diaphragm → 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

Advantages
- Flexible systems → type of seismic isolation

Disadvantages
- Large seismic displacements (danger of deck fall)
- Piers behave as cantilever → large moments at the base
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
Seismic stoppers

- Restrict the displacements in order to avoid deck fall
- Typical mechanisms:
  - Bumpers
  - Cables
  - Dowels - sockets 
    (τόρμος – εντορμία)
- 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 (κολωνοπάσσαλοι)**
  - 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 isolation of bridges”, Y.ΠΕ.ΧΩ.Δ.Ε.

Download from:
Seismic action

Elastic response spectrum ($S_e=$spectral acceleration)

\[
S_e(T) = a_g \cdot S \cdot \left[ 1 + \frac{T}{T_B} \cdot (\eta \cdot 2.5 - 1) \right] \quad \gamma \alpha \quad 0 \leq T \leq T_B
\]

\[
S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \quad \gamma \alpha \quad T_B \leq T \leq T_C
\]

\[
S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \frac{T_C}{T} \quad \gamma \alpha \quad T_C \leq T \leq T_D
\]

\[
S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \frac{T_C \cdot T_D}{T^2} \quad \gamma \alpha \quad T_D \leq T \leq 4 \text{ sec}
\]

where:
\[
a_g = \gamma_I \cdot a_{gR}
\]
\[
\eta = \sqrt{\frac{10}{\zeta + 5}} \quad \geq 0.55
\]

= damping coefficient ($\zeta$ in %)

\[
S = \text{soil factor}
\]
Seismic action

Ground acceleration

<table>
<thead>
<tr>
<th>Seismic Hazard Zone</th>
<th>$a_{gR}$ (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z1</td>
<td>0.16</td>
</tr>
<tr>
<td>Z2</td>
<td>0.24</td>
</tr>
<tr>
<td>Z3</td>
<td>0.36</td>
</tr>
</tbody>
</table>

Importance factor (E39/99)

<table>
<thead>
<tr>
<th>Bridge importance</th>
<th>$Y_I$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Less than average</td>
<td>0.85</td>
</tr>
<tr>
<td>Average</td>
<td>1.00</td>
</tr>
<tr>
<td>Greater than average</td>
<td>1.30</td>
</tr>
</tbody>
</table>

Soil factor and characteristic periods

<table>
<thead>
<tr>
<th>Ground type</th>
<th>$T_B$ (sec)</th>
<th>$T_C$ (sec)</th>
<th>$T_D$ (sec)</th>
<th>$S$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.15</td>
<td>0.40</td>
<td>2.50</td>
<td>1.00</td>
</tr>
<tr>
<td>B</td>
<td>0.15</td>
<td>0.50</td>
<td>2.50</td>
<td>1.20</td>
</tr>
<tr>
<td>C</td>
<td>0.20</td>
<td>0.60</td>
<td>2.50</td>
<td>1.15</td>
</tr>
<tr>
<td>D</td>
<td>0.20</td>
<td>0.80</td>
<td>2.50</td>
<td>1.35</td>
</tr>
<tr>
<td>E</td>
<td>0.15</td>
<td>0.50</td>
<td>2.50</td>
<td>1.40</td>
</tr>
</tbody>
</table>
Ground acceleration

\( a_{g,R} \) = 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, \( P_{\text{NCR}} \)).

\[
T_{\text{NCR}} = \frac{1}{1 - (1 - P_{\text{NCR}})^{1/t_d}}
\]

The values assigned to each seismic zone correspond to:

\( P_{\text{NCR}} = 10\%, \) i.e. \( T_{\text{NCR}} = 475 \) years.

- An importance factor \( \gamma_I = 1,0 \) is assigned to the reference return period \( T_{\text{NCR}} \).

- For return periods other than the reference, \( \gamma_I \neq 1,0 \) and the design ground acceleration on type A ground, \( a_g \), is equal to:

\[
a_g = \gamma_I \cdot a_{gR}
\]
# Ground types

<table>
<thead>
<tr>
<th>Ground type</th>
<th>Description of stratigraphic profile</th>
<th>( v_{s,30} ) (m/sec)</th>
<th>( N_{SPT} ) (bl./30 cm)</th>
<th>( c_u ) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.</td>
<td>&gt; 800</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>B</td>
<td>Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.</td>
<td>360 - 800</td>
<td>&gt; 50</td>
<td>&gt; 250</td>
</tr>
<tr>
<td>C</td>
<td>Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.</td>
<td>180 - 360</td>
<td>15 - 50</td>
<td>70 - 250</td>
</tr>
<tr>
<td>D</td>
<td>Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.</td>
<td>&lt; 180</td>
<td>&lt; 15</td>
<td>&lt; 70</td>
</tr>
<tr>
<td>E</td>
<td>A soil profile consisting of a surface alluvium layer with ( v_s ) values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with ( v_s &gt; 800 ) m/s.</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Design spectrum

Blue line: $S_e = $ elastic spectrum
For damping $\neq 5\%$, the correction factor $\eta$ must be used.

Red line: $S_d = $ design spectrum for elastic analysis with behaviour factor $q$
No correction for damping $\neq 5\%$
Seismic behaviour

![Graph showing seismic behaviour with coordinates FORCE vs DISPLACEMENT, depicting different levels of behaviour: Ideal elastic, Essentially elastic, Limited ductile, Ductile.](image-url)
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

<table>
<thead>
<tr>
<th>Type of Ductile Members</th>
<th>q</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Reinforced concrete piers:</strong></td>
<td></td>
</tr>
<tr>
<td>Vertical piers in bending ($a_s \geq 3,0$)</td>
<td>3,5 $\lambda(a_s)$</td>
</tr>
<tr>
<td>Inclined struts in bending</td>
<td>2,1 $\lambda(a_s)$</td>
</tr>
<tr>
<td><strong>Steel Piers:</strong></td>
<td></td>
</tr>
<tr>
<td>Vertical piers in bending</td>
<td>3,5</td>
</tr>
<tr>
<td>Inclined struts in bending</td>
<td>2,0</td>
</tr>
<tr>
<td>Piers with normal bracing</td>
<td>2,5</td>
</tr>
<tr>
<td>Piers with eccentric bracing</td>
<td>3,5</td>
</tr>
<tr>
<td><strong>Abutments rigidly connected to the deck:</strong></td>
<td></td>
</tr>
<tr>
<td>In general</td>
<td>1,5</td>
</tr>
<tr>
<td>“Locked-in” structures</td>
<td>1,0</td>
</tr>
<tr>
<td><strong>Arches</strong></td>
<td></td>
</tr>
</tbody>
</table>

**$a_s = L/h$** 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.

For $a_s \geq 3$ \[ \lambda(a_s) = 1,0 \]

For $3 > a_s \geq 1,0$ \[ \lambda(a_s) = (a_s/3)^{1/2} \]
**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 ($T \leq 0.03$ sec): $q = 1$

- Bridges rigidly connected to both abutments, which are laterally encased, at least over 80% of their area, in stiff natural soil formations with $T \geq 0.03$ sec: $q = 1.5$

- Vertical direction: $q = 1$