IMPROVED RELIABILITY OF (REST) SETTLEMENT PREDICTIONS OF EMBANKMENTS ON SOFT SOILS

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ABSTRACT

A 22 kilometre embankment section of the Betuweroute freight railway line has been designed and constructed by an alliance of ProRail (Dutch railway infra manager) and HBSC (a combination of 4 large Dutch and Belgium contractors) in an area of highly compressible soils. The contractual form of an alliance was chosen to maximise synergy between the various contractual parties. The objective of the alliance was to build the embankment for the lowest possible costs within the contractual time frame, guaranteeing the quality of the final product.

One of the main risks of the alliance was the uncertainty whether the post-construction settlement specifications could be met using a traditional sand fill with wick drains to accelerate settlements. Settlement predictions with various theoretical models showed an unacceptably large variation in results. Field measurements with settlement beacons also showed large variations. The combined use of an isotache settlement model and a Monte Carlo simulation has resulted in a much more reliable method for the prediction of post-construction settlements.

In November 2003, four years after the start of the alliance, the project will be completed within the contractual time frame and ample within the original project budget. Knowledge and experience on settlement predictions, gained in the Sliedrecht-Gorinchem contract of Betuweroute has already been adopted in several other projects.

KEY WORDS

RISK MANAGEMENT / SOFT SOIL / EMBANKMENT / ISOTACHE / SETTLEMENT PREDICTION / MONTE CARLO SIMULATION.

1. INTRODUCTION

Over the last 5 years, several large infrastructure projects have come into the construction phase in the Netherlands simultaneously. Among these projects Betuweroute, a 160 km double track electrificated freight railway line connecting the Rotterdam Harbour to the German Hinterland, is one of the largest projects. Betuweroute is part of the European network of Freeways and transects the Netherlands. The western sections of Betuweroute pass through a typical Dutch polder landscape and have to cope with very soft soils. As finally, rail will be rolled out from west to east, the construction of the embankments had to be finished in the western sections first.

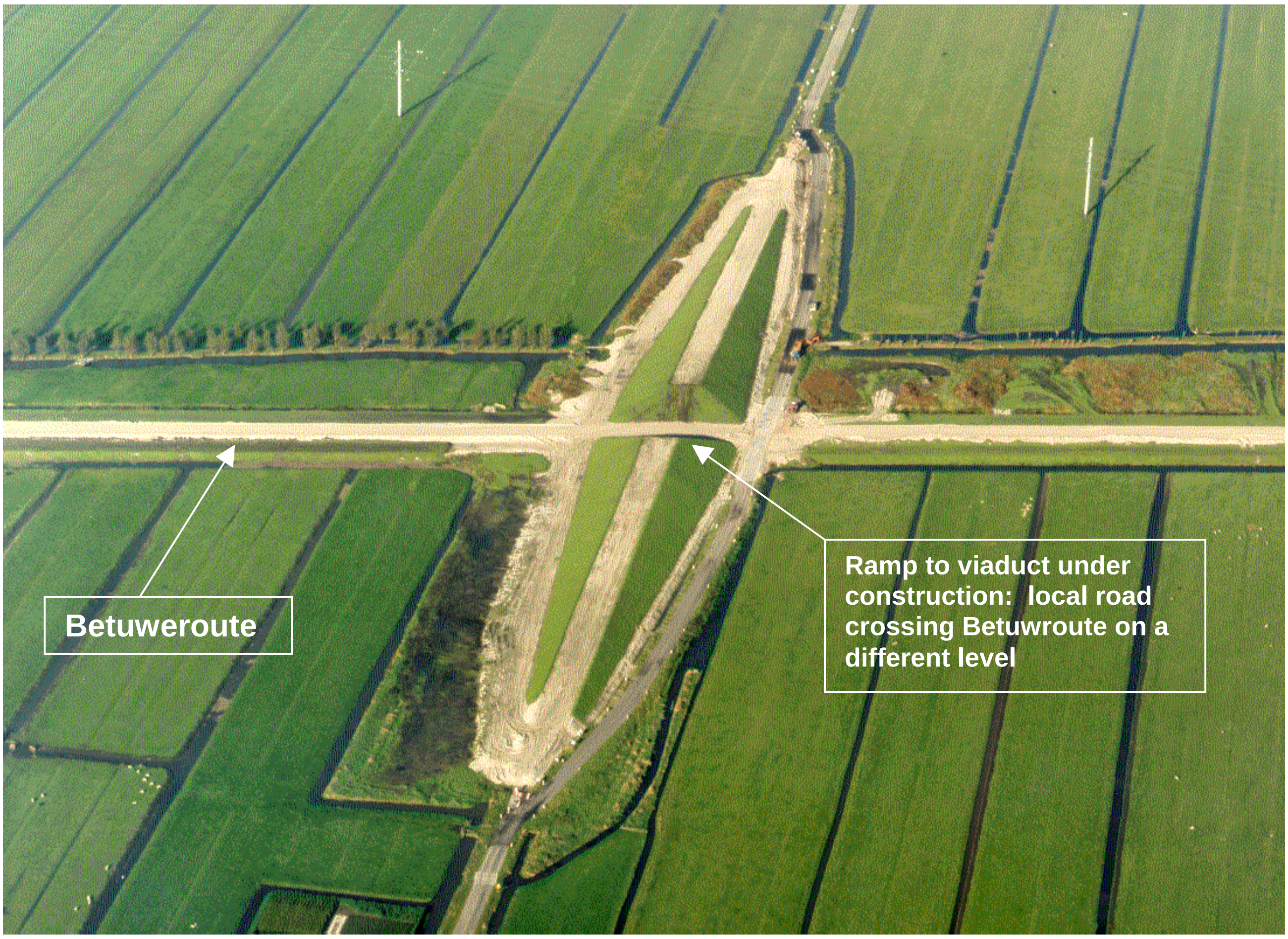

Figure 1 – Betuweroute transecting the Dutch polder landscape (© Eagle Eyes)

In the section Sliedrecht-Gorinchem described in this paper, poor soil conditions and a tight contractual time frame came together. As a simple solution for all project risks was not obvious, ProRail contracted out this section using a Design, Construct & Maintenance contract. After a pre-selection phase, five consortia were asked to the design the 22 km of the Sliedrecht-Gorinchem section keeping in mind the available time and strict directive for allowable post construction settlement. An innovative approach was stimulated by including 10 years of track maintenance in the contract sum.

Only one consortium (HBSC: Heijmans, Boskalis, Structon, CFE), dare base its design on a more or less traditional approach using a sand fill embankment and wick drains. The high geotechnical risk profile of this approach was partially compensated by a relative low bid. Confidence in the feasibility of design was gained making the geotechnical risks and possible fall back scenario transparent, however, success could only by guaranteed if both client and contractor would work together intensively.
The contract was awarded to HBSC and a close cooperation was stimulated transforming the contractual relation into an Alliance.

1.1 Soil conditions

Soil conditions in the polder section of the Sliedrecht-Gorinchem sections are extremely poor. The top 10-12 m consists of highly compressible peat and organic clays. Volumetric weights are in the range of 10.5 to 14 kN/m^3. Specific for these soils is the combination of a high compressibility with a very low permeability, leading to an extreme low coefficient of consolidation. Some examples of consolidation coefficients (c_v) determined by the Casagrande method, are shown below in Figure 1

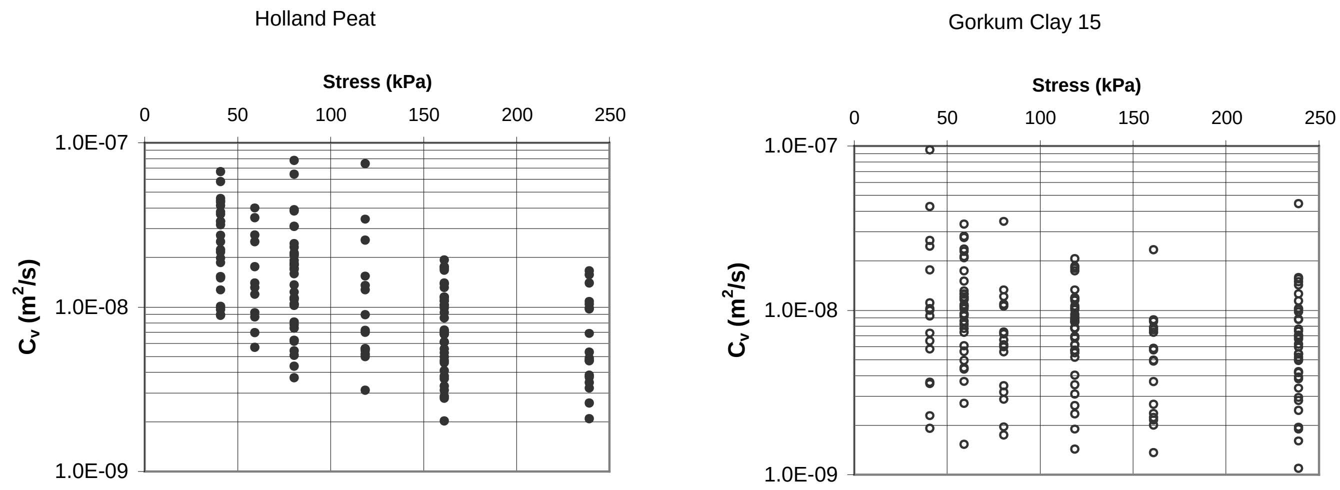

Figure 1 - Typical values of consolidation coefficient

The expected slow consolidation process introduced two major geotechnical risks, namely small construction rate and large expected creep settlement.
During construction, excess pore water pressures developed through heightening of the embankments restrict the amount of sand fill that can be placed in a certain time. To reduce post construction settlement one would like to place the sand as soon as possible and place as much sand as possible, however, the soil here dictates the maximum progress.
The second geotechnical risk is more latent in the beginning of the project however inseparable form the first, namely compliance to the contractual post construction settlement criteria. As the expected creep settlements were large, a fall back scenario had to be developed in case monitoring during construction would reveal too high creep rates. Here, an serious additional risk was the poor reliability of creep settlement predictions in general. Moreover because the future track owner had to be convinced that all basic assumptions in the design, construct and maintenance contract were indeed realised.

2. ASSESSMENT OF SOIL PARAMETERS

The soil profile typically consists of a thin (ca. 0.5 m) layer of man-made ground (agricultural clay) overlying an approximately 10 m thick sequence of Holocene peats and organic clays. Dense Pleistocene sands generally underlie the compressible layers. The Holocene units are generally soft to very soft with undrained shear strengths in the range of 5-15 kPa.

Prior to letting the Contract, the Owner conducted a large number of field and laboratory tests as shown in Table 1 Based on the results, a geologic profile was established for the route, using the local litho-stratigraphic nomenclature.

Table 1- Number of soil investigations

Test Type	Number
Dutch Cone	658
Begeman 66mm Borehole	51
Begeman 29mm Borehole	94
Laboratory Triaxial Tests	201
Laboratory Oedometer Tests	198

Statistical analysis of the available test data showed that there was no reason to assume significant spatial correlations of parameter values. Thus for each soil type only one set of parameters was required for the whole route. This considerably decreased the number of design variables and thereby reduced the required design time. A typical set of parameters for the Holland Peat is illustrated in Figure 2.

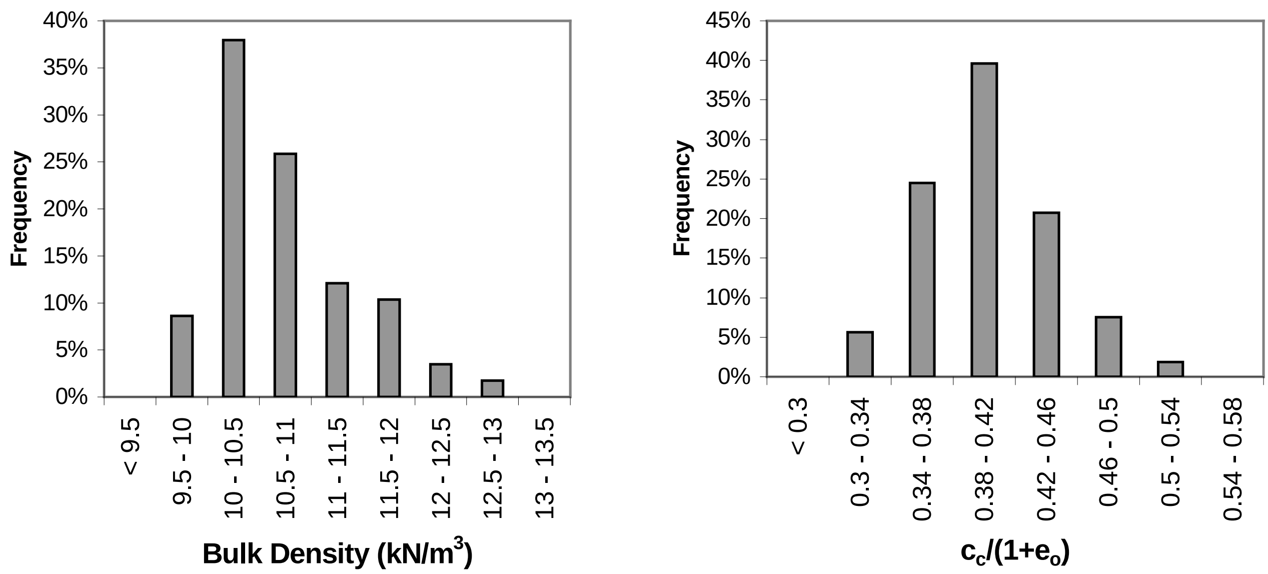

Figure 2 – Typical parameters of Holland Peat

For these soils, bulk density was found to be a good indicator of other soil characteristics. A strong correlation between the bulk density and the virgin compression ratio [=$c_c/(1+e_o)$] was observed (Figure 3).

Each step of the oedometer test was maintained for at least 24 hours. This allowed clear identification of the creep tail and quantification of the secondary compression (i.e. creep) coefficient [=$c_\alpha/(1+e_o)$]. The ratio c_α/ c_c is again strongly correlated to bulk density (Figure 4).

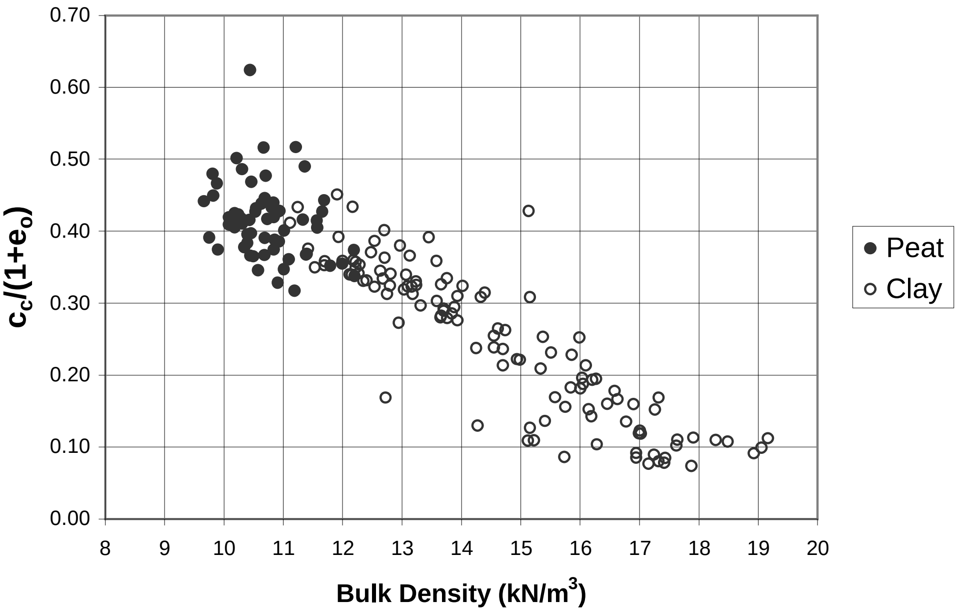

Figure 3 - Virgin compression ratio versus bulk density

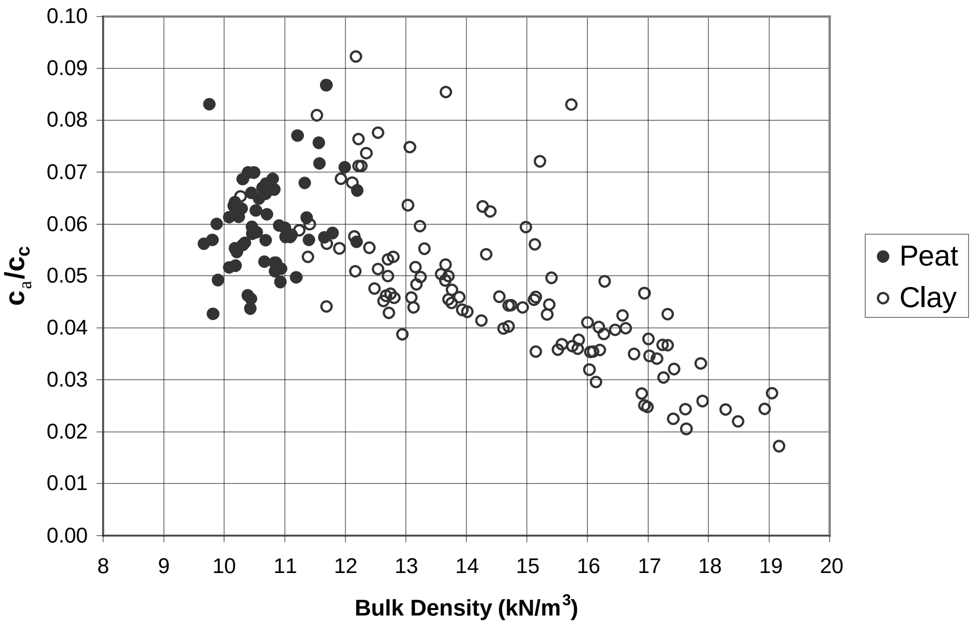

Figure 4 - C_α/C_c versus bulk density

3. SETTLEMENT PREDICTION

In the Netherlands, creep has long been recognised as having a major influence on settlement. The presence of relatively thick deposits of highly compressible material means that creep can not be neglected and must be incorporated in design calculations. The traditional approach to settlement calculations is based on the Koppejan equation.

$$\Delta s = H\left[\left(U\frac{1}{Cp}+\frac{1}{Cs}\log(\frac{t}{t_o})\right)\ln\left(1+\frac{\Delta p}{p}\right)\right]$$

where:

s	=	settlement (m)
H	=	layer thickness (m)
Cp	=	primary compression coefficient
Cs	=	secondary compression (creep) coefficient
U	=	degree of consolidation
t	=	load time (days)
t_o	=	Reference time (= 1 day)
p	=	initial effective stress
Δp	=	stress increment

The settlement is seen to be composed of two terms i.e. primary compression and secondary compression (creep). The latter is initiated simultaneously with primary compression.
The Koppejan model displays a number of shortcomings. The first is that the amount of creep is independent of the development of effective stress (i.e. consolidation). In addition, the creep increases not only with time but also with the magnitude of the total stress increment. The latter is contrary to most international practice. In the case of stepped load increments, the creep time to be used is not unequivocally defined. Lastly, the effect of a temporary surcharge (loading followed by unloading) is by no means well defined in this model.

All of these aspects are unsatisfactory while the last two introduce an unacceptable degree of arbitrariness. As more emphasis is placed on reducing post-construction settlement it is increasingly important to have a predictive model, which can be used with an acceptable degree of confidence. After due consideration, the Koppejan approach was abandoned and an in-house isotache model based largely on Yin and Graham [Yin et. al. 1999] and den Haan [den Haan, 1994] was developed. It goes too far here to describe this model in detail so it must suffice to say that this model uses the concept of equivalent creep time and isotachs (lines of constant creep strain rate). Both stepped loading and unloading conditions are clearly formulated, including an unambiguous definition of creep time.

3.1 Practical application

In practice, the minimum requirement to meet the contract specifications with respect to post-construction settlement is to ensure that consolidation is virtually complete well before construction completion. This was achieved here by placing vertical wick drains at a spacing of about 1 metre. For most of the project a small temporary surcharge of sand was also placed.
Sand was generally placed in lifts of 1-1.5 metres thick. Even with these limited lift heights stability problems were anticipated so in fact for most of the embankment a geotextile was placed on the original ground prior to sand placement. In practice very few stability problems occurred.
Settlement beacons were placed at 50 metre intervals along the axis of the embankment. The Asaoka method [Dykstra et.al., 2001] proved to be a useful tool for estimating the actual consolidation coefficient from the field settlement data. With the isotach model very good fits of the recorded settlements were generally obtained. Based on the derived fit parameters it was possible to predict the post-construction settlement at each beacon location. The model

also became an important tool to establish the required finished levels during construction (e.g. prior to placing the ballast bed).

3.2 Monte-Carlo simulation and monitoring data

Assessment of post construction settlement is very uncertain in an early stage of the project. Therefore a procedure has been developed using monitoring data to improve the accuracy of the predictions [Hölscher, 2003].
All parameters in the isotachen settlement prediction model, i.e. the time periods and height of the staged construction, the thickness of the subsoil layers and the parameters of the settlement model are stochastic values, characterised by a mean and standard deviation.
A Monte-Carlo simulation has been executed on 10.000 sets of parameters. For each case a settlement curve as function of time is calculated. In figure 5 the results of all 10.000 settlement curves are shown as mean, 5%, 50% and 95% confidence intervals. Especially the consolidation parameter increased the uncertainties in the predictions. Based on these curves the probability of a residual settlement lower than the required 0.3 m is only 30%. So the change of not meeting the requirements is 70%, which was unacceptable.

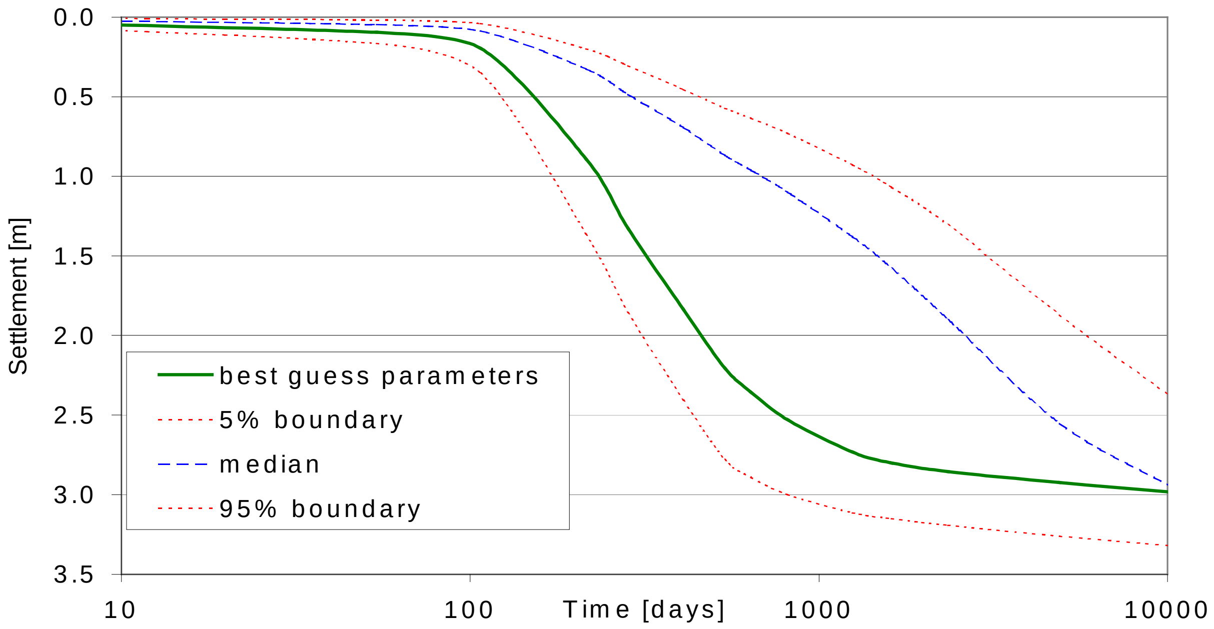

Figure 5 - Range of calculated settlement curves Monte Carlo simulation

Therefore the question raised how to increase the accuracy of the predictions by adding the monitoring data? First of all the accuracy of the monitoring data is also a stochastic value. It is therefore assumed that of the 10.000 calculated curves, those, which are within 0.1 meter of the measurements, are representing a possible and realistic curve. Then 83 of the 10.000 fit in this demand. In figure 6 the mean, 5% and 95% confidence interval of these 83 curves are shown. The range of possible curves is much smaller than in figure 5 Adding the monitoring data did decrease the range of possible setting curves. For the completion of the railroad embankment, 3,5 year after start of the work, the predicted residual settlements up to 30 years after construction are now predicted as 0,2 m with a standard deviation of 0,035. The probability of a post construction settlement lower than the required 0.3 m is 99,79%. So the change of not meting the requirements is only 0,21%, which is very acceptable.

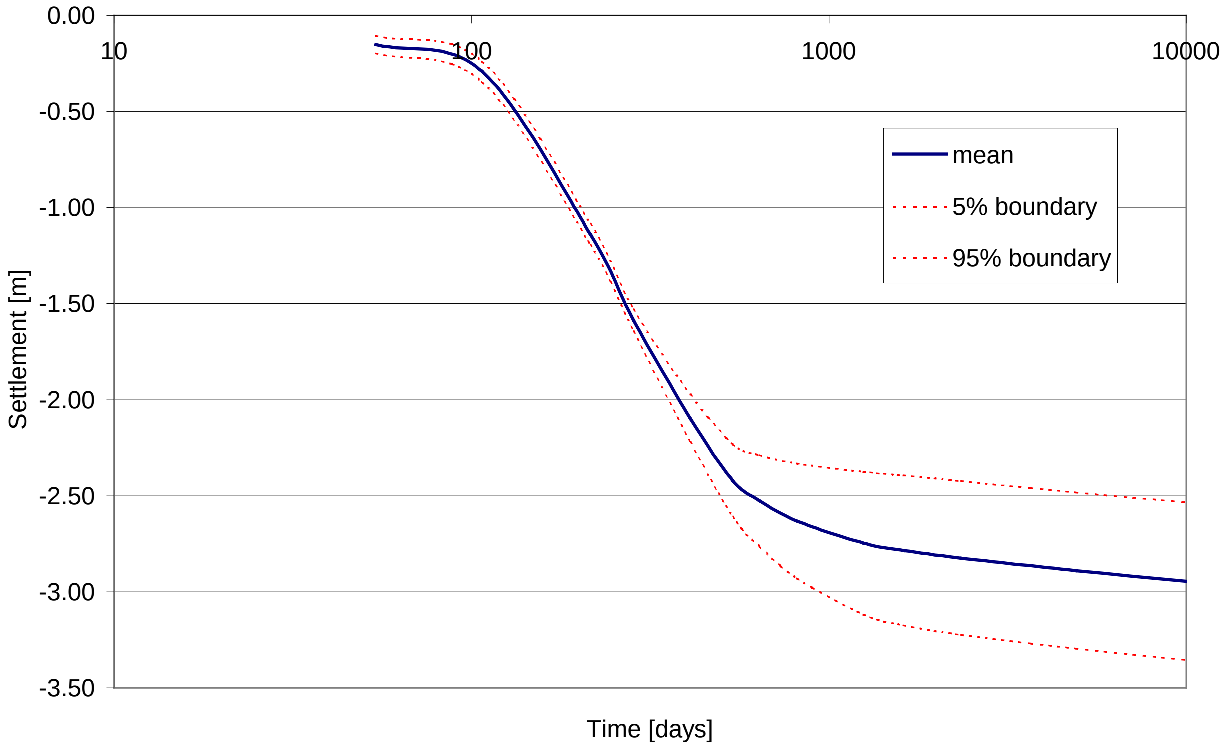

Figure 6 – Mean and 90% confidence interval for 83 selected curves

It is concluded that adding monitoring data in the prediction of post construction settlements reduces the uncertainties substantially. The Monte-Carlo simulation can be executed during the design stage and measurement data can be used from early stage of the monitoring process in order to reduce the uncertainties in the prediction of residual settlements. If the risk that requirements are not met is too large, early measures can be taken to reduce this risk and optimise the building process.

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