Geotechnical applications and conditions of the observational method

S. van Baars and J.K. Vrijling
University of Technology Delft
Faculty of Civil Engineering

Information observed during construction can be used to optimise the remaining parts of the construction or structure. In this article general conditions are indicated for the implementation of the Observational Method in geotechnical engineering. The most important condition is that the uncertain field condition (the observational data) must be clear to measure. For example in case of a brittle soil behaviour, there is no clear indicator to show that the load is approaching the maximum resistance. If a project fulfils all general conditions, a financial risk analysis is necessary to find out whether the Observational Method is profitable. The success of implementation varies strongly per geotechnical discipline. In this article a simplified approach for a risk analysis is given and a number of geotechnical cases are discussed for which the Observational Method can lead to a financial profit. Most of these cases are already informally implemented in daily practice.

Keywords: Observational method, geomechanics, design, cost and risk analysis, Bayesian updating

1. Introduction

Design calculations are commonly based on information which is available before any construction activity has started. If enough margin is built in between load and resistance, the structure will normally not fail during or after construction. In some cases, important information can be obtained on the geotechnical conditions during construction. This information might indicate that the design is too safe, which means too much money is spent. Or that the safety is insufficient, which means adjustments have to be made to avoid a disaster. If this information gathered during construction could be used in time, cheaper or safer structures could be made. If this gathering of information was part of design considerations from the beginning (*ab initio*), this method is defined as the Observational Method. In the Ninth Rankine Lecture, Peck (1969) set out procedures for the Observational Method in soil mechanics. Peck identified two applications for the Observational Method: a) *ab initio*: from inception of the project and b) *best way out*: during construction when serious site problems develop. This article only discusses the *ab initio* application.
There are at least two main options within the Observational Method:

1. Start with a standard structure, designed in a normal way. If during construction deformations or settlements indicate a more positive situation than expected, the (remaining part of the) structure can be simplified. In the same way very large deformations or settlements can lead to reinforcement of the structure.
2. Start with a lighter and cheaper structure, which is designed on basis of rather positive input data. If during construction deformations or settlements indicate a more negative situation than expected, the structure is reinforced.

Peck proposed that construction work should be started with a design based on the most probable soil conditions. However, this will not always lead to a solution with the lowest expectation value of the costs. This can only be solved with a financial risk analysis (see Benjamin and Cornell, 1970), because the consequence of failure is of paramount importance. This paper is organised as follows: firstly ten general conditions for the Observational method are mentioned, secondly a concept for a financial risk analysis is developed, thirdly two examples are presented and finally the Observational Method is discussed per geotechnical subject.

2. General conditions

In order to be able to implement this observational method, one has to meet the following ten general conditions:

1. The observational method that implies changing the design during construction should not be excluded by law or contract.
2. There must be a considerable uncertainty of the actual field conditions.
3. The uncertain field condition (the observational data) must be somehow observable during construction.
4. If the soil strength is the uncertain field condition, then the soil behaviour must not be brittle. So, after reaching its maximum value, the strength of the soil should not decrease suddenly.
5. Disappointing, expected or favourable field conditions must lead to an appreciable difference in the cost or risk of the structure or construction.
6. The design of the structure can be adapted, simplified or reinforced after the observations have been obtained.
7. This means that the construction consists of at least two (but preferably more) stages.
8. The response time for monitoring and implementation must be appropriate to control the work.

9. If the construction is started with a lighter structure (option 2), one has to be sure that during the first stage no maximum load can occur, which leads to failure before the structure can be reinforced (see point 8).
10. The costs of changing the structure (extra costs times probability of exceedance) should be less than the profit (saving by the lighter structure times probability of exceedance).

Point 3 about the uncertain field condition (the observational data) is for most geotechnical cases the biggest problem, since the observational data must lead at the right time to specific information of a specific soil layer. (For example the settlement of an embankment depends on the stiffness, the permeability and the creep of each soil layer, so the overall settlement is often not a good indicator for the degree of consolidation of one particular layer).
This point is therefore also related to point 4, the problem of brittle behaviour. In case of a sudden failure there is insufficient time to reinforce the structure between the first indication of collapse and the moment of collapse.
If the first 9 general conditions apply, the Observational Method can be implemented. According to condition 10, this will not always be cost-effective, however. For example; if the uncertainty of the actual field conditions can be solved with additional soil investigation and if this is cheaper than the additional monitoring system for the observational method, the common method will always be cheaper. So, the choice for the Observational Method depends theoretically on a financial risk analysis.

3. Financial risk analysis

The financial risk analysis presented here is based on the simplest OM (Observational Method) case possible: a case with only two construction stages (*a* and *b*), of which the second depends on the data observed during the first. The second construction stage can be build as a standard design or a lighter design. For this simple case the expectation value *C* of the total costs should be calculated for both options:
1) The standard or common design, without OM
2) The OM design, starting with a common (or even other) design

The expectation value of the cost *C* per option can be approximated by:

1) Standard: $$C_S = S_a + S_{b=C} + P_{F;b=C} \cdot F \tag{1}$$

2) OM: $$C_{OM} = S_a + P_{b=L} \cdot \left(S_{b=L} + P_{F;b=L} \cdot F\right) + \left(1 - P_{b=L}\right) \cdot \left(S_{b=C} + P_{F;b=C} \cdot F\right) + M \tag{2}$$

in which:

C_x = expectation value of the total cost for option x (Standard: $x = S$; OM: x = OM)
S_a = construction cost of the Structure of stage a (first stage)
$S_{b=y}$ = construction cost of the Structure of stage b being y (common: $y = C$; light: $y = L$)
$P_{b=y}$ = Probability of stage b being y (common: $y = C$; light: $y = L$)
$P_{F;b=y}$ = Probability of Failure of structure if stage b = y (common: $y = C$; light: $y = L$)
F = total cost of Failure
M = total cost of Monitoring

Both equation 1 and equation 2 contain the probability of failure of the common structure $P_{F;b=C}$. Since the probability of failure of equation 2 depends on the intervention criterion (unlike equation 1), these parameters are not exactly the same. However, the values are almost the same, because the probability that a light structure will not be upgraded in case a standard structure would fail, can be neglected. Also the probability of failure of the light structure $P_{F;b=L}$ and the probability that the observational data recommends a lighter structure ($P_{b=L}$) depend on the intervention criterion.
The cheapest option, i.e. the option with the lowest expected value of the costs, should be chosen.
The equation of the OM-case is merely an approximation because it already contains some simplifications, for example the total cost of failure F is regarded the same for both the light and the standard option. Another point is that the probability of failure is simply multiplied by the probability of changing the structure, while these are not completely uncorrelated.
The simplified equations are used here because it is not possible to make very accurate calculations of the different probabilities of failure with a limited amount of soil data and without the observational data. Besides, this approach results in some important general conclusions. For example, from equations 1 and 2 it follows that the Observational Method is expected to be profitable in case:

$$M + F \cdot P_{b=L} \cdot \left(P_{F;b=L} - P_{F;b=C}\right) < P_{b=L} \cdot \left(S_{b=C} + S_{b=L}\right) \qquad (3)$$

Once the decision is made to implement the Observational Method and the construction has started, a new question arises, which is whether an intervention to a lighter structure can be accepted. This intervention depends on the observational data X, which influences the posterior probability of failure of the structure $P_{F;b=x|X}$. The two possible options for the costs of construction phase b depends on the intervention:

1) intervention: standard to light: $$C_{OM;b=L} = S_{b=L} + P_{F;b=L|X} \cdot F \quad (4)$$

2) no intervention: remaining standard $$C_{OM;b=C} = S_{b=C} + P_{F;b=C|X} \cdot F \quad (5)$$

in which:

$P_{F;b=x\,|\,X}$ = the posterior Probability of Failure of the structure regarding observational data X

According to the equations 4 and 5 an intervention to a lighter structure is expected to be profitable in case:

$$\left(P_{F;b=L|X} - P_{F;b=C|X}\right) < \frac{S_{b=C} - S_{b=L}}{F} \quad (6)$$

This can be obtained by a relative low total cost of failure F, by a relative high profit of the lighter structure ($S_{b=C}$ - $S_{b=L}$) or, what is often more important, by a strongly reduced probability of failure of the lighter structure $P_{F;b=L}$, according to the observational data (point 3 of the general conditions). A strongly reduced probability of failure can only be achieved by a strong correlation between the parameter which determines the probability of failure and the parameter determined by the observational data. In other words observed deformation data will be in most cases insufficient for reducing the probability of failure caused by a lack of strength and strength data will be insufficient for reducing the probability of failure caused by a lack of stiffness.

Note that the decision to perform an observational method falls within the theoretical framework of the preposterior bayesian analysis The actual processing of the observations is called the posterior analysis. There is also a resemblance with condition based maintenance, where maintenance decisions are based upon the condition of the structure observed during inspections.

This case with two stages and two options in the second stage can be extended with more options per stage (for example an extra heavy design option) or more stages (more possible intervention times), but this will make the risk analysis far more complex. The approach described above will be the same however.

An observational approach of the soil strength is shown in the first example (Abutment foundation) of the following chapter. An observational approach of the settlement is shown in the second example (Settlement of an embankment). The probabilities of failure in both examples are calculated by numerical integration. An other calculation method is a Monte-Carlo simulation.

4. Examples of the observational method

*4.1 **Example 1: Abutment foundation***

In this example a bridge is supported by an abutment. This abutment is also loaded by the weight of the embankment next to it. The maximum height of the embankment is 4 m. The slope in cross section is 2 to 1. The angle of internal friction of the sand is not exactly known and is characterised as a normally distributed random variable with the following average and standard deviation:

$\phi \sim N(\mu,\sigma) = N(34.0°,2.8°)$

In case a shallow foundation is chosen for the abutment, there will be failure if the angle of internal friction is less than $\phi<\phi_{min}$ = 29.7°, according finite element calculations.

The probability of failure of the shallow foundation is according this data:

$P_F\,(\phi<\phi_{min}) = 0.0623$

If the structure fails, the value of the total damage will be F = k€ 1000. However, the abutment can be founded on piles instead. In this case there is almost no probability of failure. The pile foundation will cost additionally $S_{b=C}$ - $S_{b=L}$ = k€ 30. The question is whether it is profitable to install the pile foundation in order to reduce the expectation value of the total cost.

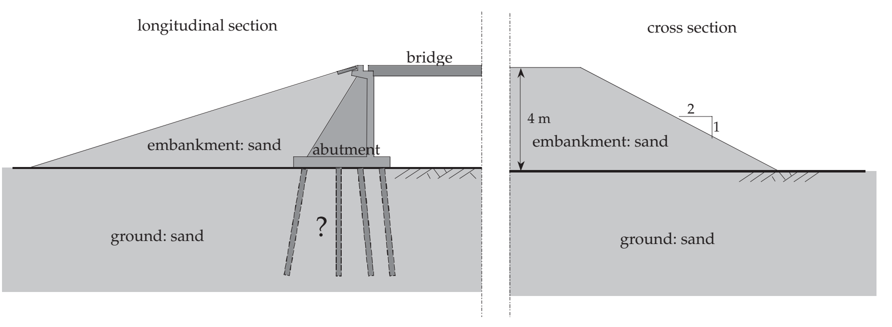

Figure 1. Example 1: Abutment foundation: situation sketch

The use of a shallow foundation is not profitable without the use of the observational data since the probability of failure is too large:

1000 k€·0.0623 > 30k€ ⇒ 62 k€ >30 k€

The Observational Method is introduced by calculating the minimum required angle of internal friction necessary for a stable slope in cross section. According to a Bishop circular slip surface calculation the slope is only stable when $\phi>\phi_{stable}$ = 29.1° (in this case $P_F\,(\phi>\phi_{stable}) = 0.0401$).

The idea is to use a shallow foundation in stead of the pile foundation only in case the slope is stable. The probability of failure of the abutment will then be:

$$P_{F;b=L|X} = P_F(\phi_{stable} < \phi < \phi_{\min}) = \frac{0.0623 - 0.0401}{1 - 0.0401} = 0.0231$$

The monitoring costs are limited, because only the stability of the slope of the embankment in cross section has to be checked visually. According to equation 3, the Observational Method is profitable, because:

$$\begin{aligned} M + F \cdot P_{b=L} \cdot \left(P_{F;b=L} - P_{F;b=C}\right) &< P_{b=L} \cdot \left(S_{b=C} + S_{b=L}\right) \\ \Rightarrow \quad 0 + 1000 \text{ k€} \times P_{b=L} \times \left(0.0231 - 0\right) &< P_{b=L} \times 30 \text{ k€} \\ \Rightarrow \quad 23 \text{ k€} &< 30 \text{ k€} \end{aligned}$$

The expected profit is in this case 30 k€ - 23 k€ = 7 k€.
A decision tree summarizing the three alternatives is shown in figure 2.

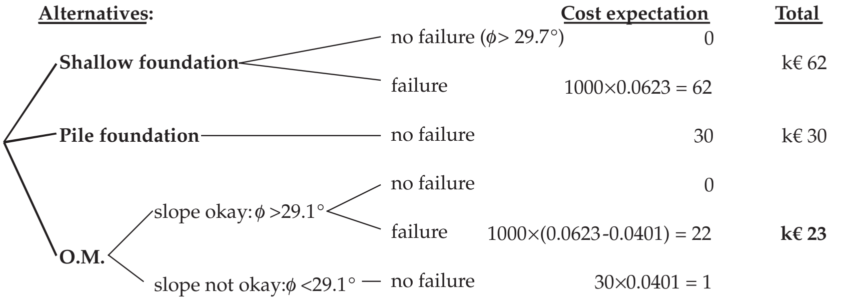

Figure 2. A decision tree with cost expectations of the alternatives

The observational method works well in this example, because the probability of failure can be reduced strongly with (almost) no monitoring costs and the mentioned intervention and failure costs have, in this case, the right ratio.

4.2 *Example 2: Settlement of embankment*

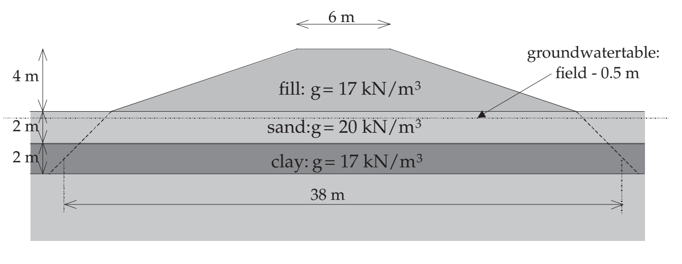

Figure 3. Embankment; geometry and specific weights

Suppose a fill embankment for an entry ramp of a High-Speed-Train bridge is made on a sand layer which is on top of a clay layer. In this example the geometry is known, see figure 3. The additional load of the new embankment is assumed to be linear over a width of 38 m of the clay layer. The initial vertical effective stress is in the centre of the clay layer is $\sigma'_{v,i}$ = 32 kPa. After loading the vertical effective stress becomes σ'_v = 83 kPa. The corresponding settlement is predicted with the method of Koppejan (1948). According to this method the strain of the clay layer is:

$$\varepsilon = U\left(\frac{1}{C'_p} + \frac{1}{C'_s}\log(t)\right)\ln\left(\frac{\sigma'_v}{\sigma'_{v,i}}\right) \tag{7}$$

In which:

$\varepsilon = \frac{\Delta H}{H}$ = strain [-]

ΔH = settlement [m]

$h=H/2$ = draining distance = half the layer thickness [m]

U = degree of consolidation [-]

C'_p = primary settlement stiffness [-]

C'_s = secondary (creep) settlement stiffness [-]

t = time after loading [d]

σ'_v = vertical effective stress after loading [kPa]

$\sigma'_{v,i}$ = initial vertical effective stress [kPa]

The degree of consolidation can be approximated by:

$$U \approx \frac{2}{\sqrt{\pi}}\sqrt{\frac{c_v t}{h^2}} \quad (U < 0{,}5) \tag{8}$$

$$U \approx 1 - \frac{8}{\pi^2}\exp\left(-\frac{\pi^2}{4}\cdot\frac{c_v t}{h^2}\right)(U > 0{,}5) \tag{9}$$

The vertical coefficient of consolidation depends on the permeability and stiffness of the soil:

$$c_v \approx \frac{k \cdot \bar{\sigma}'_v \cdot C'_p}{\gamma_w} \tag{10}$$

In which:

k = permeability of soil [m/d]

$\bar{\sigma}'_v = (\sigma'_v + \sigma'_{v;i})/2$ = average effective stress [kPa]

γ_w = specific weight of water [10 kN/m^3]

Suppose the three clay parameters are uncertain. These are random variables with a coefficient of variation of 20%, so:

$$C'_p \sim N(\mu,\sigma) = N(20,\ 4)$$
$$C'_s \sim N(\mu,\sigma) = N(100,\ 20)$$
$$k \sim N(\mu,\sigma) = N(40.0 \cdot 10^{-6}\ \text{m/d},\ 8.0 \cdot 10^{-6}\ \text{m/d})$$

Figure 4 shows the expected value of the settlement over time.

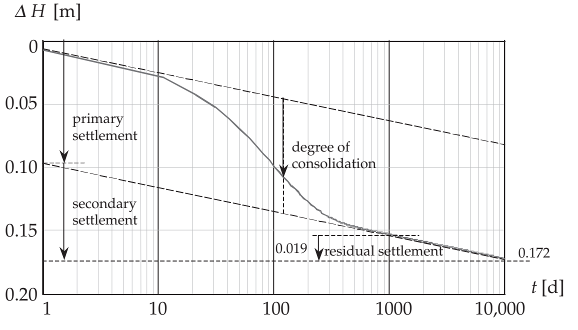

Figure 4. Embankment; settlement versus logarithmic time

The expected final settlement $\Delta\overline{H}_{t=10,000}$ is found to be 0.172 m. However not the final settlement is important in this case, but the residual settlement between the commissioning date and the final date. Suppose the following points:

- t = 100 days, which is halfway the logarithmic time scale, is the last moment to intervene.
- At t = 1000 days (the commissioning date) the rail is installed on the embankment. This is the start of the residual settlement.
- At t = 10,000 days (the final date) the rail and the embankment are reconstructed. This is the end of the residual settlement.
- There are two options: 1) No temporary surcharge is used, 2) A temporary surcharge is used between t = 100 days and 1000 days, to accelerate the settlements. The additional cost for this option is k€ 10.-.
- The residual settlement should be less than $\Delta z = \Delta H_{t=10,000} - \Delta H_{t=10,000} < 0.025$ m.
- The penalty for non-compliancy is F = k€ 100.-.

The contractor wants to use option 2 in case the probability of non-compliance is more than:
$P_{\min}$ = k€ 10/k€ 100 = 0.100
In this case the expected value of the residual settlement is only Δz = 19 mm but the prior probability of exceedance of the maximum residual settlement is:
$P_F(\Delta z > 0.025) + 0.120 > P_{\min}$
This means it is more safe and slightly cheaper (on average) to use the temporary surcharge. However the Observational Method might save some money.

In this example the settlement at t = 100 days is observed to be only 8% more than expected (i.e. 28% upper limit!). With the same method as example 1, the posterior probability of exceedance can be found:

$$P_F(\Delta z > 0.025 \,|\, \Delta H_{t=100} \approx 1.08 \cdot \Delta \overline{H}_{t=100}) = 0.139 > P_{\min}$$

So, the probability of non-compliancy has increased. This means that the observation of the settlement gives only poor information. The reason for this can be seen in figure 4. The settlement from t = 1000 d to t = 10,000 d is completely controlled by creep (C'_s), but the settlement at t = 100 days is for 71% controlled by the primary settlement (C'_p) and the consolidation (c_v or k). In other words, this problem does not satisfy condition 3 which says that the uncertain field condition (C'_s) must be observable for successful implementation of the Observational Method.

The Observational Method is more successful in case the primary and secondary stiffness parameters are strongly correlated, which is often the case for natural clays. Suppose the correlation is in this example almost 100%. The prior probability and the posterior probability of exceedance will then become:

$$P_F(\Delta z > 0.025) = 0.121 > P_{\min}$$

$$P_F(\Delta z > 0.025 \,|\, \Delta H_{t=100} \approx 1.08 \cdot \Delta \overline{H}_{t=100}) = 0.069 < P_{\min}$$

Hence, the correlation improves the estimate of the residual settlement such that in this case no temporary surcharge is necessary anymore. In order to benefit from the Observational Method the uncertain field condition (creep) must be directly on indirectly observable. Two situations apply to this. First, the intervention date is long after the primary consolidation, which is seldom the case. Second, there is sufficient correlation between the primary and secondary stiffness.

If the failure criterion was not about residual settlement, but about final settlement, than the Observational Method becomes even more useful. Suppose the same embankment must only have a height of 4 m with a tolerance of 20 mm on day t = 1000 d. Therefore the contractor restores the settled embankment back to 4 m height at day t = 100 d and leaves the site. He comes back on day t = 1000 d to restore the additional settlement. However, he wants to avoid the costs of this last restoration. Therefore he decides to apply the Observational Method by making the embankment higher on day t = 100 d with an extra height of:

$$\Delta H = \frac{\Delta \overline{H}_{t=1000}}{\Delta \overline{H}_{t=100}} \cdot \Delta H_{t=100} = 1.546 \cdot \Delta H_{t=100}$$

In other words, the expected ratio between the settlement on day t = 100 d and the final settlement on day t = 1000 d is assumed to be the real ratio. This is not completely correct, but by using the observational data (the settlement on day t = 100 d) and this expected ratio, the contractor only has to come back in 21% of the cases. In this way the Observational Method becomes both simple and useful. It can even be further optimised by measuring the pore pressures too. Suppose pore pressures indicate that the permeability and consolidation is as expected (i.e. average). Then a risk analysis shows that the contractor only has to come back in less than 10% of the cases:

$$P_r(\left\|\Delta z_{t=1000}\right\| > 0.020 \,\middle|\, k \approx \bar{k}) = 0.096$$

5. Qualitative discussion on a geotechnical observational method

The observational method in geotechnical engineering is most clearly explained if the main subjects within geotechnical engineering are treated separately:

1. Settlement and consolidation
2. Stability
3. Groundwater flow
4. Horizontally and vertically loaded piles and pile groups
5. Sheet pile walls
6. Bored tunnels

*5.1 **Settlement and consolidation***

In practice the Observational Method is often used informally for the design of structures subjected to settlement and consolidation. The uncertainty of these structures is caused by the unknown stiffness and permeability of the related soft soil layers. Therefore the design contains several options with different settlement periods during construction, an extra surcharge or a different drain distance. The choice between these options may depend on settlements observed during construction. The structures can be divided in (A) Single loaded structures and (B) Repetitive step loaded structures:

(A) The second example in this paper, a settlement caused by an embankment, falls in this category. If the failure criterion depends on the final settlement, then it is easy to benefit from the Observational Method. If it depends on the residual settlement, than there must be a strong correlation between primary and secondary settlement, or the intervention date must be long after the consolidation phase. A good example of this last condition is the Foyle Bridge East Abutment. The free-draining sandy ground conditions ensured that the embankment consolidation was completed within a few months after the fill was placed. The date of the abutment shimming option was 2.5 years later, which was enough to observe the creep behaviour (Nicholson and Low, 1994).

(B) An example of a repetitive step loaded structure is the construction of a coastal reclamation project, such as the Singapore's Changi Airport (Choa, 1994). The load is repetitive when the embankment grows horizontally in a slow process. In this case the different areas are in different consolidation phases. If the subsoil is rather homogeneous, data of filled area's can be used for areas still to fill. This is an ideal situation for the Observational Method. One can economically optimise the vertical drain distance in the process, since a 10% reduction in the drain distance results in 21% less drains. The additional conditions for implementing an Observational Method are in this case:

- The loading is repetitive.
- The time between the repetitive loadings must be a substantial part of the consolidation time, in order to have useful settlement observations.
- The soil behaviour of the different areas is strongly correlated (homogeneous).
- This correlation must be known for optimisation of the loading process.

*5.2 **Stability***

The stability of a structure depends both on the load and the resistance (strength). Observed displacements give very poor indication of the posterior probability of failure. Weak soil layers can be overlooked, the soil behaviour might be brittle and the correlation between material strength and material stiffness is rather weak. At continuous rapid loading the stability decreases while the displacement grows. Over time however, the displacement grow, due to consolidation of clay and peat layers, but the increase of the effective soil stresses increases the stability. This shows that soils have a very poor stiffness-strength relation, so the displacement is seldom a good indicator for the stability during the construction phase of a structure and is therefore in common practice seldom used.

For the calculation of the total strength of a structure, samples of all related soil layers must be taken from the site and tested in a laboratory. In order to use the Observational Method for (ground) structures, one has to obtain extra information about the uncertain field condition (Point 3: the observational data), which is in this case the strength. The first example about the abutment foundation shows that other nearby structures can provide the additional data. Also the structure itself can provide this information, for example the fact that a structure did not fail during a certain loading in the past.

For dikes the maximum load does not occur during the construction phase but after construction during high water at sea or in the river, during the exploration phase of the dike. Previous high water loads, in which there was no dike failure, can be used for calculating the proven strength of a dike for a risk analysis, see Calle & Van Der Meer (2001). The proven strength can also be used for a design of a dike enlargement.

*5.3 **Groundwater flow***

The theory of groundwater flow is important for the design of dewatering installations for building pits. The effectiveness of a pumping installation depends highly on the geometry of the soil layers and their permeability. Especially when there are sand lenses within clay layers, the uncertainty is high, since the permeability of sand is several orders of magnitude larger than the permeability of clay. The major sand lenses can be found by cone penetration tests, but a single dewatering test with pumps and water table gauges is more informative. It can be cheaper and easier than the cone penetration tests too. A small dewatering field test, as a part of a soil investigation, gives enough information for a total design (so, monitoring costs are low). It can also be part of the actual dewatering installation. Dewatering is started with a minimal number of pumps. If the result is disappointing, more pumps can be added. In this way it is a form of Observational Method which is already often used (see for example Roberts and Preene, 1994). In case of a high risk, lack of time (dewatering on critical path) or a high uncertainty (initial site investigation reveals circumstances where a dewatering system cannot be specified in detail), a start with a heavy system is appropriate. If possible some pumps can be removed early. This can be optimised with a financial risk analyses.

5.4 ***Horizontally and vertically loaded piles and pile groups***

For piles we can distinguish Observational Methods for (A) The installation phase, and (B) The loading phase (construction phase of the top structure).

(A) In the pile installation phase usually many identical piles are installed in the same area. These are driven one by one, so the pile driving can be optimised in the process. If the pile driving shows problems, the pile hammer is adjusted or a different hammer is used. This form of Observational Method is already common practice.

(B) For the loading phase the Observational Method is in most cases not applicable, since the loading of the piles mostly occurs after installation of the last pile. The installation of extra piles during construction is generally too difficult. One can apply a test load on a few piles, after their installation and before the construction of the top structure. In this way a much better design value for the strength can be obtained. The problem is that a large load has to be installed temporarily to let a pile fail. In normal cases this is more expensive than installing a few extra piles, but in case of a large number of piles this can be financially attractive. This form of Observational Method is also already common practice. The Dutch design code NEN 6740 (Geotechnics: Basic rules and loads, table 3) gives a lower material safety factor for tested geotechnical structures like piles: $\gamma_{m;b}$ = 1.25 instead of 1.40. Besides this, also a gain can be made by obtaining higher failure loads from the tests than calculated. In the Netherlands especially for sands with a high cone penetration resistance (CPT), a profit can be expected, since values of q_c > 15 MPa are not allowed to be used in design, unless test loading has proven the pile resistance (NEN 6743 Geotechnics: Pile foundations, fig. 5). Also in case of a sand layer with only a few weak spots (CPT with low q_c) the reality can be less negative than the design rules for piles assume.

A good example of test loading is the underpinning of an existing three-storey underground shopping mall, while a new subway tunnel was to be excavated (see Iwasaki, 1994). The cast-in-place concrete piles, designed as friction piles, were all preloaded. Since the large load (the mall itself) was already above the piles, only a load cell was needed for testing the piles.

5.5 *Sheet pile walls and diaphragm walls*

In the same way as for piles we can distinguish Observational Methods for sheet pile walls in methods for (A) The installation phase, and (B) The loading (excavation) phase.

(A) The installation phase for sheet pile walls is the same as for piles. During the installation phase usually many identical sheet piles are installed in the same area. These are driven one by one, so the sheet pile driving can be optimised in the process. This form of Observational Method is already common practice. If the sheet pile driving shows problems, the vibrator or pile hammer is adjusted or a different vibrator or hammer is used.

(B) For the excavation or loading phase the Observational Method can be used in case repetition is possible. A good example is a long excavation (for example a cut and cover tunnel) with many parts or sections, that are excavated subsequently in rather homogeneous soil conditions. In this case observational data (stresses and displacements) of the first part of the excavation can be used in the design of the following excavation parts. Sometimes the repetition is created by making an additional test excavation first (see Young and Ho, 1994). One has to be prepared for less homogeneous soil conditions though.

For a single excavation there is strictly speaking no repetition. This makes the use of the Observational Method very complex. During excavation and dewatering of the excavation pit the loading is increased step by step and several observations can be made during the construction phases, like:

1. Bending of the sheet pile wall (displacement)
2. Stresses in the sheet pile wall (strain gauges)
3. Stresses in the anchors or struts (strain gauges)

These obtained data can be used in a Bayesian updating (see Ikuta et al, 1994), but this leads seldom to profit, because there are at least five serious problems for single excavations:

1. The three characteristics mentioned above (displacement of sheet pile, moment in sheet pile and anchor or strut forces) are highly correlated. This means there is much less data than variables of the different soil parameters, so the observational data can not lead to a more accurate parameter set to improve the predictions of the coming construction phases.
2. If there are several soil layers, then excavation of the first phase of a excavation gives extra data about the upper soil layers, but not of the deeper soil layers.
3. During the first part of an excavation the soil is mainly behaving elastically, so mainly extra information about the elastic parameters (stiffness) is obtained. However, during the last part of the excavation the soil is behaving more plastically, so extra information about the plastic parameters (strength) is in fact needed.
4. The only way to save money is by reducing the number of anchors, struts or their supporting waling beams, since the sheet pile wall is already installed. The reduction can be in horizontal and in vertical direction. An increase of the horizontal distance l between

the anchors or struts leads to an even larger increase in the bending moment of the waling beams (quadratic relation!), which can be unacceptable, especially when the waling beams have been designed and made already.

5. A reduction in vertical direction sounds more interesting since this does not only reduce anchors or struts, but also gives less hindrance during construction. The problem is however that a reduction from 3 to 2 layers or from 2 to 1 layer of supports causes an increase in the stresses in the sheet pile wall or diaphragm wall, which is in most cases too large.

The first three problems are related to the general condition about the uncertain field condition (Point 3: The observational data). Problem 3 becomes especially clear in cases of a high ground water table. Most excavation pits are in this case made by first excavation and finally dewatering. The active lateral earth pressure coefficient is often low ($K_a < \frac{1}{3}$) while the pore pressure is the same in all directions (K_{water} = 1). Therefore the relative load on the sheet pile wall after excavation and before dewatering is less than a quarter of the final load:

$$\frac{\sigma_{\text{hor; exc.}}}{\sigma_{\text{hor; exc.+dew.}}} = \frac{K_a}{K_a + K_{water}} < \frac{1}{4} \tag{11}$$

Because of the non-linear stress-strain behaviour of soil this load after excavation and before dewatering leads to displacements, forces and bending moment of only 10% to 20% of the values of the final phase. This is too small for an accurate extrapolation or Bayesian updating for the final construction phase. This means that the Observational Method is not attractive for most sheet pile wall or diaphragm wall excavations. This explains the statement of Ikuta et all (1994) that the observational data are rarely used to modify the design of subsequent stages of construction. Only very exceptional cases satisfy all conditions mentioned above. One of these is for example an underground construction with a single homogeneous soil layer, with little or no groundwater, supported by horizontal main struts which are supported by smaller diagonal struts (between diaphragm wall and horizontal strut). In this case the diagonal supports can be subject to change on the basis of observational data. See for example the deep basement excavation discussed by Ikuta et all (1994).

5.6 *Bored Tunnels*

The construction of a bored tunnel in rock with a low permeability (Switzerland) is rather different from a bored tunnel in soft clay or highly permeable sand under the water table (Netherlands).

In the first case very often the New Austrian Tunnelling Method (NATM) is used, which is in fact an Observational Method, since the experience of the previous tunnelling is constantly used for adjusting the process. Already in 1939 the Observational Method led, for the construction of the Chicago Subway, to a reduction of the total settlement of the centre of the street from 9 inch to 3 inch (Peck 2001). The use of this method is correct as long as the load is not changing too fast in the process (depth of tunnel and rock weight) and the strength of the rock is relatively constant. Therefore one should be careful with the NATM in a city, since the load (for example a truck or a bus) can change relatively fast, which can lead to a sudden failure (see figure 5). Then the NATM does not satisfy general conditions 3 and 4 (clear observational data / brittle behaviour) and condition 8 (enough response time).

In case of a combination of soft soils (sand and clay) and a high water table, an open method as the NATM is very difficult, because the soil has to be supported during construction and the working area should be free of groundwater. Freezing is a possibility but mostly waterproof methods with tunnel boring machines and elements for the tunnel lining are used. Since the elements for the tunnel lining are prefabricated, only two types of cases remain useful for the Observational Method:

1. The adjustment of the tunnel boring machine in the process, to optimise the boring speed, for example by altering the cutter teeth at the front, the bore front supporting pressure, the bentonite supporting mixture at the front (only for slurry shield machines) or by using the overcutters (increasing diameter).
2. The control of deformations with compensation grouting to limit settlements. For example the new tunnels under Waterloo Station in London (see Harris et all, 1994) or the new tunnel under the Mansion House in London (see Powderham, 1994).

Both cases are common practice.

Figure 5. Bus waiting above a metro tunnel face, München 1997 (T&T International)

6. Conclusions

Information observed during construction can be used to optimise the remaining part of the construction or structure. This procedure is indicated as the Observational Method. In this article ten general conditions are named for the implementation of this Observational Method in geotechnical engineering. The most difficult condition to fulfil is that the uncertain field condition (the observational data) must be somehow observable during construction. A strongly reduced probability of failure can only be achieved by a strong correlation between the parameter which determines the probability of failure and the parameter determined by the observational data. In other words observed deformation data will be in most cases insufficient for reducing the probability of failure caused by a lack of strength and strength data will be insufficient for reducing the probability of failure caused by a lack of stiffness. If a project fulfils all general conditions mentioned in this article, a financial risk analysis is necessary to find out if the Observational Method is profitable.

Two examples are given to show that the Preposterior Bayesian Analysis is useful in the implementation of the Observational Method. The success of implementation varies strongly per geotechnical discipline. Especially situations with a large number of repetitions are suitable for the Observational Method. For instance optimising the drain distance during a large land reclamation, optimising pile or sheet pile installations and optimising compensation grouting for bored tunnels. Most of these cases are already informally implemented in daily practice.

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