Probabilistic analysis of settlements under a pile foundations of a road bridge pylon

Karol Winkelmann, Kamil Żyliński, Jarosław Górski

Soils and Foundations 2020

HOSTED BY
ELSEVIER

Available online at www.sciencedirect.com

ScienceDirect

Soils and Foundations xxx (xxxx) xxx

www.elsevier.com/locate/sandf

Technical Paper

Probabilistic analysis of settlements under a pile foundation of a road bridge pylon

Karol Winkelmann [a,*], Kamil Żyliński [a,b], Jarosław Górski [a]

[a] *Faculty of Civil and Environmental Engineering, Gdańsk University of Technology, Narutowicza 11/12, Gdańsk 80-233, Poland*
[b] *Industria Engineering, Azymutalna 9, Gdańsk 80-298, Poland*

Received 20 April 2020; received in revised form 6 November 2020; accepted 12 November 2020

The paper addresses **the reliability change of a road bridge pile foundation due to the unpredictable increase of settlements in time**.

The analysis is based on the **Rędziński Bridge in Wrocław**, Poland, its design assumptions, and **monitoring results**.

The bridge foundation rests on a **multi-layered subsoil assumed random**.

To simplify the probabilistic approach, **substitute soil strata stiffness** parameters are adopted.

Tracing their time decrement allows for a **comprehensive definition of the entire foundation over-settlement** produced by numerous factors.

The Serviceability Limit State helps to assess **the foundation reliability index**, compared with the condition in the EN 1990:2002+A1:2005 standard.

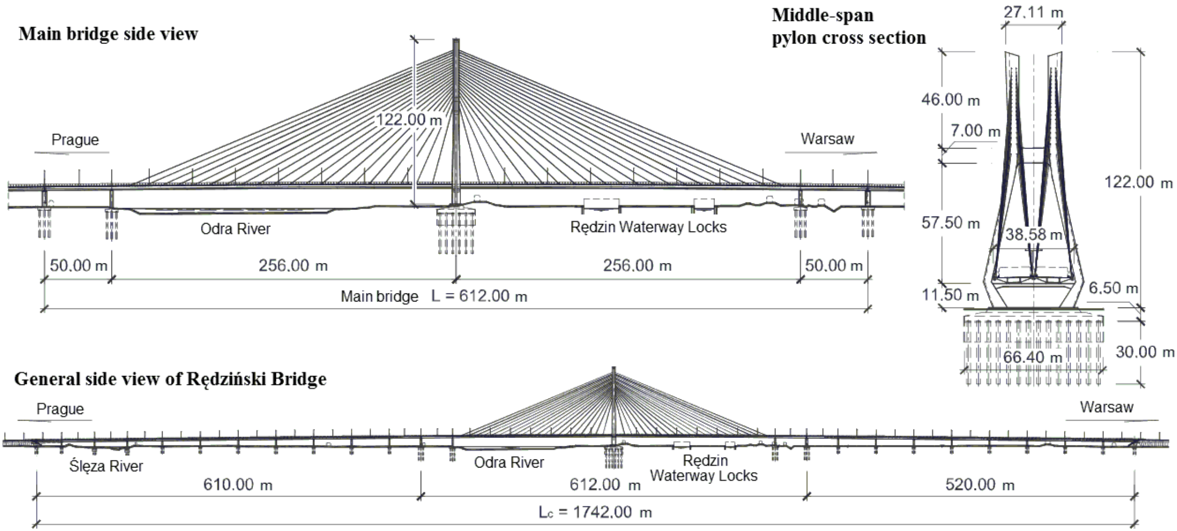
Real-life settlements are measured in the first year of bridge operation, they **are used to calibrate the reliability index assessment**.

The real-life survey database of settlements makes it possible **to validate the results** of probabilistic calculations.

A dedicated flowchart is devised to support further analysis of a wide structural domain.

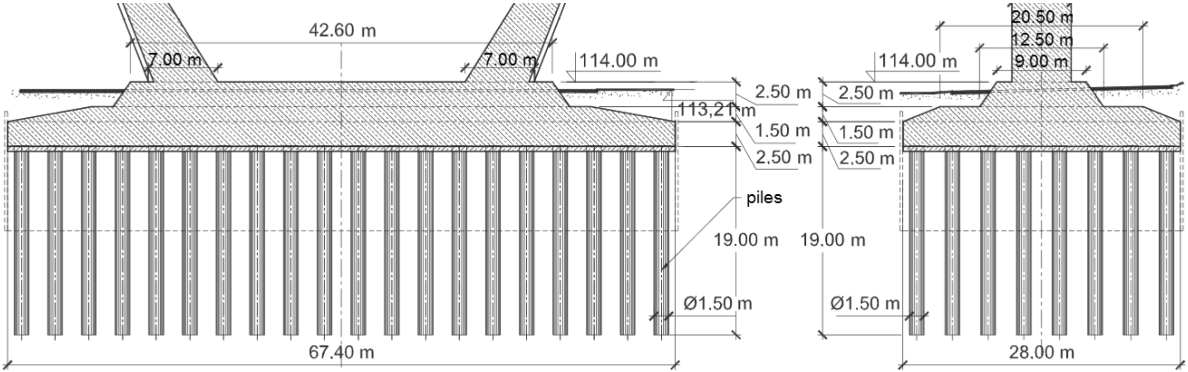
The analysed pile foundation of a road bridge pylon

The investigated **pile foundation** is a part of the second-largest cable-stayed bridge in Poland, the **Rędziński Bridge in Wrocław** (Biliszczuk et al., 2012).



The bridge superstructure consists of two separate prestressed concrete box girders, suspended to an pylon.

The H-shaped hybrid pylon foundation is a concrete massive slab, its base dimensions are 67.4×28.0 m with variable thickness (2.5 to 6.5 m), *supported on 160 reinforced concrete piles* (8 rows of 20 piles, in a 3.4×3.6 m rectangular grid), each 18.0 m long and 1.5 m in diameter.



The subsoil comprises of seven distinct layers.

The layer (IIa/IIb) of normally consolidated river accumulations lies on three layers (IIIa, IIIb, IIIc) of dense coarse material, with an unconfined water table at the elevation of 107.5 m a.s.l.
Below them, a layer (Va) consisting of fine soils (clays, silty sands) with local water percolations is observed.

It contains two thin lentils (Vc) of silty sands – both with a confined water table under high pressure situated on top of them (at 89.0 m a.s.l. and 71.0 m a.s.l., respectively.

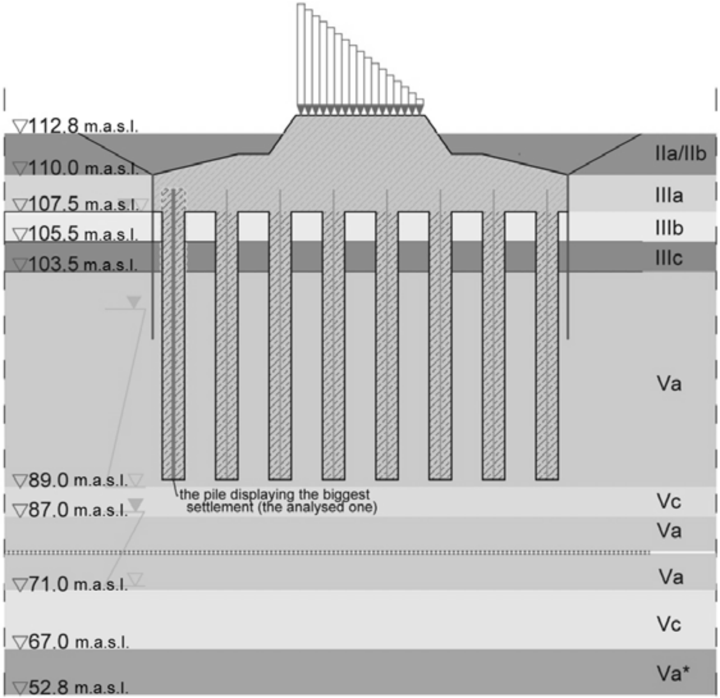
The Va layer is classified as the most relevant one, due to its high thickness (in two appearance levels) as well as lower stiffnesses and strength parameters than those of the grained soils of IIIa, IIIb, IIIc, and Vc layers.

Moreover, the geotechnical conditions of the **Va layer** are compromised by **local water percolations and high water pressures in confined aquifers**.

Layer	Soils present	Top	Bottom
IIa, IIb	clay, loamy clay, loam	112.8 m.a.s.l.	110.0 m.a.s.l.
IIIa	fine, medium and coarse sands	110.0 m.a.s.l.	107.5 m.a.s.l.
IIIb	medium and coarse sands with gravel mixes	107.5 m.a.s.l.	105.5 m.a.s.l.
IIIc	gravel mix, gravel and coarse-sands with gravel mixes	105.5 m.a.s.l.	103.5 m.a.s.l.
Va	loam, silty loam, silty boulder clay, boulder clay	103.5 m.a.s.l.	89.0 m.a.s.l.
Vc	silty sand	89.0 m.a.s.l.	87.0 m.a.s.l.
Va*	loam, silty loam, silty boulder clay, boulder clay	87.0 m.a.s.l.	---

The FE model of the pile foundation - ZSoil® system

Two numerical models of the foundation were generated to represent two cross-sections (both separately analysed).



The numerical models were implemented in a plane strain regime, introducing **stiffness corrections in order to automatically account for pile spacing**.

Quadrilateral, 4-node continuous elements were applied to form a **two-dimensional (2-D) soil FE mesh**.

The **piles were modelled as beam elements**, rigidly fastened to the slab.

Their weight was assumed to reflect the difference between the specific weight of concrete of the piles and soil specific weight in the pile contact zones.

The numerical routines incorporate the augmented **Mohr-Coulomb (M-C) model** and a **higher-order Hardening Soil (HS) model**.

The M-C constitutive model was applied for three top unsaturated, low-cohesive strata IIa, IIb, and IIIa.

These layers are situated over the pile-slab connection level, thus they may be identified as an indirect form of a dead load.

The explicit HS model predicts precisely the structure-soil interaction by capturing the stress-strain relation in the subsoil and acquiring a wide database on soil plastic behaviour (including soil dilatancy and yield cap).

The HS model was chosen here to analyse strata IIIb, IIIc, Va, Va*, and Vc, situated under the foundation bottom.

The values of HS model reference stiffness parameters were adopted on the basis of **the tri-axial tests** on small strain stiffness moduli **performed by (Dembicki et al., 2013)**.

Layer	Volum. weight γ / γ' [kN/m^3]	Effect. internal friction angle ϕ' [°]	Effect. cohesion c' [kPa]	Oedom. modulus M_0 [kPa]	Poisson's ratio ν [-]	Reference oedom. modulus M_o^{ref} [kPa]	Sec. mod. at 50% of max. dev. stress E_{50}^{ref} [kPa]	Secant modulus for soil loading E_{ur}^{ref} [kPa]	Reference precon-solidation p^{ref} [kPa]	Reference mod. depend. on stress level m [-]
IIa	21.0/11.0	15.0	5.0	30 000	0.20	---	---	---	---	---
IIb	21.0/11.0	15.0	5.0	30 000	0.20	---	---	---	---	---
IIIa	19.0/10.0	33.0	1.0	85 000	0.20	---	---	---	---	---
IIIb	20.0/10.0	35.0	1.0	150 000	0.15	140 000	110 000	250 000	100	0.5
IIIc	20.0/10.0	35.0	1.0	220 000	0.15	150 000	100 000	300 000	100	0.5
Va	21.5/11.5	23.0	18.0	40 000	0.20	40 000	35 000	100 000	100	0.5
Va*	21.5/11.5	23.0	18.0	100 000	0.20	100 000	70 000	200 000	100	0.5
Vc	20.5/11.0	32.0	1.0	85 000	0.15	85 000	75 000	150 000	100	0.5

Hydrogeological conditions are modelled as two water levels in the layer Vc.

The soil pore pressure is taken into account directly in the calculation of the effective stresses of the subsoil.

In all layers, the zero **dilatancy angle** was assumed ψ=0°.

The preconsolidation ratio (OCR) was taken 1.0 for layers IIIb and IIIc and 2.0 for layers Va, Va* and Vc, in accordance with **(Dembicki** et al., 2013).

The concrete of foundation slab and piles was adopted as linearly elastic with the following material parameters: Young's modulus $E_c = 30 \cdot 10^7$ kPa, Poisson's ratio $\nu = 0.1$, and specific weight $\gamma_c = 24\,\mathrm{kN/m^3}$.

Nine construction stages were analysed in the FE model:

the initial scenario of a flat surface at 112.8 m a.s.l., the excavation to 110.0 m a.s.l., the sheet wall driving, the deeper excavation to 107.0 m a.s.l., the driving of the piles, the casting and hardening of the concrete, the introduction of dead loads of the pylon and the deck, and the introduction of variable traffic loads in all most unfavourable combinations.

In the FE model, **the stiffnesses and specific weights of groups of 1-D or 2-D finite elements are modified** to reflect the operations performed in subsequent construction stages – either reduced to infinitesimal values if excavations are carried out to match the concrete parameters if concrete works are conducted.

The addition of the sheet walls and piles is simulated by introducing appropriate 1-D FE with set stiffnesses and specific weights.

The analysis of **the most unfavourable dead and live loads characteristic combination** is necessary to determine the final construction stage.

The database on the combination, i.e. the dead and live loads, the envelopes of support reactions, and key internal forces were acquired from the outlines of Rędziński Bridge designers, presented in e.g. (**Biliszczuk** et al., 2014; 2016).

The most unfavourable bulk pressure function $q(d)$ was determined:

$$q(d) = -2.8587d + 26.79 \ \text{[MN/m]} \tag{1}$$

where d [m] denotes the distance from the left-side edge of the pylon.

The work addresses t**he assessment of key settlements of the pile-foundation subsystem – vertical displacements at the pile head level**.

The displacements and rotations of the foundation indicate a **rigid motion of the slab**; its bottom surface remains flat in the deformed configuration.

Displacement identification is based on a simple data extrapolation from two distinct FE models (representing both cross-sections) to each point on the foundation bottom.

The left-corner pile exhibits the highest settlement of 0.080 m.

The inquiries were limited to the analysis of **the perpendicular model only**, and the assessment of **the extreme left-corner pile displacement**.

The calculated numerical settlements were confirmed by the respective survey results (0.0808 m), available from the geodetic monitoring of six nodes of the slab (at four corners and two middle nodes of longer edges).

Adoption of the input random variables

A probabilistic FE model to effectively describe the settlements of a structural foundation over a period of several years is created.

The FEM software allows to analyse **a multitude of factors that affect structural settlements**, e.g. the subsoil-pile system geometry, arching, the coefficient of skin friction, pile spacing, the pile cap width, the pile to subsoil modulus ratio on the vertical stress-settlement response, and many others.

When dominant origins of foundation settlement cannot be identified**, a single substitute parameter is defined** – the E_{ur}^{ref} modulus; its time-dependent variation is bound **to reflect all factors leading to over-settlements**.

Due to a complex, geotechnical profile variation in E_{ur}^{ref} moduli **is distinguished in each respective soil strata** of the HS model.

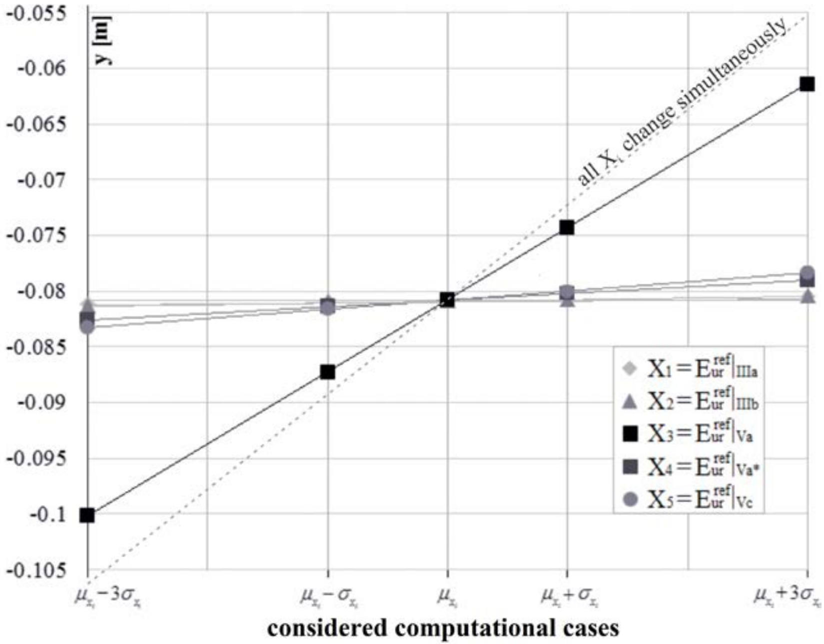
Five random variables reflecting the E_{ur}^{ref} moduli change are adopted in the task, all of them **are assumed Gaussian**.

Their **mean values** $\mu_{x_i} \equiv E_{ur}^{ref}\big|_i$ are consistent with the data from (Dembicki et al., 2013); the same **coefficient of variation** $v_{x_i} = 0.1$ was applied.

Variable	Layer	Mean value μ_{x_i} [kPa]	Variation v_{x_i}	Standard deviation σ_{x_i} [kPa]	$\mu_{x_i} - \sigma_{x_i}$	$\mu_{x_i} + \sigma_{x_i}$	$\mu_{x_i} - 3\sigma_{x_i}$	$\mu_{x_i} + 3\sigma_{x_i}$
X_1	IIIb	250000	0.1	25000	225000	275000	175000	325000
X_2	IIIc	300000	0.1	30000	270000	330000	210000	390000
X_3	Va	100000	0.1	10000	90000	110000	70000	130000
X_4	Va*	200000	0.1	20000	180000	220000	140000	260000
X_5	Vc	150000	0.1	15000	135000	165000	105000	195000

Sensitivity analysis to the soil secant elastic modulus variation

A series of numerical model simulations was performed, where **one variable was adopted with its mean value corrected** by one $x_i = \mu_{x_i} \pm \sigma_{x_i}$ or three standard deviations $x_i = \mu_{x_i} \pm 3\sigma_{x_i}$; 20 different FE samples were created.



The analysis shows an approximately **linear change of pile settlements** parameters over the entire $\pm 3\sigma_{x_i}$ range.

Displacements	Change in the layer-wise soil secant elastic moduli					
	all layers at once	only in IIIa $\overline{x_1} \mp \sigma_1$	only in IIIb $\overline{x_2} \mp \sigma_2$	only in Va $\overline{x_3} \mp \sigma_3$	only in Va* $\overline{x_4} \mp \sigma_4$	only in Vc $\overline{x_5} \mp \sigma_5$
$y_{\min}(y \text{ for } x_i - \sigma_i)$	-0.0889 m	-0.0801 m	-0.0801 m	-0.0870 m	-0.0807 m	-0.0810 m
$y_{\max}(y \text{ for } x_i + \sigma_i)$	-0.0727 m	-0.0800 m	-0.0799 m	-0.0742 m	-0.0794 m	-0.0792 m
$\Delta y = \lvert y_{\max} - y_{\min} \rvert$	$\Delta y(\Sigma x_i) =$ 0.0162 m	$\Delta y(x_1) =$ 0.0001 m	$\Delta y(x_2) =$ 0.0002 m	$\Delta y(x_3) =$ 0.0128 m	$\Delta y(x_4) =$ 0.0013 m	$\Delta y(x_5) =$ 0.0018 m
α_i	-----	0.62 %	1.23 %	79.01 %	8.02 %	11.12 %

The **percentage impact factors** of respective variables on the settlement

$$\alpha_i = \frac{\Delta y(x_i)}{\Delta y(\Sigma x_i)} \times 100\% \qquad (2)$$

The sensitivity analysis **indicates the** $X_3 = E_{ur}^{ref}\big|_{Va}$ **variable of the greatest impact** on overall displacements of the foundation.

Variables $X_4 = E_{ur}^{ref}\big|_{Va*}$ and $X_5 = E_{ur}^{ref}\big|_{Vc}$ show a **lower impact** here than X_3; the influence of higher strata, $X_1 = E_{ur}^{ref}\big|_{IIIa}$ and $X_2 = E_{ur}^{ref}\big|_{IIIb}$ **is negligible**.

Upon comparing the obtained α_i values, **a possible reduction of the number of basic random variables of the problem seems possible**.

Thus in reliability assessment, t**wo cases were initially regarded for comparative purposes**: adopting either **all five variables** or **only the three variables** with the biggest impact on settlements (X_3, X_4, X_5).

Standard SLS-based reliability assessment of a pile foundation

When designing structures, **validating all Ultimate Limit State** (ULS) and **Serviceability Limit State** (SLS) criteria is mandatory.

In the case of **bridge foundations**, verification of the **SLS becomes crucial**.

In the case of Rędziński Bridge, **the SLS limit value of** $u_{\lim} = 0.10\,\mathrm{m}$ **was applied by the designers**.

This value is not related to any structural failure (the collapse of the bridge superstructure) but **is determined by acceptable tolerance of road grade line positioning stated by national regulations**, the Technical Conditions issued by the Ministry of Infrastructure in Poland in this case.

On the basis of the deterministic analysis, **the left-edge pile was indicated as the one undergoing maximum settlements in the entire pile formation**.

Thus, **only this variate extreme key displacement** (y_{extr}) **will be referred to the limit value** $(u_{\lim})$, resulting in a simple formulation of the final SLS criterion, $y_{extr} \le u_{\lim}$.

The SLS-based reliability index calculation

The reliability analysis is conduct using **a joint approach of the Point Estimate Method (PEM) and the Response Surface Method (RSM)**.

A parallel application allows for a cross-check of the reliability estimators assessed with both approaches, **with no additional numerical cost** (RSM re-uses the same data needed to complete the PEM calculations).

The scenarios of the reliability assessment due to bridge settlements concern a total of either **all five or three variables**.

The first scenario incorporating five variables uses 32 samples in PEM computations.

A set of $2n = 2 \times 5 = 10$ samples required for PEM calculations **is identical to the ones used in standard** $(\pm\sigma_{x_i})$ **sensitivity analysis**; thus we consider only $2^n - 2n = 32 - 10 = 22$ samples linked with variability combination in further FE model calculations.

The reliability index $\beta_{PEM-5} = 2.929$ was estimated.

The same set of 32 samples **was re-used to approximate the first-order response surface**, hence no additional computations are required.

The following form of **the structural response** $\hat{y}(\mathbf{x})$ (the largest settlement of the foundation slab, displayed by its left-corner node) was estimated

$$\hat{y}(\mathbf{x})_{RSM-5} = -1.62\times10^{-1} + 4.55\times10^{-9}x_1 + 5.04\times10^{-9}x_2 + 6.46\times10^{-7}x_3 + 2.98\times10^{-8}x_4 + 5.43\times10^{-8}x_5$$

This allows for the determination of **the reliability index** $\beta_{RSM-5} = 2.946$.

A slight difference between the PEM and RSM results is observed.

While only three variables are considered the number of PEM samples equals 8; the reliability index $\beta_{PEM-3} = 2.928$.

The same set of 8 samples allows forming **the first-order response surface**

$$\hat{y}(\mathbf{x})_{RSM-3} = -1.59\times10^{-1} + 6.42\times10^{-7}x_3 + 2.96\times10^{-8}x_4 + 5.48\times10^{-8}x_5$$

which results in a **reliability index** of $\beta_{RSM-3} = 2.936$.

While no relative difference of β_{PEM-3} and β_{PEM-5} occurs, the reduction of the random problem to three variables is justified.

Thus, **the index value $\beta = \beta_{PEM-3} = 2.928$ is adopted as the final result of the SLS-based reliability assessment**.

VERIFICATION OF THE RELIABILITY INDEX ADMISSIBILITY DUE TO DESIGN STANDARDS

Similarly to the simple formulation of the SLS displacement criterion, the **reliability index verification may be also defined as** $\beta \geq \beta_{\lim}$.

Such a simple-form reliability check **is recommended by a majority of design standards**, as part of the general structural verification.

The standards define various reference β **values**, dependent on e.g. the failure consequence class, the structure execution class, and the supervision levels of both design and execution processes:
EN 1990:2002+A1:2005 (EN 1990, 2005), EN-ISO 2394:2015 (EN-ISO 2394, 2015), *fib* Model Code for Concrete Structures 2010 (*fib* MC, 2012), JCSS Probabilistic Model Code (JCSS, 2001).

They are also fully related to **discerned time periods of a planned non-failure structural operation**.

Two crucial reference periods are mostly indicated: **the 1-year (initial)** period (Serviceability Limit State verification) **and the 50-year (long-term)** period (the Ultimate Limit State verification).

The table lists **the suggested target reliability indices** for these explicit time periods presented in various standards.

Time-wise reliability check	**Calc. index β=2.928**	Considered standards defining the anticipated reliability indices				
		EN 1990 CC3	EN 1990 CC2	JCSS	*fib* MC	EN-ISO 2394
Initial (1-year period)	Adm. value	4.7	2.9	4.2	3.0	1.5
	Fulfilled?	**no**	yes	**no**	**no**	yes
Long-time (50-year period)	Adm. value	3.8	1.5	4.2	1.5	1.5
	Fulfilled?	**no**	yes	**no**	yes	yes

In the real-life design of the Rędziński Bridge, the **EN 1990:2002+A1:2005** standard was referenced, **obligatory in Poland**.

As the **Rędziński bridge** was planned a decisive element in the Wrocław transportation grid, it **was classified to CC2**.

Thus, in the initial verification (a theoretical, 1-year scenario of the bridge operation under all dead and exploitation loads), the Rędziński Bridge fulfilled its **SLS-based reliability criterion** $(\beta = 2.928 \geq \beta_{\mathrm{lim,EN\ CC2}} = 2.9)$.

The table shows that **the index does not meet the demanded level according to other standards in a 1-year period scenario**.

Time-wise prediction of SLS-based reliability variation

The settlement variation is attributed to the change in representative soil stiffness parameter E_{ur}^{ref} of each key strata.

The time-related change in the decisive three moduli $(X_3 = E_{ur}^{ref}\big|_{Va}$, $X_4 = E_{ur}^{ref}\big|_{Va*}$, $X_5 = E_{ur}^{ref}\big|_{Vc})$ yields an auxiliary **time fluctuation function** $n(t)$

This function $n(t)$ **addressed the percentage change** in the two first probabilistic moments of all three variables.

The function governs **the mean value decrease of the variables**

$$\Delta\mu_{x_i}(t) = \mu_{x_i}(1 - 0.01\, n(t)) \qquad (12)$$

and **the increase in all standard deviations of the variables**

$$\Delta\sigma_{x_i}(t) = \sigma_{x_i}(1 + 0.01\sqrt{|n(t)|}) \qquad (13)$$

where: t – time (given in days).

Each random variable x_i becomes time-related, regarding variations of mean values $\mu_{x_i}(t)$ and standard deviations $\Delta\sigma_{x_i}(t)$, by means of the $n(t)$ function.

The RSM approximation, allows the assessment of **the time-variant response**

$$\hat{y}(\mathbf{x}(t)) = b_0 + \sum_{i=1}^{n} b_i x_i(t) + \varepsilon \tag{14}$$

At each time step $t_i \in \langle 0, 365 \rangle$ [days], the settlement $y(t_i)$ taken in the form of $\Delta y(t)$ function on the basis of real settlement survey

$$y(t_i) = \Delta y(t)\Big|_{t=t_i} = b_0 + \sum_{i=1}^{n} b_i \, \Delta\mu_{x_i}(t)\Big|_{t=t_i} = b_0 + \sum_{i=1}^{n} b_i \, \mu_{x_i}(1 - 0.01\, n(t_i)) \tag{15}$$

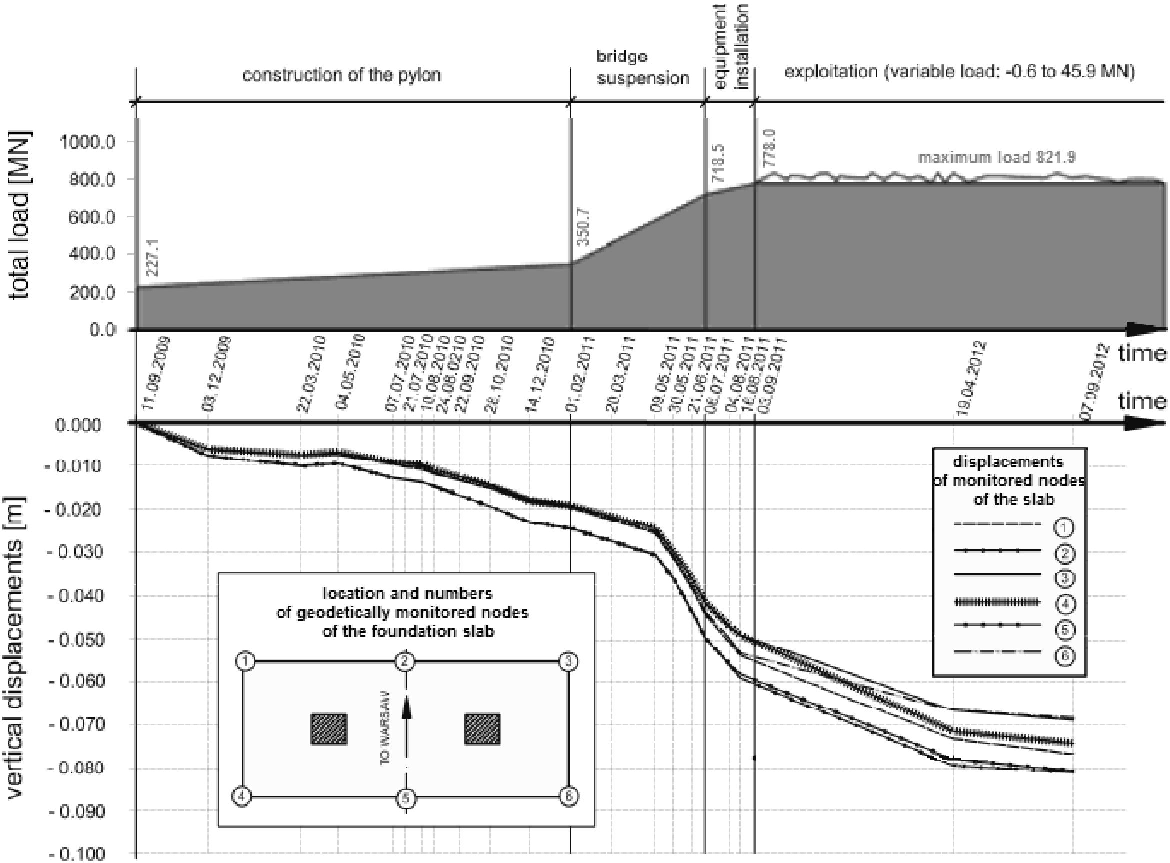
Performing the calculation for each *i*-th time step (t_i) of **the analysis allows for the global approximation of the foundation response function**.

Incorporating the **Hasofer–Lind procedures** it is possible **to forecast the future reliability index decrease**.

The procedure makes it possible **to extrapolate the trends of settlement increase or reliability decrease in subsequent years**, based on measurements made in the **first year of bridge operation**.

SLS-based reliability decrease of Rędziński Bridge in the 1-year period

The fluctuation function $n(t)$, may be determined by means of the **geodetically surveyed change** checked during the first year of the bridge operation.


construction of the pylon
bridge suspension
equipment installation
exploitation (variable load: -0.6 to 45.9 MN)
total load [MN]
maximum load 821.9
227.1
350.7
718.5
778.0
time
vertical displacements [m]
location and numbers
of geodetically monitored nodes
of the foundation slab
TO WARSAW
displacements
of monitored nodes
of the slab

On **day 0** (3rd September 2011), the largest settlement of the foundation slab (displayed by the left-edge one on the perpendicular section, denoted by number 5 in Figure) was 0.0600 m, in the **day 229** it was equal to 0.0780 m, next it rose to 0.0808 m in **day 369 (ca. one year)**.

A **significant settlement change** occurs in the first year of bridge operation, it can be approximated by a cubic polynomial

$$\Delta y(t) = -1.858\times10^{-10}\, t^3 + 2.718\times10^{-7}\, t^2 - 1.329\times10^{-4}\, t \qquad (16)$$

Introducing time t into Eqn. (11)

$$\hat{y}(\mathbf{x}(t))_{RSM-3} = -1.59\times10^{-1} + 6.42\times10^{-7}\, x_3(t) + 2.96\times10^{-8}\, x_4(t) + 5.48\times10^{-8}\, x_5(t) \qquad (17)$$

Next, substituting the real-life displacements and the $n(t)$ **function to the RS equation** (15), results in

$$y(t_i) = -1.59\times10^{-1} + (1-0.01\, n(t_i))\left[6.42\times10^{-7}\, \mu_{x_3} + 2.96\times10^{-8}\, \mu_{x_4} + 5.48\times10^{-8}\, \mu_{x_5}\right] \qquad (18)$$

Hence **the** $n(t_i)$ **is determined** according to every time step and approximated to a universal form

$$n(t) = 2.372\times10^{-7}\, t^3 - 3.469\times10^{-4}\, t^2 + 1.697\times10^{-1}\, t - 27.237 \qquad (19)$$

The settlement increase is permanent, thus it is important **to extrapolate the reliability decrement**.

Taking the time step $t = 730$ (two years from the bridge operation start) the extreme settlement is bound to increase to 0.0844 m.

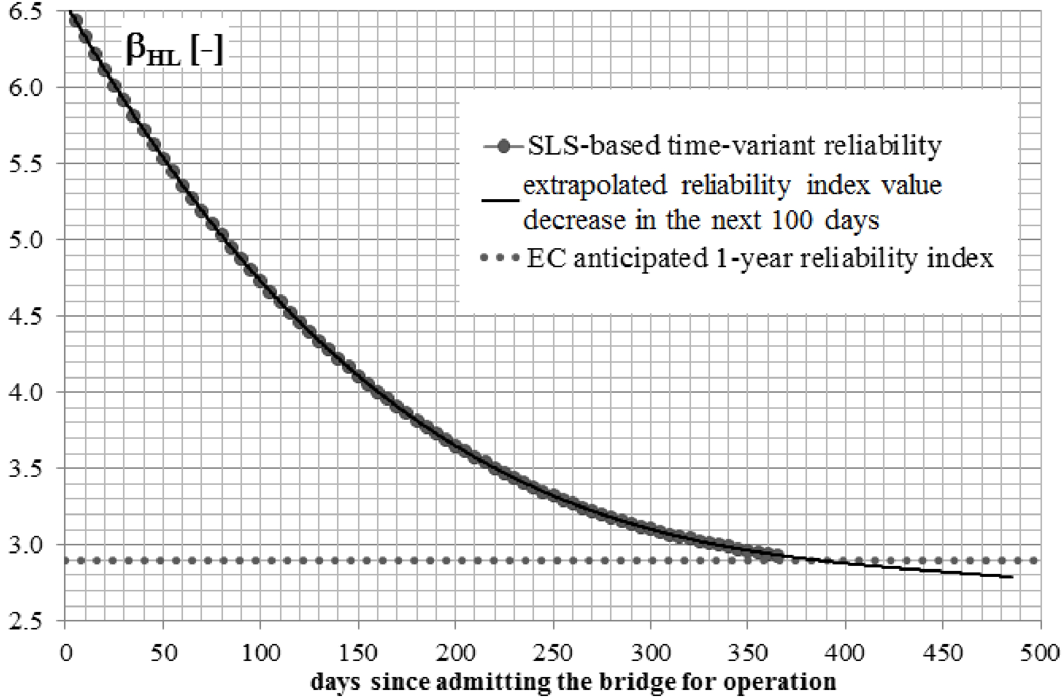
In turn, Eqn. (19) is satisfied with the value of $n(t = 730) = 4.004$.

Based on Eqn. (12), **mean secant elastic moduli** of three respective soil strata **reduce to** $\mu_{x_3}(t = 730) = 95996 \text{ kPa}$, $\mu_{x_4}(t = 730) = 191992 \text{ kPa}$, and $\mu_{x_5}(t = 730) = 143994 \text{ kPa}$, whereas according to Eqn. (13), **their standard deviations increase** to $\sigma_{x_3}(t = 730) = 10\,200 \text{ kPa}$, $\sigma_{x_4}(t = 730) = 20\,400 \text{ kPa}$, and $\sigma_{x_5}(t = 730) = 15\,300 \text{ kPa}$.

The time-variant response surface equation, given in Eqn. (17), allows for a step-wise determination of the Hasofer-Lind **time-variant reliability indices** β_{HL}, according to Eqn. (9)

$$\beta_{HL}(t)_{RSM-3} = -3.483 \times 10^{-8} t^3 + 4.755 \times 10^{-5} t^2 - 2.263 \times 10^{-2} t + 6.554 \qquad (20)$$

Variation in the reliability index β_{HL} is displayed in the figure.

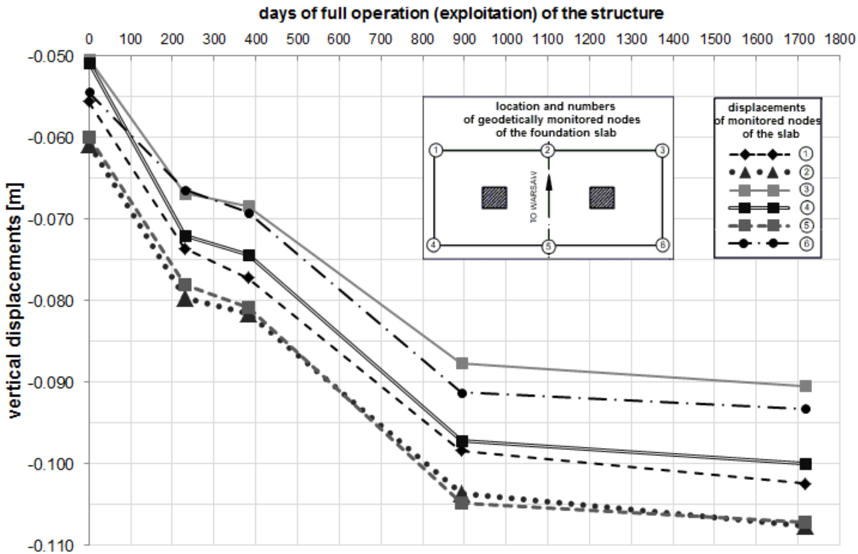


On the 384th day of bridge operation actual reliability reaches the limit value of $\beta_{EN} = 2.9$ stated by EN 1990:2002+A1:2005, provided no accidental loads will appear during the operation.

Long-term SLS-based reliability of Rędziński Bridge

The variation in reliability index β_{HL} defined with (20) **may be extrapolated to the following years**.

In the Rędziński Bridge case the **re-evaluation of the index was performed**, including settlements data up to 4.5 years after the bridge admission date (the geodetic measurements are presented in the figure).



Incorporating additional data points the adjusted settlement change was approximated as

$$\Delta y(t) = -7.304\times10^{-12}\,t^{3} + 4.151\times10^{-8}\,t^{2} - 7.782\times10^{-5}\,t \qquad (21)$$

In this case, the adjusted fluctuation function $n(t)$ was approximated as

$$n(t) = 9.323\times10^{-9}\,t^{3} - 5.299\times10^{-5}\,t^{2} + 9.934\times10^{-2}\,t - 29.652 \qquad (22)$$

The predefined variation parameters of random variables, their values in time steps of geodetic surveys are defined in the table, the example of $X_3 = E_{ur}^{ref}\big|_{Va}$.

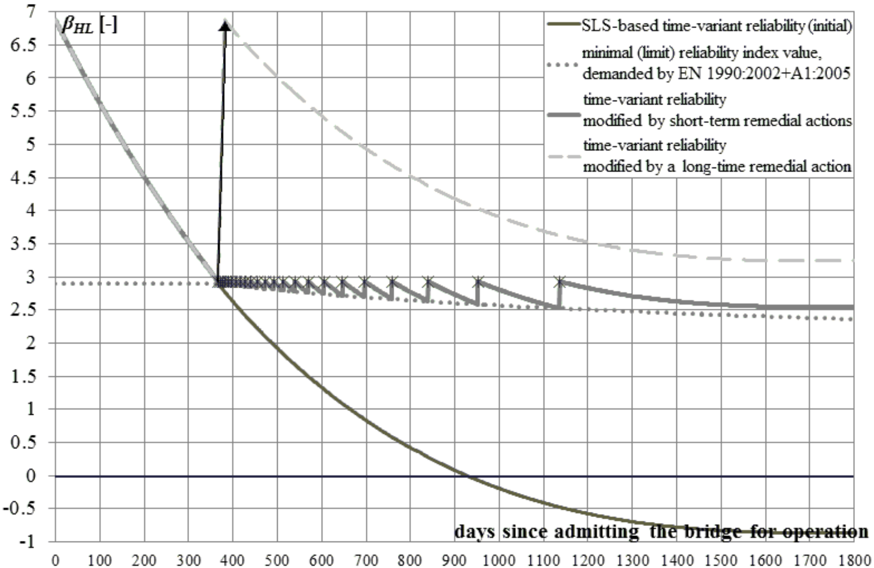
Time step	Respective extreme displacement $y(t)$ [m]	Change coefficient $n(t)$	$E_{ur}^{ref}\big\|_{Va}$ mean value $\overline{x_3}$	$E_{ur}^{ref}\big\|_{Va}$ variation v_3	$E_{ur}^{ref}\big\|_{Va}$ standard dev. σ_3
0 (0 days)	-0.0600	-29.652	129651.5	0.081	9455.5
1 (229 days)	-0.0780	-9.570	109570.2	0.094	9690.6
1-year	-0.0800	0	100000	0.1	10000
2 (382 days)	-0.0808	1.082	98917.9	0.102	10104.1
3 (893 days)	-0.1048	23.435	76565.1	0.137	10484.2
4 (1716 days)	-0.1072	31.867	68132.8	0.155	10564.5

The adjusted Hasofer-Lind β_{HL} reliability index change

$$\beta_{HL}(t)_{RSM-3} = -1.577\times10^{-9}t^3 + 7.882\times10^{-6}t^2 - 1.341\times10^{-2}t + 6.887 \qquad (23)$$

The outcome of long-term reliability analysis – remedial actions planning

To showcase **the outcome of the long-term analysis**, the variation of the SLS-based β_{HL} index is presented in the figure in different scenarios.



A hypothetical scenario, showing that **if no remedial actions were undertaken**, **a zero value of the SLS-based foundation reliability** would be reached after 925 days (2.5 years) of bridge operation (a maximum displacement of the foundation equal to the SLS design limit $u_{lim} = 0.10\,\mathrm{m}$).

The reliability index limit function due to the EN 1990:2002+A1:2005 standard regulations is also showcased in the figure **with a dotted line**.

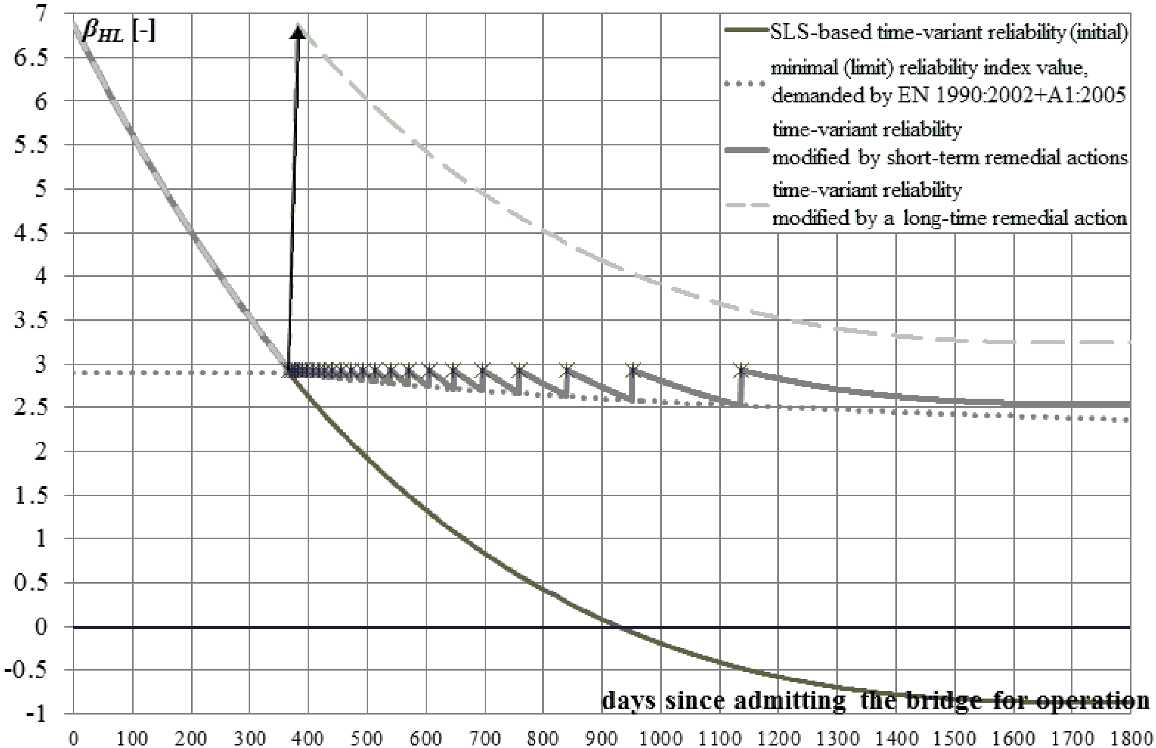
The constant value $\beta_{\mathrm{EN,lim,1}} = 2.9$ is adopted only for a 1-year period, then it decreases to $\beta_{\mathrm{EN,lim,50}} = 1.5$ over 50 years.

The decrement is assessed (EN 1990:2002+A1:2005)

$$\Phi(\beta_{\mathrm{EN,lim,}Y}) = \left[\Phi(\beta_{\mathrm{EN,lim,1}})\right]^{Y} \tag{24}$$

where $\beta_{\mathrm{EN,lim,}Y}$ is the limit reliability index in a given point in time Y (in years), $\beta_{\mathrm{EN,lim,1}}$ is the 1-year reliability index, and Y denotes the length of the time period (in years).

Two possible scenarios addressing courses of remedial actions and their resultant influence on the time-variant reliability were presented in the figure.



The first remedial action course is to undertake short-time actions, to maintain the bridge reliability on a 1-year operation level $(\beta = 2.928)$.

While over-consolidation makes the reliability drop below the value of $\beta_{EN} = 2.9$ demanded by EN 1990:2002+A1:2005, stay cables should restore the 1-year reliability level (proper road grade line elevation); the cable tension control is made possible for Rędziński Bridge

The time steps of these actions are denoted in the figure by asterisks.

As they do not require bridge closure, the initial reliability index graph is split and vertically elevated in all single-day time steps when the repairs are undertaken.

The second remedial action course concerns undertaking one-time long-term repair, to ensure an identical bridge reliability level to its value of the operation start ca. $(\beta = 6.887)$.

Given the settlements increment stopping at some point in time, such high reliability provides a proper distance from the EC-based limit.

Due to the time-consuming nature of such large scale repairs, a temporary bridge closure due to repairs is assumed.

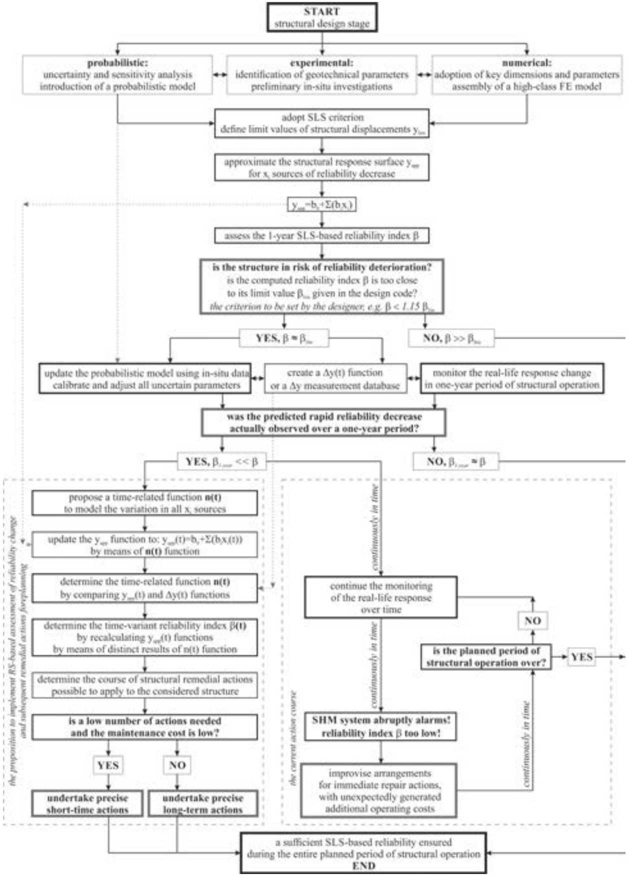
Procedure extension to other structural types

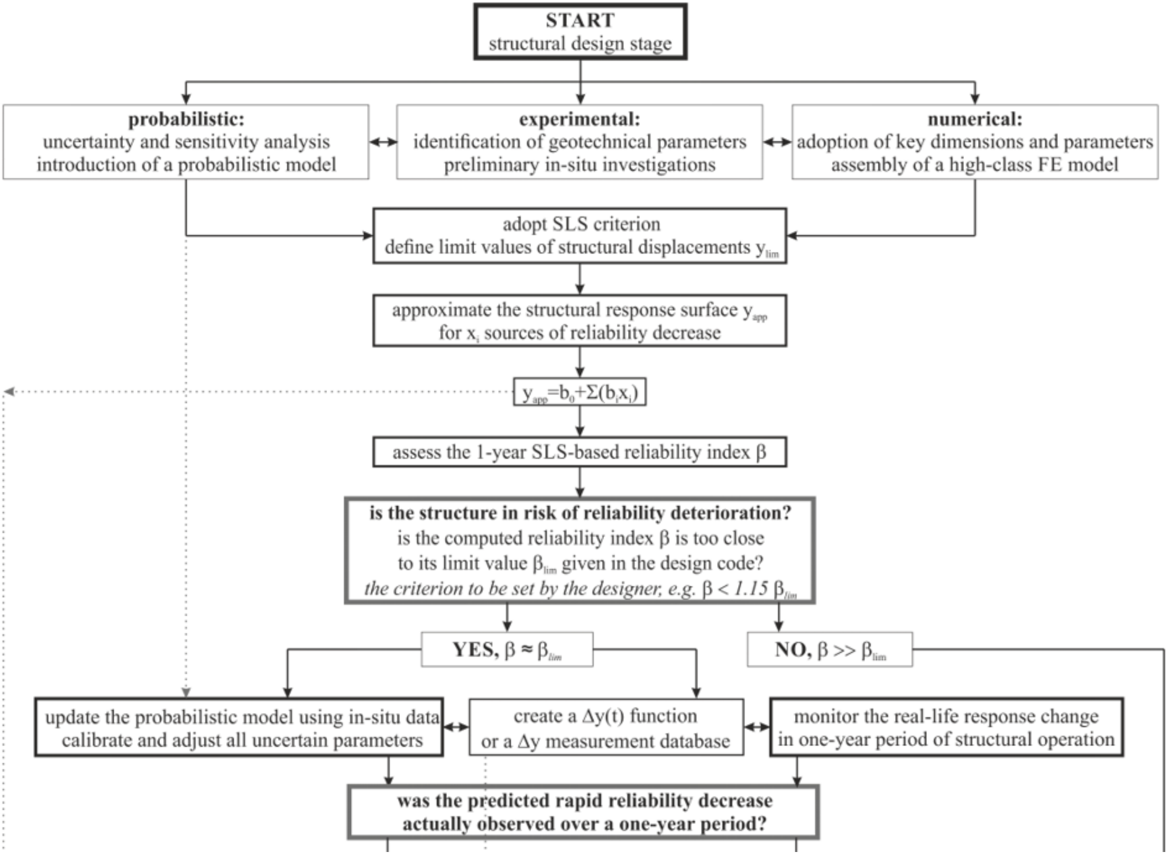
A threat of a rapid decrement in the SLS-based reliability in time is induced in many structures of social and infrastructural importance, e.g. bridges, dams, telecommunication towers, wind turbines, buildings with structural roofing, silos, and tanks.

In the majority of such structures, **the standard analysis is divided into three main sections**:
– probabilistic modelling is applied to the initial design stage to account for the variation in structural response induced by uncertainty sources;
– the in-situ investigations are performed to calibrate and adjust the uncertainties' parameters;
– structural health monitoring systems are implemented to continuously collect and analyse the data on the mechanical response.

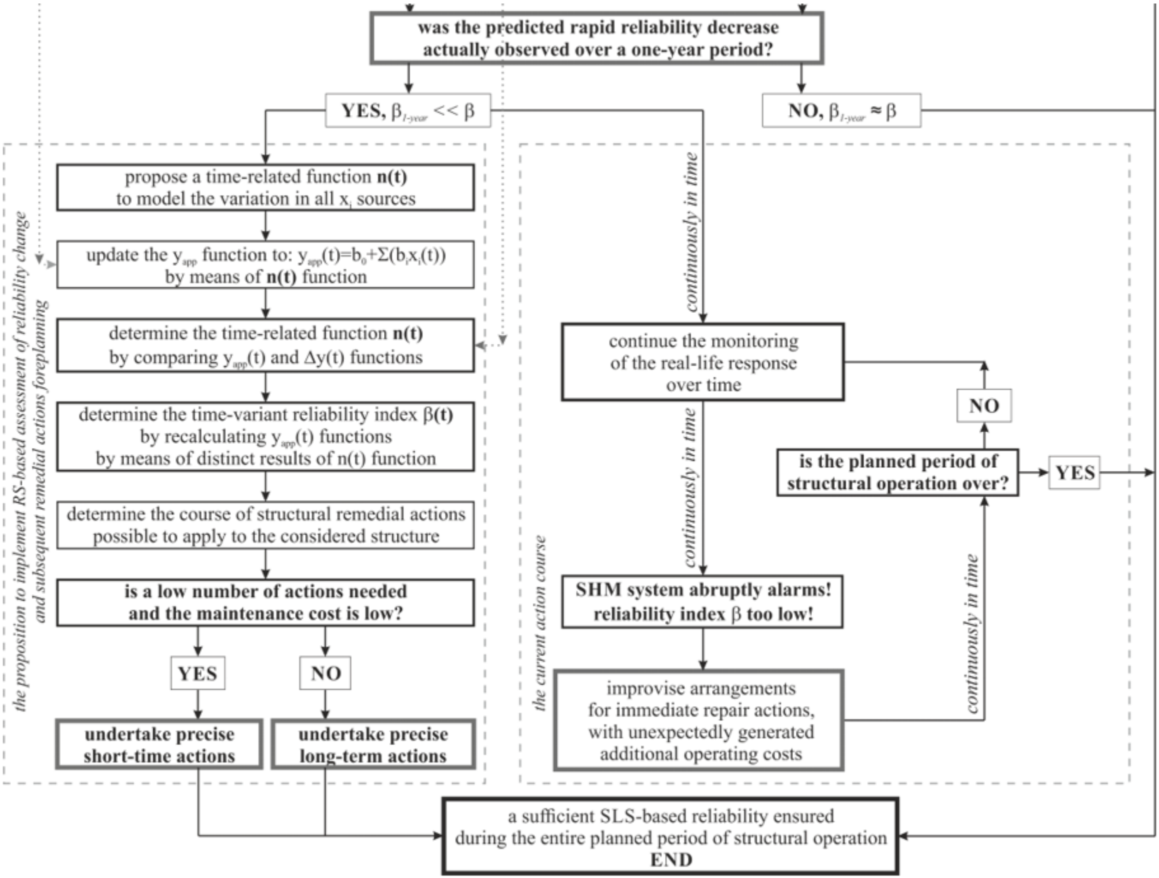
Thus, a proposition follows, **to conduct an updated reliability assessment using a probabilistic approach, in favour of continuous monitoring**.

A flowchart of the current engineering process is devised and presented in the figure, where an action course benefitting from the proposed probabilistic procedure is highlighted.


START
structural design stage
probabilistic:
uncertainty and sensitivity analysis
introduction of a probabilistic model
experimental:
identification of geotechnical parameters
preliminary in-situ investigations
numerical:
adoption of key dimensions and parameters
assembly of a high-class FE model
adopt SLS criterion
define limit values of structural displacements
approximate the structural response surface
sources of reliability decrease
assess the 1-year SLS-based reliability index β
is the structure in risk of reliability deterioration?
is the computed reliability index β is too close
to its limit value given in the design code?
the criterion to be set by the designer, e.g.
YES
NO
update the probabilistic model using in-situ data
calibrate and adjust all uncertain parameters
create a Δy(t) function
or a Δy measurement database
monitor the real-life response change
in one-year period of structural operation
was the predicted rapid reliability decrease
actually observed over a one-year period?
YES
NO
the proposition to implement RS-based assessment of reliability change
and subsequent remedial actions forplanning
propose a time-related function n(t)
to model the variation in all sources
by means of n(t) function
determine the time-related function n(t)
by comparing and Δy(t) functions
determine the time-variant reliability index β(t)
by recalculating functions
by means of distinct results of n(t) function
determine the course of structural remedial actions
possible to apply to the considered structure
is a low number of actions needed
and the maintenance cost is low?
YES
NO
undertake precise
short-time actions
undertake precise
long-term actions
the current action course
continuously in time
continue the monitoring
of the real-life response
over time
NO
is the planned period of
structural operation over?
YES
continuously in time
SHM system abruptly alarms!
reliability index β too low!
improvise arrangements
for immediate repair actions,
with unexpectedly generated
additional operating costs
continuously in time
a sufficient SLS-based reliability ensured
during the entire planned period of structural operation
END



START
structural design stage
probabilistic:
uncertainty and sensitivity analysis
introduction of a probabilistic model
experimental:
identification of geotechnical parameters
preliminary in-situ investigations
numerical:
adoption of key dimensions and parameters
assembly of a high-class FE model
adopt SLS criterion
define limit values of structural displacements y_{lim}
approximate the structural response surface y_{app}
for x_i sources of reliability decrease
$y_{app}=b_0+\Sigma(b_i x_i)$
assess the 1-year SLS-based reliability index β
is the structure in risk of reliability deterioration?
is the computed reliability index β is too close
to its limit value β_{lim} given in the design code?
the criterion to be set by the designer, e.g. $\beta < 1.15\ \beta_{lim}$
YES, $\beta \approx \beta_{lim}$
NO, $\beta >> \beta_{lim}$
update the probabilistic model using in-situ data
calibrate and adjust all uncertain parameters
create a Δy(t) function
or a Δy measurement database
monitor the real-life response change
in one-year period of structural operation
was the predicted rapid reliability decrease
actually observed over a one-year period?


was the predicted rapid reliability decrease actually observed over a one-year period?
YES, β1-year << β
NO, β1-year ≈ β
the proposition to implement RS-based assessment of reliability change and subsequent remedial actions foreplanning
propose a time-related function n(t) to model the variation in all xi sources
update the yapp function to: yapp(t)=b0+Σ(bixi(t)) by means of n(t) function
determine the time-related function n(t) by comparing yapp(t) and Δy(t) functions
determine the time-variant reliability index β(t) by recalculating yapp(t) functions by means of distinct results of n(t) function
determine the course of structural remedial actions possible to apply to the considered structure
is a low number of actions needed and the maintenance cost is low?
YES
NO
undertake precise short-time actions
undertake precise long-term actions
the current action course
continuously in time
continue the monitoring of the real-life response over time
continuously in time
SHM system abruptly alarms! reliability index β too low!
improvise arrangements for immediate repair actions, with unexpectedly generated additional operating costs
continuously in time
is the planned period of structural operation over?
NO
YES
a sufficient SLS-based reliability ensured during the entire planned period of structural operation END