Probabilistic Settlement Analysis For The Botlek Lifting Bridge Design

Analyse probabiliste de tassement pour la conception du pont levant Botlek

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ABSTRACT: A new lifting bridge is being constructed crossing the river Oude Maas in the Rotterdam harbour area in the Netherlands. For the deformation analysis deterministic 3D FEM calculations were performed. In order to take the effect of soil heterogeneity on the deformation behaviour of the bridge piers into account, a probabilistic model has been developed. This model and the applications are described in this paper. The application of a simplified stochastic subsoil model enables a quantitative risk analysis in order to deal with this uncertainty. Furthermore the model is used to determine design values of the deformations of several components of the bridge.

RÉSUMÉ : Au port de Rotterdam aux Pays-Bas on construit un nouveau pont levant qui traverse la rivière Oude Maas. Des calculs déterministes 3D FEM sont effectués pour analyser la déformation. On a développé un modèle probabiliste pour tirer l'effet de l'hétérogénéité du sol sur la déformation des piles du pont. Cet article décrit ce modèle et ses applications. L'application d'un sous-sol simplifié et stochastique permet une analyse de risque quantitative qui sait régler l'incertitude des paramètres du sous-sol. En outre le modèle est utilisé pour déterminer la valeur de calcul des déformations des différentes pièces du pont.

KEYWORDS: Foundation design, shallow foundation, soil heterogeneity, probabilistic deformation analysis, quantitative risk analysis

MOTS-CLES: Calcul de fondations, foundation superficielle, hétérogénéité, analyse probabiliste, analyse quantitative, analyse de risques

1 INTRODUCTION

The Dutch highway A15 in the Rotterdam harbour area is being widened due to an increase in traffic load. One of the main challenges in this project is the construction of a new lifting bridge over the river Oude Maas. Consisting of two lifting spans of approximately 100 m and pylons reaching over 60 m above water level, this new bridge will be one of the largest lifting bridges in Europe (see Figure 1).

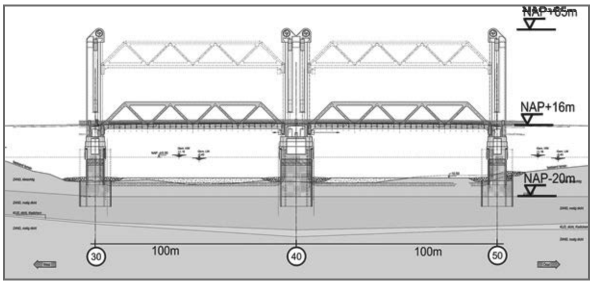

Figure 1. Typical cross section of new Botlek Lifting Bridge

The three main bridge piers (from left to right in Figure 1: Pier 30, Pier 40 and Pier 50) are founded on rigid concrete blocks with footing dimensions of 15 x 60 m, at 8 m below river bed at the top of the first dense (Pleistocene) sand layer. For the geotechnical design the foundation was essentially treated as a shallow foundation.

At a depth of approximately 16 m below the foundation footing a relatively soft clay layer is present with varying thicknesses between 0 and 4 m. This stratum complicated the design, especially with respect to the settlement behaviour which has a major impact on the performance of the total bridge and influences the different design disciplines (e.g. mechanical, electrical and structural). This article considers the risk analysis with respect to the deformation behaviour of the subsoil which was undertaken as part of the foundation design.

2 SOIL INVESTIGATION AND PARAMETER DETERMINATION

For the determination of the soil parameters an extensive soil investigation has been performed. A relatively dense grid of Cone Penetration Tests (CPT's) with a mutual distance of about 15 m was executed to a depth of about 3 times the foundation width. In addition, a number of boreholes were drilled and undisturbed samples were taken at regular intervals for geotechnical laboratory tests by means of light percussion drilling in combination with thin-walled samplers. From the CPT's and borehole logs the soil stratigraphy is determined, see Table 1.

Table 1. General soil stratigraphy

Top of layer [m NAP]	Soil description	Soil layer
-7 à -14	SAND, clayey	cover layer
-14 à -20	SAND, (medium) dense	1st sand layer
-33 à -39	CLAY, stiff	deep clay layer
-34 à -42	SAND, (medium) dense	2nd sand layer
-60	Max. investigation depth	

The thickness of the deep clay layer varies strongly. At some locations the thickness is about 4 m, whereas this layer was not encountered at other locations.

Classification tests, such as particle size distribution (granular layers) and volumetric weight and water content (cohesive layers), were performed on samples from the different soil layers. In order to determine the strength properties of the second sand layer isotropically consolidated drained triaxial tests were performed. The samples were prepared in the laboratory at relative densities of 40%, 60% and 80%. The in situ relative density was determined from the CPT's from the correlation deduced by Baldi (Lunne, 1997), and turned out to be approximately 70% for this sand stratum. The characteristic strength properties were determined from statistical analyses on the results from the triaxial tests. For the effective angle of shearing resistance of the sand below foundation level a representative value of 33° was determined at higher axial strain levels, which corresponded well with the cone resistance as can be found in literature, e.g. in the Dutch Code (NEN-EN 1997-1, 2005).

Since deformations of the deep clay layer were expected to have a relatively large influence on the superstructure, additional oedometer tests were performed on samples from the deep clay layer. From experience in the area, it is known that this layer is overconsolidated, which was confirmed by the CPT results. However, the overconsolidation ratio (OCR) could not be accurately determined from the oedometer tests, most likely due to relaxation of the samples. Therefore the OCR is determined from the following correlation with the cone resistance (Lunne, 1997):

$$OCR \approx \frac{c_{u;oc}}{c_{u;nc}} \approx \frac{(q_c - \sigma_v)/17}{0.3\sigma'_v} \quad (1)$$

In which:

- OCR = overconsolidation ratio [-]
- $c_{u;oc}$ = in situ (overconsolidated) undrained shear strength [kPa]
- $c_{u;nc}$ = normally consolidated shear strength [kPa]
- q_c = cone resistance [kPa]
- σ_v = vertical total stress [kPa]
- σ'_v = vertical effective stress [kPa]

The calculated OCR corresponded well with experience from other projects in the area and geological information.

The virgin stiffness and unloading/reloading stiffness was determined from the oedometer tests, which included an unloading/reloading step. The determination of these stiffness parameters from the laboratory tests was expected to be reliable, since these were determined beyond the preconsolidation stress, so relaxation effects are expected to be minimal.

Based on the soil investigation and laboratory tests, representative values for the soil stiffness's were determined. A representative elasticity modulus ($E_{oed;ref}$) of approximately 40 MPa and 3.5 MPa was determined for respectively the 1st sand and deep clay layer. This is the oedometer stiffness at a reference vertical effective stress of 100 kPa. For the stress-stiffness relationship a power law was adopted (Brinkgreve, 2011), with a power 1.0 for sand and 0.8 for the stiff clay (based on oedemeter tests). An unloading/reloading oedometer stiffness ratio of 4 is applied.

3 DETERMINISTIC DEFORMATION ANALYSIS

During the design process it was recognised that deformations of the foundation have a large influence on the design and construction of the superstructure, especially for the mechanical and structural design. Due to the large ratio between the height of the pylons and the width of the foundation, a small rotation of the foundation base results in a large deflection of the pylon heads. This effect has a significant influence on the design of the superstructure. In order to determine safe tolerances which have to be taken into account by the other design disciplines, a thorough deformation analysis was performed.

First step in the deformation analysis was to perform 'best estimate' deformation calculations. In the early design stages analytical 2D settlement calculations were performed. In the detailed design phase additional 3D FEM calculations were performed. The software program Plaxis was used for these calculations.

In the calculations the Hardening Soil (HS) model is used for the deep clay layer. Aspects of this model include:

- Stress dependent stiffness of the soil
- Plastic straining due to primary deviatoric loading
- Plastic straining due to primary compression
- Elastic unloading/reloading
- Failure according to the MC criterion

The Hardening Soil model does not take creep effects into account. However, from the laboratory tests it turned out that about 80% of the settlements are primary and only 20% of the settlements are related to secondary compression. Therefore the choice was made to consider the creep effect separately, instead of applying a soft soil creep model. Soft soil creep models are especially useful if the influence of creep is more pronounced.

For the sand layers underneath the foundation surface the Hardening Soil Small-Strain Stiffness (HSSmall) model is used. This model is similar to the HS model, but additionally takes the higher stiffness of the soil at small strain levels into account. For the clayey sand layer above the foundation layer, also the Hardening Soil (HS) model is used.

The serviceability limit state (SLS) foundation pressures for the different main piers are in the range between 500 to 700 kPa. The calculated 'best estimate' final settlements of the foundation footings range from 0.10 to 0.25 m. For the rotations maximum values in the order of 1/1000 were calculated. These calculated rotations are mainly the result of the bending moments loads, rather than soil heterogeneity.

In general, soil deformations are difficult to predict accurately, since various uncertainties can be present. See for instance Figure 2 where the thickness of the deep (Kedichem) clay layer is plotted over the footprint of the main bridge piers. The variation is based on factual data from CPT's and boreholes with interpolation between these data.

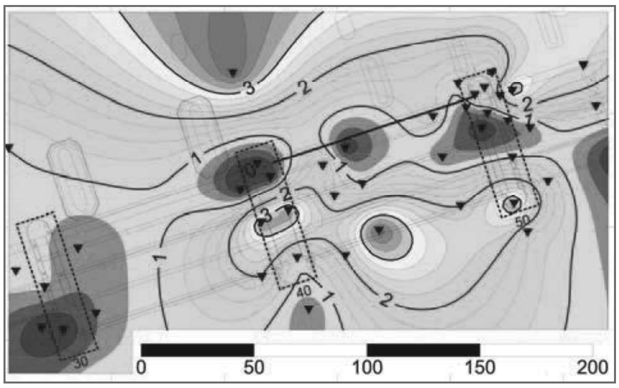

Figure 2. Thickness Kedichem clay layer [m]

To get a better understanding with regard to subsoil uncertainties, a sensitivity analysis with the 3D FEM model was performed. The influence of variations in OCR, (virgin) stiffness and the thickness of the deep Kedichem clay layer between the soil investigation points were considered.

From the sensitivity analysis it was concluded that the variation in calculated average settlement was about 25% for the different subsoil scenarios. The variation in the calculated rotations was small. The calculated absolute rotations of the piers were still in the range of 1/1000.

4 PROBABILISTIC DEFORMATION ANALYSIS

The sensitivity analysis with the 3D FEM results in a better understanding in the range of settlements which could be expected. However, the variation of soil properties within one homogeneous soil layer is hardly taken into account in standard 3D FEM calculations. Since this effect can have a large influence on the rotations of the foundation, a practical stochastic subsoil model was set up to take this effect into account.

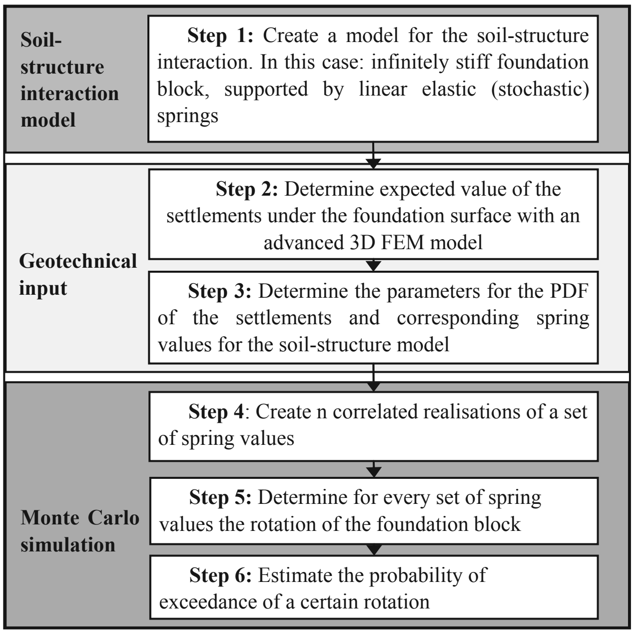

Figure 3. Description of probabilistic model

With this probabilistic model it was possible to determine the probability of exceedance of a certain design rotation. With this model it was also possible to perform a quantitative risk analysis, regarding the effects of foundation rotations. The model is described section 3.1 to 3.6 and schematically presented in Figure 3.

4.1 *Step 1 - Soil-structure interaction*

The foundation is modelled as an infinitely stiff foundation block, supported by linear elastic (stochastic) springs at a spacing of about 3 m. Since the foundation consists of a massive concrete block with a thickness of about 20 m the assumption of a stiff foundation is considered reasonable.

The linear elastic soil springs are stochastic, representing the uncertainty in soil behaviour. The stochastic (correlated) stiffness $k_{i;j}$ of the springs $S_{i;j}$ under the foundation is determined according to:

$$k_{i.j} = \frac{q}{z_{i,j}} \cdot \frac{L}{I} \cdot \frac{W}{J} \tag{2}$$

In which:

- q = uniform distributed foundation load, P_z /(L·W) [kN/m^2]
- $z_{i;j}$ = settlement at location ($x_{i;j}$;$y_{i;j}$) [m]
- L = length of the foundation [m]
- W = width of the foundation [m]
- I = number of equally spaced springs along the length of the foundation [-]
- J = number of equally spaced springs along the width of the foundation [-]
- P_z = Vertical foundation load [kN]

All parameters in eq. (2) are deterministic, except for the settlements. A linear transformation between the probability density function (PDF) of the settlements (see section 4.3) and the soil stiffness is applied.

4.2 *Step 2 - Settlements*

For the determination of the expected value of the (residual) settlements the results of the 3D FEM model are used. Since the soil stiffness is primarily a soil property, the influence of the stiff foundation should not be taken into account in the determination of the settlements. Therefore the stiffness of the foundation block is neglected for these settlement calculations, by using a flexible footing in the 3D FEM model. The settlements are calculated with a uniform load on the foundation surface.

4.3 *Step 3 – Parameters probability density function*

4.3.1 *Model parameters*

The settlements are modelled as random variables with a lognormal distribution. The lognormal distribution is often used to model non-negative random variables, such as thickness of layers and soil properties. The calculation results from 3D FEM model are interpreted as the expected value μ_z of the PDF of the settlements.

The parameters of the lognormal distribution of the settlements has the following parameters (i.e. Fenton and Griffiths, 2008):

$$\sigma_{\ln(z)} = \sqrt{\ln(1+V_z^2)} \tag{3}$$

$$\mu_{\ln(z)} = \ln(\mu_z) - 1/2 \cdot \sigma_{\ln(z)}^2 \tag{4}$$

The coefficient of variation of the settlements is based on the assumption of 30% inaccuracy in the settlement calculations. That means that there is a probability of about 5% that the settlements will be 30% larger than the calculated average settlements. This is a generally applied rule of thumb in the Netherlands. So:

$$z_{95\%} = 1.3 \cdot \mu_z \tag{5}$$

Based on the lognormal distribution the $z_{95\%}$ can estimated by:

$$z_{95\%} \approx e^{(\mu_{\ln(z)} + 1.65 \cdot \sigma_{\ln(z)})} \tag{6}$$

Equating eq. (5) and (6) in combination with eq. (3) and (4) the coefficient of variation (V_z) of the PDF of the settlements can be estimated by:

$$e^{-\frac{1}{2} \cdot \ln(1+V_z^2)} e^{1.65\sqrt{\ln(1+V_z^2)}} = 1.3 \tag{7}$$

Eq. (7) results in a coefficient of variation V_z of approximately 0.17.

4.3.2 *Correlation parameters*

Due to its natural fabric, the soil properties, can be considered as spatially correlated. Different autocorrelation functions are known from literature, from which the functions that adopt an exponential shape are commonly used. In this case the following expression is used (Breysse, 2004 and DeGroot, 1993):

$$\rho_{\ln(z1);\ln(z2)} = e^{-d/L_c} \tag{8}$$

In which:
- d = horizontal distance between two springs [m]
- L_c = autocorrelation length of ln(z) [m]

The covariance can be determined according to (CUR190, 1997):

$$Cov(\ln(z_1), \ln(z_2)) = \rho_{\ln(z1);\ln(z2)} \cdot \sigma_{\ln(z1)} \cdot \sigma_{\ln(z2)} \tag{9}$$

From this the covariance matrix **C** can be constructed.

4.3.3 *Autocorrelation length*

The autocorrelation length L_c can be interpreted as the distance over which a certain parameter is significantly correlated. In literature several indicative values for the horizontal and vertical correlation length for soil parameters are given. In this case especially the horizontal correlation length is relevant. Typical values for the horizontal autocorrelation length for soil properties are in the range $L_c \approx 20$ to 100 m (DeGroot, 1993; TAW, 2001 and Gruijters, 2009).

Before determining the autocorrelation length the influence of this parameter is checked. If $L_c \to 0$, the logarithm of the settlement at two locations is independent. Because of the averaging effect of the stiff foundation, the rotation is expected to approach the value as found in a deterministic approach, e.g. zero rotation if a homogeneous soil is modelled. If $L_c \to \infty$, the logarithm of the settlement at two locations is fully correlated. In this case the rotation is also expected to approach the value as found in an deterministic approach, e.g. zero rotation if a homogeneous soil is modelled. The maximum rotation is found for an intermediate value of L_c, typically half the foundation size.

For the deformation analysis especially the spatial variation of the compressibility of the different soil layers and the thickness of the clay layer are important. From the soil investigation it turned out that the horizontal correlation length with respect to the thickness of the clay layer is typically in the order of 10 to 20 m. However, in general the horizontal correlation length with respect to soil properties is typically in the order of 50 to 100 m. Therefore the most critical value for the horizontal correlation length within the range between 10 to 100 m was selected. In this case a correlation length of 20 m has been used.

4.4 *Step 4 - Realisations of spring values*

For the probabilistic analysis a Monte Carlo (MC) procedure is used (CUR190, 1997 and Haugh, 2004). To generate correlated values for the spring values an algorithm in a spreadsheet program was set up. The following procedure is applied for each realisation 1 to *n*:

1) Generate a vector with realization of the standard normal distribution $\boldsymbol{X} \sim N(0,\boldsymbol{I})$. In which $\boldsymbol{I}$ is the identity matrix and the size of the vector is equal to the number of springs $s = I.J$
2) Decompose the covariance matrix $\boldsymbol{C}_{ln(z)} = \boldsymbol{A}.\boldsymbol{A}^T$ (Cholesky decomposition (Haugh, 2004))
3) Determine the correlated vector $\boldsymbol{Z}' = \boldsymbol{A}.\boldsymbol{X} + \boldsymbol{\mu}_{ln(z)} \sim N(\mu_{ln(z)}, C_{ln(z)})$
4) Determine the vector with correlated settlement values $Z_{i;j} = exp(Z_{i;j}')$
5) Determine the vector with the correlated spring values from eq. (1)

4.5 *Step 5 – Determining settlement and rotation foundation*

Since an infinitely stiff foundation is assumed in step 1, the rotation of the foundation can be determined from the vertical force equilibrium and the moment equilibrium in 2 directions:

$$\sum F_z = \sum_{i=1}^{I} \sum_{j=1}^{J} R_{i.j} = qLW \tag{10}$$

$$\sum M_y = \sum_{i=1}^{I} \sum_{j=1}^{J} R_{i.j} x_{i.j} = 1/2\, qLW^2 - M_{y\_a} \tag{11}$$

$$\sum M_x = \sum_{i=1}^{I} \sum_{j=1}^{J} R_{i.j} y_{i.j} = 1/2\, qL^2W - M_{x\_a} \tag{12}$$

In which:
- $R_{i;j}$ = force in spring $S_{i;j}$ [kN]
- $x_{i;j}$ = x coordinate of spring $S_{i;j}$ [m]
- $y_{i;j}$ = y coordinate of spring $S_{i;j}$ [m]
- M_{y_a} = acting bending moment around the y axis [kNm]
- M_{x_a} = acting bending moment around the x axis [kNm]

The force in every spring can be determined according to:

$$R_{i;j} = k_{i;j} u_{i.j} \tag{13}$$

In which:
- $u_{i;j}$ = deformation in spring $S_{i;j}$ [m]

The deformation in every spring can be expressed as:

$$u_{i;j} = u_{0;0} + \theta_x x_{i;j} + \theta_y y_{i;j} \tag{14}$$

In which:
- $u_{0;0}$ = deformation in the point $x = 0$, $y = 0$ [m]
- θ_x = rotation around the y axis, long axis [-]
- θ_y = rotation around the x axis, short axis [-]

These are exactly the variables of interest, which can be filled in into the equilibrium equations. This leads to a system of linear equations, which can be presented in matrix notation:

$$\begin{bmatrix} A1 & A2 & A3 \\ A2 & A6 & A7 \\ A3 & A7 & A9 \end{bmatrix} \begin{bmatrix} u_{0;0} \\ \theta_x \\ \theta_y \end{bmatrix} = \begin{bmatrix} A4 \\ A5 \\ A8 \end{bmatrix} \tag{15}$$

Wherein the parameters A1 to A9 can be derived from eq. (10), (11) , (12), (13) and (14).

The matrix equation can be solved by Cramer's rule (Lay, 2003), which states that:

$$U_{0;0} = D_{u0;0} / D \tag{16}$$

$$\theta_x = D_{\theta x} / D \tag{17}$$

$$\theta_y = D_{\theta y} / D \tag{18}$$

In which:
- D = determinant of the coefficient matrix
- $D_{u0;0}$ = determinant of the matrix formed by replacing the $u_{0;0}$ column of the coefficient matrix by the answer matrix
- $D_{\theta x}$ = determinant of the matrix formed by replacing the θ_x column of the coefficient matrix by the answer matrix
- $D_{\theta y}$ = determinant of the matrix formed by replacing the θ_y column of the coefficient matrix by the answer matrix

4.6 *Step 6 – Determination probability of exceedance*

For every simulation a set of spring values was generated. With the soil-structure interaction model the bending moment and vertical force equilibrium the rotation and (average) settlement was calculated for every set of springs. The probability of exceedance for a certain rotation can be estimated by:

$$P(\theta > \theta_r) \approx n_f / n \qquad (19)$$

In which:

- $P(\theta > \theta_r)$ = exceedance probability of rotation θ_r [-]
- n_f = number of simulations for which the calculated rotation is larger than the reference rotation [-]
- n = total number of simulations [-]

The accuracy of this estimate strongly depends on the number of simulations in relation to the probability of exceedance; for smaller probabilities, a higher number of simulations is necessary to reach the same reliability of the estimation. The relative error ε is given by (CUR190, 1997):

$$\varepsilon = (n_f / n - P(\theta > \theta_r) / P(\theta > \theta_r)) \qquad (20)$$

For a certain value of the relative error E with an accuracy of 95% can be estimated by:

$$E \approx \sqrt{(4 \cdot (n / n_f - 1) / n)} \qquad (21)$$

For this study a relative error E of maximum 20% is assumed to be acceptable. In order to be able to determine probabilities of exceedance of $1 \cdot 10^{-4}$ sufficiently accurate, therefore at least $1 \cdot 10^{6}$ simulations are necessary.

5 RESULTS AND APPLICATION

5.1 *Results*

Figure 4 shows the results of realisations for the residual rotations of pier 40. In this figure the combined realisations of rotation around the long axis (θ_x) and the rotations around the short axis (θ_y) are shown.

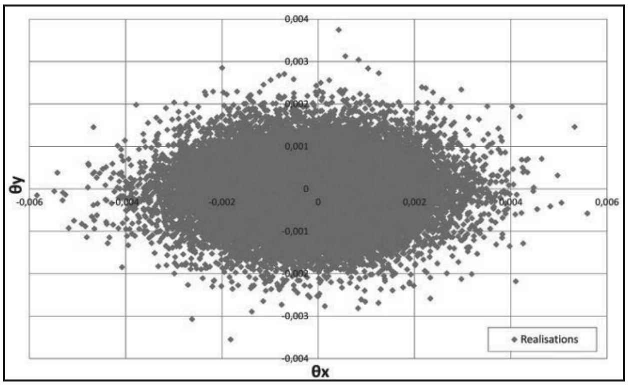

Figure 4. Results Monte Carlo analysis pier 40, residual rotations

Figure 4 shows that the distribution of realisation is located around the origin what means that the expected rotation is more or less equal to zero. This is in line with the deterministic settlement calculations. It is also shown that rotation around the long axis has a higher probability than rotation around the short axis. The shortest side (width) of the foundation block is more sensitive for rotation.

The calculated probabilities of exceedance for different rotations are presented in Figure 5 for the rotation around the long axis (θ_x). The results for pier 30 and 50 are almost equal because the calculated deformations with the FEM model are also almost equal for these piers. From Figure 5 related to the average settlement of the piers it can be concluded that larger average settlements (pier 40) result in a higher probability of larger rotations, which is reasonable.

Important for the bridge deck is the combined rotation of two piers. Based on the results of the individual piers also the probability of a combined rotation of two piers could be determined. For the combined rotation it is assumed that the deformation behaviour of the piers is uncorrelated.

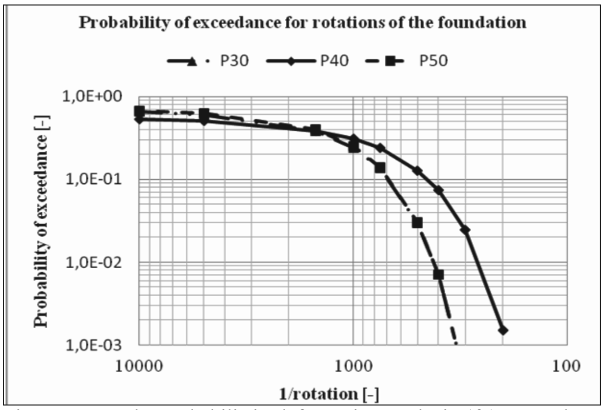

Figure 5. Results probabilistic deformation analysis (θ_x). Note that the results for P30 and P50 are almost identical

5.2 *Application of results*

The results of the model are used for the design of the different components of the bridge which are influenced by the settlement and rotation of the pier.

Based on the calculated probability of exceedance of a certain rotation, safe boundary conditions for the other design disciplines could be determined. Relevant components are the towers with the guiding system, the deck, the expansion joints for the deck and the supports of the deck. A design value of the deformation is derived for these components based on the acceptable probability of exceedance.

During construction the deformations will be monitored and control measures can be applied if necessary.

6 CONCLUSION

For the design of the new Botlek Lifting Bridge soil deformations can potentially have a major affect on one of the most critical design requirements, which is a limited rotation of the large foundation footing.

Alongside a well designed site investigation campaign, laboratory tests and the application of appropriate constitutive models, a quantification of the probability of exceedance of soil deformations was desired. Application of a simplified stochastic subsoil model enabled a quantitative risk analysis in order to deal with the uncertainties described in this paper.

Based on the calculated probability of exceedance of a certain rotation, safe boundary conditions for the other design disciplines could be determined.

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