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Conditions for the use of the observational method in geotechnical engineering

Conditions pour l'application de la méthode d'observation pour le dessin géotechnique

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ABSTRACT
Information observed during construction can be used to optimise the rest of the construction or structure. In this article nine general conditions are named for the implementation of the Observational Method in geotechnical engineering. The most important general condition is that the uncertain field condition (the observational data) must be clear to measure. The problem of brittle behaviour is related to this point. If a project fulfils all general conditions, a financial risk analysis is necessary to find out if the Observational Method is profitable. In this article a simplified approach for a risk analysis is given, inclusive two examples, which show that the success of implementation varies strongly per geotechnical discipline.

RÉSUMÉ
Information observé pendant la construction peut être utilisé pour optimiser le restant de la construction. Dans cette article neuf conditions général sont donnés pour l'implémentation de la méthode d'observation pour la géotechnique. La condition le plus important est que la condition inconnu du terrain (les data de l'observation) doit être clair à mesuré. La problème de la conduite de frêle est apparenté de ce point. Si un projet suffi tout les conditions général, un analyse de risque financier est nécessaire pour savoir si la méthode d'observation est profitable. Dans cette article une méthode simplifiée est donnée, inclusive deux exemples qui montrent que le succès d'implémentation dépends beaucoup par discipline géotechnique.

1 INTRODUCTION

Common design calculations are only based on information which is available before any construction has started. If enough safety is built in between load and resistance, the structure will not fail during or after construction. In some cases, important information can be obtained during construction. This information might indicate that there is too much safety, which means too much money is spent, or too little safety, which means adjustments have to be made to avoid a disaster. If this information could be used in time, cheaper and safer structures can be made. The information observed during construction can be used to optimise the rest of the construction or structure. If this was designed from the beginning (ab initio), this method is defined by the author as the Observational Method. In the Ninth Renkine Lecture, Peck (1969) set out procedures for the Observational Method as applied to 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 common structure, which is 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.
2. Start with a lighter and cheaper structure, which is designed on base of rather positive input data. If during construction deformations or settlements indicate a more negative situation than expected, the structure is re-enforced.

Peck proposed that construction work should be started using the most probable design. 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).

2 GENERAL CONDITIONS

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

1. The observational method 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 clear to measure. The problem of brittle behaviour is related to this point (see below).
4. Disappointing, expected or favourable field conditions must lead to an appreciable difference in the cost or risk of the structure or construction.
5. The structure or construction must be able to be simplified or re-enforced after the data has been obtained.
6. This means that the construction consists of at least two (but rather more) stages.
7. The response time for monitoring and implementation must be appropriate to control the work.
8. 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 re-enforced (see point 7).
9. The costs of changing the structure (extra costs times probability of exceedance) should be less than the profit (saved money for 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 a specific characteristic value of a specific soil layer. This point is therefore also related to the problem of brittle behaviour, in which the stress/strength ratio is not indicated by a more than linear growth of the displacement, unlike ductile behaviour.

If the first 8 general conditions apply, an Observational Method can be implemented. According condition 9, this will not always lead to a solution with the lowest expectation value of the costs, however. For example; if the uncertainty of the actual field conditions can be solved with additional soil investigation and laboratory tests 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. The expectation value C of the total costs should be calculated for at least three options: Common design, OM starting with a common design and OM starting with a light design. The cheapest option should be chosen. The expectation value of the cost per option can be approximated by:
0) Standard: completely common design:

$$C_0 = S_0 + P_{F;0} \cdot F \tag{1}$$

1) OM, start: common design:

$$C_1 = S_0 + P_{0\to1} \cdot S_{0\to1} + M + \left(P_{0\to1} \cdot P_{\mathrm{F};1} + \left(1 - P_{0\to1}\right) \cdot P_{\mathrm{F};0}\right) \cdot F \tag{2}$$

2) OM, start: light design:

$$C_2 = S_1 + P_{1\to0} \cdot S_{1\to0} + M + \left(P_{1\to0} \cdot P_{\mathrm{F};0} + \left(1 - P_{1\to0}\right) \cdot P_{\mathrm{F};1}\right) \cdot F \tag{3}$$

in which:

S_{x}	=	total construction cost of the Structure (common: x = 0; light: x = 1)
$S_{\mathrm{x}\bullet\mathrm{y}}$	=	total construction cost for changing the structure from x to y (common = 0; light = 1)
$P_{\mathrm{F;x}}$	=	Probability of Failure of structure
$P_{\mathrm{x}\bullet\mathrm{y}}$	=	Probability of changing the structure from x to y
M	=	total cost of Monitoring
F	=	total cost of Failure

These equations are merely approximations because the probability of failure is simply multiplied by the probability of changing the structure, while these are not completely uncorrelated. The probability of failure of the common structure $P_{\mathrm{F};0}$ can be regarded the same in all three options. This is because the probability a light structure will not be upgraded in case even a common structure would fail, can be neglected.

Because a light structure has a much higher probability of failure ($P_{\mathrm{F};1} >> P_{\mathrm{F};0}$), two remarks can be made:

1. The only parameter which can be negative is $S_{0\bullet1}$, which means that money is saved by downgrading the structure during construction from common to light. If this is not the case then OM option 1) is always more expensive ($C_1 > C_0$) than the standard design option 0). Even if money is saved by downgrading, it can still be insufficient to pay for the extra monitoring costs and extra risk of failure.
2. In case the Observational Method is cheaper, a light start is cheaper than a common start ($C_2 < C_1$) in case:
   - a light design is much cheaper than a common design ($S_1 << S_0$),
   - and downgrading gives low profit or even high cost ($S_{0\bullet1}$ >>),
   - and good soil conditions are expected,
   - so the probability of upgrading is low ($P_{1\bullet0}$ <<),
   - and therefore the probability of downgrading is high ($P_{0\bullet1}$ >>).

The decisions between an OM common start (1) or cheaper start (2) does not depend on:

- The total cost of monitoring M.
- The total cost of failure F. The quality of the monitoring is in most cases such that it leads to a correct decision. So, the sum of the probabilities of upgrading a light structure and downgrading a common structure is about one ($P_{0\bullet1} \approx 1 - P_{1\bullet0}$). In this case the total cost of failure F is not important, under general condition number 8), which states that it must be sure that during the first stage no maximum load can occur, which leads to failure before the structure can be re-enforced.

Once the decision is made to implement the observational method and the construction has started, a new question arises, which is whether an intervention is required. This intervention depends on the observational data X, which influences the à posteriori probability of failure of the structure $P_{\mathrm{F;x|X}}$:

1) common to light intervention:

$$S_{0\to1} + P_{\mathrm{F};1|\mathrm{X}} \cdot F < P_{\mathrm{F};0|\mathrm{X}} \cdot F \tag{4}$$

2) light to common intervention:

$$S_{1\to0} + P_{\mathrm{F};0|\mathrm{X}} \cdot F < P_{\mathrm{F};1|\mathrm{X}} \cdot F \tag{5}$$

in which:
$P_{\mathrm{F;x|X}}$ = A posteriori Probability of Failure of the structure regarding observation X

This means it is important to be able to obtain an accurate description of the relation between the probability of failure ($P_{\mathrm{F};0|\mathrm{X}}$ or $P_{\mathrm{F};1|\mathrm{X}}$) and the observational data X.

This case with three options and two stages can be extended with extra 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 method described above will be the same however.

3 EXAMPLES

In order to show how this can be applied, two examples are given . Both fulfil the first 8 conditions, the question is in both cases whether the ninth condition about the financial profit is fulfilled as well.

3.1 *Example 1: Loaded beam*

Suppose we have a beam which is loaded. The load q , length l, breaking stress (f = 20 MPa) and material stiffness ($E = 1{\cdot}10^5$ MPa) are deterministic variables. The uncertainty is about the height h and width b of the beam. These are random variables with a standard deviation of 10%, so:

$$b \sim N(\mu,\sigma) = N(0.350, 0.035)$$
$$h \sim N(\mu,\sigma) = N(0.450, 0.045)$$

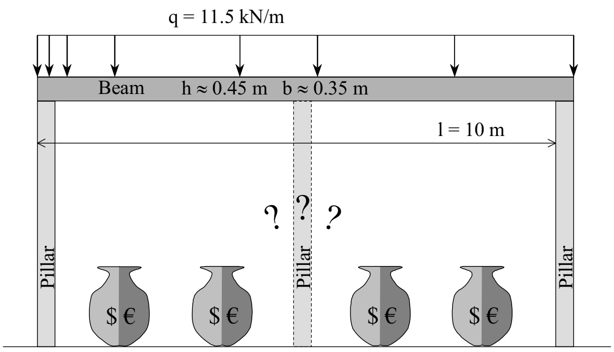

Figure 1. Example 1: Loaded beam: situation sketch

If the beam fails, four expensive vases with a total value of c_0 = € 100,000.- will be destroyed. The beam is supported by two pillars, but a third one can be installed in the centre for c_1 = € 1500.-. The question is whether it is profitable to install the extra pillar in order to reduce the risk.

Since the modulus of section $Z = \frac{1}{6} bh^2$ depends on height h and width b, also failure depends on these two random variables. With only two pillars, failure occurs when:

$$z_0 < 0 \quad \text{with:} \quad z_0 = f \cdot Z - \tfrac{1}{8} q l^2 \tag{1}$$

and with three pillars when:

$$z_1 < 0 \quad \text{with:} \quad z_1 = f \cdot Z - \tfrac{1}{32} q l^2 \tag{2}$$

The probability of failure depends on the number of pillars. The value of the probability of failure is equal to the volume of the probability density function for the combinations of the height and width, which leads to failure, see figure 2:

$P_r(z_0 < 0) = 17.1‰$

$P_r(z_1 < 0) = \quad 0.0‰$

The average à priori costs are:

$C_0 = c_0 \cdot P_r(z_0 < 0) = € \ 1711.-$

$C_1 = c_0 \cdot P_r(z_1 < 0) + c_1 = € \ 1500.-$

Which means an extra pillar in the centre is profitable ($C_0 > C_1$).

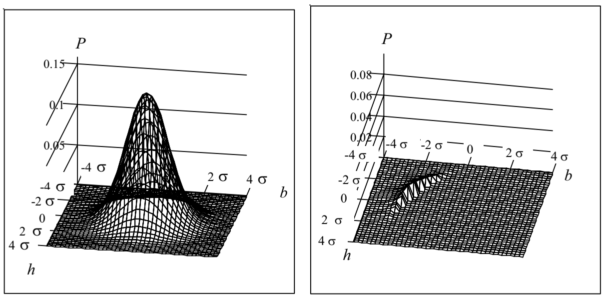

Figure 2. Probability density of h and b;
(Left: $P_r(h,b) = 1$, Right: $P_r(h,b \,|\, z_0 < 0) = 0.0171$)

The observational method is introduced by implementing a load test. The test load is small enough to avoid failure, so we can start with a light structure with 2 pillars. Suppose, the actual deflection can be observed with 10% accuracy. The deflection depends on the width and height of the beam:

$$w(b,h) = \frac{5}{384} \cdot \frac{q l^4}{EI(b,h)} \quad \text{with:} \quad I(b,h) = \frac{bh^3}{12} \tag{3}$$

so, the deflection provides info about the actual height en width. The average value or expectation value of this deflection is:

$\bar{w} = w(\bar{b}, \bar{h}) = 5.477 \text{ mm}$

Suppose the test result is a deflection which is even 80% higher than the expectation value, which is in this case a 4.5% upper limit. One might think that a third pillar is even more necessary in this case, but the à posteriori probability of failure is:

$$P_r(z_0 < 0 \,|\, w \approx 1.80 \cdot \bar{w}) = \frac{P_r(z_0 < 0 \cap w \approx 1.80 \cdot \bar{w})}{P_r(w \approx 1.80 \cdot \bar{w})}$$

$$= 0.338‰ / 53.79‰ = 6.3‰$$

This results in the following average à posteriori costs:

$C_0 = c_0 \cdot P_r(z_0 < 0 \,|\, w \approx 1.80 \cdot \bar{w}) = € \ 628.-$

$C_1 = c_0 \cdot P_r(z_1 < 0 \,|\, w \approx 1.80 \cdot \bar{w}) + c_1 = € \ 1500.-$

This means an extra pillar is far from profitable ($C_0 << C_1$) for all cases in which the deflection is up to 80% more than expected. So, in 95.5% of the cases the observational method saves € 1500 for an extra pillar (minus the costs of the extra load test).

The reason why the observational method works so well in this example, is that the randomness of both the strength and the stiffness of the beam depend on the uncertainty of the geometry of the beam (*b* and *w*). If the geometry would have been deterministic (fixed) and the breaking strength (*f*) and material stiffness (*E*) would have been uncorrelated random variables, the observational method would have been completely useless, because the deflection does not give any additional information about the breaking strength. Soil itself has only a poor correlation between strength and stiffness, therefore it will be difficult to make the Observational Method profitable for strength cases in which the geometry is well known. This problem does not apply to stiffness cases (deformations and settlements).

3.2 *Example 2: Settlement of embankment*

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.

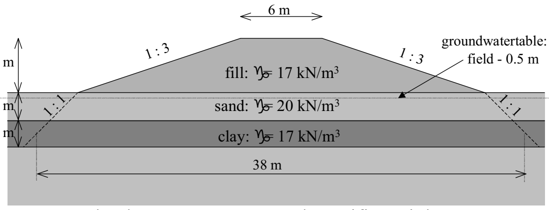

Figure 3. Embankment; geometry and specific weights

The settlement is predicted with the method of Koppejan (1948). According to this method the strain of the clay layer is:

$$\varepsilon = U \left(\frac{1}{C'_p} + \frac{1}{C'_s} \log(t) \right) \ln \left(\frac{\sigma'_v}{\sigma'_{v,i}} \right) \tag{4}$$

In which:

$\varepsilon = \Delta H / H$ = strain [-]
ΔH = settlement [m]
$h = H/2$ = drain 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 effect. stress after loading (83 kPa) [kPa]
$\sigma'_{v,i}$ = initial vertical effective stress (32 kPa) [kPa]

The degree of consolidation can be approximated by:

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

$$U \approx 1 - \frac{8}{\pi^2} \exp\left(-\frac{\pi^2}{4} \cdot \frac{c_v t}{h^2} \right) \quad \text{(for: } U > 0{,}5\text{)} \tag{6}$$

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

$$c_v \approx \frac{k \cdot \bar{\sigma}'_v \cdot C_p}{\gamma_w} \tag{7}$$

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) [kN/m^3]

Suppose the uncertainty is about the three clay parameters. These are random variables with a standard deviation 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 expectation value of the settlement over time.

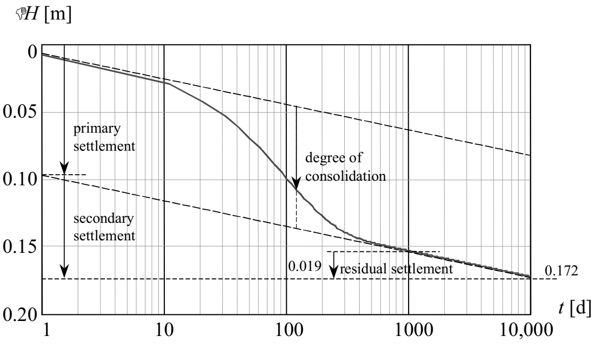

Figure 4. Embankment; settlement versus logarithmic time

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:

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

The contractor wants to use the temporarily surcharge of option 2 in case the chance of non-compliance is more than:

$$P_{\max} = \text{k€ } 10 / \text{k€ } 100 = 10\%$$

For this case the expectation value of the residual settlement is only $\Delta z = 19$ mm but the à priori probability of exceedance of the maximum residual settlement is calculated to be:

$$P_r(\Delta z > 0.025) = 12.0\% > P_{\max}$$

This means it is slightly cheaper (on average) and more safe to use the temporarily surcharge. However the Observational Method might save some money. It looks as if this residual settlement problem applies to all 9 conditions.

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

$$P_r(\Delta z > 0.025 \,|\, \Delta H_{t=100} \approx 1.08 \cdot \Delta\overline{H}_{t=100}) = 13.9\% > P_{\max}$$

So, the risk of non-compliance has even become more. 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 to t = 10,000 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 clear to measure.

The Observation Method can be improved for this case by measuring the pore pressures which gives extra information about the degree of consolidation. Suppose the development of the pore pressures indicate that the permeability is as expected ($k = \overline{k}$). The à priori probability and the à posteriori probability of exceedance will become in this case:

$$P_r(\Delta z > 0.025) = 11.9\% > P_{\max}$$

$$P_r(\Delta z > 0.025 \,|\, \Delta H_{t=100} \approx 1.08 \cdot \Delta\overline{H}_{t=100}) = 15.4\% > P_{\max}$$

Hence, pore pressure meters do not improve the estimate of the residual settlement in this case.

The reason why the à posteriori probability is even larger than before, is as follows: The 8% extra settlement, at t = 100 days (i.e. 24% upper limit), can not be caused by a faster consolidation ($k = \overline{k}$), which does not influence the creep. It can only be caused by extra primary settlement (C_p) or creep (C_s). And extra creep increases the à posteriori probability of exceedance.

In order to benefit from the Observational Method the uncertain field condition (creep) must be clear to measure. This example shows that for situations with a single settlement caused by primary settlement, consolidation and creep this condition is difficult to fulfil. The intervention date must be long after primary consolidation, which is not often the case.

4 CONCLUSIONS

Information observed during construction can be used to optimise the rest of the construction or structure. In this article nine general conditions are named for the implementation of the Observational Method in geotechnical engineering. The most difficult condition to fulfil is the condition about the uncertain field condition (the observational data), which must be clear to measure during construction. If a project fulfils all nine conditions, a financial risk analysis is necessary to find out if the Observational Method is profitable.

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