Overview of Seismic issues for bridge design (EN1998)

Basil Kolias
DENCO SA
Athens

Overview of the presentation

1. **Example of ductile piers**

   Special example of seismic design of a bridge with concrete deck rigidly connected to piers designed for ductile behaviour
2. **Example of limited ductile piers**

   Seismic design of the general example: Bridge on high piers designed for limited ductile behaviour
3. **Example of seismic isolation**

   Seismic design of the general example: Bridge on squat piers designed with seismic isolation

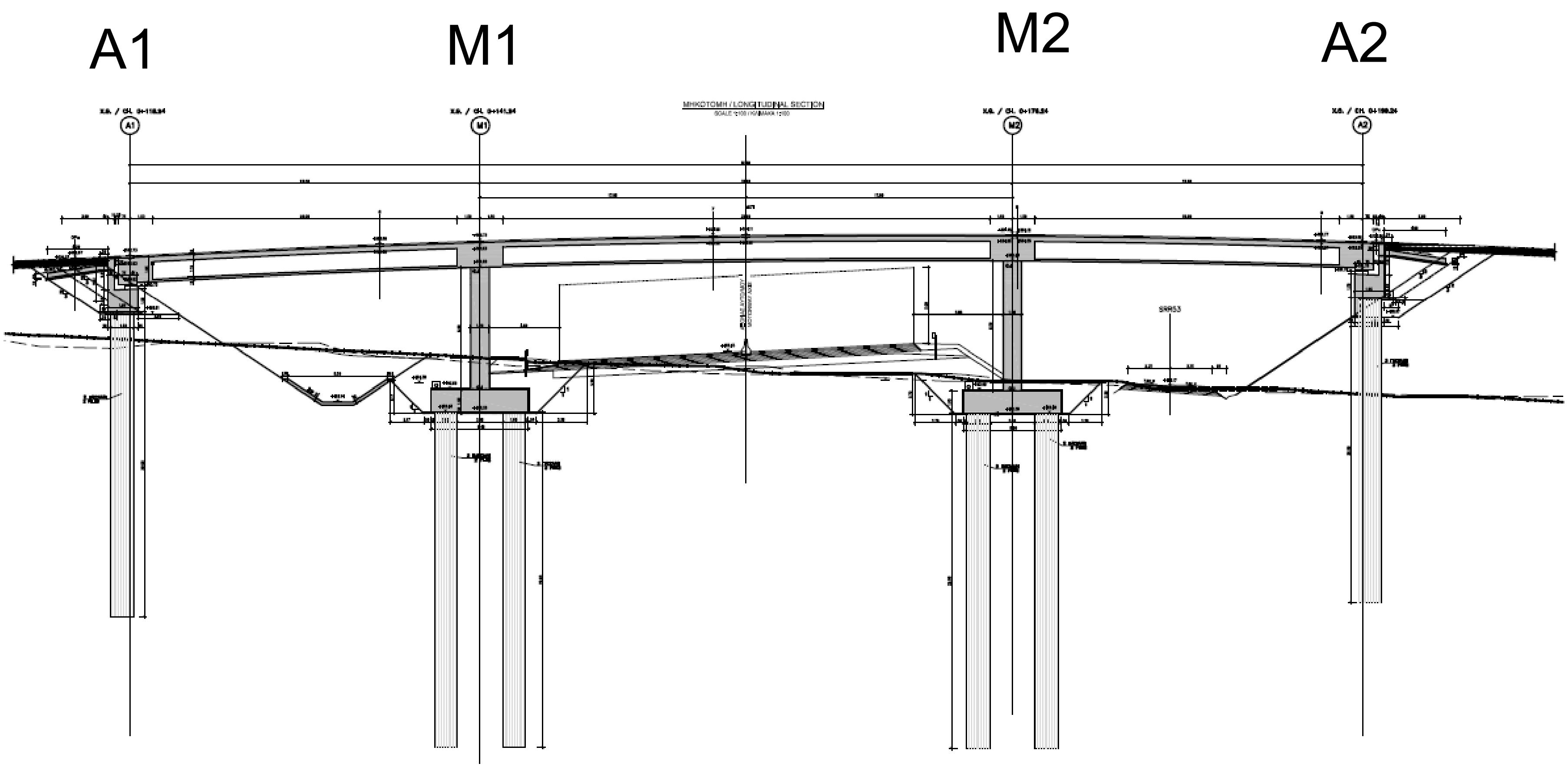
1 Example of ductile piers

Bridge description

3 span voided slab bridge (overpass), with spans 23.0m, 35.0m and 23.0m. Total length of 82.50m.
Piers: single cylindrical columns D=1.20m, rigidly connected to the deck. Pier heights 8.0m for M1 and 8.5m for M2.
Simply supported to abutments through a pair of sliding bearings.
Foundation of piers and abutments through piles.

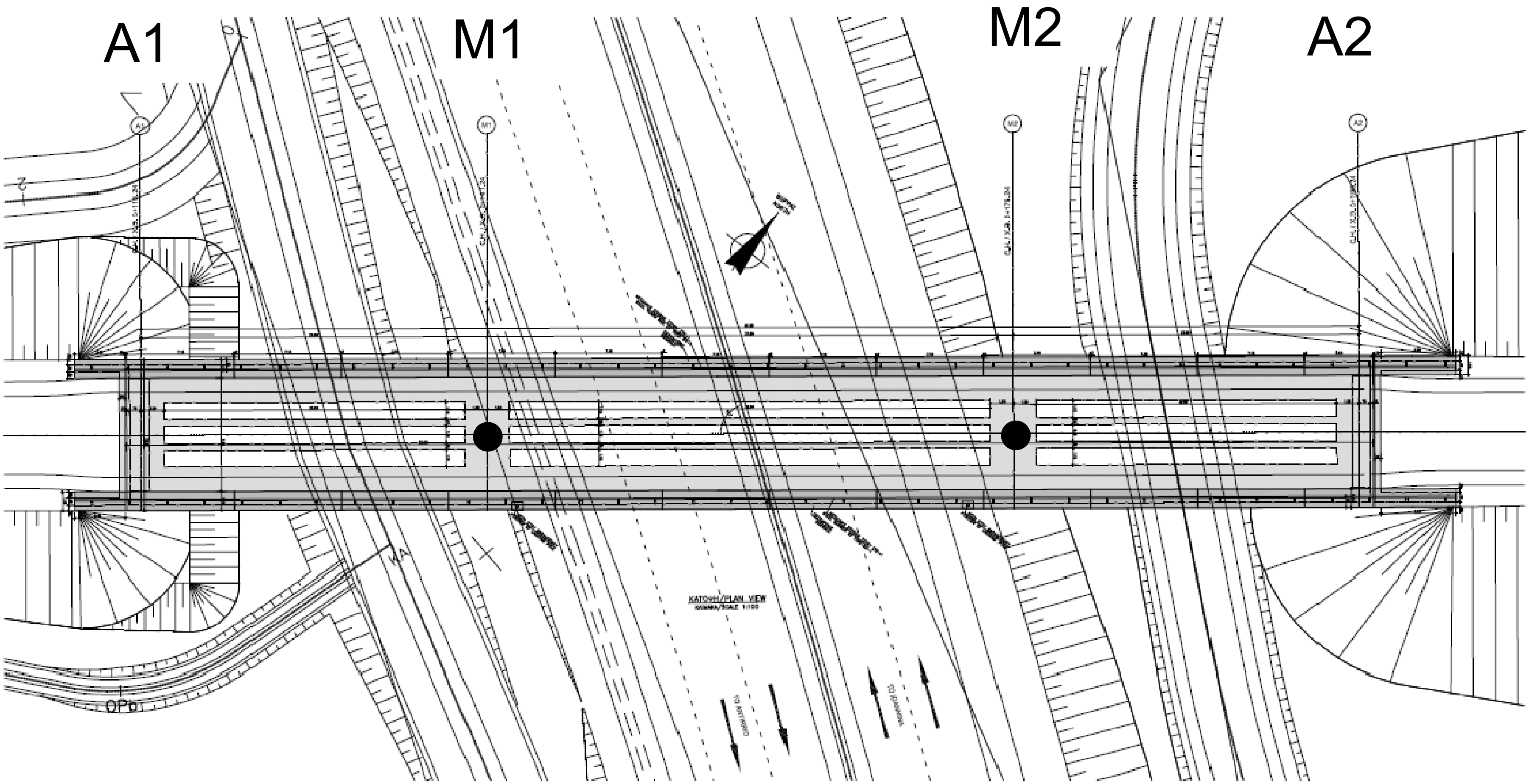
Example of ductile piers

Longitudinal section

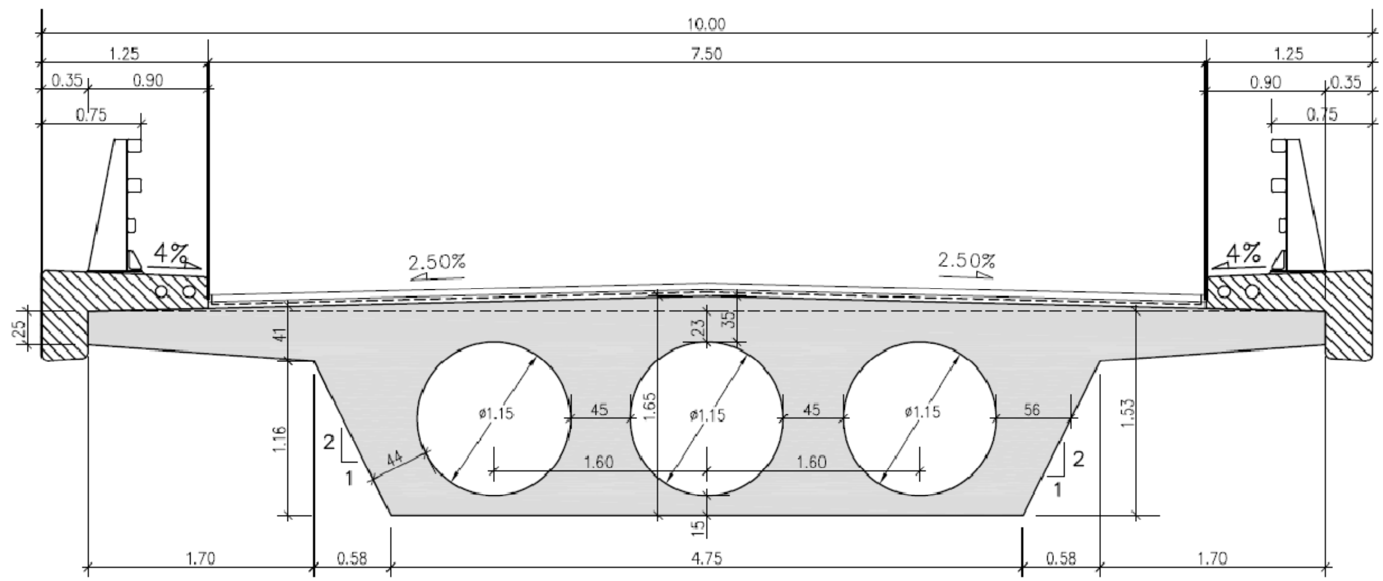


Example of ductile piers

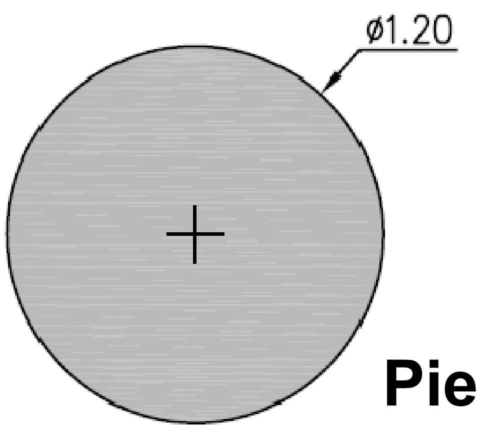
Plan view



Example of ductile piers



Cross section of deck d = 1.65m



Pier cross section D = 1.20m

Linear Analysis Methods

- **Equivalent Linear Analysis:**
  Elastic force analysis with forces from unlimited elastic response divided by the global behaviour factor = ***q***.
  design spectrum = elastic spectrum / q
  (except for very low periods $T < T_B$)

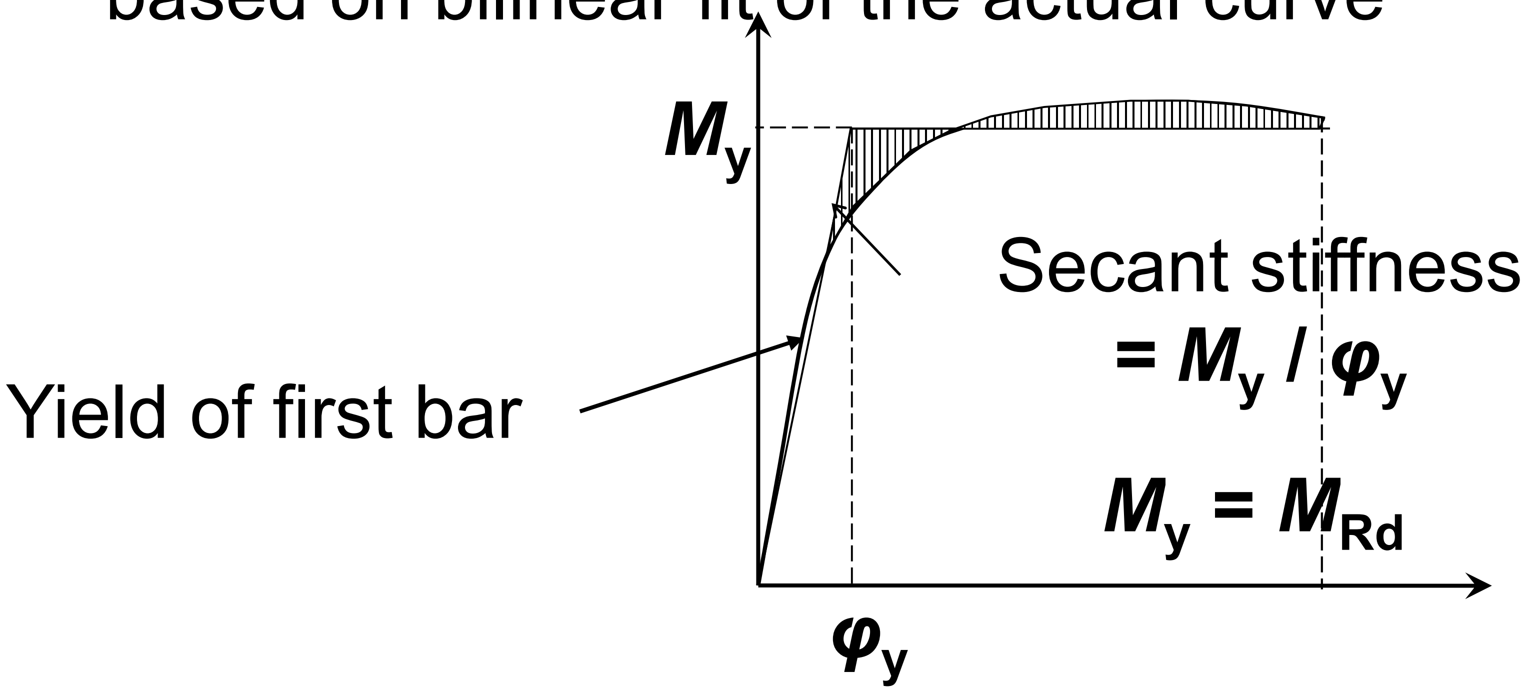
  - **Response spectrum analysis**:
    Multi-mode spectrum analysis
    Reference method

  - **Fundamental mode analysis**
    Equivalent static load analysis
    With several models of varying simplification

Linear Analysis Methods

Basic prerequisite:
Stiffness of Ductile Elements (piers):

secant stiffness at the theoretical yield,
based on bilinear fit of the actual curve



Guidance for estimation of stiffness in Annex C

Stiffness of concrete deck

Bending stiffness: uncracked stiffness both for prestresed and non-prestressed decks

Torsional stiffness:

- open sections or slabs: may be ignored
- prestressed box sections: 50% of uncracked
- reinforced box sections: 30% of uncracked

Linear Analysis Methods

<u>Ductility Classes</u>

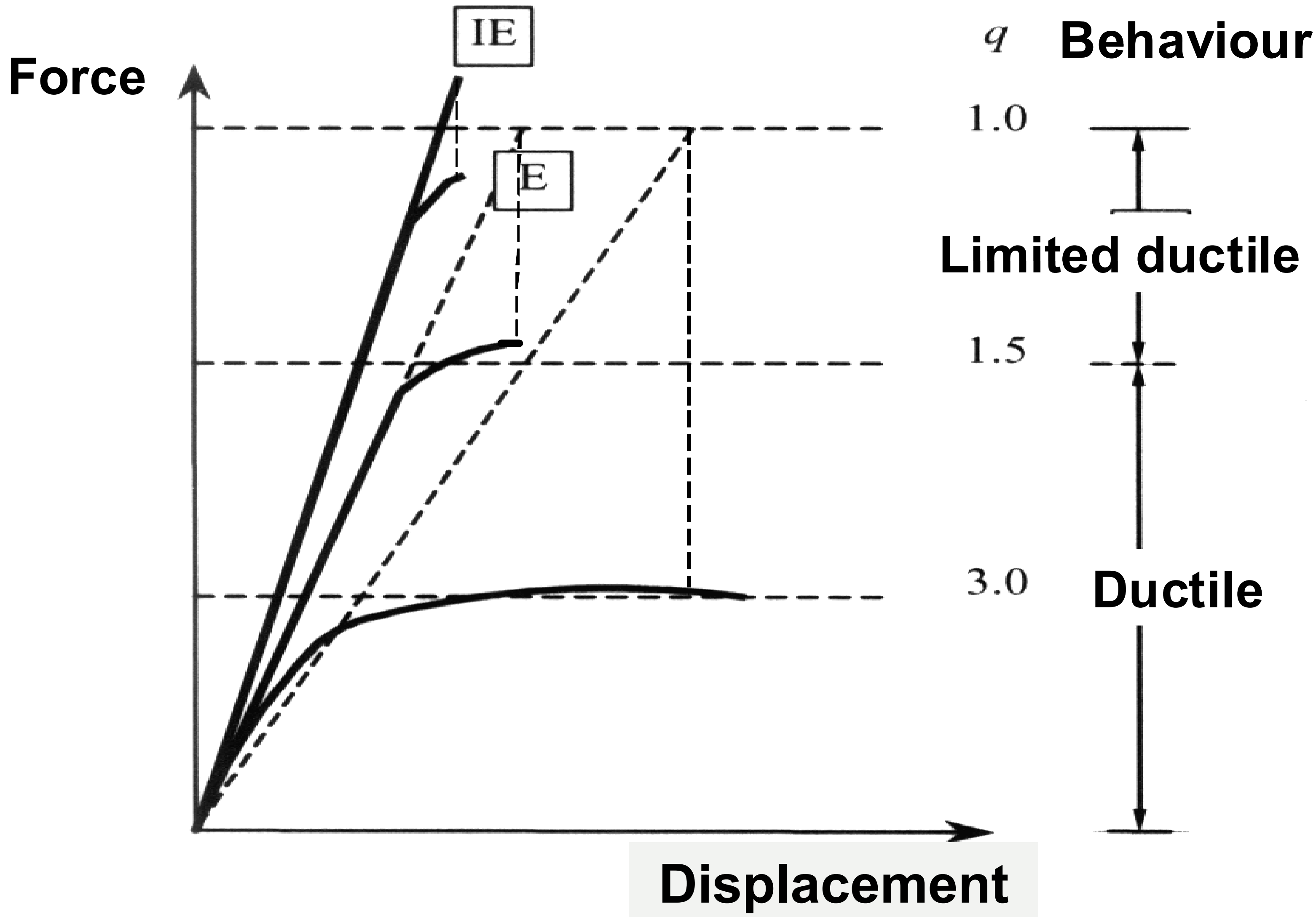
Limited Ductile Behaviour:

$\boldsymbol{q} \leq 1.50$ *(behaviour factor)*
$min\boldsymbol{\mu}_{\boldsymbol{\Phi}} = 7$ *(curvature ductility)*

Ductile Behaviour:

$1.50 < \boldsymbol{q} \leq 3.50$
$min\boldsymbol{\mu}_{\boldsymbol{\Phi}} = 13$

Ductility Classes



Behaviour factors

Type of Ductile Members	Seismic Behaviour	
	Limited Ductile	Ductile
Reinforced concrete piers:		
Vertical piers in bending	**1.5**	**3.5 $\lambda(\alpha_s)$**
Inclined struts in bending	**1.2**	**2.1 $\lambda(\alpha_s)$**
Steel Piers:		
Vertical piers in bending	**1.5**	**3.5**
Inclined struts in bending	**1.2**	**2.0**
Piers with normal bracing	**1.5**	**2.5**
Piers with eccentric bracing	**-**	**3.5**
Abutments rigidly connected to the deck:		
In general	**1.5**	**1.5**
Locked-in structures (see. 4.1.6(9). (10))	**1.0**	**1.0**
Arches	**1.2**	**2.0**

Shear ratio $\alpha_s = L_s/h$. For $\alpha_s \geq 3 \Rightarrow \lambda_s = 1$, for $1 \leq \alpha_s < 3 \Rightarrow \lambda_s = \sqrt{\alpha_s/3}$

Regular / Irregular bridges

- Criterion based on the variation of local required force reduction factors r_i of the ductile members i :

$$r_i = qM_{Ed,i} / M_{Rd,I} =$$

q x Seismic moment / Section resistance

- A bridge is considered regular when the "irregularity" index:

$$\rho_{ir} = max(r_i) / min(r_i) \leq \rho_0 = 2$$

- Piers contributing less than 20% of the average force per pier are not considered

Regular / Irregular bridges

- **For regular bridges:** equivalent elastic analysis is allowed with the ***q***-values specified, without checking of local ductility demands

- **Irregular bridges are:**
  - either designed with reduced behaviour factor:

$$q_r = q\, \rho_o / \rho_{ir} \geq 1.0$$

  - or verified by non-linear static (pushover) or dynamic (time history) analysis

Design Seismic Action to EN 1998

- **Two types of elastic response spectra:**
  - Types: 1 and 2.
- **5 types of soil:**
  - A, B, C, D, E. (+ S1,S2)
- **4 period ranges:**

  short < T_B < constant acceleration < T_C < const. velocity < T_D < const. displacement.
- **Design spectrum = elastic spectrum / *q*.**
- **3 importance classes:**

Class	III	II	I
γ_I =	1.30	1.00	0.85.

  Reflect reliability differentiation

Seismic action: Elastic response spectra $S_e(T)$

Ground motion: Dependence on ground type (A, B, C, D, E)

max. values: $A_g = a_g S$, $v_g = a_g S T_C/(2\pi)$, $d_g = 0.025\, a_g S T_C T_D$

Response spectrum: Acceleration $S_e(T)$ as a function of T

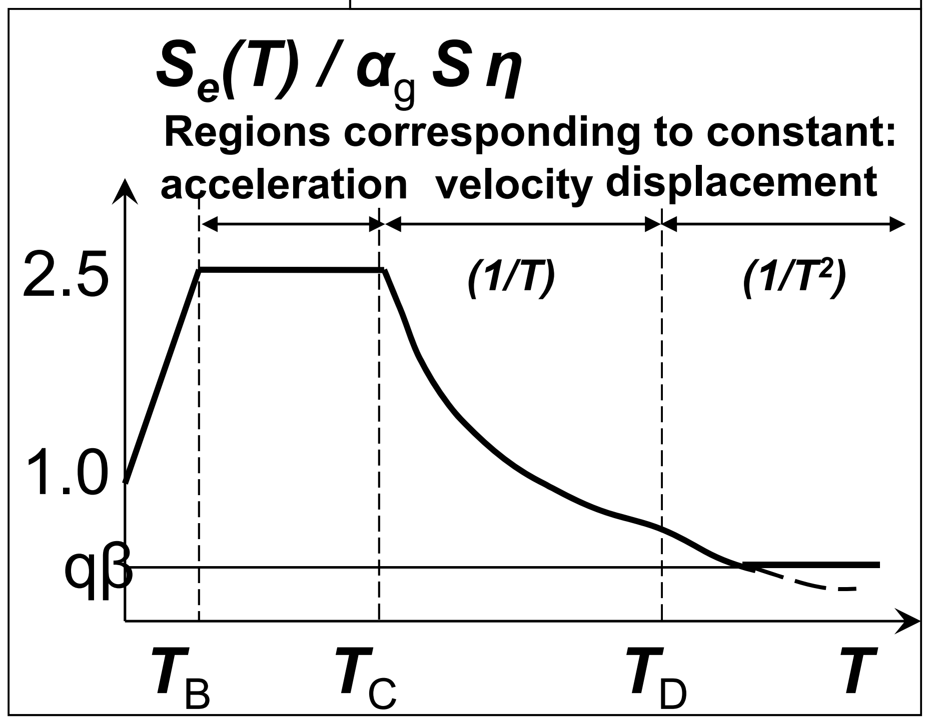
a_g: P.G.A.
S: soil factor
$\eta(\xi) = 1$, $\xi = 0.05$

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

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

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

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



Seismic action

- **Response spectrum type 1**
- **Ground type C: Characteristic periods T_B=0.20s, T_C=0.60s and T_D=2.50s. Soil factor S=1.15.**

- **Seismic zone Z1: Reference peak ground acceleration a_{gR} = 0.16g**
- **Importance factor γ_I = 1.0**
- **Lower bound factor β = 0.20**

- **Seismic actions in horizontal directions $a_g = \gamma_I \cdot a_{gR} = 1.0 \cdot 0.16g = 0.16g$**
- **Behaviour factors (§4.1.6(3) of EN 1998-2) :**
  q_x=3.5 (α_s=3.3>3, $\lambda(\alpha_s)$=1.0) and
  q_y=3.5 (α_s=6.7>3, $\lambda(\alpha_s)$=1.0)

Methods of analysis

- **Multi mode spectrum response analysis**
  - Combination of modes with CQC rule.
  - Combination of responses in 3 directions through the rule specified in Eq. (4.20)-(4.22) of EN1998-1 (1 "+" 0.3 "+" 0.3 – rule).

  30 modes considered, Σ(modal masses)>90%

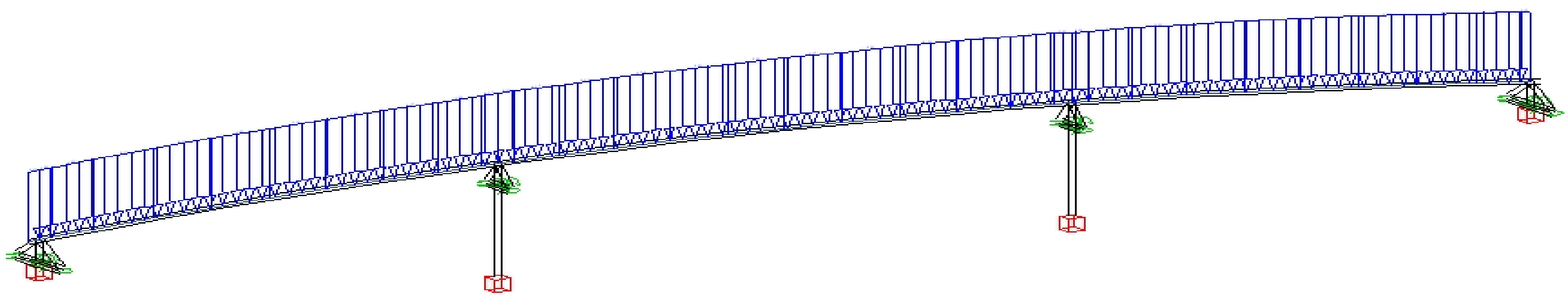
  Program: SOPHISTIK

- **Fundamental mode method**

  In the longitudinal direction

  (hand calculation for comparison/check)

Deck weights and other actions



1. **Self weight (G):** $(6.89m^2 \cdot 73.5m + 9.97m^2 \cdot 9.0m) \cdot 25kN/m3 =$ 14903kN
2. **Additional dead (G_2):** $q_{G2} = 43.65kN/m =$

$$\overset{\text{sidewalks}}{2 \cdot 25kN/m^3 \cdot 0.50m^2} + \overset{\text{safety barriers}}{2 \cdot 0.70kN/m} + \overset{\text{road pavement}}{7.5m \cdot (23kN/m^3 \cdot 0.10m)}$$

3. **Effective seismic live load (L_E):** (20% of uniformly distributed traffic load) $q_{LE} = 0.2 \times 45.2\ kN/m = 9.04kN/m$
4. **Temperature action (T)*:** $+52.5^oC$ / -45^oC
5. **Creep & Shrinkage (CS)*:** Total strain: -32.0×10^{-5}

* Actions 4, 5 applicable only for bearing displacements

Deck seismic weight:

$W_E = 14903kN + (43.65 + 9.04)kN/m \times 82.5m = 19250kN$

Fundamental mode method

- **Horizontal Stiffness in longitudinal direction**

  For cylindrical column of diameter 1.2m, $J_{un} = \pi \cdot 1.2^4/64 = 0.1018\text{m}^4$

  Assumed effective stiffness of piers J_{eff}/J_{un} = 0.40 (to be checked later).

  Concrete grade C30/37 with E_{cm} = 33GPa

  Assuming both ends of the piers fixed, the horizontal stiffness of each pier in longitudinal direction is:

  - $K_1 = 12EJ_{eff}/H^3 = 12 \cdot 33000\text{MPa} \cdot (0.40 \cdot 0.1018\text{m}^4)/(8.0\text{m})^3 = 31.5\text{MN/m}$
  - $K_2 = 12EJ_{eff}/H^3 = 12 \cdot 33000\text{MPa} \cdot (0.40 \cdot 0.1018\text{m}^4)/(8.5\text{m})^3 = 26.3\text{MN/m}$
  - Total horizontal stiffness: K = 31.5 + 26.3 = 57.8MN/m

- **Total seismic weight:** W_E = 19250kN (see loads)
- **Fundamental period:**

$$T = 2\pi\sqrt{\frac{m}{K}} = 2\pi\sqrt{\frac{(19250\text{kN}/9.81\text{m/s}^2)}{57800\text{kN/m}}} = 1.16\text{s}$$

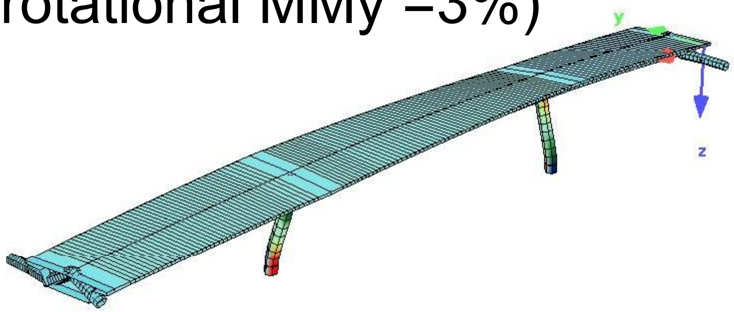
Fundamental mode method

- Spectral acceleration in longitudinal direction:
  $S_e = a_g S(\beta_0/q)(T_C/T) = 0.16g \cdot 1.15 \cdot (2.5/3.5)(0.60/1.16) = 0.068g$
- Total seismic shear force in piers:
  $V_E = S_e W_E/g = 0.068g \cdot 19250kN/g = 1309kN$

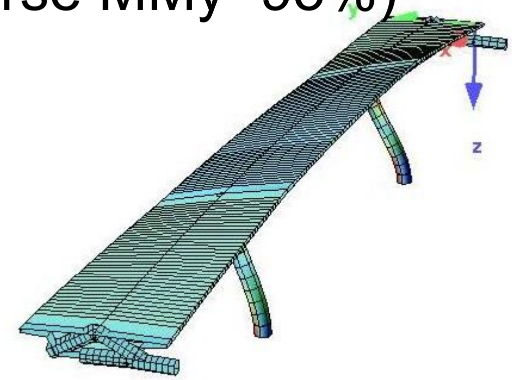
  Distribution to piers M1 and M2 proportional to their stiffness:
  $V_1 = (31.5/57.8) \cdot 1309kN = 713kN$
  $V_2 = 1309 – 713 = 596kN$
- Seismic moments My):
  (full fixity of pier columns assumed at top and bottom)
  $M_{y1} \approx V_1 \cdot H_1/2 = 713kN \cdot 8.0m/2 = 2852kNm$
  $M_{y2} \approx V_2 \cdot H_2/2 = 596kN \cdot 8.5m/2 = 2533kNm$

Multimode response spectrum analysis

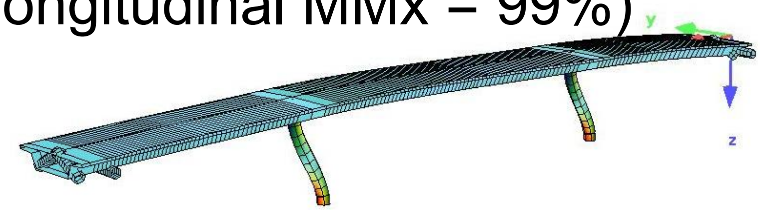
1st mode (T=1.75s)
(rotational MMy =3%)



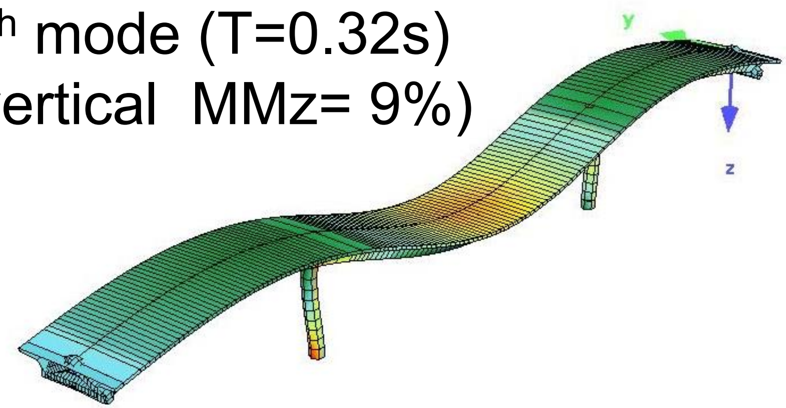
2nd mode (T=1.43s)
(transverse MMy=95%)



3rd mode (T=1.20s)
(longitudinal MMx = 99%)



4th mode (T=0.32s)
(vertical MMz= 9%)



Comparison in longidudinal direction

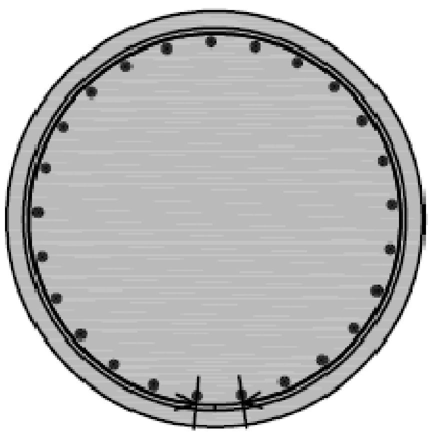
		Fundamental mode analysis	Multimode response spectrum analysis
Effective period T_{eff} for longitudinal direction		1.16s	1.20s (3rd mode)
Seismic shear, V_z	M1 M2	713kN 596kN	662kN 556kN
Seismic moment, M_y	M1 M2	2852kNm 2533kNm	2605...2672kNm 2327...2381kNm (values at top and bottom)

Required Longitudinal Reinforcement in Piers

Pier concrete C30/37, f_{ck}=30MPa, E_c=33000MPa
Reinforcing steel S500, f_{yk}=500MPa
Diameter D=1.20m
Cover to center of reinforcement c=8.2cm

Required reinforcement in bottom section of Pier M1

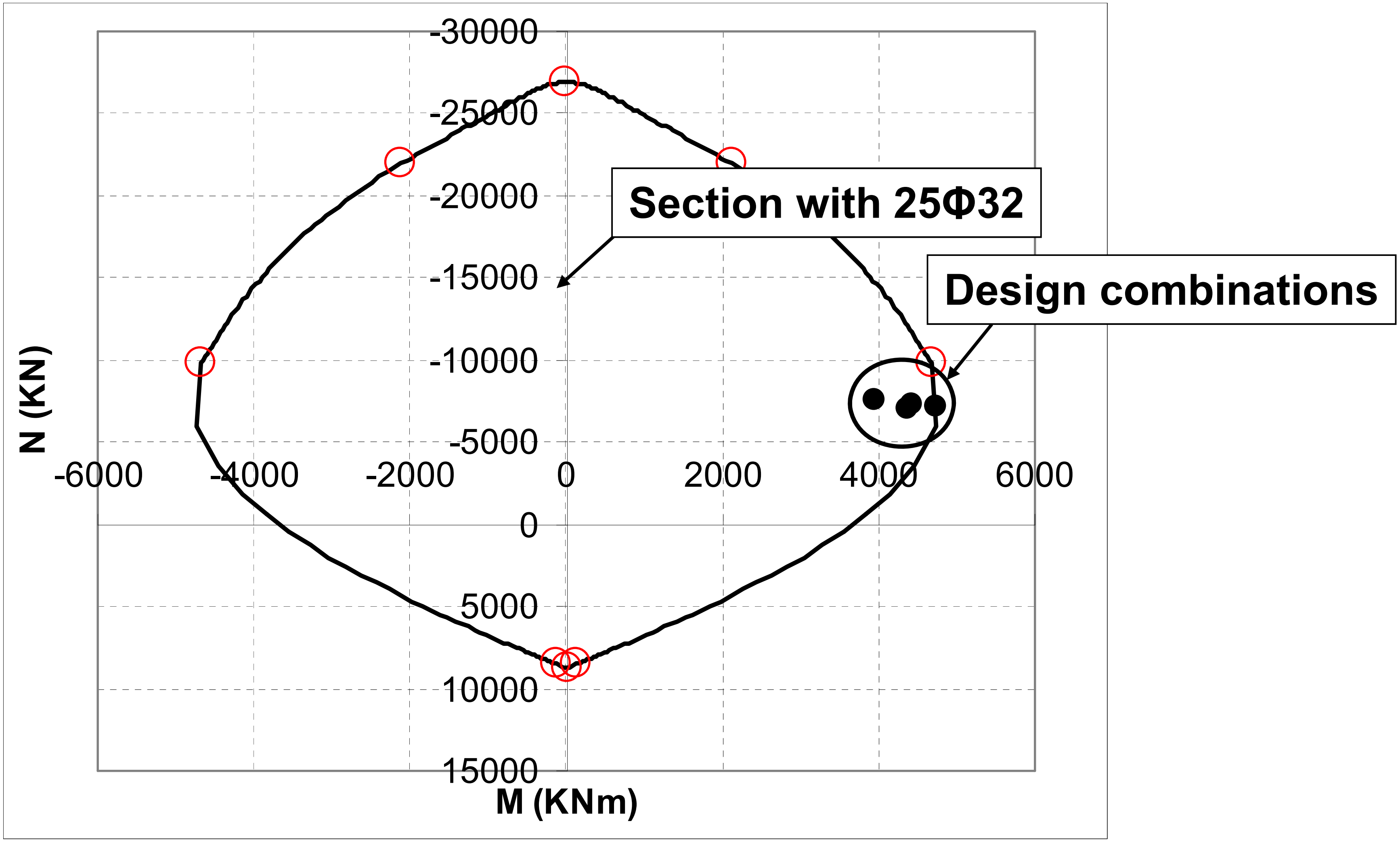
Combination	N	M_y	M_z	A_s
	kN	kNm	kNm	cm^2
maxM_y + M_z	-7159	4576	-1270	198.7
minM_y + M_z	-7500	-3720	1296	134.9
maxM_z + M_y	-7238	713	4355	172.4
minM_z + M_y	-7082	456	-4355	170.0



Final reinforcement: 25Φ32 (201.0cm^2)
M_{RD} = 4779kNm

Verification of Longitudinal Reinforcement in Piers

Moment – Axial force interaction diagram for the bottom section of Pier M1

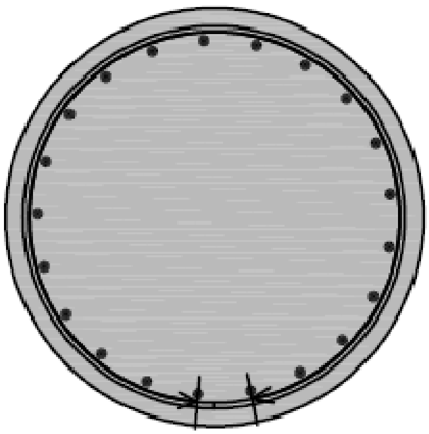


Required Longitudinal Reinforcement in Piers

Pier concrete C30/37, f_{ck}=30MPa, E_c=33000MPa
Reinforcing steel S500, f_{yk}=500MPa
Diameter D=1.20m
Cover to center of reinforcement c=8.2cm

Required reinforcement in bottom section of Pier M2

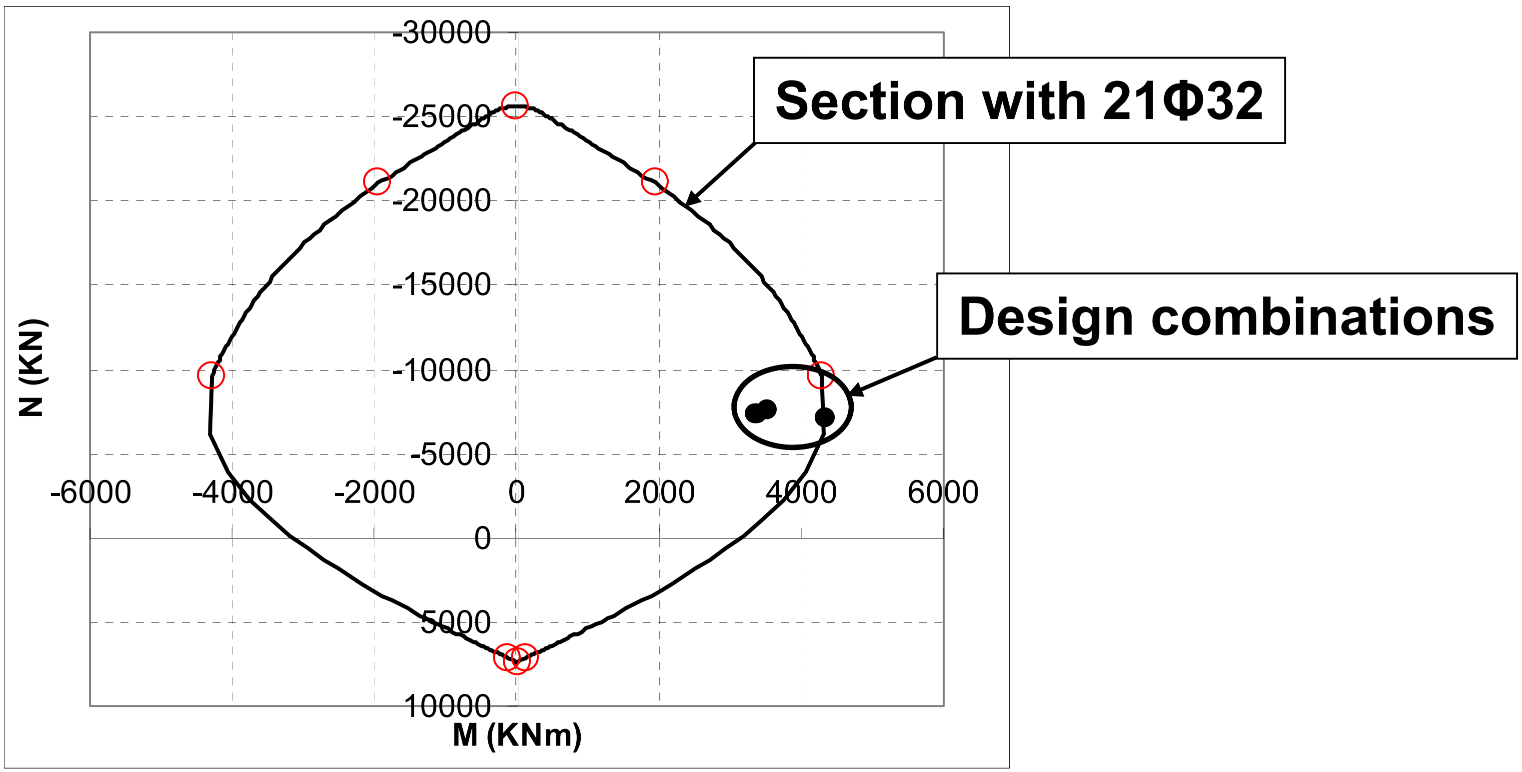
Combination	N	M_y	M_z	A_s
	kN	kNm	kNm	cm²
maxM_y + M_z	-7528	3370	-1072	103.2
minM_y + M_z	-7145	-4227	1042	168.0
maxM_z + M_y	-7317	-465	3324	89.8
minM_z + M_y	-7320	-674	-3324	92.5



Final reinforcement: 21Φ32 (168.8cm²)
M_{RD} = 4366kNm

Verification of Longitudinal Reinforcement in Piers

Moment – Axial force interaction diagram for the bottom section of Pier M2



Verification of sections

- **Ductile Behaviour:**
  - Flexural resistance of plastic hinge regions with design seismic effects $\boldsymbol{A_{Ed}}$ (only in piers).

$$A_{Ed} \leq A_{Rd}$$

  - All other regions and non-ductile failure modes (shear of elements & joints and soil) are checked with capacity design effects $\boldsymbol{A_{Cd}}$.
  - For non-ductile failure modes:

$$A_{Cd} \leq A_{Rd}/\gamma_{Bd}$$

$$1.0 \leq \gamma_{Bd} \leq 1.25 \text{ (depending on } A_{Cd}\text{)}$$

  - Local ductility ($\boldsymbol{\mu_\Phi}$) ensured by special detailing rules (confinement, restraining of compressed bars etc), i.e. without direct assessment of $\boldsymbol{\mu_\Phi}$.

Capacity Design Effects

- $\boldsymbol{A_{Cd}}$ correspond to the section forces under permanent loads and a seismic action creating the assumed pattern of plastic hinges, where the flexural over-strength:

$$\boldsymbol{M_o = \gamma_o M_{Rd}}$$

has developed with: $\boldsymbol{\gamma_o}$ = 1.35 and $\boldsymbol{\gamma_{Bd}}$ = 1.0

However $\boldsymbol{A_{Cd} \leq qA_{Ed}}$

when $\boldsymbol{A_{Cd} = qA_{Ed}}$ then $1.0 \leq \boldsymbol{\gamma_{Bd}} \leq 1.25$

- Simplifications for $\boldsymbol{A_{Cd}}$ satisfying the equilibrium conditions are allowed.
- Guidance is given in normative Annex G

Shear verification of piers (Capacity effects)

Over strength moment $\boldsymbol{M_o = \gamma_o \cdot M_{Rd}}$

Over strength factor $\boldsymbol{\gamma_o}$ = 1.35 is increased due to $\boldsymbol{\eta_k}$ = 0.22 > 0.1 according to (4) of 5.2.3 to:

$\boldsymbol{\gamma_o} = 1.35 \cdot [1+2 \cdot (0.22-0.1)^2] = 1.35 \cdot 1.029 = 1.39$,

Over strength moments:

$\boldsymbol{M_{o1}} = 1.39 \cdot 4779 = 6643\text{kNm}$

$\boldsymbol{M_{o2}} = 1.39 \cdot 4366 = 6069\text{kNm}$

Longitudinal direction (seismic actions in negative direction)

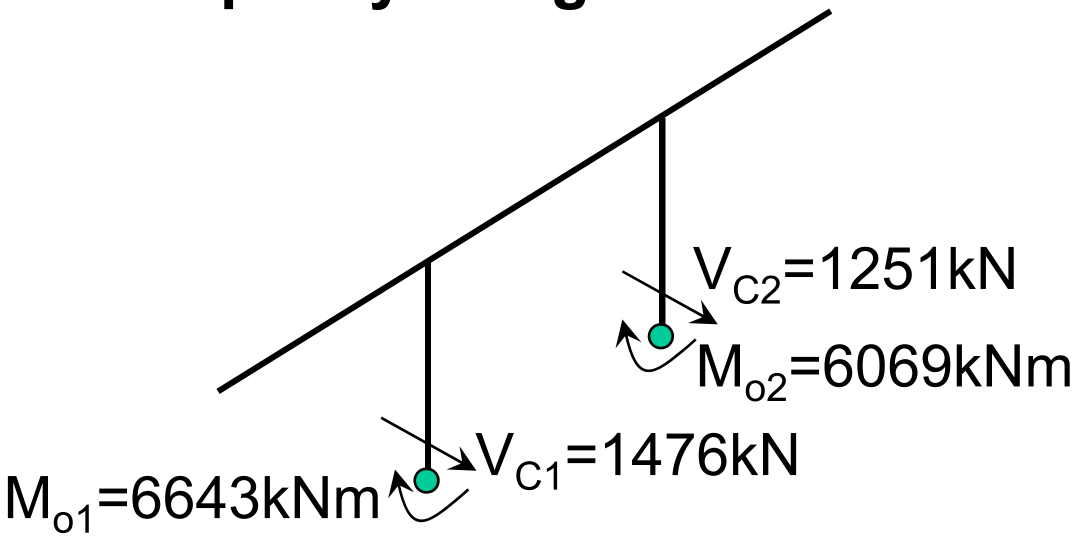
Capacity shear forces (over strength moments in both ends):

$\boldsymbol{V_{C1} = 2M_{o1}/H_1} = 2\text{x}6643/8.0 = 1661\text{kN}$

$\boldsymbol{V_{C2} = 2M_{o2}/H_2} = 2\text{x}6069/8.5 = 1428\text{kN}$

Shear verification of piers (Capacity effects)

Capacity design in transverse direction



M_{E1}=3061kNm
V_{E1}=680.3kN

M_{E2}=2184kNm
V_{E2}=450.2kN

The base shear on each pier is calculated by $\boldsymbol{V_{Ci}=(M_o/M_{Ei}) \cdot V_{Ei}}$ according to simplifications of Annex G of EN 1998-2.

Design for shear

Design shear force $\boldsymbol{V_{C1}}$ = 1661kN and $\boldsymbol{V_{C2}}$ = 1428kN. $\boldsymbol{\gamma_{Bd}}$ = 1.0

For circular section effective depth: $\boldsymbol{d_e}$ = 0.60 + 2·0.52/π = 0.93m,

Internal lever arm: $\boldsymbol{z}$ = 0.90 · $\boldsymbol{d_e}$ = 0.75 · 0.93m = 0.84m

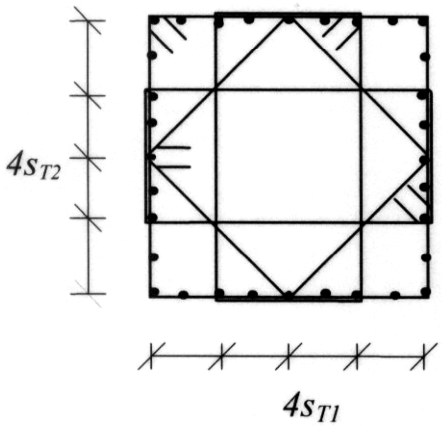
$\boldsymbol{V_{Rd,s} = (A_{sw}/s) \cdot z \cdot f_{ywd} \cdot \cot\theta / \gamma_{Bd}}$, **$\boldsymbol{\cot\theta}$ = 1** ➔

Pier M1: $\boldsymbol{A_{sw}/s}$ = 1661kN / (0.84m · 50kN/cm²/1.15) = <u>**45.5cm²/m**</u>

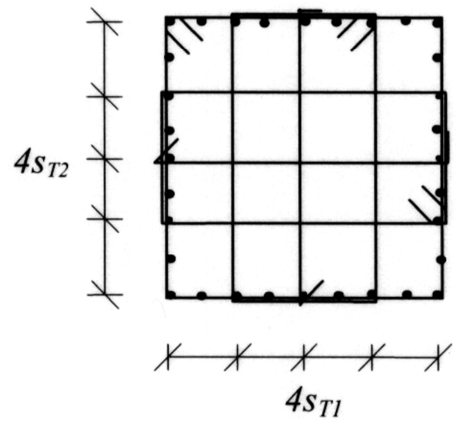
Pier M2: $\boldsymbol{A_{sw}/s}$ = 1428kN / (0.84m · 50kN/cm²/1.15) = <u>**39.1cm²/m**</u>

Confinement reinforcement

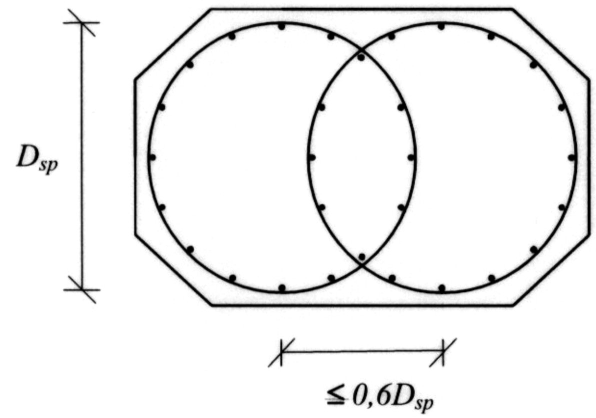
Typical arrangements



Closed hoops

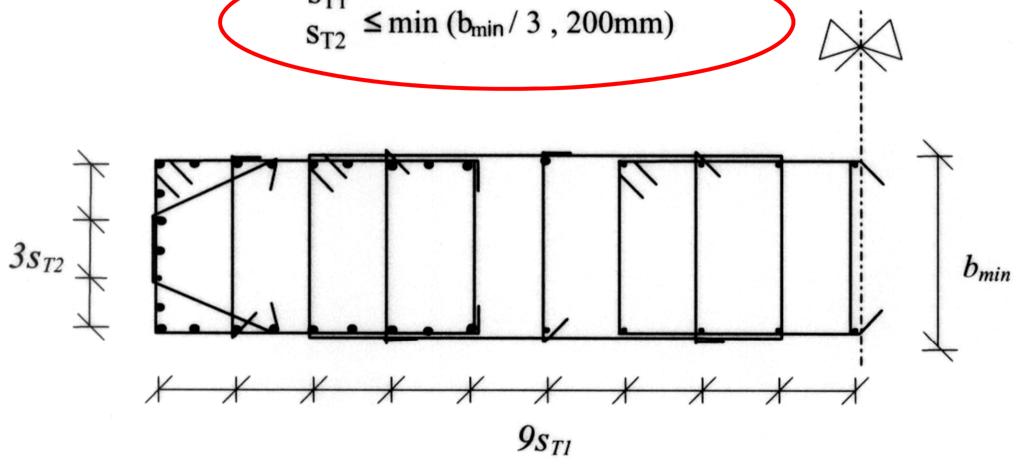


Hoops + crossties



Overlapping hoops

s_{T1}, s_{T2} $\leq \min(b_{min}/3, 200\text{mm})$



Mechanical confinement ratio ω_{wd}

$$\omega_{wd} = \rho_w f_{sd}/f_{cd}$$

Geometric reinforcement ratio ρ_w

	Rectangular	Circular
$\rho_w =$	$A_{sw}/(s_L b)$	$4A_{sp}/(s_L D_w)$

Requirement

$$\omega_{w,req} = \frac{A_c}{A_{cc}} \lambda \eta_k + 0{,}13 \frac{f_{yd}}{f_{cd}} (\rho_L - 0{,}01)$$

$$\omega_{wd,r} \geq \max\left(\omega_{w,req};\ \frac{2}{3}\omega_{w,min} \right) \qquad \omega_{wd.c} \geq \max\left(1{,}4\omega_{w,req};\ \omega_{w,min}\right)$$

Seismic Behaviour	λ	$\omega_{w,min}$
Ductile	0,37	0,18
Limited ductile	0,28	0,12

Special detailing rules: Confinement reinforcement

Confinement reinforcement according to §6.2.1 of EN 1998-2

Normalized axial force:
$\eta_k = N_{Ed}/A_c \cdot f_{ck}$ = 7600kN / 1.13m$^2 \cdot$ 30MPa = 0.22 > 0.08 →
Confinement of compression zone is required

For ductile behaviour: λ = 0.37 and $\omega_{w,min}$ = 0.18
Longitudinal reinforcement ratio:
- Pier M1: ρ_L=201.0cm^2/11300cm^2=0.0178
- Pier M2: ρ_L=168.8cm^2/11300cm^2=0.0149

Distance to spiral centerline c=5.8cm (D_{sp}=1.084m) →
A_{cc}=0.923m^2
Required mechanical reinforcement ratio $\omega_{w,req}$:
- Pier M1: $\omega_{w,req} = (A_c/A_{cc}) \cdot \lambda \cdot \eta_k + 0.13 \cdot (f_{yd}/f_{cd})(\rho_L - 0.01)$ =
$(1.13/0.923) \cdot 0.37 \cdot 0.22 + 0.13 \cdot (500/1.15)/(0.85 \cdot 30/1.5) \cdot (0.0178-0.01)$ = 0.126
- Pier M2: $\omega_{w,req}$ = 0.116

Confinement Reinforcement

For circular spirals

Mechanical reinforcement ratio (for worst case: pier M1)
$\omega_{wd,c} = \max(1.4 \cdot \omega_{w,req};\ \omega_{w,min}) = \max(1.4 \cdot 0.126;\ 0.18) = 0.18$

Required volumetric ratio of confining reinforcement
$\rho_w = \omega_{wd,c} \cdot (f_{cd}/f_{yd}) = 0.18 \cdot (0.85 \cdot 30/1.5)/(500/1.15) = 0.0070$

Required confining reinforcement
$A_{sp}/s_L = \rho_w \cdot D_{sp}/4 = 0.0070 \cdot 1.084\text{m}/4 = 0.00190\text{m}^2/\text{m} =$ $\underline{19.0\text{cm}^2/\text{m}}$

Req. spacing for Φ16 spirals $s_L^{req} = 2.01/19.0 = \underline{0.106\text{m}}$

Allowed max. spacing
$s_L^{allowed} = \min(6 \cdot 3.2\text{cm};\ 108.4\text{cm}/5) = \min(19.2\text{cm};\ 21.7\text{cm}) =$ $19.2\text{cm} > 10.6\text{cm}$

Buckling of longitudinal bars

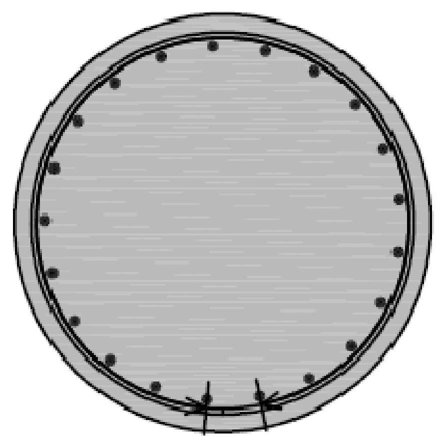
Avoidance of buckling of longitudinal bars according to §6.2.2 of EN 1998-2.

For steel S500 the ratio $f_{tk}/f_{yk} = 1.15$

$$\delta = 2.5 \cdot (f_{tk}/f_{yk}) + 2.25 = 2.5 \cdot 1.15 + 2.25 = 5.125$$

Maximum required spacing of spirals

$$s_L^{req} = \delta\, d_L = 5.125 \cdot 3.2\text{cm} = \underline{16.4\text{cm}}$$



Transverse reinforcement of piers

Comparison of requirements for Φ16 spiral

Requirement	Confinement	Buckling of bars	Shear design
A_t/s_L (cm^2/m)	2x19.0=38	-	M1: 45.5 M2: 39.1
maxs_L (cm)	19.2	16.4	8.5

The transverse reinforcement is governed by the shear design.

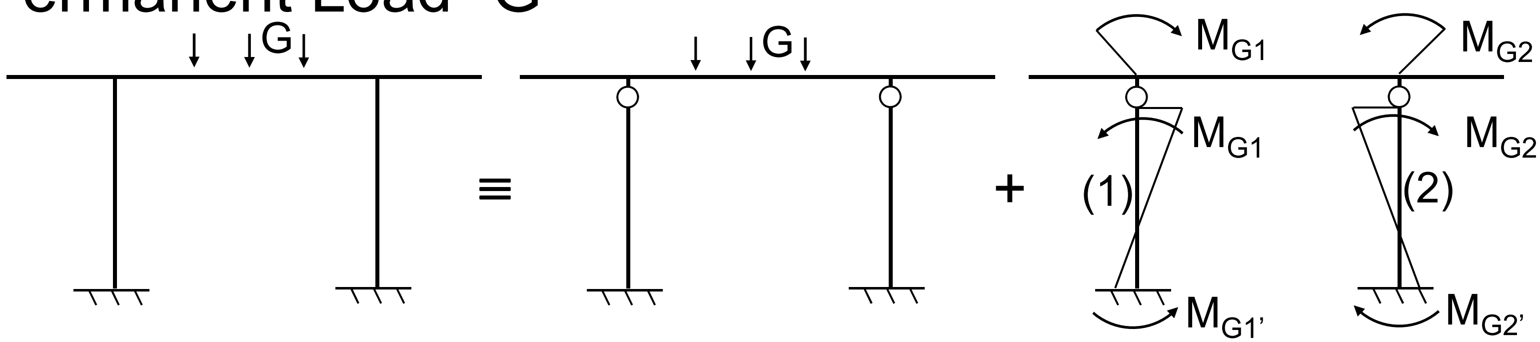
Reinforcement selected for both piers is one spiral of Φ16/8.5 (47.3cm^2/m)

Calculation of capacity effects

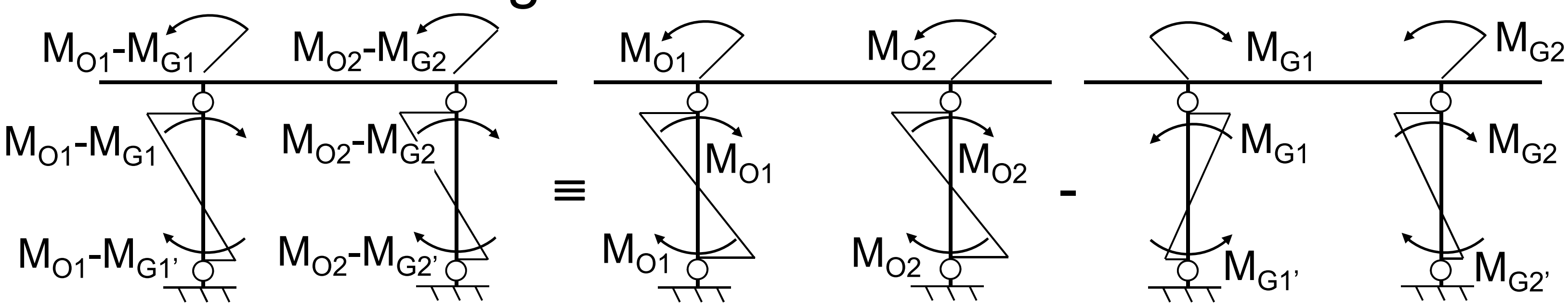
General procedure: combination “G” + “Mo-G” (acc. to §G1 of Annex G of EN 1998-2)

Alternative: “G” + “Mo” on continuous deck articulated to piers

Permanent Load “G”



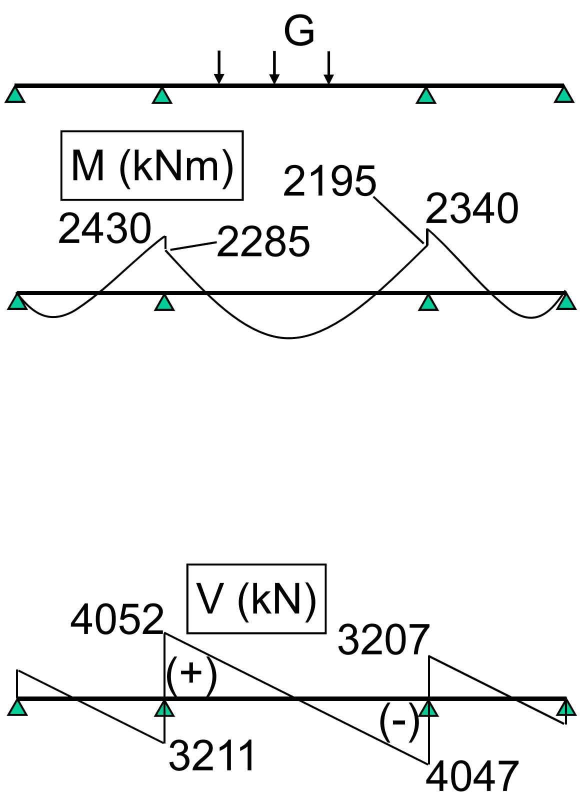
ΔAc: Over strength – “G”



general procedure ≡ alternative procedure

Calculation of capacity effects

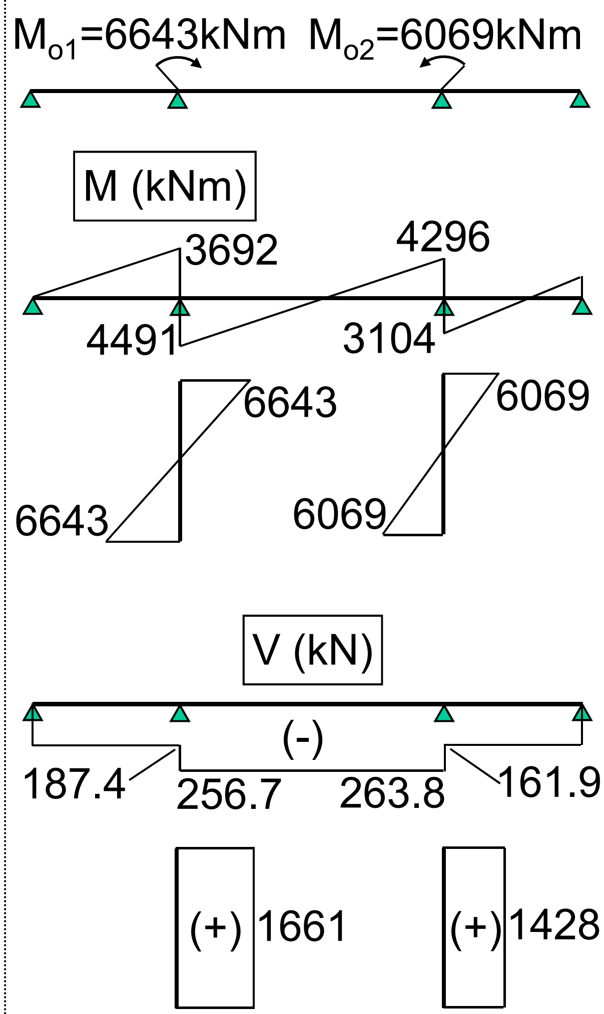
"G" loading



alternative procedure is applied

Overstrength "M$_o$"

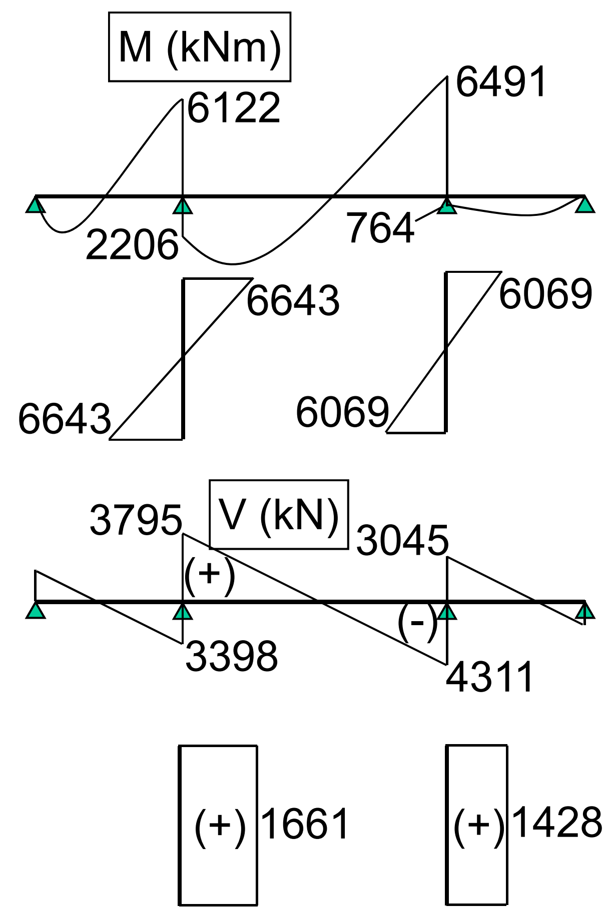
(+x direction)



Reversed signs for –x direction

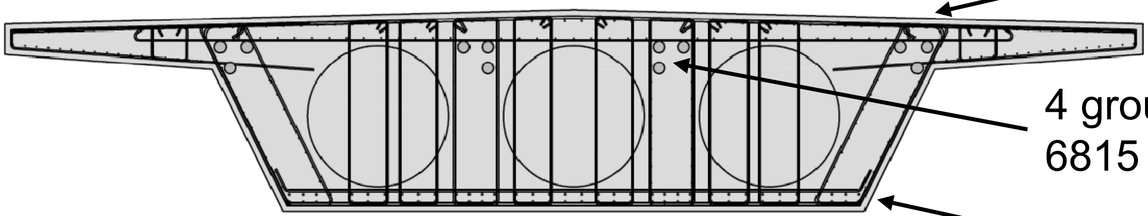
Capacity effects "G" + "M$_o$"

(+x direction)



Verification of deck section for capacity effects

Deck section, reinforcement and tendons



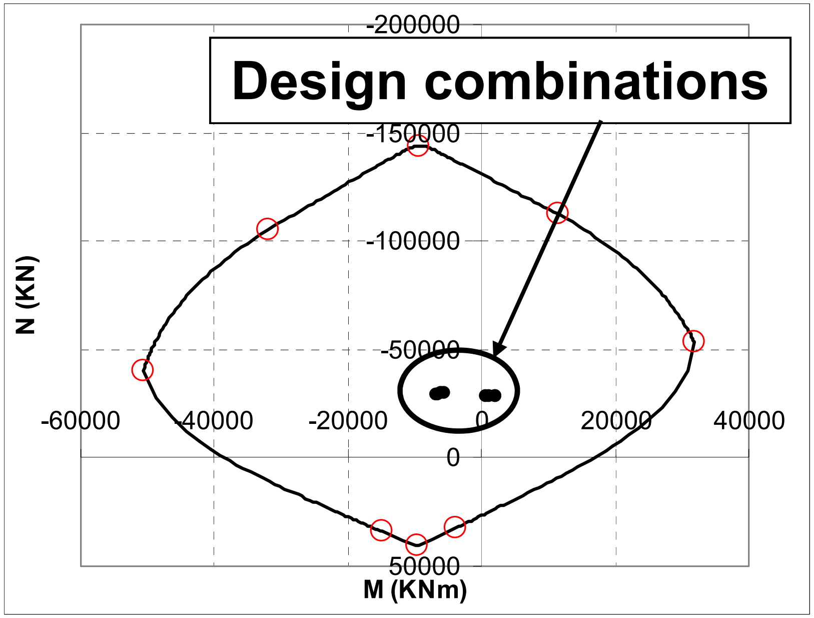
Top layer: 46Φ20 + 33Φ16 (210.8cm^2)

4 groups of 3 tendons of type DYWIDAG 6815 (area of 2250mm^2 each)

Bottom layer: 2x33Φ16 (182.9cm^2)

Combination / location	M_y (kNm)	N (kN)
Pier M1 – left side (+x)	**-6122**	**-29900**
Pier M1 – right side (+x)	**2206**	**-28300**
Pier M2 – left side (+x)	**-6491**	**-29500**
Pier M2 – right side (+x)	**764**	**-28100**
Pier M1 – left side (-x)	**1262**	**-28100**
Pier M1 – right side (-x)	**-6776**	**-29500**
Pier M2 – left side (-x)	**2101**	**-28300**
Pier M2 – right side (-x)	**-5444**	**-29900**

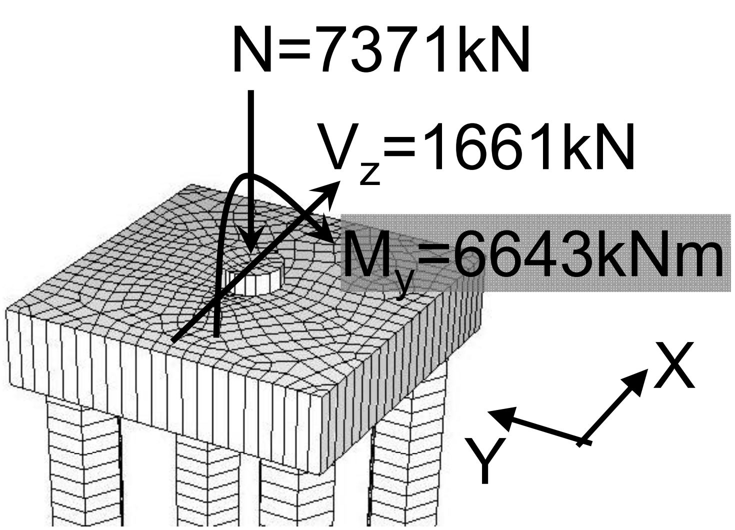
Moment – Axial force interaction diagram for deck section



Capacity effects on pier foundation

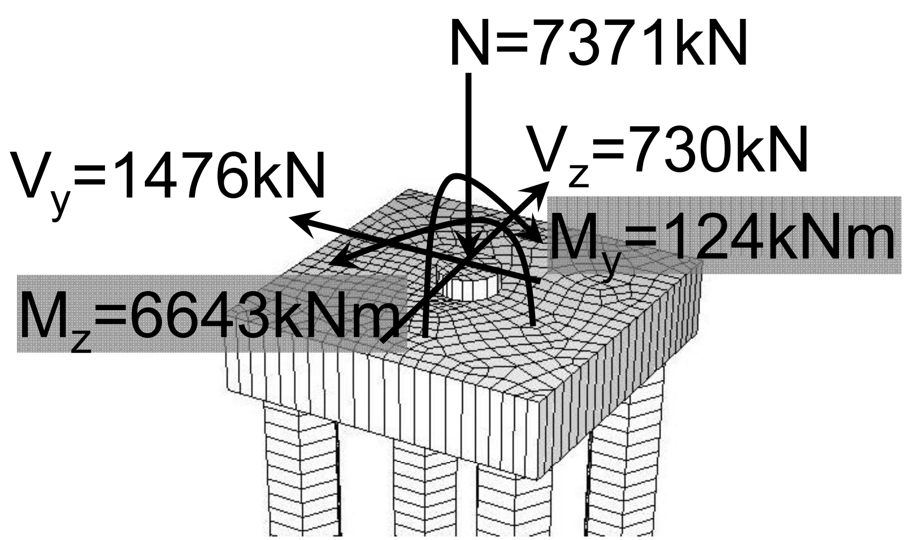
Longitudinal direction

(pier M1 foundation for +x direction)



Transverse direction

(pier M1 foundation for +/- direction)



Displacements in linear analysis

Control of Displacements

Assessment of seismic displacement d_E

$$d_E = \eta \mu_d d_{Ee}.$$

d_{Ee} = result of elastic analysis.

η = damping correction factor for $\xi \neq 0.05$.

$$\eta = \sqrt{\frac{0.10}{0.05 + \xi}}$$

μ_d = displacement ductility as follows:

when $T \geq T_0 = 1.25 T_C$: $\mu_d = q$

when $T < T_0$: $\mu_d = (q-1) T_0 / T + 1 \leq 5q - 4$

Displacements in linear analysis

- **Provision of adequate structure clearance for the total seismic design displacements:**

$$d_{Ed} = d_E + d_G + \psi_2 d_T \quad \text{with} \quad \psi_2 = 0.5$$

d_G due to permanent and quasi-permanent actions (mainly shrinkage + creep).
d_T due to thermal actions.

- **Roadway joint displacements:**

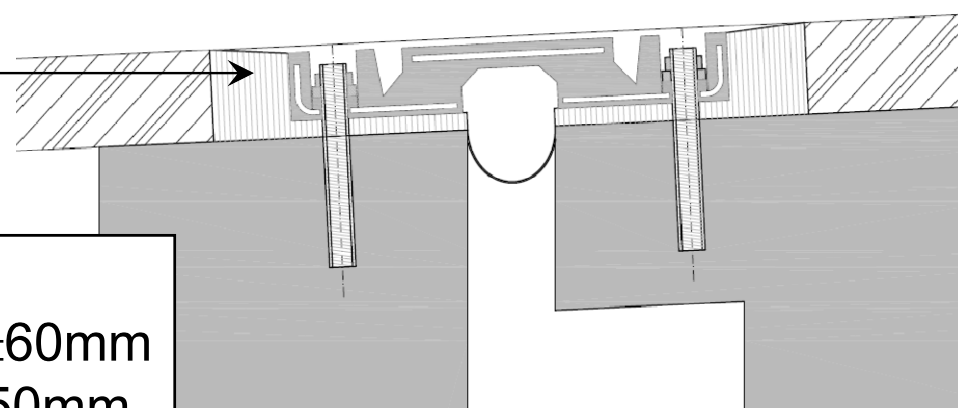
$$d_{Ed} = 0.4 d_E + d_G + \psi_2 d_T$$

Roadway joints over abutments

Roadway joint: $d_{Ed,J} = 0.4d_E + d_G + \psi_2 d_T$, where $\psi_2 = 0.5$

Structure clearance: $d_{Ed} = d_E + d_G + \psi_2 d_T$

Displacements (mm)		d_G Shrinkage +Creep	d_T Temper. variation	d_E Earth quake	$d_{Ed,J}$ Roadway joint	d_{Ed} clearance
Longitudinal	opening	+18,7	+10,7	+76,0	+54,5	+100,7
	closure	0	-8,5	-76,0	-34,7	-80,3
Transverse		0	0	±109,9	±44,0	±109,9



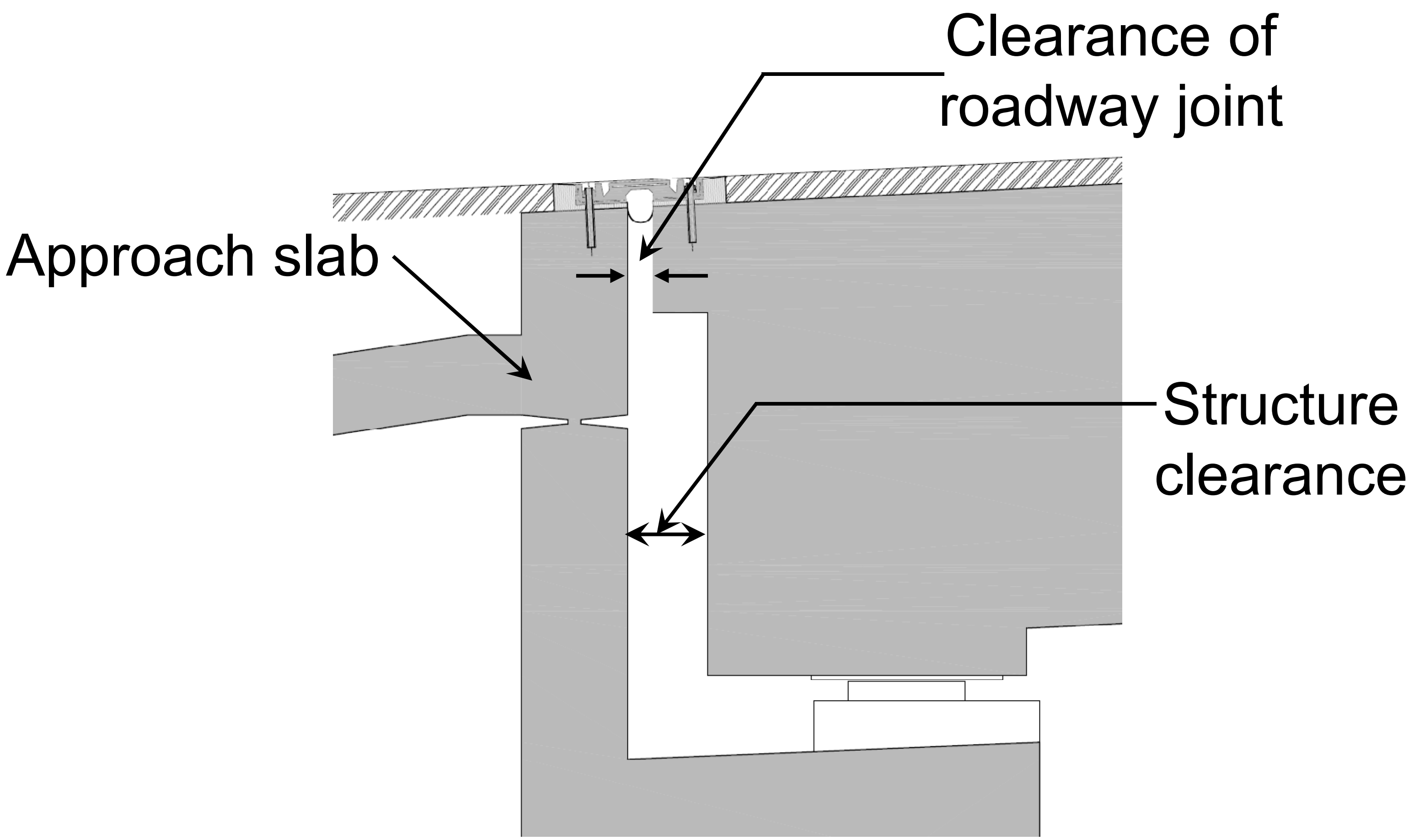
Roadway joint type: T120

Capacity in longitudinal direction: ±60mm

Capacity in transverse direction: ±50mm

Roadway joints over abutments

Detailing of back-wall for predictable (controlled) damage. (2.3.6.3 (5), EN 1998-2))



Minimum overlapping length

Minimum overlapping (seating) length at moveable joint (according to 6.6.4 of EN 1998-2)

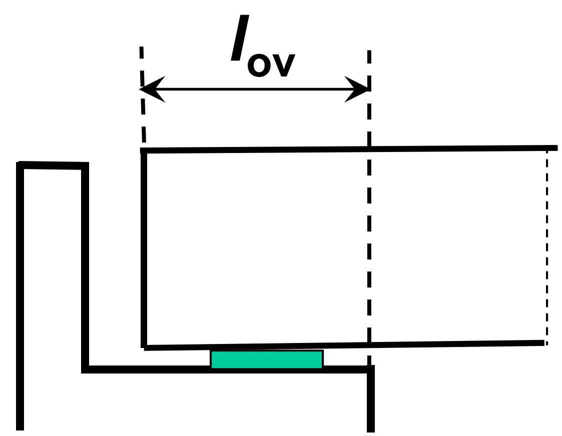
$l_m = 0.50\text{m} > 0.40\text{m}$

$d_g = 0.025 \cdot a_g \cdot S \cdot T_C \cdot T_D = 0.025 \cdot 0.16 \cdot 9.81\text{m/s}^2 \cdot 1.15 \cdot 0.60\text{s} \cdot 2.50\text{s} = 0.068\text{m}$
$L_g = 400\text{m}$ (Ground type C)
$L_{eff} = 82.50/2 = 41.25\text{m}$
No proximity to fault
Consequently $d_{eg} = (2 \cdot d_g/L_g)L_{eff} = (2 \cdot 0.068/400) \cdot 41.25 = 0.014\text{m} < 2\text{d}_g$

$d_{es} = 0.101\text{m}$ (deck effective seismic displacement)

$\boldsymbol{l_{ov} = l_m +} \ d_{eg} + d_{es} =$
$= 0.50 + 0.014 + 0.101 = 0.615\text{m}$

Available seating length: $1.25\text{m} > l_{ov}$



Conclusions for design concept

- **Optimal cost effectiveness of a ductile system is achieved when all ductile elements (piers) have dimensions that lead to a seismic demand that is critical for the main reinforcement of all critical sections and exceeds the minimum (e.g:ρ_{min}= 1%)**
- **This is difficult to achieve when the piers resisting the earthquake:**
  - **have substantial height differences, or**
  - **have section larger than required.**
- **In such cases it may be economical to use:**
  - **limited ductile behaviour for low a_{gR} values**
  - **flexible connection to the deck (seismic isolation)**

2 Example of limited ductile piers

Seismic design of the general example: Bridge on high piers designed for limited ductile behaviour

Bridge description

Composite steel & concrete deck

Three spans: 60m+80m+60m

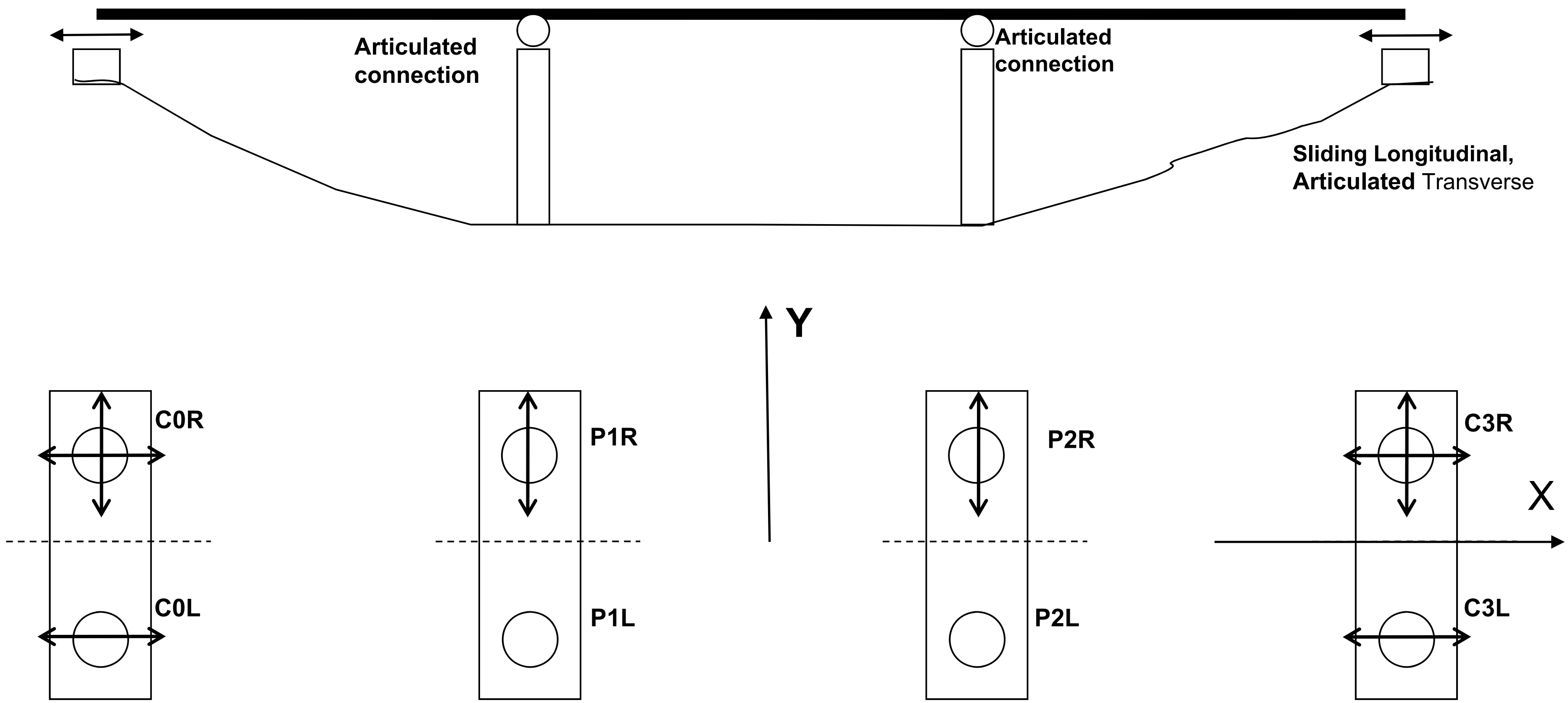
Pier dimensions:

Height 40 m,

External/internal diameter 4.0 m/3.2 m

Pier head: 4.0 m width x 1.5 m height

Bridge elevation and arrangement of bearings



Example of limited ductile piers

Design Concept

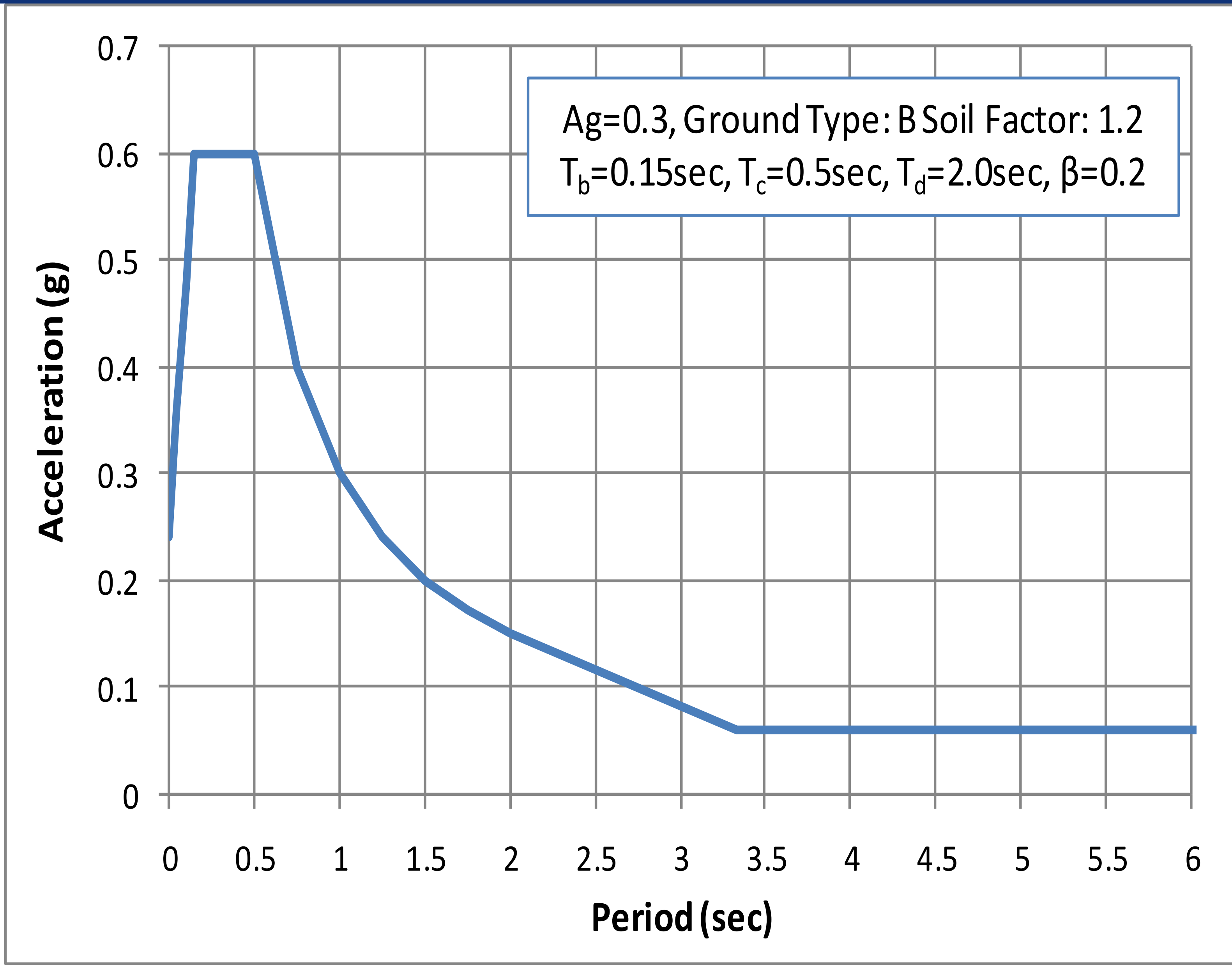
Due to the high pier flexibility:

- Hinged connection of the deck to both piers
  ►both piers resist earthquake without excessive restraints
- High fundamental period
  ► Low spectral acceleration
- No need of high ductility or seismic isolation
  ► $\boldsymbol{q}$ = 1.5 (limited ductility)

Design seismic action

- Soil type B
- Importance factor γ_I = 1.00
- Reference peak ground acceleration: a_{Gr} = 0.30g
- Soil factor: S = 1.20, $a_{Gr}S$ = 0.36g
- Limited elastic behaviour is selected
  q = 1.50
- Lower spectral boundary β = 0.2

Design seismic spectrum



Seismic analysis

Quasi permanent traffic load for the seismic design situation (4.1.2 (4):

For bridges with severe traffic

$$\boldsymbol{\psi_{2,1} Q_{k,1}} \quad \text{with} \quad \boldsymbol{\psi_{2,1}} = 0.20$$

$\boldsymbol{Q_{k,1}}$ is the characteristic load of UDL system of Model 1 **.**For the 4 lanes of the deck:

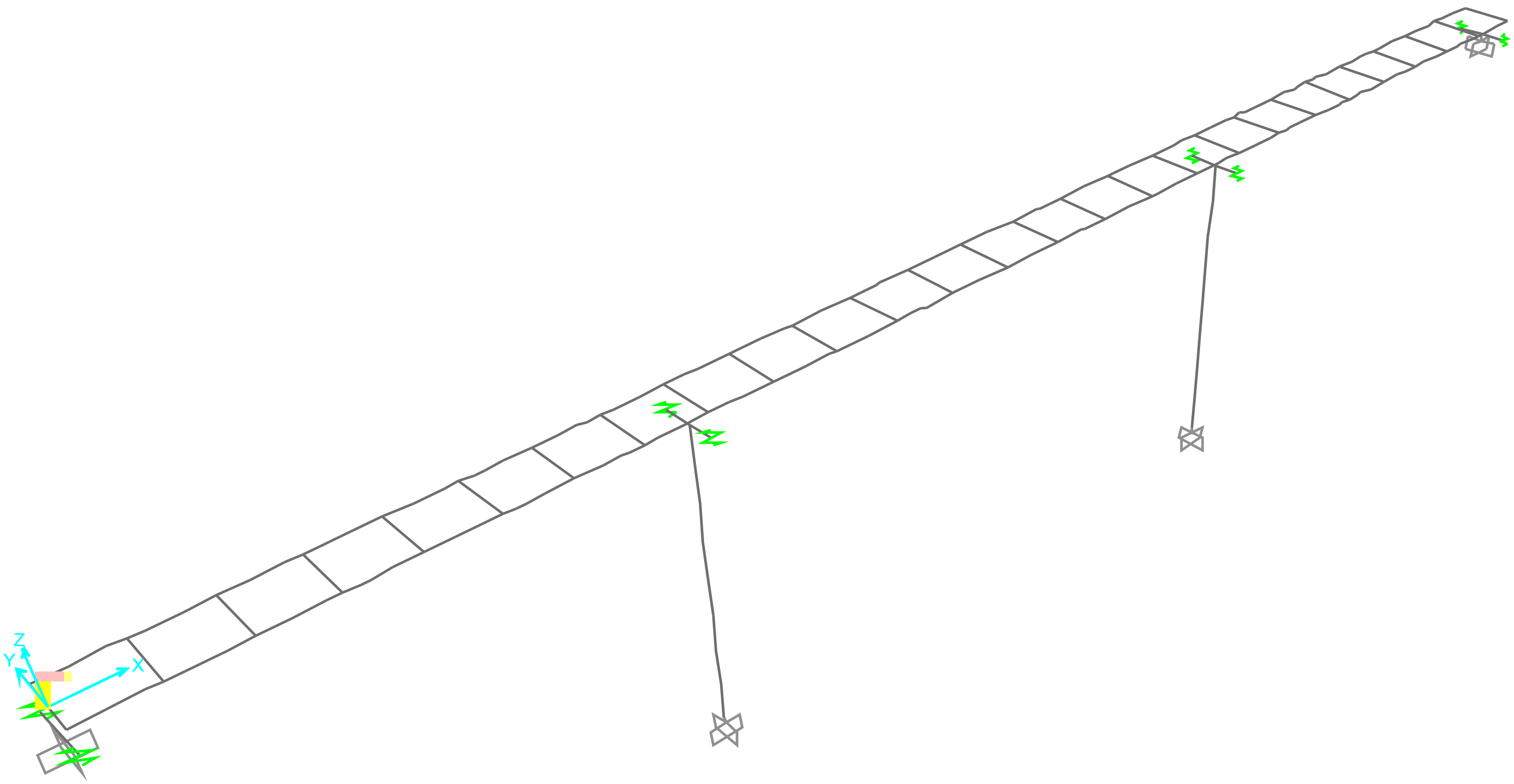
3 m x 9 + 3 m x 2.5 + 3 m x 2.5 + 2 m x 2.5

$$Q_{k,1} = 47.0 \text{ kN/m}$$

$$\psi_{2,1} Q_{k,1} = 9.4 \text{ kN/m}$$

Seismic analysis

Structural Model



Seismic analysis

- A beam finite element model of the bridge is used with program SAP 2000.
- Each composite steel concrete beam and cross beam is modeled as a beam element.
- The effective width of the main beams is assumed equal to the total geometric width.
- The bending stiffness about the vertical axis of the two main beam sections is modified so that the sum of the stiffnesses is equal to the relevant stiffness of the entire composite deck.

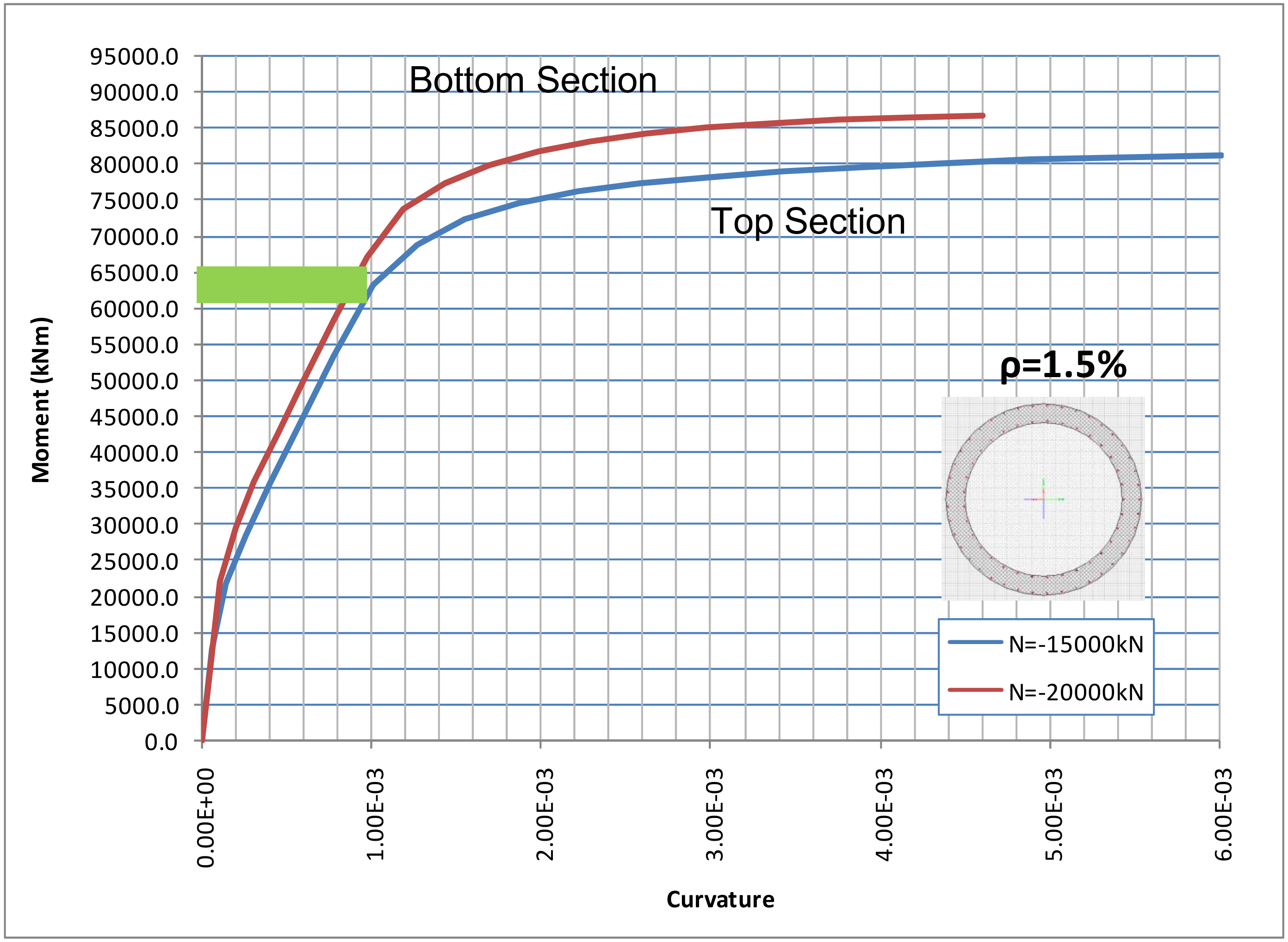
Seismic analysis

Effective Pier Stiffness

- 2.3.6.1 (1) of EN1998-2 defines the stiffness of the ductile elements (piers) to correspond to the secant stiffness at the theoretical yield point.
- This value depends on the axial force and on the final reinforcement of the element.
- It is estimated from the moment-curvature and the moment – Jeff/Jgross ratio curves of the piers for the estimated final reinforcement ρ = 1.5% and the seismic axial force, as

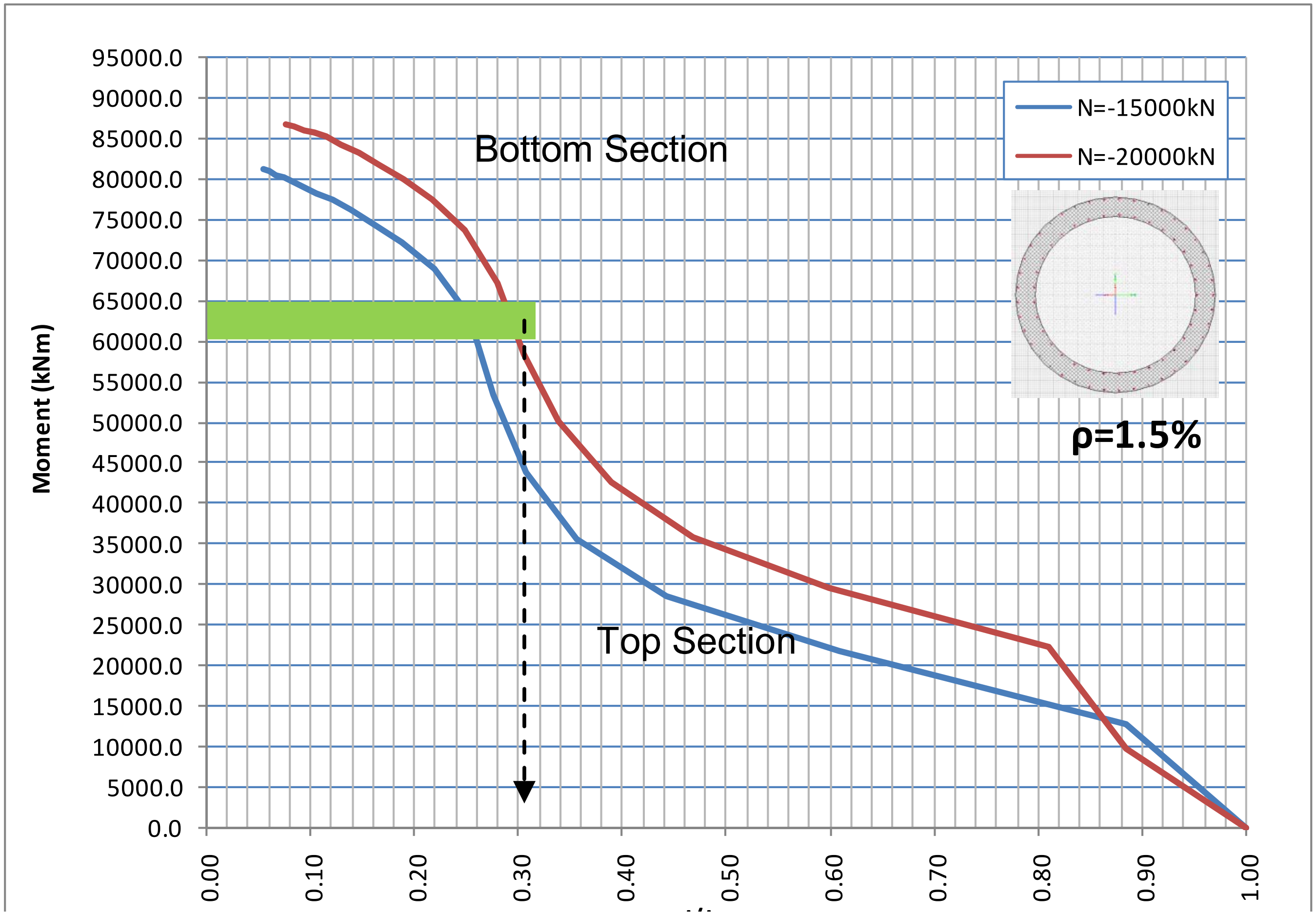
Jeff/Jgross ≈ 0.30

Seismic analysis



Moment-Curvature M-Φ curve of Pier Section for ρ = 1.5%

Seismic analysis



Moment –$(EI)_{eff}/(EI)$ Curve of Pier Section for ρ = 1.5%

$$(EI)_{eff}/(EI) = (M/\Phi)/(EI)$$

Seismic analysis

Eigenmodes – Response Sectrum Analysis

- The first 30 eigenmodes of the structure were calculated and considered in the response spectrum analysis.
- The sum of modal masses of these modes amounts to: 97.1% in the X and 97.2% in the Y direction respectively.
- Combination of modal responses was carried out using the CQC rule.

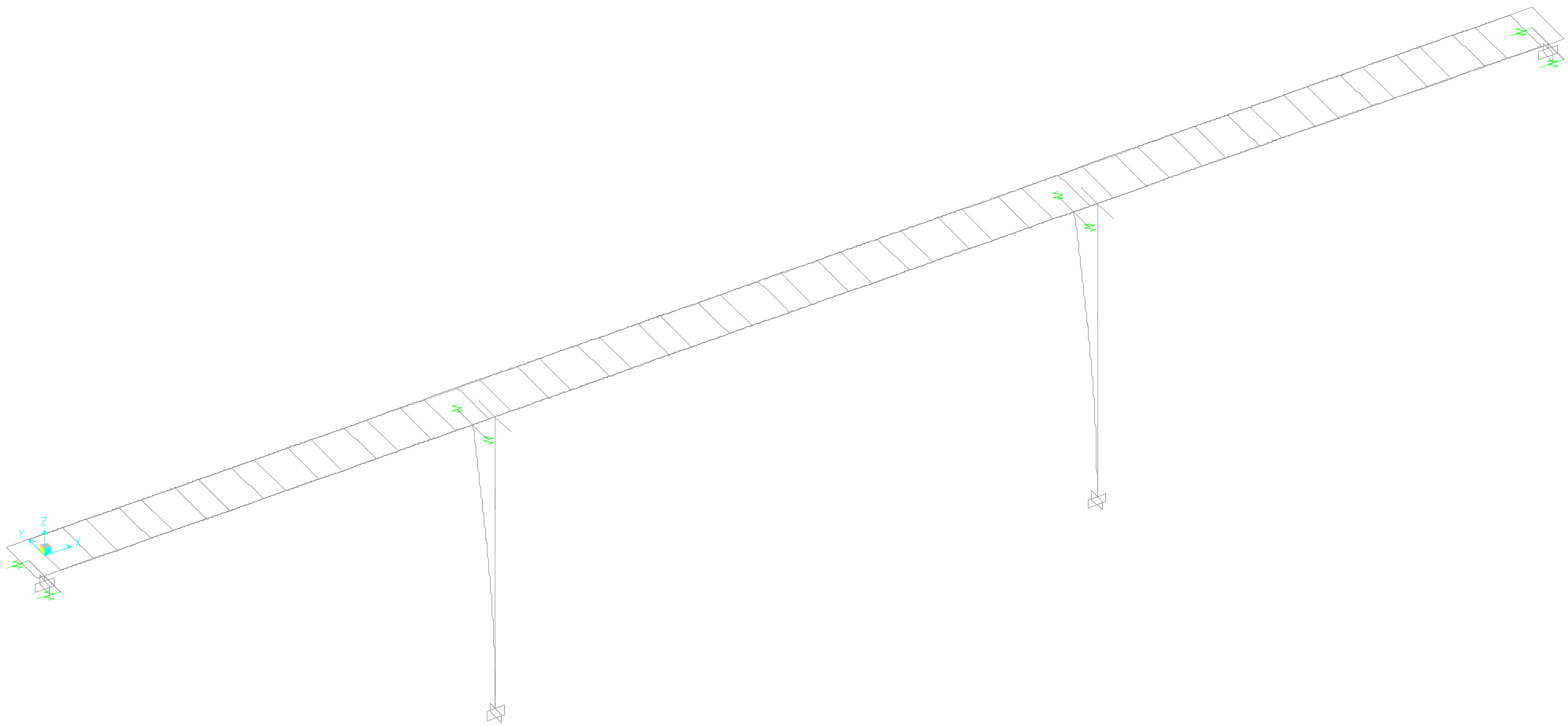
Following table shows the characteristics of the first 10 eigenmodes.

Seismic analysis

First 10 eigenmodes

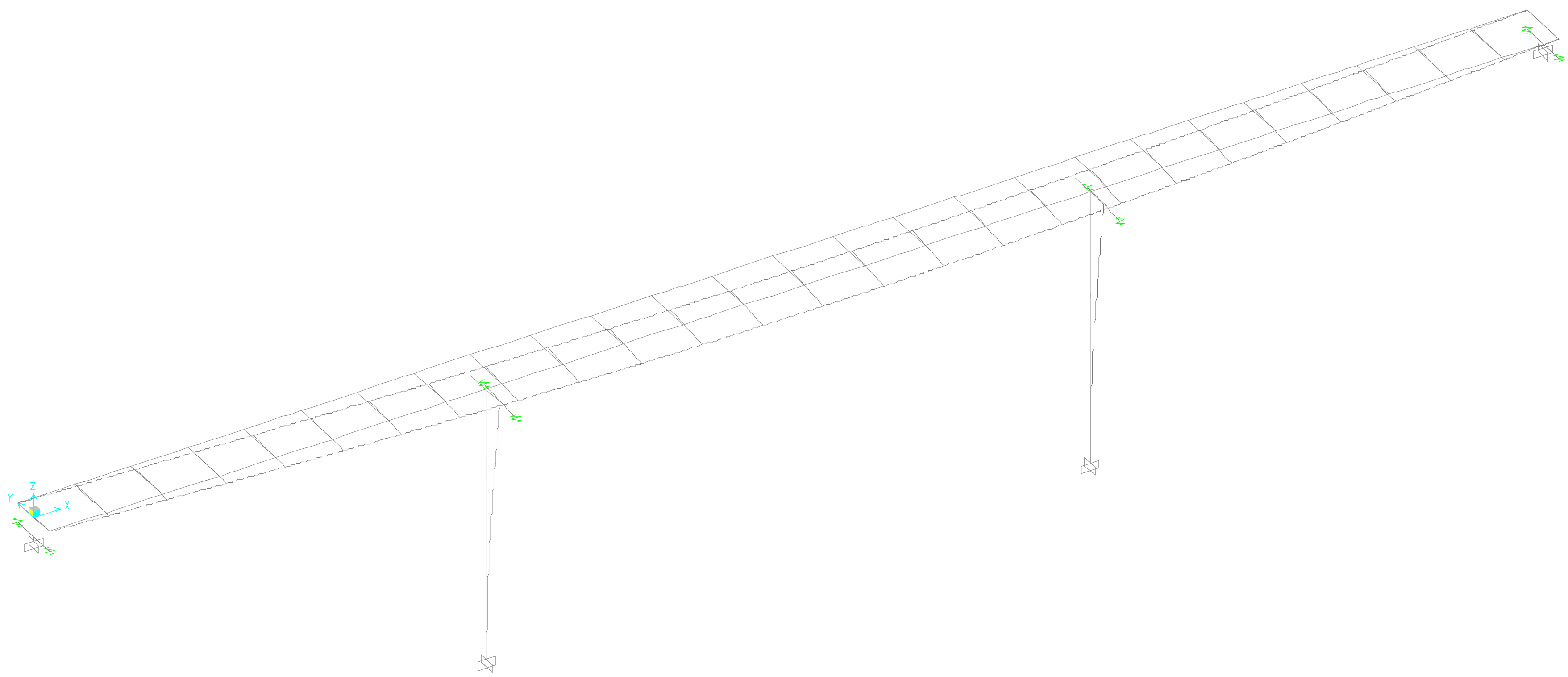
No	Period Sec	Modal Mass % X	Modal Mass % Y	Modal Mass % Z
1	5.03	92.5%	0.0%	0.0%
2	3.84	0.0%	76.8%	0.0%
3	1.49	0.0%	0.0%	0.0%
4	0.79	0.0%	0.0%	1.2%
5	0.71	0.0%	0.5%	0.0%
6	0.66	0.0%	8.4%	0.0%
7	0.52	0.0%	0.0%	0.0%
8	0.50	0.0%	0.0%	0.0%
9	0.48	0.0%	2.1%	0.0%
10	0.46	0.0%	0.0%	0.0%

Seismic analysis



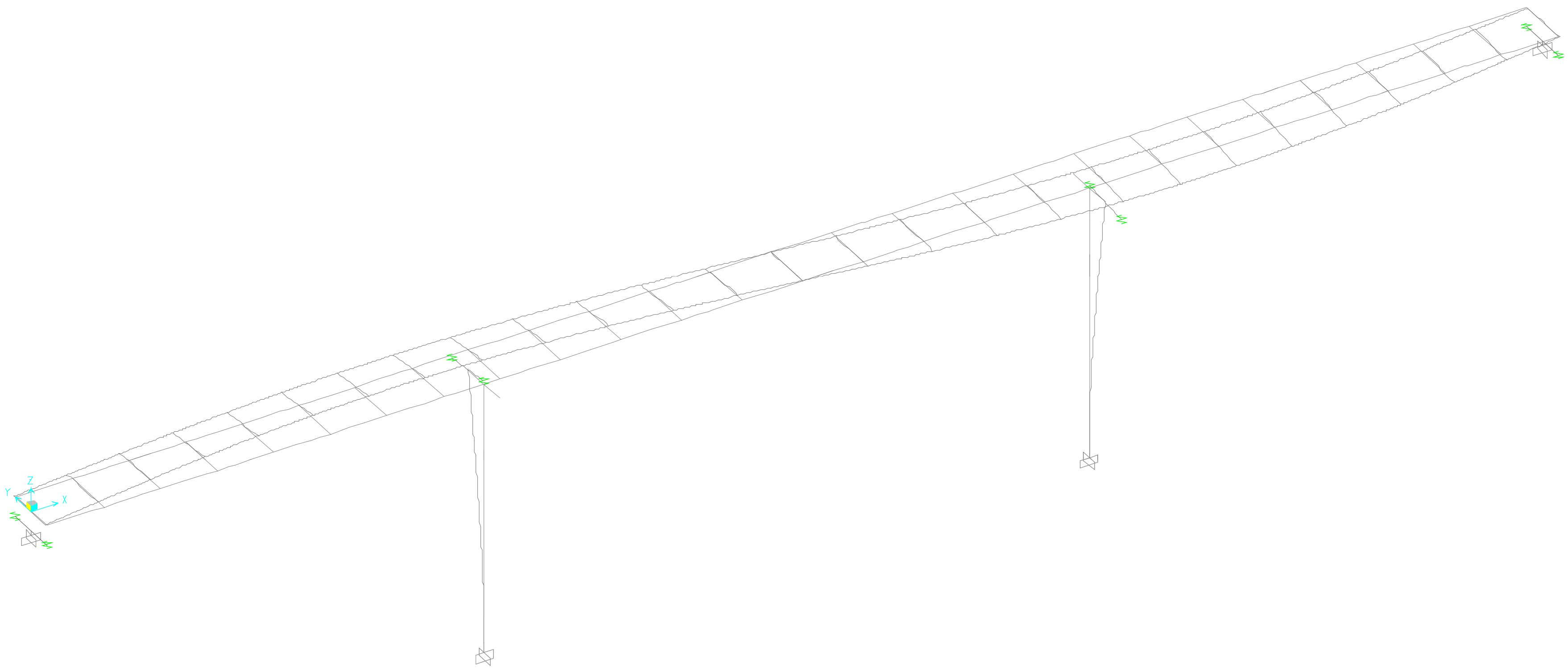
1st Mode - Longitudinal – Period 5.02 sec c
(Mass Participation Factor Ux:93%)

Seismic analysis



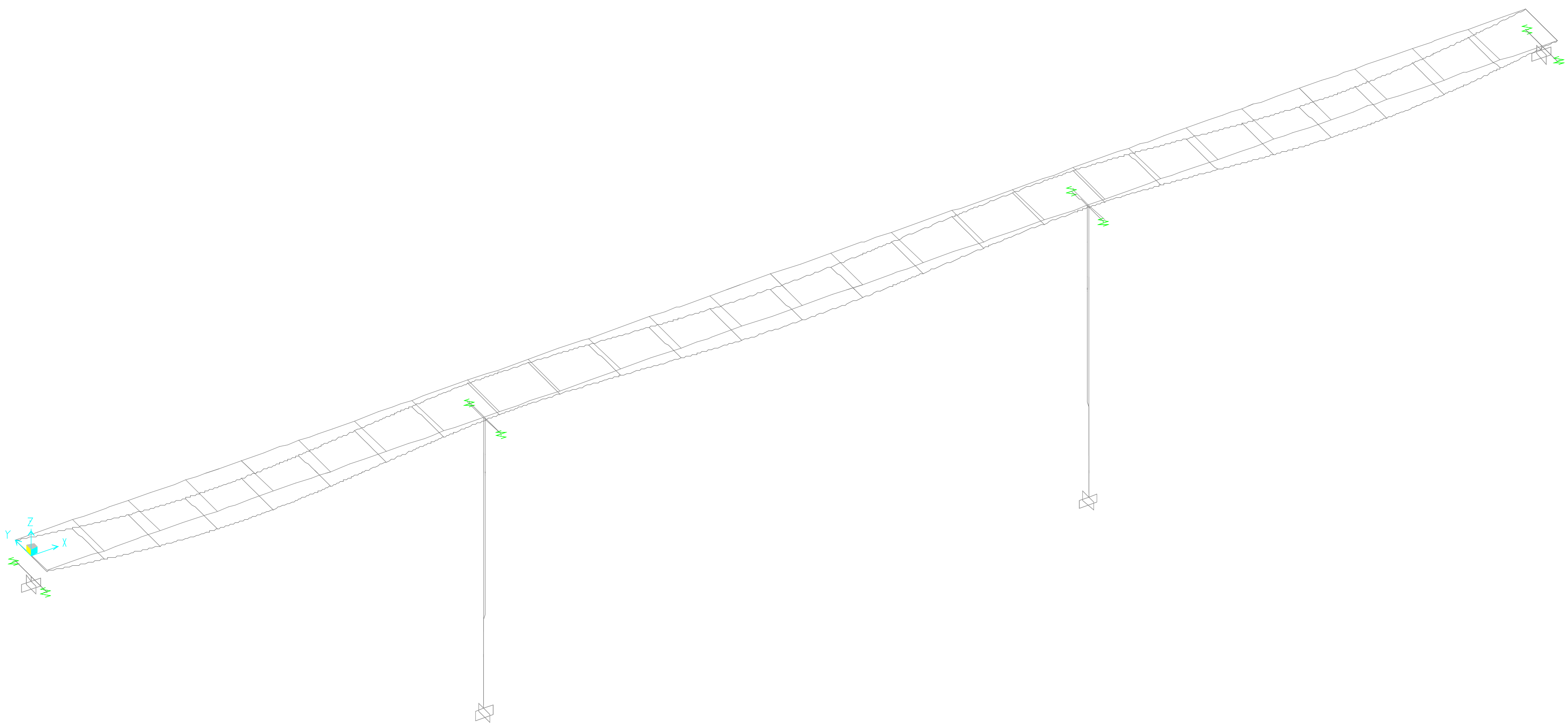
2nd Mode -Traqnsverse– Period 3.84 sec
(Mass Participation Factor Uy:77%)

Seismic analysis



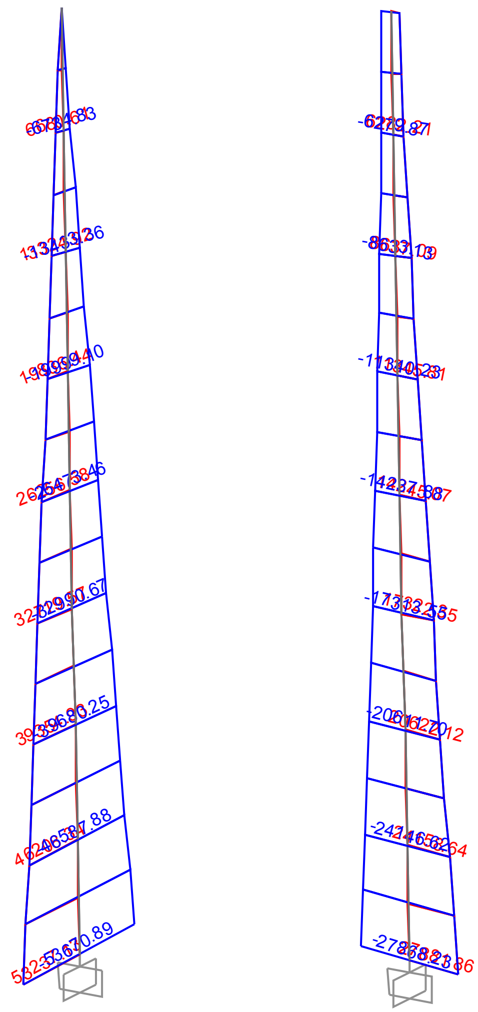
3rd Mode - Rotation– Period 1.49 sec

Seismic analysis



11th Mode - Vertical – Period 0.42sec
(Mass Participation Factor Uz:63%)

Seismic analysis



Max bending moment distribution along pier P1

2nd order effects for the seismic analysis

Geometric Imperfections of Piers

According to 5.2 of EN 1992-2:2005:

$$\theta_i = \frac{1}{200}\frac{2}{\sqrt{l}} = 1.58 \cdot 10^{-3}$$

$$e_i = \theta_i \frac{l_0}{2}$$

Imperfection Eccentricities

Direction	l_0 (m)	e_i (m)
X	80	0.063
Y	40	0.032

2^{nd} order effects for the seismic analysis

The first and second order effect of imperfection eccentricities $e_{i,II}$ including creep for $\varphi = 2.0$

$$e^{II}_{imp,\varphi} = e_{imp}\left(1 + \frac{1+\varphi}{v-1}\right)$$

where

$v = N_B/N_{Ed}$

N_B: buckling load according to 5.8 of EN1992-1-1

Direction	e_i	$v = N_B/N_{ED}$	$e_{i,II}/e_i$	$e_{i,II}$
x	0.063	19.65	1.161	0.073
y	0.032	78.62	1.039	0.033

2^{nd} order effects for the seismic analysis

Second order effects due to seismic first order effects

Two alternative approaches:

1. To 5.8 of EN 1992-1-1 (nominal stiffness method)

Use of nominal stiffness method (5.8.7) for $(EI)_{eff} = 0.30\ (EI)$

Moment magnification factor:

$$MF = 1+[\beta/((N_B/N_{Ed})-1)]$$

$$N_B = \pi^2(EI)_{eff}/(\beta_1 L_0)^2 \quad \beta_1 = 1$$

- Longitudinal direction $MF = 1.154$
- Transversal direction $MF = 1.034$

2^{nd} order effects for the seismic analysis

2nd order effects due to seismic 1st order effects

2. To 5.4 of EN 1998-2 (followed in the example)

Increase of bending moments at the plastic hinge section

$$\Delta M = 0.5\,(1+q)d_{Ed}N_{Ed}$$

d_{Ed}: seismic displacement of pier top

2^{nd} Order Effects on Pier Base Moments

iCombination		EN1992-1-1	EN1998-2
***Ex*+0.3*Ey*+2nd Ord**	***My***	**58576.4**	**58298.5**
***Ey*+0.3*Ex*+2nd Ord**	***Mx***	**27178.8**	**30508.3**

► The Eff. Stiffness method of EN1992-1-1 may be unsafe for $1.5 < \boldsymbol{q} \le 3.5$

Verification of piers

Flexure and axial force for pier base section

Design action effects:

N_{Ed} = 19568 kN

M_{Ey} = 59610 kNm

M_{Ex} = 9833 kNm

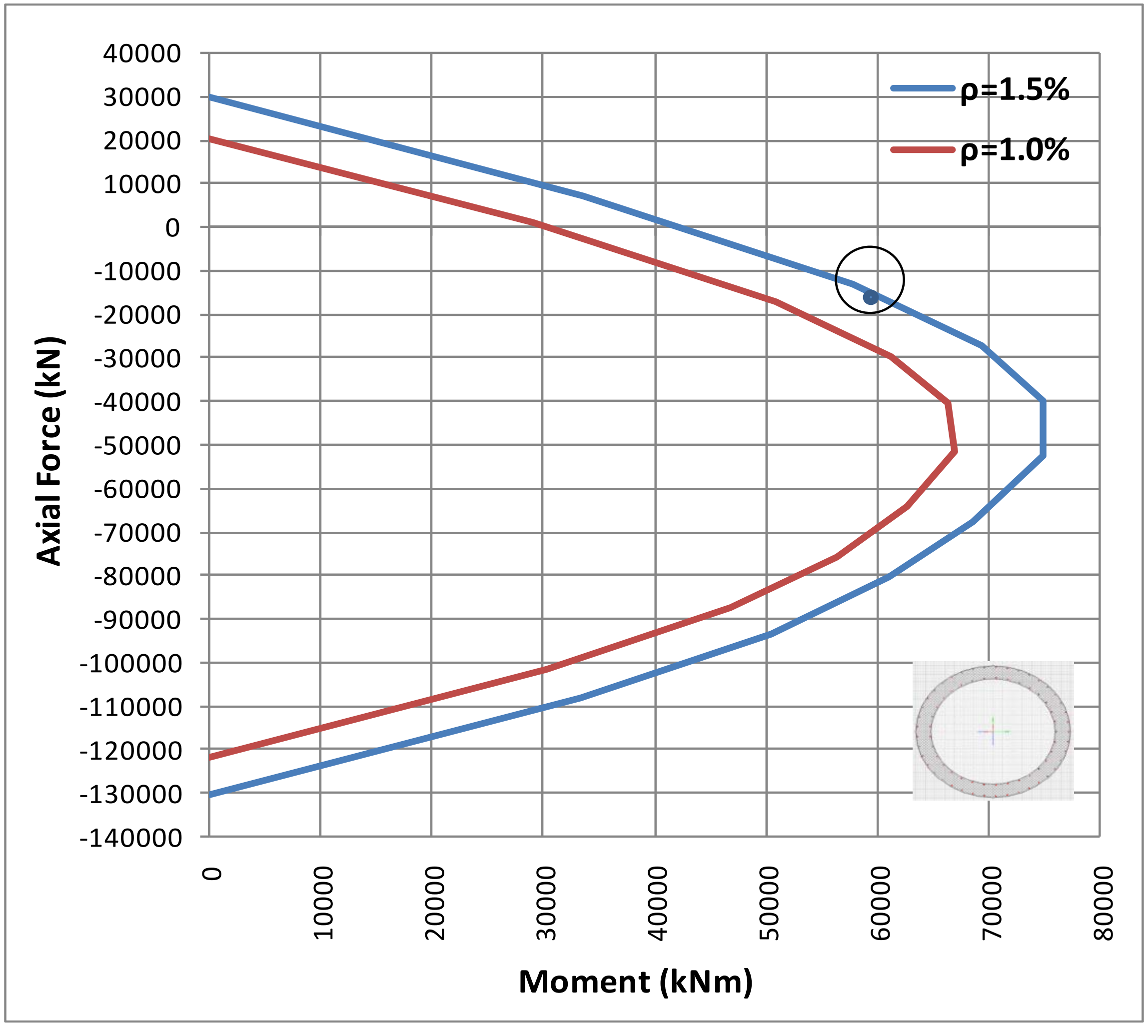
► $A_{s,req}$ = 678 cm^2

External perimeter 62Φ28 (381 cm^2)

Internal perimeter 49Φ28 (301 cm^2) $\rho \approx 1.5\%$

See design interaction diagram next page

Verification of piers



Design Interaction Diagram of Pier Base Section

Verification of piers

Shear verification

Design shear: $V_{x,d}$ = 1887 $V_{y,d}$ = 281

$$V_d = \sqrt{V_{x,d}^2 + V_{y,d}^2} = 1908kN$$

$$V_{Rd,c} = [C_{Rd,c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d$$

$$C_{Rd,c} = \frac{0.18}{\gamma_c} = \frac{0.18}{1.5} = 0.12$$

$$d_e = r + \frac{2 \cdot r_s}{\pi} = 2.0 + \frac{2 \cdot 1.8}{\pi} = 3.15$$ (EN1998-2:2005, 5.6.3.3.(2))

$$k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{3150}} = 1.25 \qquad k_1 = 0.15 \qquad \sigma_{cp} = \frac{N_{Ed}}{A_c} = \frac{15}{4.52} = 3.32$$

$V_{Rd,c}$ = 2670kN $\qquad \dfrac{V_{Rd,c}}{\gamma_{Bd1}} = \dfrac{2670}{1.25} = 2136kN > V_d$

No shear reinforcement required

Verification of piers

Ductility requirements

- Confining reinforcement

To 6.2.1.4 of EN 1998-2 for limited ductile

$$\omega_{wd,c} \geq \max(1.4 \cdot \omega_{w,req};0.12) = \max(1.4 \cdot 0.058;0.12) = 0.12$$

$$\omega_{w,req} = \frac{A_c}{A_{cc}} \cdot 0.28 \cdot \eta_k + 0.13 \cdot \frac{f_{yd}}{f_{cd}} \cdot (\rho_l - 0.01) =$$

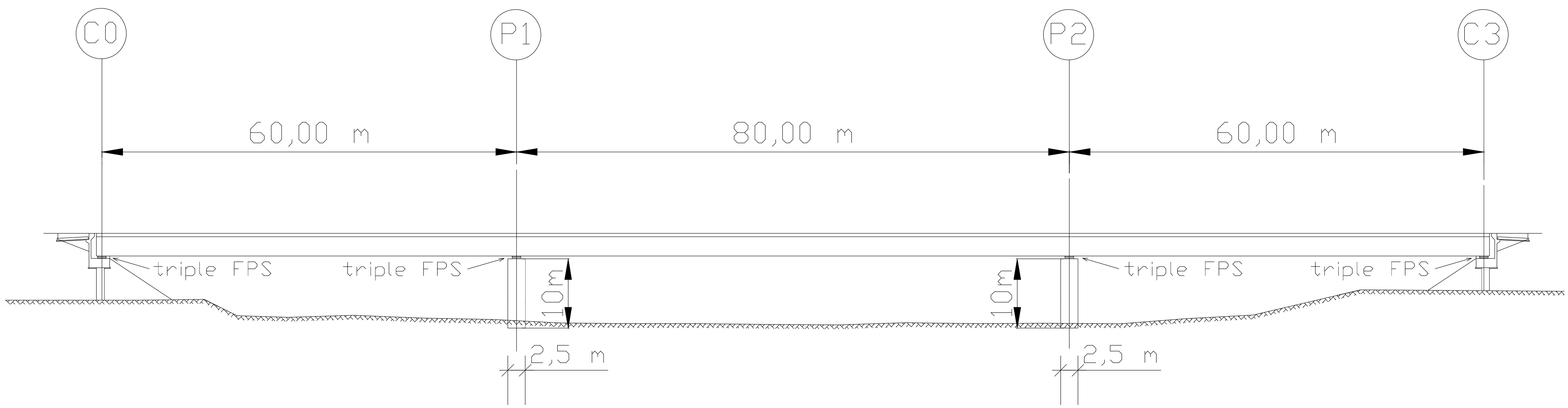
$$= \frac{4.52}{3.39} \cdot 0.28 \cdot \frac{19580}{35000 \cdot 4.52} + 0.13 \cdot \frac{500000 \cdot 1.5}{35000 \cdot 1.15} \cdot (0.015 - 0.01) = 0.058$$

$$\rho_w = \omega_w \frac{f_{cd}}{f_{yd}} = 0.12 \cdot \frac{35000 \cdot 1.15}{500000 \cdot 1.5} = 0.0064 \qquad \rho_w = \frac{\pi \cdot D_{sp} \cdot A_{sp}}{A_{cc} \cdot s_l} \triangleright \Phi 16 / 11$$

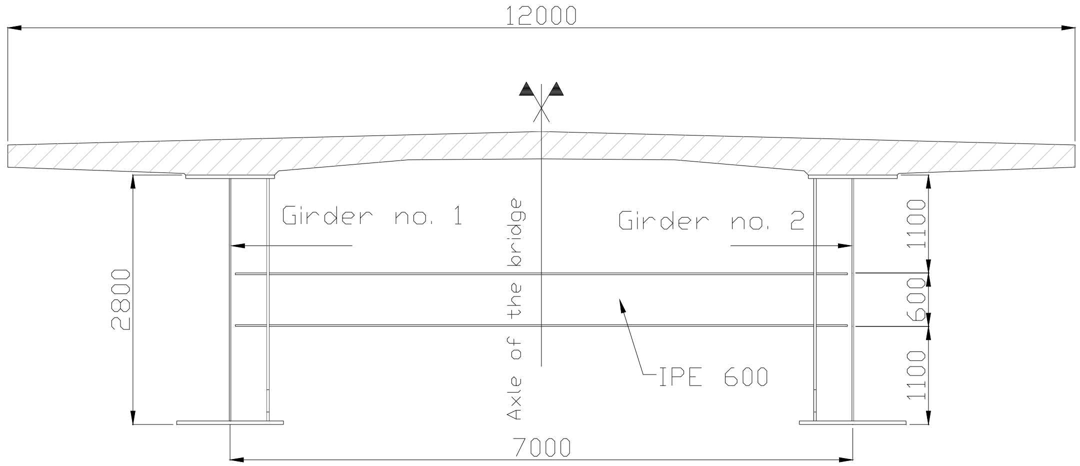
- No buckling of reinforcement $s_L < 5d_{bL} = 14$ cm

3 Example of seismic isolation

Bridge Elevation:



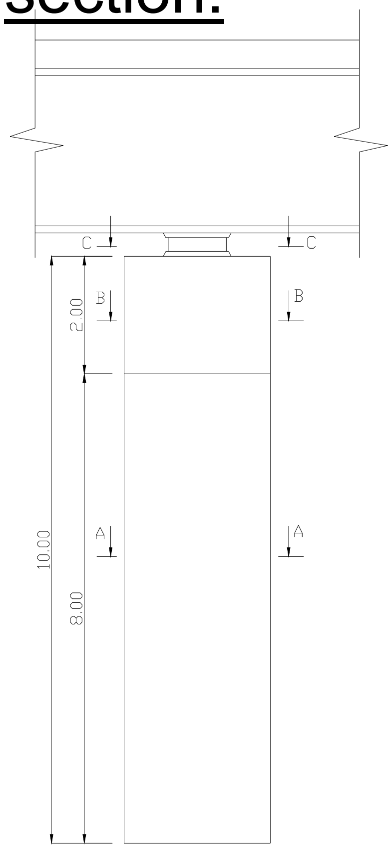
Typical deck cross-section:



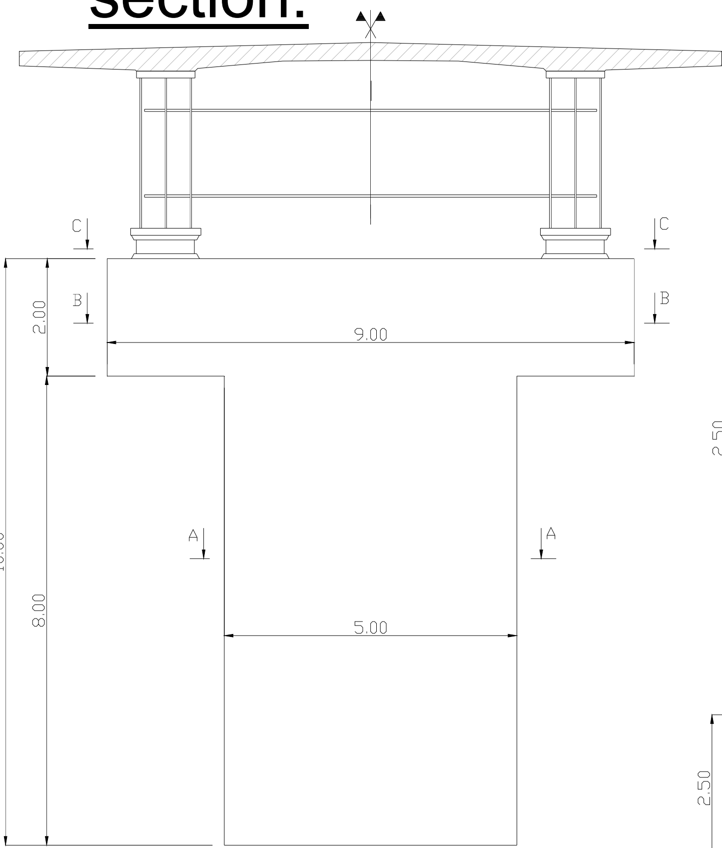
- **Layout based on general example bridge**
- **Three spans (60m+80m+60m)**
- **Composite steel & concrete deck**

Pier Layout

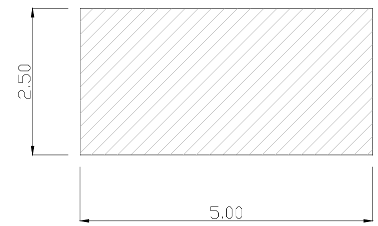
Longitudinal section:



Transverse section:

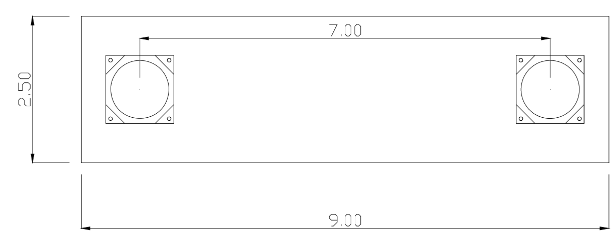


Section A-A:



Section B-B:

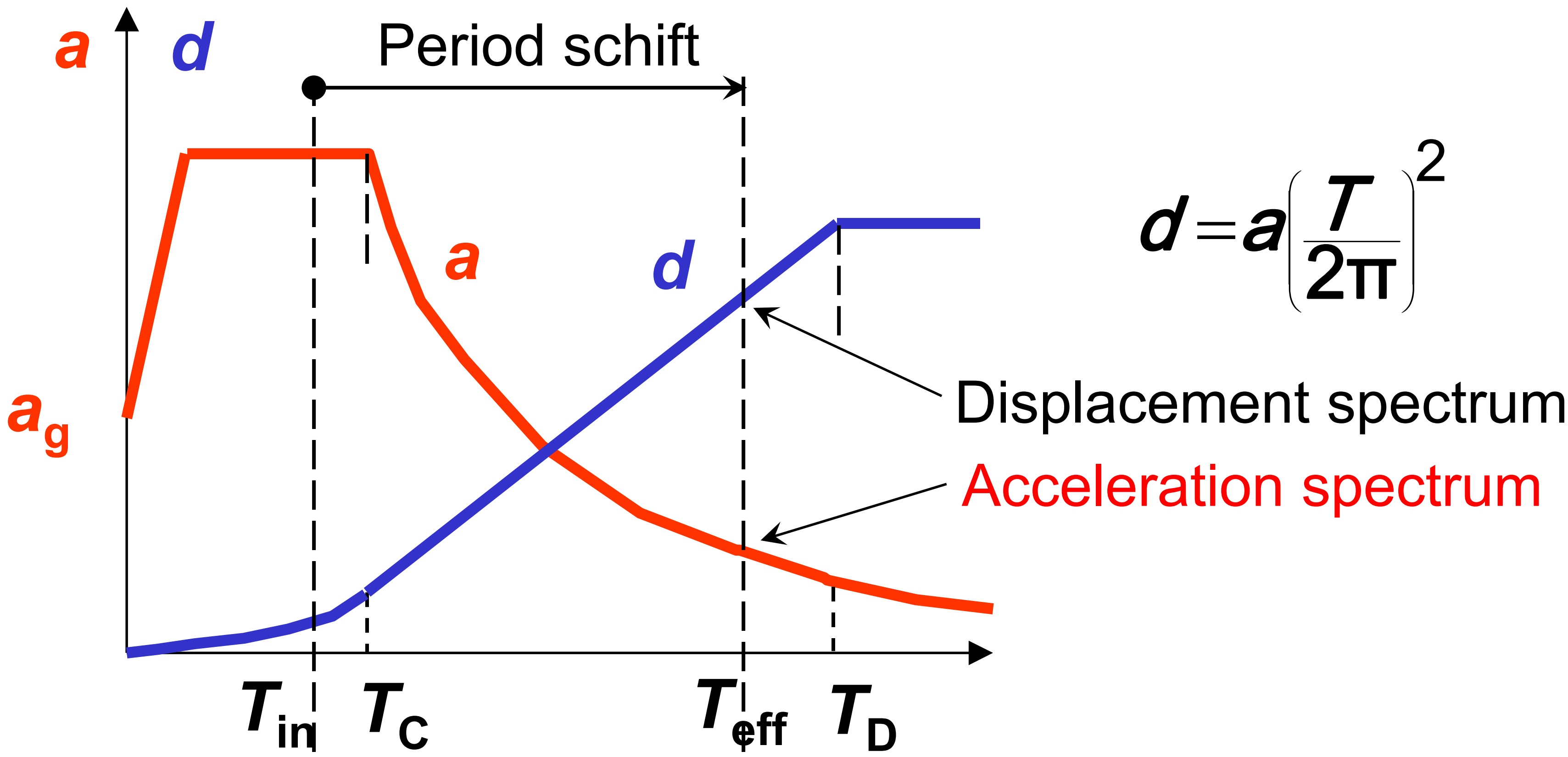
Section C-C:



- **Short piers h=10m**
- **Rectangular cross-section 5.0m x 2.5m**
- **Enlarged pier head 9.0m x 2.5m to support bearings**

Bridges with Seismic Isolation

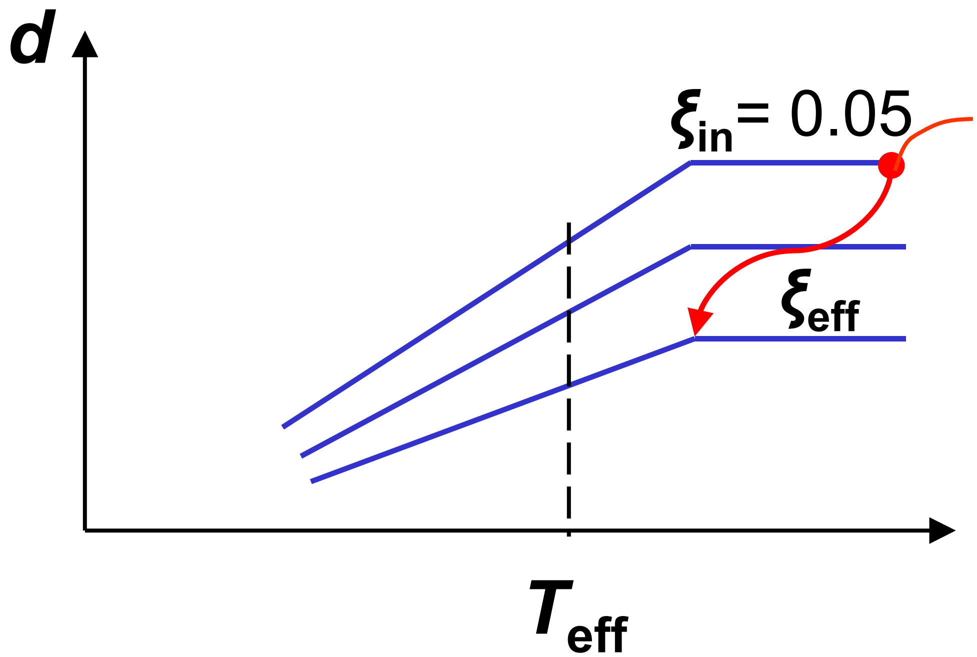
Effect of period shift from T_{in} to T_{eff}



$$d = a\left(\frac{T}{2\pi}\right)^2$$

Bridges with Seismic Isolation

Effect of increasing damping (ξ_{eff})

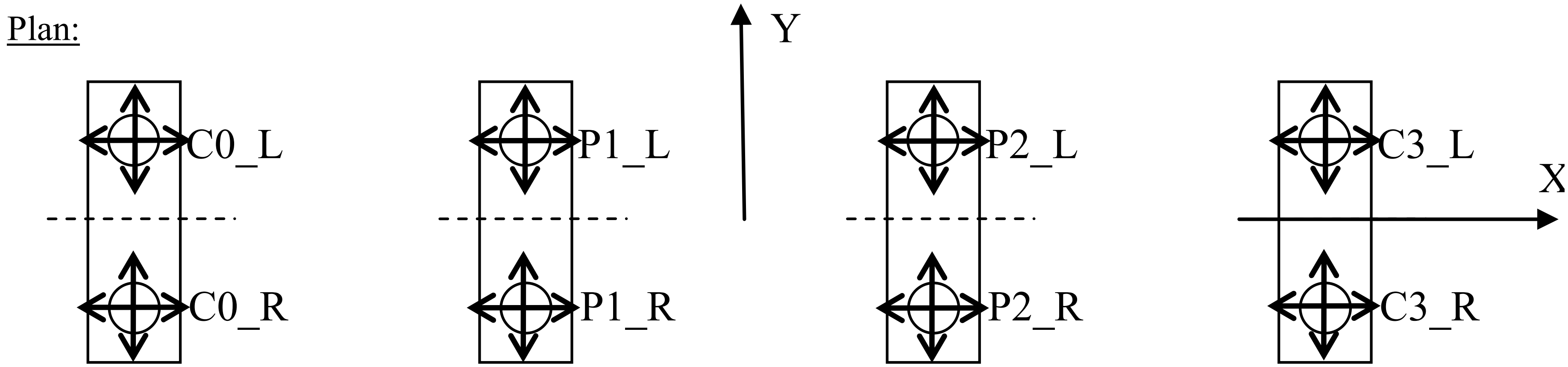


Increasing ξ_{in} to ξ_{eff} reduces displacements but not necessarily forces

$$d_{\xi eff} = \eta \cdot d_{0,05}$$

$$\eta = \sqrt{\frac{0{,}10}{0{,}05 + \xi_{eff}}} \geq 0{,}4$$

Layout of Seismic Isolation

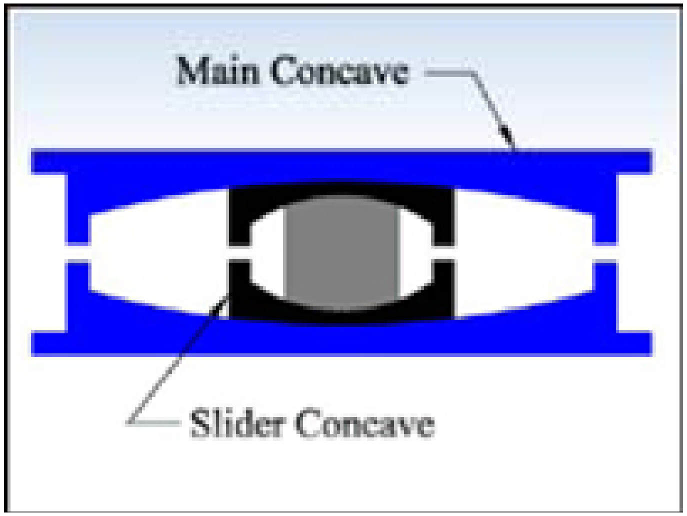


- 8 bearings of type Triple Friction Pendulum System (Triple FPS)
- 2 bearings at each abutment 0.9m x 0.9m x 0.4m
- 2 bearings at each pier 1.2m x 1.2m x 0.4m

- Allow displacements in all horizontal directions with non-linear frictional force-displacement law

Triple FPS bearings

Photo of Triple Pendulum™ Bearing



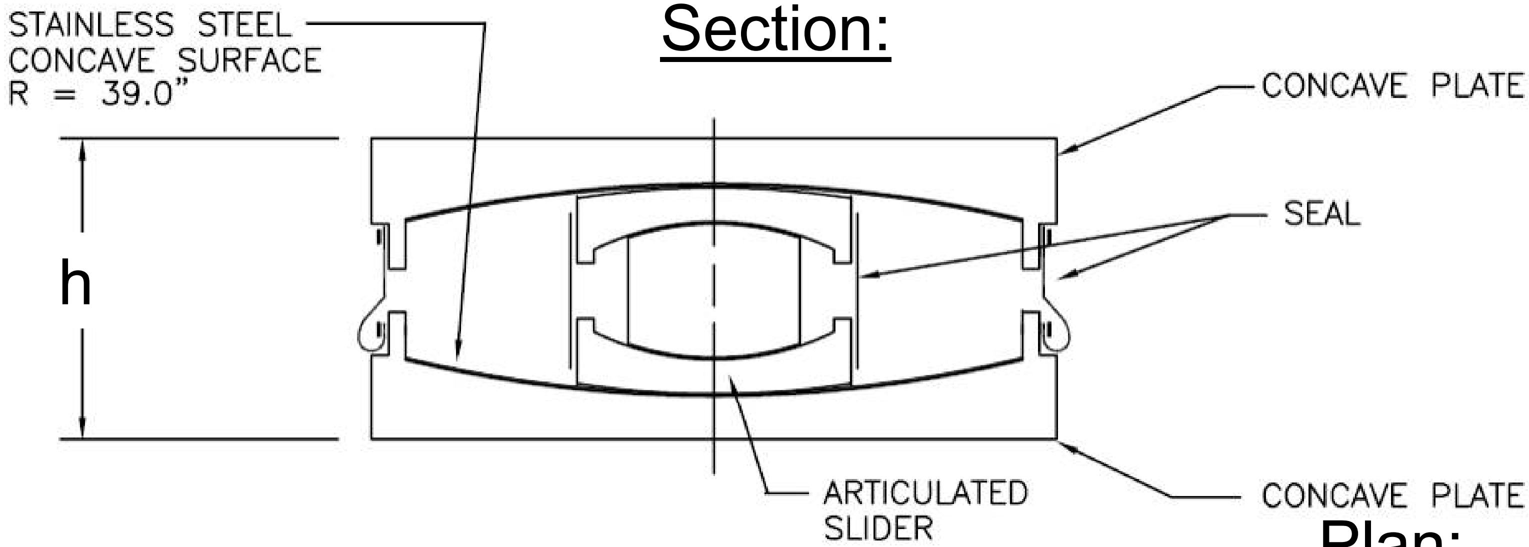
Schematic Cross Section

Concaves and Slider Assembly

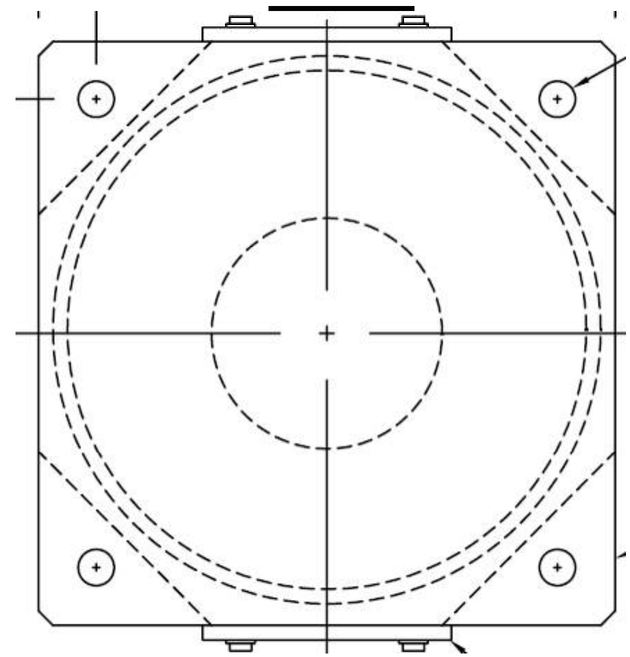
Concaves and Slider Components

Triple FPS bearings

Section:

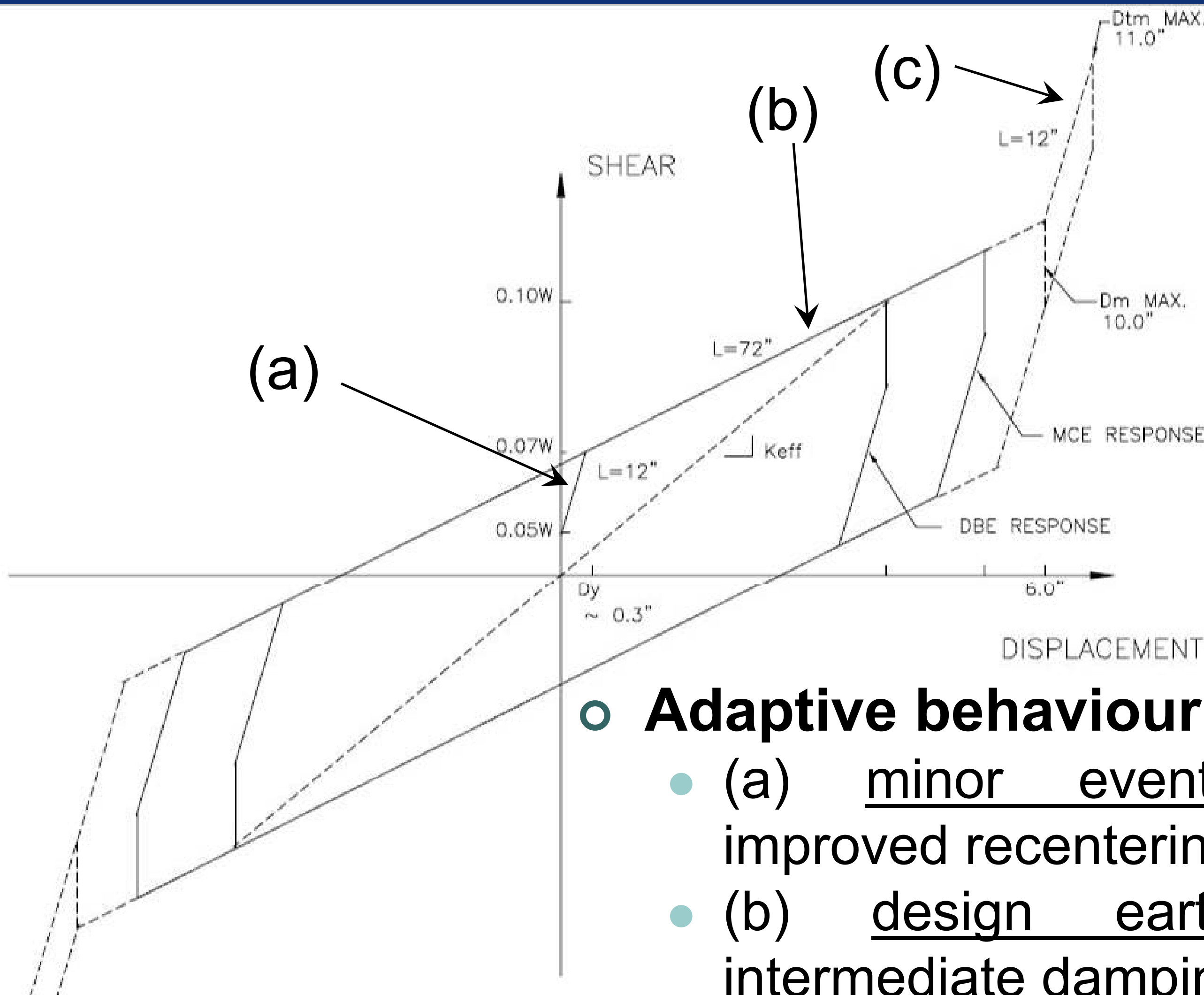


Plan:



- **4 Stainless steel concave sliding surfaces & articulated slider**
- **Special low-friction sliding material**
  - low horizontal stiffness (increased flexibility)
  - high vertical stiffness and load bearing capacity
- **Protective seal**

Triple FPS – Force-Displ. relation



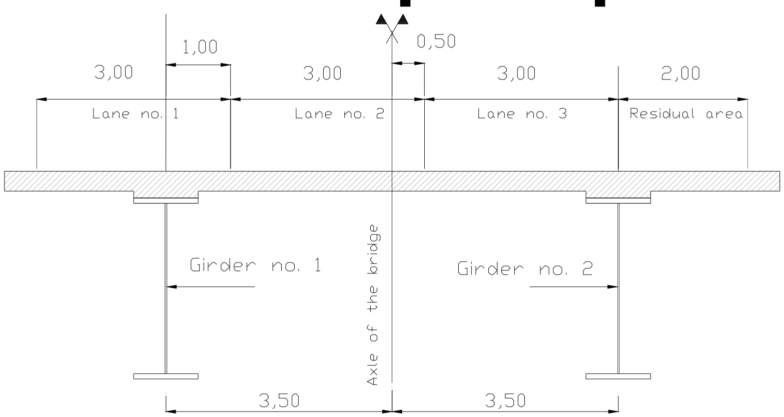
- **Adaptive behaviour**
  - (a) minor events: high stiffness, improved recentering
  - (b) design earthquake: softening, intermediate damping
  - (c) extreme events: stiffening, increased damping

General Loads

1. Permanent loads: (from general example)

Total support loads in MN (both beams)	Self weight after construction	Minimum equipment load	Maximum equipment load	Total with minimum equipment	Total with maximum equipment	Time variation due to creep & shrinkage
C0	2.328	0.664	1.020	2.993	3.348	-0.172
P1	10.380	2.440	3.744	12.819	14.123	0.206
P2	10.258	2.441	3.745	12.699	14.003	0.091
C3	2.377	0.664	1.019	3.041	3.396	-0.126
Sum of reactions	25.343	6.209	9.528	31.552	34.871	0.000

2. 20% of quasi-permanent traffic load:



Lane Number 1: $\alpha_q q_{1,k}$ = 3 m x 9 kN/m^2 = 27.0 kN/m

Lane Number 2: $\alpha_q q_{2,k}$ =3 m x 2.5 kN/m^2 = 7.5 kN/m

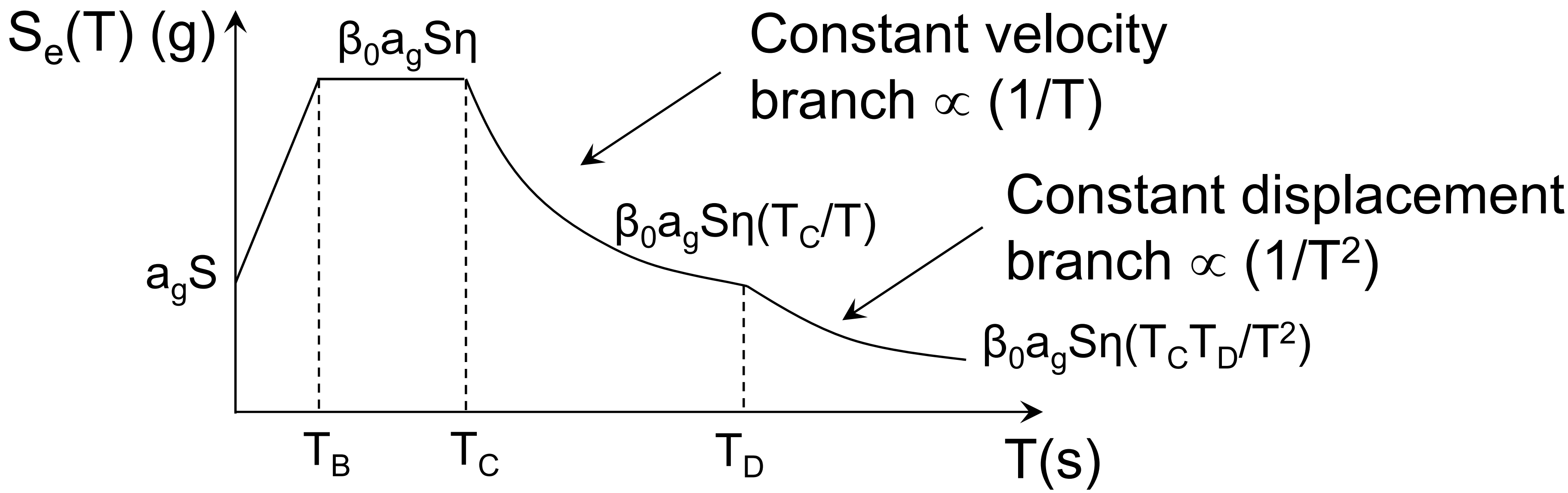
Lane Number 3: $\alpha_q q_{3,k}$ =3 m x 2.5 kN/m^2 = 7.5 kN/m

<u>Residual area: $\alpha_q q_{r,k}$ =2 m x 2.5 kN/m^2 = 5.0 kN/m</u>

Total load = 47.0 kN/m

3. 50% of thermal action: +25°C / -35°C

Seismic Action – Design Spectra



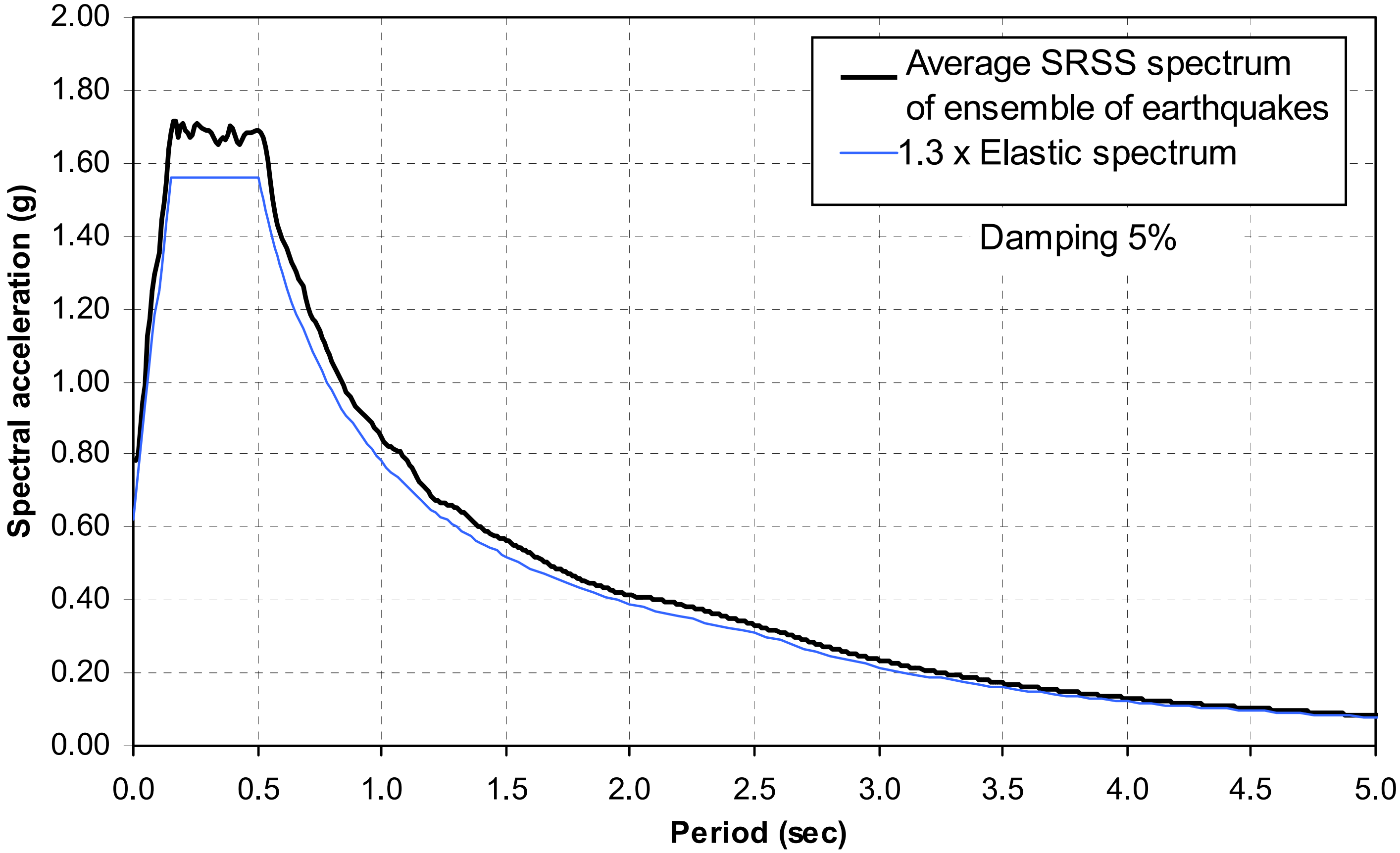
- **Elastic response spectrum EN1998-2 §7.4.1, EN1998-1 §3.2.2.2 and §3.2.2.3**
- **Average importance:** γ_I=1.00
- **High Seismicity Zone:** a_{gR}=0.40g

$\Big\}$ $a_g = a_{gR} \times \gamma_I = 0.40g$

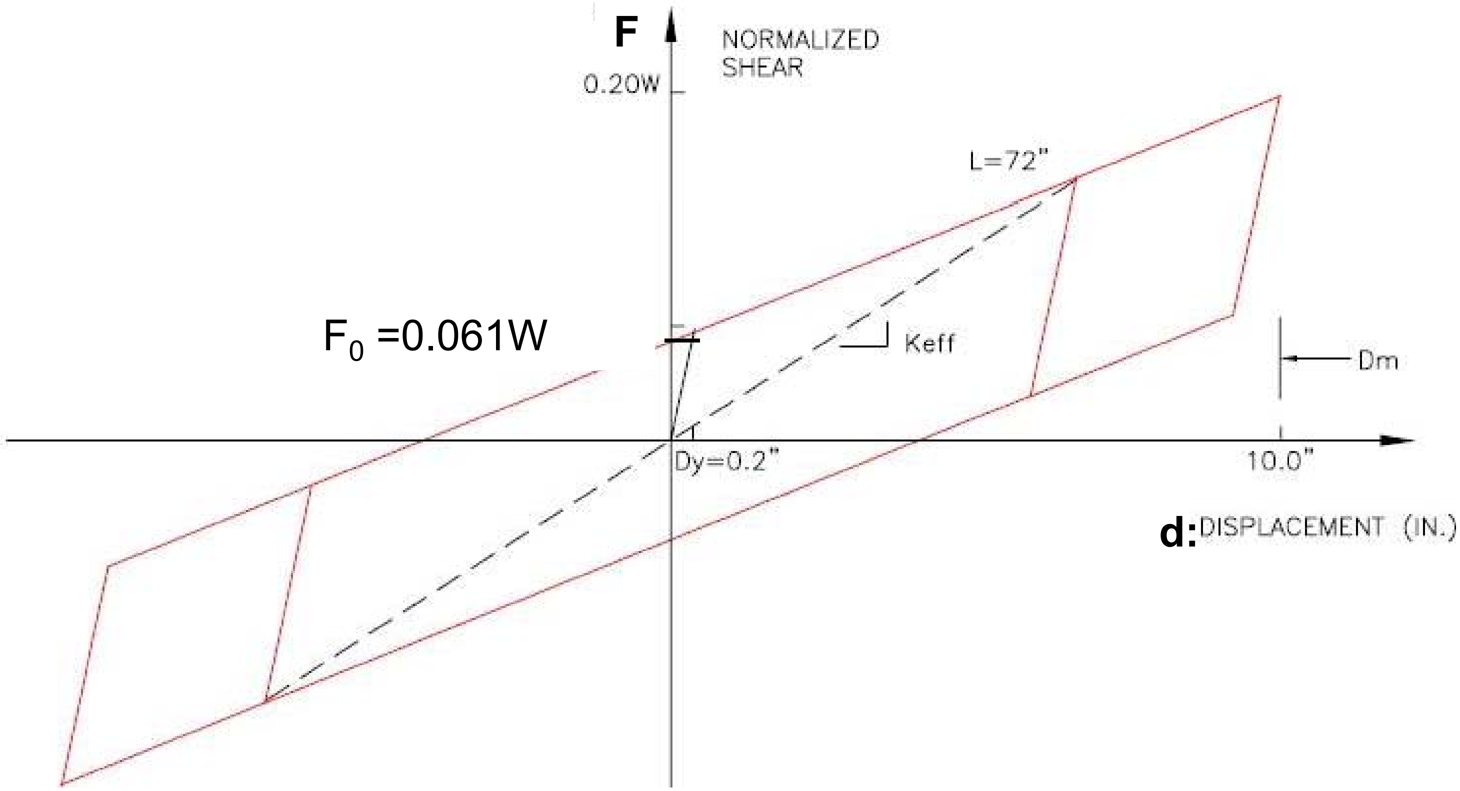
- **Ground Type B:** S=1.20, T_B=0.15s, T_C=0.5s, T_D=2.5s
- **Horizontal spectrum:** spectral amplification β_0=2.5
- **Vertical spectrum:** β_0=3.0, S=1.00, a_{vg}=0.9a_g, T_B=0.05s, T_C=0.15s, T_D=1.0s

Seismic Action – Ground Motions

- 7 Ground Motions: 2 horizontal & 1 vertical components
- Semi-artificial accelerograms from modified records
- Consistency with design spectrum:
  EN 1998-2, 3.2.3(6):



Triple FPS - Equivalent bilinear model



- **$F = W \times [\,(F_o/W) + d / R\,]$ EN1998-2, 7.5.2.3.5(2)**
  - $\mu_d = F_o/W$ = effective friction coefficient = 0.061 ± 16%
  - R = effective pendulum length = 1.83m
  - D_y = effective yield displacement = 0.005m
  - F = shear force
  - d = displacement

Design properties of isolators

Design properties of the isolating system

- Nominal design properties (NDP) assessed by prototype tests, confirming the **range** accepted by the Designer.
- Variation of design properties due to external factors (aging, temperature, contamination, cummulative travel/wear)
- Design is required for:
  - Upper Bound design properties (UBDP).
  - Lower Bound design properties (LBDP).
- Bounds of Design Properties result either from tests or from modification **λ**-factors (Annexes J & JJ) .

Isolator design properties

- **Effective pendulum length:** R=1.83m
- **Yield displacement:** D_y=0.2”=0.005m
- **Force at zero displacement F_0/W:** Upper bound design properties (UBDP) and Lower bound design properties (LBDP) in accordance with EN 1998-2, 7.5.2.4
- **Nominal value range:** $F_0/W = 0.061 \pm 16\% = 0.051 \sim 0.071$
- **LBDP:** $(F_0/W)_{min} = minDP_{nom} = 0.051$
- **UBDP:** According to EN 1998-2 Annexes J and JJ
  - <u>Minimum isolator temperature for seismic design:</u> $T_{min,b}=\psi_2 T_{min}+\Delta T_1 = 0.5 \times (-20^oC) + 5.0^oC = -5.0^oC$ where ψ_2=0.5 is the combination factor for thermal actions, T_{min}=-20°C the minimum shade air temperature at the site, ΔT_1=+5.0°C for composite deck.
  - <u>**λ_{max} factors**:</u>
    - **f1 - ageing: $\lambda_{max,f1}$=1.1 (Table JJ.1, for normal environment, unlubricated PTFE, protective seal)**

Isolator design properties

- f2 - temperature: $\lambda_{max,f2}$=1.15 (Table JJ.2 for $T_{min,b}$=-10.0°C, unlubricated PTFE)
- f3 - contamination $\lambda_{max,f3}$=1.1 (Table JJ.3 for unlubricated PTFE and sliding surface facing both upwards and downwards)
- f4 – cumulative travel $\lambda_{max,f4}$=1.0 (Table JJ.4 for unlubricated PTFE and cumulative travel ≤ 1.0 km)

- <u>Combination factor ψ_{fi}:</u> ψ_{fi}=0.70 for Importance class II (Table J.2)
- <u>Combination value of λ_{max} factors:</u> $\lambda_{U,fi}=1+(\lambda_{max,fi}-1)\psi_{fi}$ (J.5)
  - f1 - ageing: $\lambda_{U,f1}$ = 1 + (1.1 - 1) x 0.7 = 1.07
  - f2 - temperature: $\lambda_{U,f2}$ = 1 + (1.15 – 1) x 0.7 = 1.105
  - f3 - contamination $\lambda_{U,f3}$ = 1 + (1.1 – 1) x 0.7 = 1.07
  - f4 – cumulative travel $\lambda_{U,f4}$ = 1 + (1.0 – 1) x 0.7 = 1.0
- <u>Effective UBDP:</u>

  UBDP = $maxDP_{nom} \cdot \lambda_{U,f1} \cdot \lambda_{U,f2} \cdot \lambda_{U,f3} \cdot \lambda_{U,f4}$ (J.4)

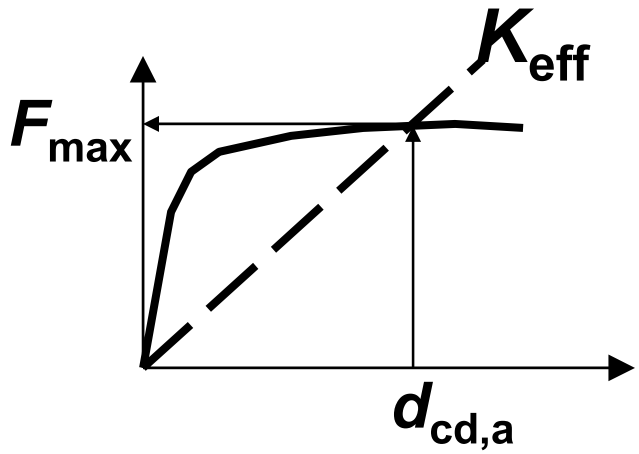
  $(F_0/W)_{max}$ = 0.071 x 1.07 x 1.105 x 1.07 x 1.0 = 0.09

Fundamental mode spectrum analysis

Estimation of $d_{max} = d_{cd}$ from dynamic equilibrium

- Assume $d_{cd,a}$
- From static *F-d relation* : F_{max}

$$K_{eff} = F_{max}/d_{cd,a}$$



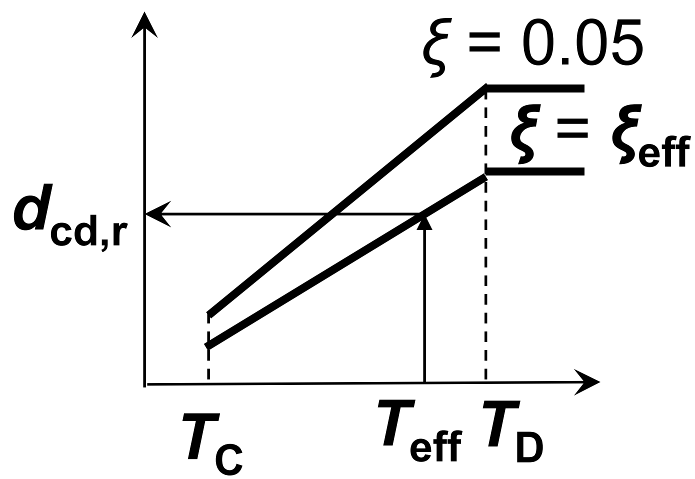
- Effective period and damping

$$T_{eff} = 2\pi\sqrt{\frac{M}{K_{eff}}} \qquad \xi_{eff} = \frac{1}{2\pi}\frac{\Sigma E_{D,i}}{K_{eff}d_{cd}^{2}}$$

- Displacement spectrum

$$d_{0,05} = a_{0,05}\left(\frac{T}{2\pi}\right)^2 \qquad d_{\xi eff} = \eta \cdot d_{0,05}$$

$$\eta = \sqrt{\frac{0,10}{0,05+\xi_{eff}}} \geq 0,4$$



- Iterations for $d_{cd,r} = d_{\xi eff} = d_{cd,a}$

Fundamental Mode analysis (LBDP)

Fundamental mode analysis (EN1998-2, 7.5.4)

- **Seismic weight:** W= 36751kN (see loads)
- **Assume value for design displacement:**
  - Assume d_{cd}=0.15m
- **Effective Stiffness of Isolation System (ignore piers):**
  - $K_{eff} = F / d_{cd} = W \times [(F_0/W) + d_{cd} / R] / d_{cd} =$
    $36751kN \times [0.051+0.15m/1.83m] / 0.15m =$
    $\Rightarrow K_{eff} = 32578$ kN/m
- **Effective period of Isolation System: eq. (7.6)**

$$T_{eff} = 2\pi\sqrt{\frac{m}{K_{eff}}} = 2\pi\sqrt{\frac{(36751kN/9.81m/s^2)}{32578kN/m}} = 2.13s$$

- **Dissipated energy per cycle: EN1998-2, 7.5.2.3.5(4)**
  - $E_D = 4 \times W \times (F_0/W) \times (d_{cd}-D_y) =$
    $4 \times 36751kN \times (0.051) \times (0.15m-0.005m)$
    $\Rightarrow E_D = 1087.09$ kNm

Fundamental Mode analysis (LBDP)

- **Effective damping: eq. (7.5), (7.9)**
  - $\xi_{eff} = \Sigma E_{D,i} / [2 \times \pi \times K_{eff} \times d_{cd}^2] = 1087.09\text{kNm} / [2 \times \pi \times 32578\text{kN/m} \times (0.15\text{m})^2] = 0.236$
  - $\eta_{eff} = [0.10 / (0.05 + \xi_{eff})]^{0.5} = 0.591$
- **Calculate design displacement d_{cd} (EN 1998-2 Table 7.1)**
  - $d_{cd}=(0.625/\pi^2) \times a_g \times S \times \eta_{eff} \times T_{eff} \times T_C = (0.625/\pi^2) \times (0.40 \times 9.81\text{m/s}^2) \times 1.20 \times 0.591 \times 2.13\text{s} \times 0.50\text{s} = 0.188\text{m}$
- **Check assumed displacement**
  - Assumed displacement 0.15m
  - Calculated displacement 0.188m

$\Rightarrow$ **Do another iteration**

Fundamental Mode analysis (LBDP)

- **Assume value for design displacement:**
  - Assume d_{cd}=0.22m
- **Effective Stiffness of Isolation System (ignore piers):**
  - K_{eff} = F / d_{cd} = W x [(F_0/W) + d_{cd} / R] / d_{cd} = 36751kN x [0.051+0.22m/1.83m] / 0.22m =
    $\Rightarrow$ K_{eff} = 28602 kN/m
- **Effective period of Isolation System: eq. (7.6)**

$$T_{eff} = 2\pi\sqrt{\frac{m}{K_{eff}}} = 2\pi\sqrt{\frac{(36751kN/9.81m/s^2)}{28602kN/m}} = 2.27s$$

- **Dissipated energy per cycle: EN1998-2, 7.5.2.3.5(4)**
  - E_D = 4 x W x (F_0/W) x (d_{cd}-D_y) = 4 x 36751kN x (0.051) x (0.22m-0.005m)
    $\Rightarrow$ E_D = 1611.90 kNm

Fundamental Mode analysis (LBDP)

- **Effective damping: eq. (7.5), (7.9)**
  - $\xi_{eff} = \Sigma E_{D,i}$ / [2 x π x K_{eff} x d_{cd}^2] = 1611.90kNm / [2 x π x 28602kN/m x $(0.22m)^2$] = 0.1853
  - $\eta_{eff} = [0.10 / (0.05 + \xi_{eff})]^{0.5} = 0.652$
- **Calculate design displ. d_{cd} (EN1998-2 Table 7.1)**
  - $d_{cd}=(0.625/\pi^2)$ x a_g x S x η_{eff} x T_{eff} x T_C = $(0.625/\pi^2)$ x (0.40 x 9.81m/s²) x 1.20 x 0.652 x 2.27s x 0.5s = 0.22m
- **Check assumed displacement**
  - Assumed displacement 0.22m
  - Calculated displacement 0.22m

  ⇒ **Convergence achieved**
- **Spectral acceleration S_e (EN 1998-2 Table 7.1)**
  - S_e= 2.5 x (T_C/T_{eff}) x η_{eff} x a_g x S = 2.5 x (0.5s/2.27s) x 0.652 x 0.40g x 1.20 = 0.172g
- **Isolation system shear force (EN 1998-2 eq. 7.10)**
  - V_d= K_{eff} x d_{cd} = 28602 kN/m x 0.22m = 6292kN

Fundamental Mode analysis (UBDP)

- **Fundamental mode analysis: Summary of results**
  - Seismic weight: W= 36751kN (see loads)
  - Design displacement d_{cd}=0.14m
  - Effective stiffness K_{eff} = 435410 kN/m
  - Effective period T_{eff} = 1.84s
  - Dissipated energy per cycle E_D = 1799.32 kNm
  - Effective damping ξ_{eff} = 0.331, η_{eff} = 0.512
  - Spectral acceleration S_e= 0.166g
  - Isolation system shear force V_d= 6096kN
- **Pier shear forces for averaged vertical load**
  - Abutments C0, C3: V_d= 570kN
  - Piers P1, P2: V_d= 2480kN

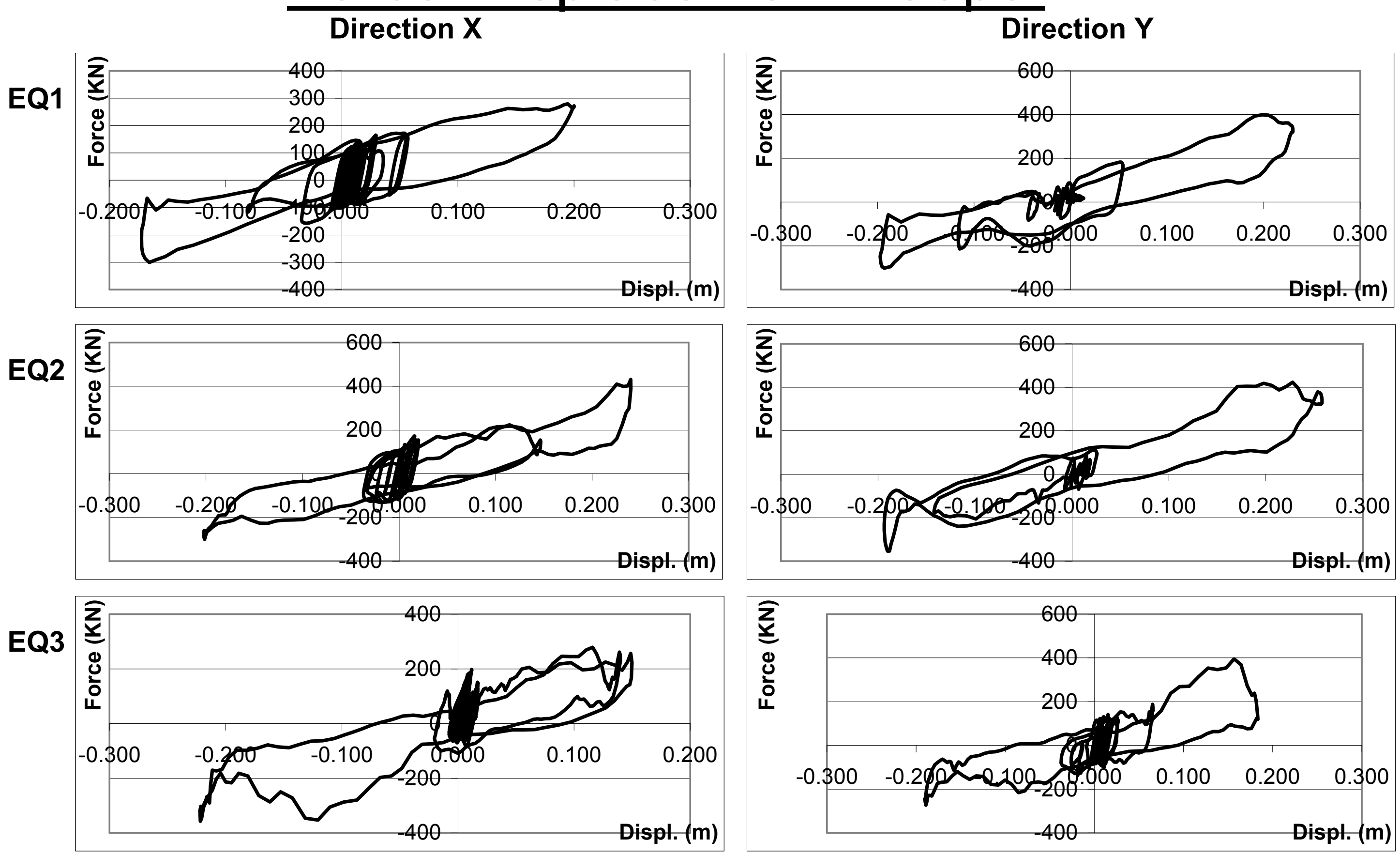
Comparison with non-isolated bridge

- **Non-isolated bridge with fixed bearings at pier top**
  - Period: longitudinal T_x = 0.33s, transverse T_y= 0.17s
  - Behaviour factor q (EN1998-2, Table 4.1):
    q = 3.5 x $\lambda(\alpha_s)$, where $\alpha_s = L_s/h$ the shear ratio
    Long. q_x=3.5x1.0=3.5, Transverse: q_y=3.5x0.82= 2.87
  - Spectral acceleration in constant acceleration branch of spectrum S_e= 2.5Sa_g/q
    Longitudinal: S_e = 2.5x1.2x0.40g/3.5 = 0.34g$\Rightarrow$V=12495kN
    Transverse: S_e = 2.5x1.2x0.40g/2.87 = 0.42g$\Rightarrow$V=15435kN
- **Isolated bridge with UBDP**
  - Spectral acceleration S_e= 0.166g $\Rightarrow$ V= 6096kN
  - Reduction of forces with respect to non-isolated:
    - at 49% in longitudinal direction
    - at 40% in transverse direction

Non linear Time-History Analysis

- **Newmark constant acceleration method**
- **Rayleigh damping**
- **Bouc-Wen model for FPS isolators**

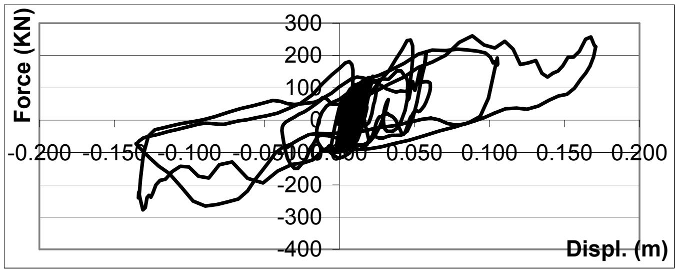
<u>Force-Displacement Loops:</u>

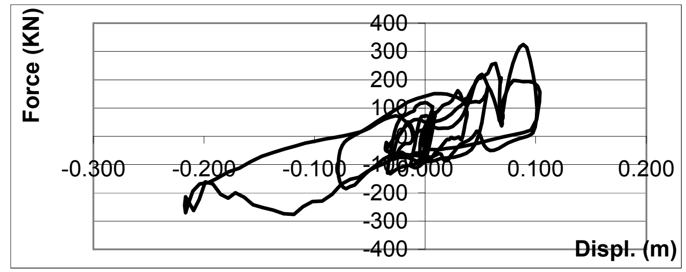


Non linear Time-History Analysis

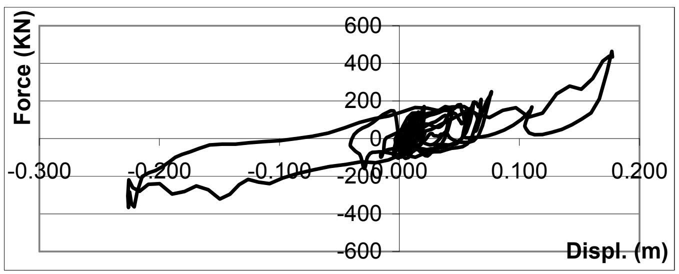
Force-Displacement Loops (continued):

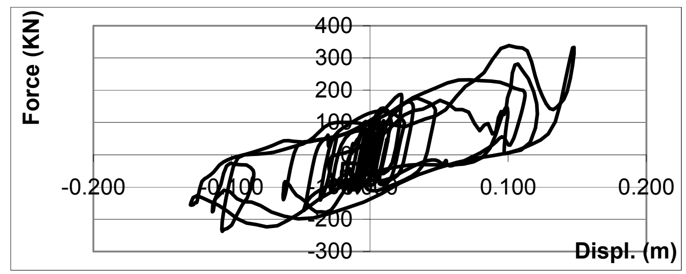
EQ4



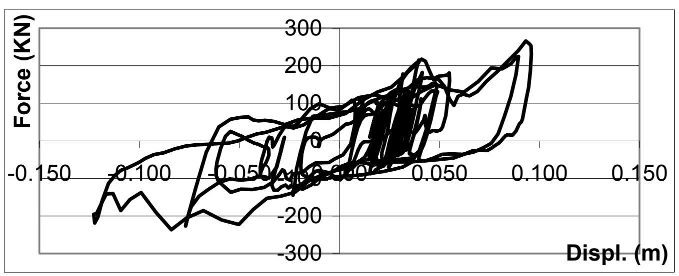


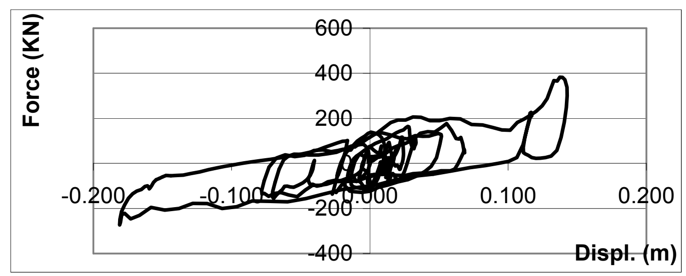
EQ5



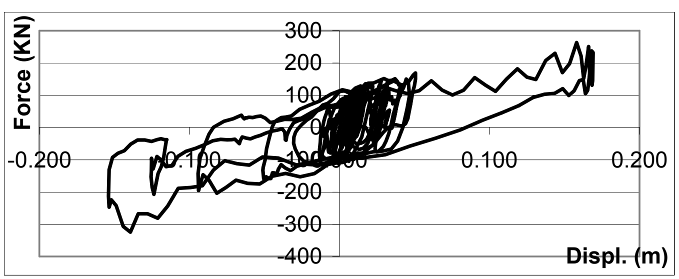


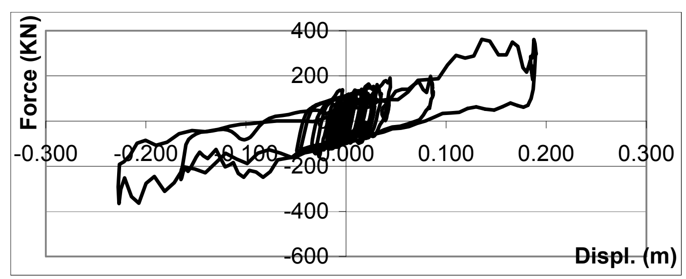
EQ6





EQ7





Check of Lower Bound Action Effects

- **Lower bound of action effects = 80% of corresponding Fundamental Mode method results (EN 1998-2, 7.5.6(1) & 7.5.5(6):**
- **Applicable for design displacement d_{cd} and total shear force V_d**

- Displacement in X direction: $\rho_d = d_{cd} / d_f$ = 0,193m / 0,22m = 0,88 > 0,80 $\Rightarrow$ ok
- Displacement in Y direction: $\rho_d = d_{cd} / d_f$ = 0,207m / 0,22m = 0,94 > 0,80 $\Rightarrow$ ok
- Total shear in X direction: $\rho_v = V_d / V_f$ = 6929,3kN / 6292kN = 1,10 > 0,80 $\Rightarrow$ ok
- Total shear in Y direction: $\rho_v = V_d / V_f$ = 6652,1kN / 6292kN = 1,06 > 0,80 $\Rightarrow$ ok

Bridges with Seismic Isolation

Compliance criteria

- **Isolating system:**

Increased reliability is required for the isolating system

Seismic displacements increased by factor:

$$\gamma_{IS} = 1.50$$

- Sufficient lateral rigidity under service conditions is required.
- Adequate self-restoring capability

- **Substructure**

Design for limited ductile behaviour: $q \leq 1.50$

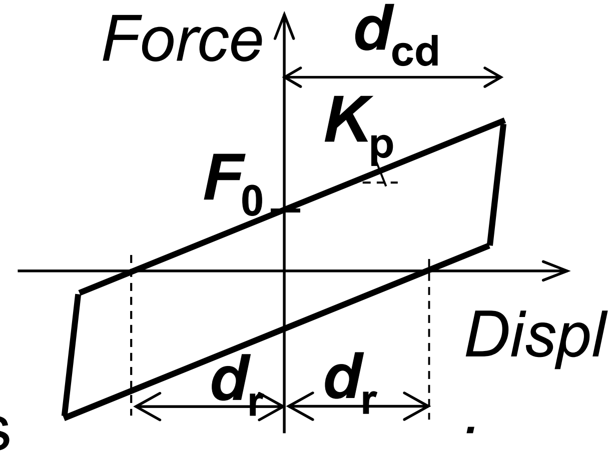
Displacement demand of isolators

- **Max. total displacement EN 1998-2, 7.6.2(1) &(2):**
  $d_{tot} = \gamma_{IS} \times d_{bi,d} + d_{o,i}$
  - $d_{bi,d}$ = design displacement of isolator i
  - γ_{IS} = 1.50 seismic displacement amplification factor
  - $d_{o,i}$ = offset displacement due to permanent actions, long term actions, 50% of thermal action

- **Abutment C0 bearings:**
  - Longitudinal: d_m = 1.50 x 193mm + 25.5mm = 315mm
  - Transverse: d_m = 1.50 x 207mm = 311mm

- **Pier P1 bearings:**
  - Longitudinal: d_m = 1.50 x 188mm + 14.5mm = 208mm
  - Transverse: d_m = 1.50 x 193mm = 290mm

Lateral restoring capability



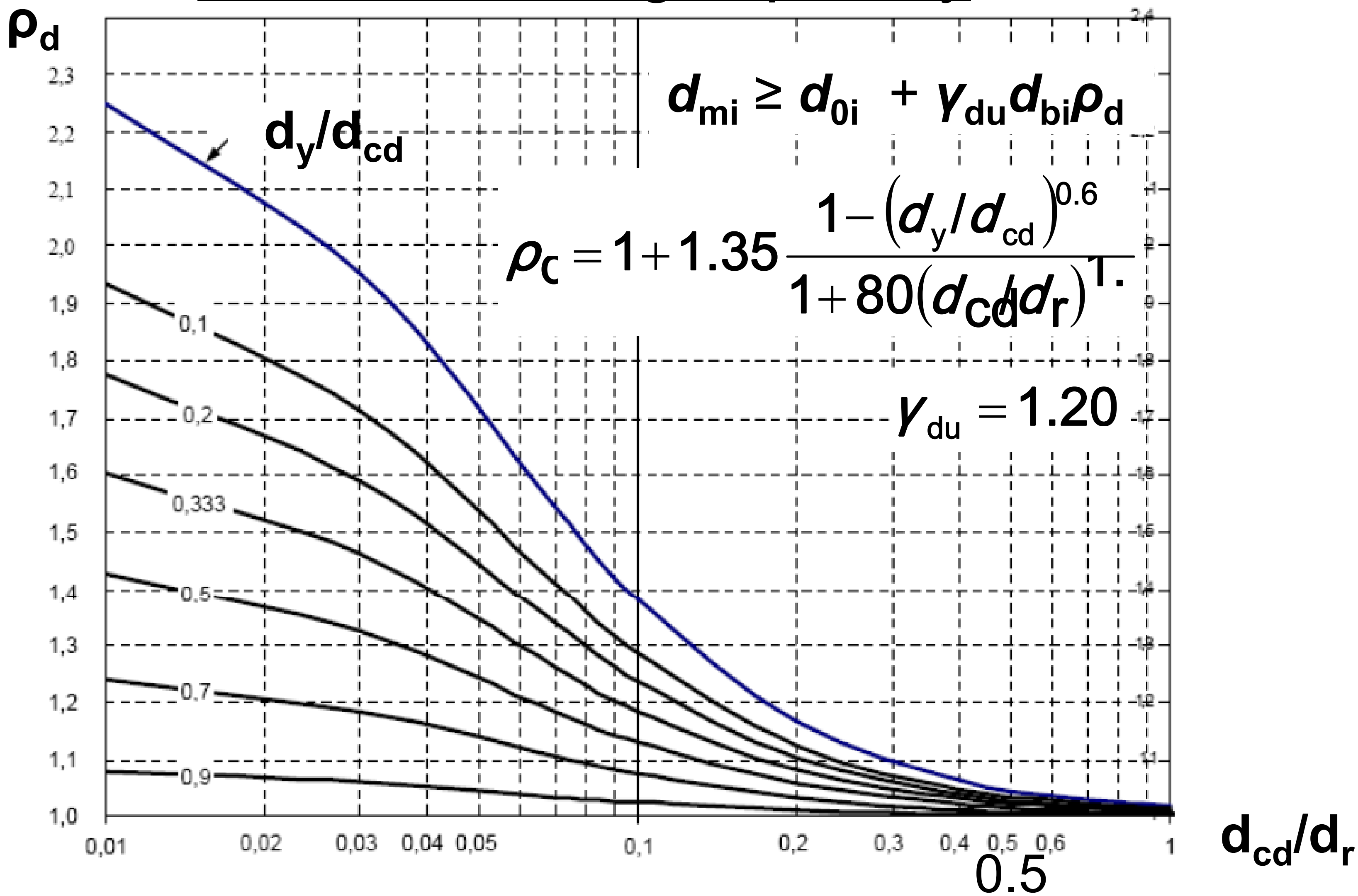
EN 1998-2:2005 + Amendment A1:2009, 7.7.1

- **Check ratio $d_{cd}/d_r \geq 0.5$ (§7.7.1)**
  - UPDP give most unfavorable results
  - Post-elastic stiffness K_p = W/R
  - Force at zero displacement $F_0 = W \times (F_0/W)$
  - Maximum static residual displacement d_r
    $d_r = F_0 / K_p = W \times (F_0/W) / (W/R) = (F_0/W) \times R = 0.09 \times 1.83m = 0.165m$
  - Check ratio d_{cd}/d_r = 0.139m / 0.165m = 0.84 > 0.5

⇒ **Adequate lateral restoring capability without increase of displacement demand (EN1998-2, 7.7.1(2))**

Bridges with Seismic Isolation

Lateral restoring capability



$$d_{mi} \geq d_{0i} + \gamma_{du} d_{bi} \rho_d$$

$$\rho_C = 1 + 1.35 \frac{1-(d_y/d_{cd})^{0.6}}{1+80(d_{cd}/d_r)^{1.}}$$

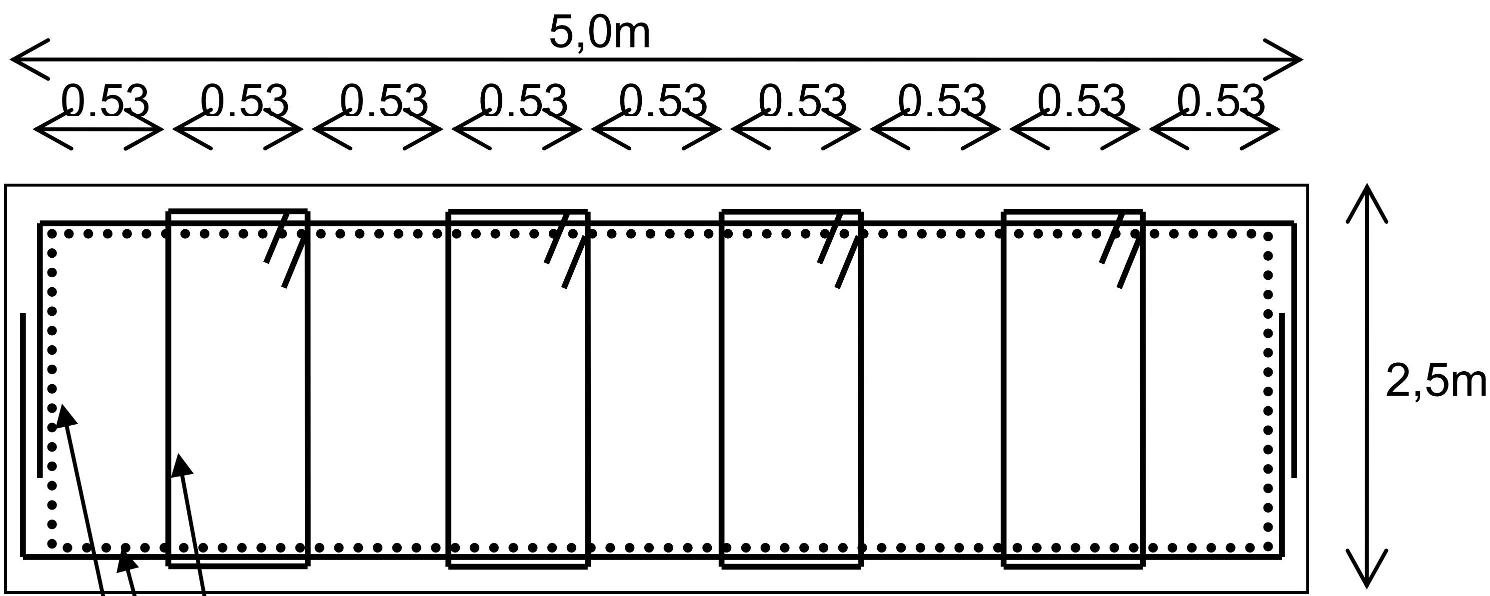
$$\gamma_{du} = 1.20$$

Design of Piers

- **Action effects for pier design: EN 1998-2, 7.6.3(2)**
- **Flexural design: q-factor corresponding to limited ductile / essentially elastic behaviour, i.e. $q \leq 1,50$**
- **Confinement reinforcement not required when $M_{Rd} / M_{Ed} < 1,30$**
- **Shear design with $q = 1$ and additional safety factor γ_{Bd1}=1,25 (EN 1998-2, 5.6.2(2)P)**
- **Minimum longitudinal reinforcement to avoid brittle failure: ρ ≈ 0.5% in total**

Design of Piers

- **Provided reinforcement:**

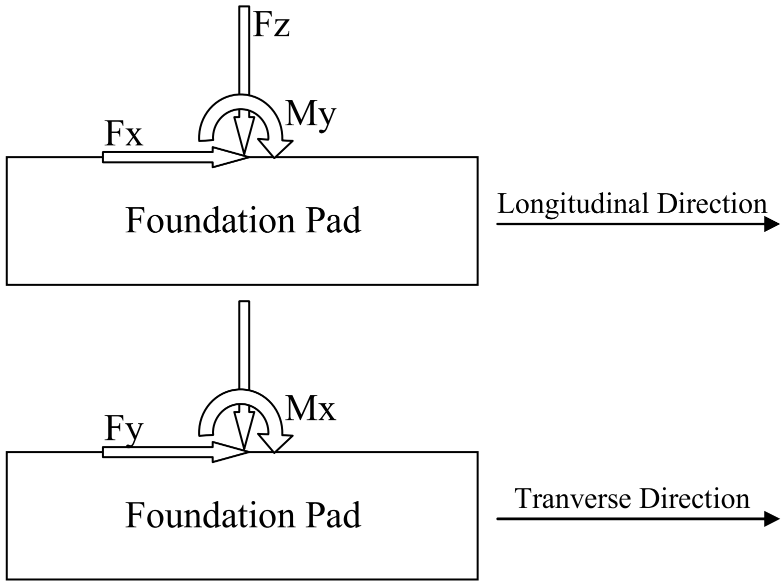


Stirrups : 4 two-legged Φ12/15 = 4 x 2 x 7,54cm^2/m = 60,3 cm^2/m

Perimetric Hoop: 1 two-legged Φ16/15 = 2 x 13,40cm^2/m = 26.8 cm^2/m

Longitudinal reinforcement: 1 layer Φ28/13,5 = 45,6 cm^2/m

Action Effects on Foundation



- **Action effects for foundation design: EN 1998-2, 7.6.3(4)P and 5.8.2(2)P for bridges with seismic isolation.**
- **Analysis results multiplied by the q-factor used (i.e. effectively using *q* = 1).**

Location	Envelope	Fx (kN)	Fy (kN)	Fz (kN)	Mx (kNm)	My (kNm)	Mz (kNm)
C0, C3	Max Fx envelope	783	111	4242	329	78	57
	Max Fy envelope	470	695	4124	2003	47	191
P1, P2	Max My envelope	3625	162	15110	2070	32494	74
	Max Mx envelope	1095	2624	16394	33331	10950	747

Special Features of FPS isolators

- Isolator horizontal forces are proportional to the vertical isolator load.

► Minimization of horizontal eccentricities

- Both the inertial and the isolator horizontal forces are proportional to the mass.

► Motion characteristics (period, displacement acceleration etc) are ~ independent of the mass

- Vertical seismic motion causes short period positive and negative variation of vertical isolator load

► May lead to an increase of maximum forces

Special Features of FPS isolators

- **Horizontal seismic components cause continuous coupling of horizontal eccentricities of the isolators**
- ► **Coupled isolator model should be used for T.H. non linear analysis**
- ► **Increase displacement demand of FMM as a stand alone analysis**
  - **Assuming 0.5d_{cd} to occur *simultaneously* transverse to max d_{cd}, as estimated by the FMM, the displacement demand should be 1.15d_{cd}**
  - **Increase max forces b the same factor.**

Comparison of TH and FMM analyses

Seismic displacement and shear demand at the abutments

Method of analysis	**Displacement demand** (mm)	**Total shear in longitudinal direction** (kN)	**Total shear in transverse direction** (kN)
Time-history analysis	393	783	695
Fundamental Mode Method (FMM)	405	683	683

Other subjects covered by EN 1998-2

Main design issues covered by EN 1998-2, not dealt with in the present:

- Non linear analysis of bridges: Static (push-over) and dynamic (time history)
- Spatial variability of the seismic action (for long bridges)
- Hydrodynamic interaction for immersed piers
- Verification of joints adjacent to plastic hinges
- Design rules for bearings, holding down devices and shock transmission units
- Abutments and culverts with large overburden

Thank you !!!