General Report of Technical Session 3B

Monitoring, Performance and Evaluation

M.A. Van [1] and F.B.J. Barends [2]

[1] *Deltares, Delft, The Netherlands*
[2] *TUDelft & Deltares, Delft, The Netherlands*

ABSTRACT
Most geotechnical behavior is unseen, under the ground surface. Monitoring, physical and numerical modeling, and the evaluation of performance in hind cast studies are valuable options to measure and understand the behavior of a structure and its subsoil. It is not sufficiently recognized that a geotechnical design based on simple rules will suffer from lack of safety and excessive expenses. Monitoring and physical modeling should be always part of a design framework in order to convince stakeholders and decision makers to care for realistic geotechnical uncertainties. This general report elaborates on the uncertainties our profession has to deal with and describes a framework to determine and explain the various geotechnical uncertainties and a framework to decrease uncertainties by proper understanding of the behavior of geotechnical structures.

RÉSUMÉ
La plupart des événements géotechniques restent invisibles, sous la surface de la terre. Surveillance, modélisation physique et numérique, et l'évaluation des performances dans les études hindcast sont de bons moyens de mesurer et comprendre le comportement d'une structure et de son sous-sol. On ne reconnaît pas suffisamment qu'une conception géotechnique se basant sur des règles simplistes souffrira d'un manque de sécurité et d'un coût excessif. La surveillance et la modélisation physique devraient toujours faire partie d'un cadre de travail afin de convaincre les responsables et les décideurs d'accorder de l'importance aux incertitudes géotechniques réalistes. Ce rapport général décrit les incertitudes que notre profession rencontre et propose un cadre de travail pour réduire ces incertitudes grâce à une compréhension appropriée du comportement des structures géotechniques.

Keywords : Uncertainties, engineering factor, monitoring, observational method, innovation cycle

1 INTRODUCTION

Most geotechnical behavior is unseen, under the ground surface. Monitoring, physical modeling, numerical modeling and evaluation of performance in hind cast studies are valuable options to measure and understand the real behavior of a structure and the subsoil. Norbert Morgenstern stated in his keynote lecture Common Ground (2000): Notwithstanding the achievements of the past and the exciting new developments provoking change in geotechnical engineering in recent years, the way in which geotechnical engineering adds value is not adequately understood, recognized and rewarded." Littlejohn, chairman of the Ground Board, stated in 1991: Continuing disquiet concerning the late completion of construction projects and high-cost overruns which have been attributed to inadequate site investigations cannot be ignored. Something positive must be done to improve the situation." Cummings et al. (2004) and Staveren (2006) looking at various geotechnical failures conclude that human error is a culprit. Tol (2008) reviewing several building pit failures observed that in 80% of the cases relevant knowledge existed but was not available at the spot in proper time.

The society has high expectations. The perception of stakeholders about our profession is low. Facilities for general information about our added value are inadequate. Crises preparation and management, particular in complex construction need more attention. We need to deal with these facts. We should export our pride. Ground should be recognized as a vital element of most structures and similar care must be given as is commonly done to other aspects of engineered structures. In this respect, monitoring of performance during construction and sometimes afterwards is a worthy and invaluable aspect, a subject which deserves continuing attention.

This general report is based on the articles in 'session 3B Monitoring, Performance and Evaluation' of the XVIIth ISSMGE conference in Alexandria and on personal experience of the authors of this general report. A general overview of the monitoring issues and geotechnical structures in the papers in session 3B is given in appendix A. The following items are distinguished:

- settlement and dynamic behavior of roads and railways;
- vertical and horizontal displacements and water pressures or flow in dams and slopes;
- deformation including swell, and stresses in tunnel projects;
- horizontal deformations of deep excavations and retaining walls;
- settlement and dynamic behavior of pile, raft and gravity foundations;
- settlement and water quality of land fills and dredge material.

Almost all monitoring results in the reviewed papers are hind cast studies based on Finite Element Methods. First class prediction, according to Peck, are rare. Apart from the more complex FEM applications, a single paper includes an analytical or neural networks method for evaluation. If these FEM hind casts results, which are the best our profession can do to postdict, are within a scatter of 10% to 50% of the measurements predictions, the authors (including the authors of

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this report) are satisfied with the results. And probably, in first class predictions, most authors would be satisfied to be within a 50% range.

This general report will further elaborate on the uncertainty our profession has to deal with and describes a framework to determine and explain the geotechnical uncertainties and a framework to decrease these uncertainties by proper understanding the behavior of geotechnical structures.

2 OBSERVATIONAL METHOD

The Eurocode 7 includes the so-called observational method (citation):

"The complexity of the interaction between ground and the retaining structure sometimes makes it difficult to design a retaining structure in detail before the actual execution begins. When prediction of geotechnical behavior is difficult, it can be appropriate to apply the approach known as the observational method, in which the design is reviewed during construction."

Here, two aspects are worthwhile emphasizing. The ground behavior is officially recognized as a difficult material. This is not the case for other materials addressed in the Eurocode. Ground is a natural material, whereas steel and concrete are fabricated. The second aspect is the word "reviewed". It implies that a proper design, which fulfills all requirements and foreseeable uncertainties and risks, will be reviewed during execution, in case foreseen uncertainties are less severe, and corresponding contract restriction may be released, according to a protocol defined and accepted beforehand by all parties involved.

In this respect, the method of observation is, of course, specific monitoring, dedicated to the aspects of concern of the original design.

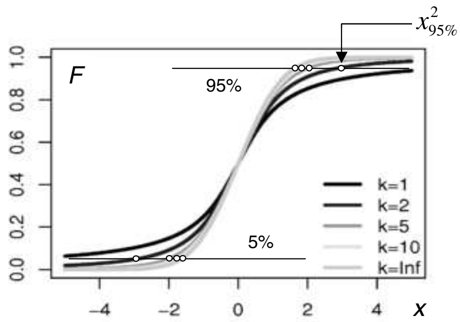
Another aspect, which is not well covered in the Eurocode, concerns temporary structures. Often, contractors have to comply with the codes of practice meant for permanent structures, which are safe and to some extent conservative, but which prevent them for exploiting their experience, for their benefit, in particular situations. Also here, specific monitoring could be a solution. However, the uncertainties of geo-engineering should not be underestimated.

3 UNCERTAINTIES GEOTECHNICAL ENGINEERS HAVE TO COPE WITH[1]

Peck (1980) proclaims that sound engineering judgment, which means by a well-trained and experienced engineer, is of prime importance above sophisticated approaches.

Here, the role of the engineer is considered with regard to his capability to predict and decide, i.e. to apply quantifying prediction models adequately. In general, a prediction model is qualified and accepted when it has a record of successful applications. It becomes even more accredited, if the applier can use it easily and if all model's possibilities and limitations have been unveiled in practice. An essential aspect arises, that is, the calculated outcome for a prediction will provide information in balance with the chosen problem schematizations and choices, objective choices so to say. Often the result looks in good agreement to expectation, but that may just be misleading.

For the estimation of the engineering accuracy of geotechnical predictions Student's t-distribution is applied. It is a probability distribution that arises in the problem of estimating the mean of a normally distributed population when the sample size is small. It is particularly useful when the standard deviation σ is unknown and has to be estimated from the data. The corresponding distributions are shown in the next graph.



For a limited set of samples k the average is equal to $\mu = (Max + Min) / 2$. For the 95% interval $Max = \mu + x^k_{95\%}\,\sigma$ and $Min = \mu + x^k_{5\%}\,\sigma$. From symmetry, $x^k_{95\%} = -\,x^k_{5\%}$. Hence, the variation coefficient becomes:

$$v = \frac{\sigma}{\mu} = \frac{Max - Min}{Max + Min} / x^k_{95\%}$$

The variation coefficient is used to characterize the accuracy of engineering prediction.

3.1 *uncertain soil properties*

Geo-engineers know that uncertainty of soil properties (stratification and model parameters) is relatively high with regard to other common building materials like concrete and steel. Uncertainty[2] for steel it is not beyond ±5%, for concrete it is about ±15%, for soil it is usually beyond ±50%. Outside our geotechnical society this aspect is hardly recognized. The fact is that we deal with a natural and not fabricated material like steel, bricks and concrete. We should inform our stakeholders and decision makers clearly about this aspect in order to inspire realistic expectations.

Figure 1 shows our limited capability indicating soil conditions for some common site investigation methods by non-destructive techniques: NDT. If this would be a capability profile of a dentist, one would, no doubt, look for another doctor. In soils this situation is common, and obviously the engineer dealing with it has to add a lot of interpretation and experience when defining relevant soil conditions.

NDT	Depth	Piles	Stone	Peat	Clay	Loam	Silt	Gravel	Gas	Sand
		Hole	*Boulder*	*Lence*	*Lence*	*Lence*	*Lence*	*Bed*	*Pocket*	*Type*
From surface or borehole										
CPT + coring	N-F	+?	+!?	+?	+?	+?	+?	+!	+?	+
Seismic	M-F	?	?	?	?	?	?	+?	+?	?
Electro-magnetic	N-M	-	-	+?	-	-	?	?	-	-
Ground radar	N	+?	+?	-	-	-	-	-	?	-
Geoelectric	M-F	-	-	+	+?	?	+?	?	+?	?

Figure 1. Capability of site investigation methods in geo-engineering. *Legenda: + OK, ? unknown, +? probable, ! damage, - not possible; N: near (1 – 5 m), M: medium distance (5 - 20 m), F: far (more than 20m).*

The range of intrinsic and characteristic soil property values gathered from common field and laboratory tests is well studied. In codes of practice corresponding partial safety factors are defined related to actual circumstances and related risks. For geotechnics values of 1.1 up to 1.6 are mentioned (Eurocode 7).

3.2 *unknown boundary conditions*

When modeling the real geometry, a choice has to be made about dimensions (1D, 2D or 3D), and the stratification. At the borders of chosen domains suitable conditions are to be chosen with regard to groundwater and soil matrix in terms of stresses, fluxes and/or displacements. This also holds to some extent for

[1] This chapter is based on a recent article of Barends (2009).

[2] Corresponding variation coefficients assuming normal distribution and a 5%-95% interval yields: steel < 0.03, concrete ~0.09, soil > 0.30.

interfaces within the considered domain; by example, soil-structure interaction. For non-linear and time-dependent problems the initial state has to be defined. In fact, the determination of the initial state is sometimes more difficult than the actual problem itself.

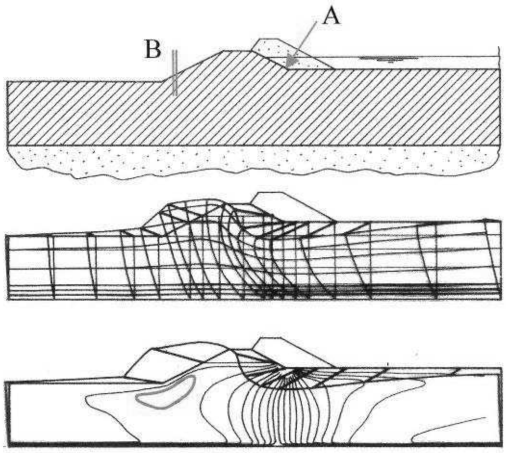


Figure 2.. A river embankment subjected to sudden water level rise.

Figure 2 shows the effect of unknown boundary conditions at the edge of a chosen domain: a two-layered (top clay, bottom sand) soil mass with a dike subjected to a sudden river level rise. The problem arose as a local pore-pressure meter (position B) in the lee-side slope of the dike showed an immediate response in the measurements, which did not comply with the hydrodynamic period of the clay layer (months). The measured immediate response was modeled by FEM considering 2D linear-elastic consolidation. Various boundary conditions at the sides and bottom have been adopted: fixed, slip, pressure, impermeable, in 6 scenarios. The deformation of the toe of the dike (point A) shows a horizontal deformation[3] varying between 5 and 40 cm and vertical between 1 and 2.5 cm. The immediate pore pressure response varied not so much (less than 10%). Figure 2 shows one such scenario: deformations and excess pore-pressure contours. The immediate response at point B is due to horizontal (total) loading by the river water on the dike, which is reflected in the pore water. The vertical effective stresses are slightly affected (by about 25% due to 2D-effects), so slope stability was not really at risk. When the water dropped the response disappeared.

Boundary conditions have distinct effects to different field variables. Particularly deformations are sensitive to subjective choices for unknown boundary conditions.

3.3 *various constitutive behavior models*

For the improvement of the stability of a LNG reservoir design, in case the inner metallic tank suddenly ruptures, a soil embankment is placed around the outer concrete tank wall (Sweet etal. 1980). To prevent (explosive) gas escape, the roof structure must remain in tact under dynamic forces caused by induced dynamic liquid flow pressures.

The critical issue is the maximum horizontal displacement U of the roof edge (Figure 3). Structure and soil properties and initial state are chosen and various constitutive soil behavior models are selected: elastic, von Mises (elasto-plastic, no friction), Mohr-Coulomb (elasto-plastic and friction), and endochronic (visco-elastic including creep). The choice of characteristic parameter values for different constitutive models, based on available soil tests, has been taken identical when appropriate. The results show a wide range of the critical displacement U varying by a factor 3, from 6 to 16 cm.[4] The residual deformation (just one, the Mohr-Coulomb model, is shown) varies even more. As in this case there is not yet a structure, practical validation or monitoring is not possible.

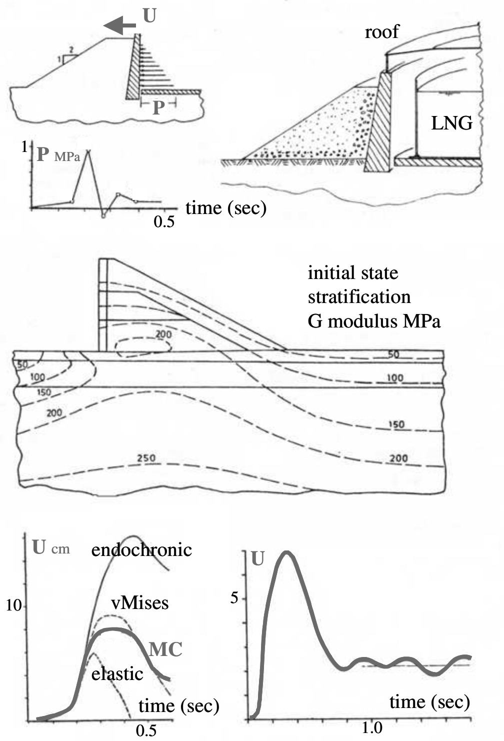


Figure 3. Effect of the choice of a constitutive model

The client was not pleased with a range by a factor 3. Here, it is up to the engineer's intuition and experience, if any, to decide how to appreciate this outcome and what practical maximum displacement should be adopted for the design.

3.4 *choice of physical processes*

In reality, all possible and relevant physical processes do play a role, always. For a simulation or prediction the engineer has to optimize his approach by making a selection of the dominant processes and disregard physical processes, which are not relevant. It is recommended to support such a choice by an elementary analysis or by specific experiments. A striking example of the effect of choices of physical processes is found in the application of well-functions in geohydrology.

For a constant well in a semi-confined aquifer system, see Figure 4, the Hantush-Jacob well-function is commonly applied. It encompasses flow and storage in the aquifer and leakage through the adjacent aquitard. The outcome after calibration reveals the production capacity Q and the corresponding area of influence, the radius λ. This approach is applicable for (deep) reservoirs and thin or rigid adjacent aquitards.

Table 1. Physical processes; well production in a semi-confined aquifer

Choice	aquifer		aquitard		λ/λ_1
	permeable	storage	permeable	consolidation	
1	yes	yes	no	no	1.00
2	yes	yes	yes	no	1.25
3	yes	no	yes	yes	3.45

well-functions: (1) Theis, (2) Hantush-Jacob, (3) Barends

[3] Using Student's t Distribution it corresponds, for a 5%-95% interval and 6 samples, to a variation coefficient of $v = \sigma/\mu = 0.40$, if this variation is considered as the effect of choices of scenarios.

[4] Using Student's t Distribution it corresponds, for a 5%-95% interval and 4 samples, to a variation coefficient of 0.22.

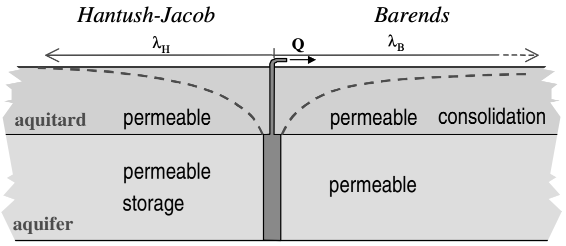


Figure 4. Physical processes in a semi-confined aquifer system

In shallow regions where semi-permeable aquitards are relatively young deposits, the storage is due to induced consolidation (storage) in the aquitard; storativity of the aquifer is less relevant. By applying a well-function including the consolidation process (Barends et al. 1987) the outcome shows a value for λ being about 2.8 times larger than with the Hantush-Jacob function (see Table 1). That the outcome of a constant well in a shallow semi-confined aquifer system by applying Hantush-Jacob well-function represents all expected phenomena (storage, flow, leakance), is in fact misleading. Fortunately, it has little implication on the production Q, but the influence area is largely underestimated, which can have dramatic effect on for instance wooden pile foundations in old cities.

3.5 *lessons from prediction contests*

Validation and calibration are essential for the approval of the quality and applicability of simulation or prediction models. But, as has been indicated above, also the user, i.e. his choices and assumptions, is a distinctive factor. This can be elucidated by looking at prediction contests, reported in literature.

3.5.1 Pile load-settlement behavior

During the Penetration Testing Symposium ESOPT-II, in 1982, a prefabricated pre-stressed concrete pile (15 m, 0.25x0.25 m^2) was driven and statically loaded till failure. Long before, extensive site and lab investigations had been performed. With the results of these tests 15 international experts made a prediction, using their best methods and experience. The predictions together with the real result are shown in Figure 5 (Weele 1989).

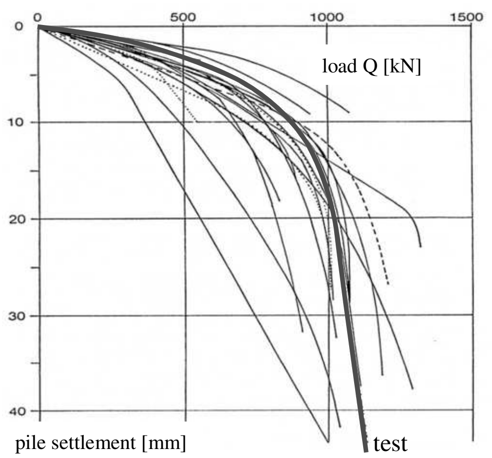


Figure 5. Predicted and actual load-settlement of a concrete pile

The predicted pile loading at 10 mm settlement varies from 360 to 1110 kN, and predicted failure load ranged from 600 to 1500 kN. It corresponds to a variation coefficient of 0.29 and 0.25, respectively, using Student's t Distribution for a 5% - 95% interval and 15 samples.

During the 4th conference of the Application of Stress-Wave Theory to Piles, in 1992, a similar contest was organized. Four concrete piles with different shape were dynamically tested and 9 international experts made a prediction for the static bearing capacity (Test Report 1996), each using his tool and interpretation method. In Table 2 results are presented.

The Test Report states that the interpretation method and type of software were more important for the variation in answers than the difference in monitoring systems. Moreover, local experience was not significant for obtaining more reliable predictions.

Table 2. Ratio between predicted and measured displacement at 50% of the ultimate bearing capacity (static loading test)

Expert	Pile 1	Pile 2	Pile 3	Pile 4
1	-	0.59	1.32	0.70
2	3.32	0.80	1.53	1.08
3	-	-	-	1.06
4	1.20	1.10	1.29	1.25
5	2.00	1.06	1.38	1.50
6	0.60	0.63	1.35	1.22
7	1.03	0.79	1.26	0.80
8	1.42	1.46	1.04	0.71
9	1,21	0.74	1.41	1.00
Max/Min	5.5	1.9	1.4	2.1
Average	1.54	0.90	1.32	1.04
$v_{5\%-95\%}$	0.36	0.26	0.15	0.29

$v_{5\%-95\%}$: the variation coefficient of Student's t Distribution

3.5.2 Slope stability

Since the eighties the phenomenon uplift became a dominant dike failure mechanism and a validated method for design has been developed (Van et al. 2005). The last stage was an in-situ test at Bergambacht in the Netherlands, in 2001. Uplift occurs when high river waters induce pressures under top clay layers at the dike's lee side, larger than its weight. A thin film of water then seeps into the interface affecting slope stability.

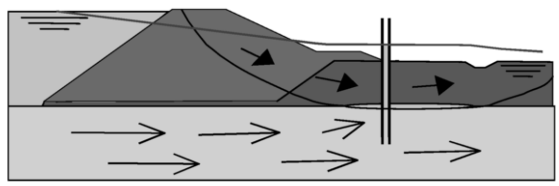


Figure 6. The uplift phenomenon affecting river embankments

Before the test, five independent experts have been asked to perform predictions in three stages: (I) general information was given and two CPTs, (II) in addition lab test results, and (III) a 3rd CPT and a boring, all according to official standards. In table 3 the results of the predicted stability factors are compiled.

Table 3. Predicted stability factors (using Eurocode 7)

Expert	Stage I	Stage II	Stage III	Max/Min
1	0.52	0.86	0.85	1.65
2	0.63	-	-	-
3	0.84	0.82	0.80	1.05
4	0.75	0.92	0.91	1.23
5	-	0.82	0.87	-
Max/Min	1.62	1.12	1.14	
Average	0.69	0.85	0.84	
$v_{5\%-95\%}$	0.11	0.05	0.06	

In subsequent stages results converged. A post-diction using measured pore pressures indicated a stability factor of 1.02. Apparently, the predictions are conservative (about 15%). The choice of the slope stability model appears less important when compared to the completeness of information, the individual interpretation and the uncertainties regarding shear strength.

In 2008 a large dike on soft clay and peat was tested for slope stability (Figure 7). An international prediction contest was organized and 40 independent experts took a chance to estimate at which of the 8 phases (see also paragraph 3.3) the dike would collapse.

Table 4. Predictions made by experts of the phase of collapse

Phase	Phase description	Experts	Method
1	before the test	1	-
2	digging the ditch	1	E
3	deepening the ditch	1	-
4	filling the sand core	6	E S P
5	emptying the ditch	10	E S
6	filling containers	13	E S P
7	saturating the dike	4	E S
8	no failure	4	E

E: educated guess, S: slip circle analysis, P: FEM (Plaxis)

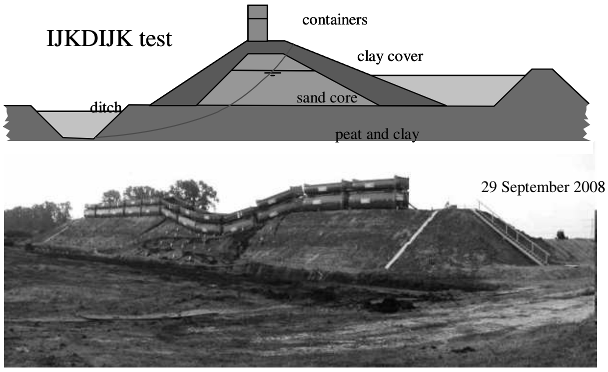


Figure 7. The real scale slope stability test

The collapse occurred in phase 4, The prediction results show that the wide spread[5] is mainly related to engineering choices and that the use of standard or sophisticated models does not make much of a difference.

3.5.3 *Sheetpiling design*

In 1993 in Karlsruhe a sheetpiling test was organized (Wolffersdorff 1991). The behavior of a strutted sheetpiling retaining 4 m of non-saturate sand was approached by 43 international experts using FEM, subgrade reaction and other methods to predict deformations, moments and forces. The outcome showed a wide spread: three quarter of the experts obtained an answer beyond 50% of the test result. Those who disregarded capillary cohesion were completely out of range.

In 1999 an anchored sheetpiling test was organized in Rotterdam. 6.5 m of soft clay and peat were retained at high groundwater table. 23 international experts made a prediction for horizontal displacement, plastic hinge and oblique bending, applying FEM and subgrade reaction methods. Results showed again a wide spread: for the displacement, by FEM between 45 and 210 mm and by subgrade reaction between 62 and 173 was calculated, while 107 mm was measured.[6] Surprisingly, FEM was less accurate. It was observed that data interpretation for parameter values for strength and stiffness varied significantly ($19^o < \phi < 35^o$, $2 < c' < 10$ kPa), and that different sets of parameter values could produce the same result.

3.6 *the engineering factor*

The examples mentioned above indicate that a significant part of the spread in geotechnical prediction is due to lack of information, different interpretations and subjective choices. If more specific information becomes available, the spread will decrease, but always uncertainty about soil stratification, initial state, soil behavior and boundary conditions remains. We could refer to that as the engineering factor, since it is related to individual knowledge and experience.

Adopting Student's t Distribution and accounting for limited sampling, a relevant variation coefficient can be found when assuming the test results are within a probability range of 5% to 95%. For the examples discussed in this article this variation coefficient varies somewhere between 0.20 and 0.45, which is quite large. In this respect, further investment in improvement of prediction models from case studies is in fact practically not worthwhile, unless the size of this large variation is drastically reduced or unless this variation is abandoned in physical testing with known materials and boundary conditions, further elaborated in the next section 3.

To reduce the engineering factor, always ask at least three engineers to make an independent prediction. If possible, harmonize parts of the subjective interpretation by information and education; include it in terms of reference.

Calle (2008) suggests weighing probabilities of scenarios by specific experience. In other words, when considering several soil stratifications or several constitutive models, independent experts should be asked to give these scenarios (S_i) a particular value of likelihood. The prediction of the reliability of a design for various weighted scenarios can be expressed by:

$$P(F < 1) = \Sigma_i [\, P(F < 1; S_i)\, P(S_i)\,]$$

Modern ICT developments could very well support the implementation of this procedure in practice. In this manner the engineering factor can be reduced structurally. Moreover, by this procedure available expertise (sound engineering judgment, as proclaimed by Peck) is mobilized, and multi-disciplinary cooperation is stimulated. Furthermore, the resulting variation is less out of range with regard to other building materials, the value of eventually obtaining additional information becomes emphasized, investments and efforts towards intrinsic prediction model improvements do make sense, and our image outside the profession will improve.

4 DECREASE THE UNCERTAINTIES BY BETTER UNDERSTANDING OF THE BEHAVIOR OF GEOTECHNICAL STRUCTURES

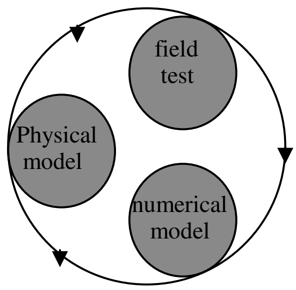


Figure 8. Innovation cycle.

A framework for better understanding the behavior of geotechnical structures and reducing the uncertainties is the so-called innovation cycle, see Figure 8. This cycle normally starts with numerical analyses of the structure. Prediction with finite element codes in feasibility studies will lead to directions how problems can be solved. Then physical modeling with known materials and under known boundary conditions will give the validation of the theory. Field tests and/or monitored projects will show the behavior in real practice. The physical model results and the field test results have to be evaluated and reanalyzed in postdictions to create better understanding, reduce

[5] Using Student's t Distribution, a variation coefficient of 0.46 is found, assuming a 5%-95% interval for 40 samples and 8 phases.

[6] Idem, for 23 samples the FEM method gives a variation coefficient of 0.38, and subgrade reaction method gives 0.28.

uncertainties and develop general design rules.

The physical modeling step is often skipped in practice, which introduces large uncertainties of soil heterogeneity and boundary conditions, as mentioned in the previous section. In hind cast studies, parameters are adjusted and the theory is made fit for a specific location. It means that exclusive validation of the theory for that specific field condition is performed. However, a step to generic knowledge is only possible when there are results of many similar projects on different locations. It is much more efficient to reduce uncertainties by general verification of theory in physical models. However, this is valuable for a client if he has more similar projects, in future.

Physical modeling can start with a series of small tests by which mechanisms are qualitatively understood and shown: i.e. does the soil collapse or how is the contamination dispersed. When the behavior is qualitatively understood, detailed measurements can be added to the tests, and theory can be verified. If the test model is rather expensive detailed measurements should be applied already in the first test. Proper preparation is essential.

Subsequently, more advanced and quantitative tests are executed. Boundary conditions and materials must be well known and consistent with the calculation model. In this manner, the calculation model is validated by adequate physical model measurements. The result is better predictions of processes in the field and less fitting parameters for a specific locations.

The innovation cycle can also start from a monitored field application. Sometimes, a calculation model is outside its range of validity and its application may create excessive uncertainties, which therefore can be risky. Then, the observational method can help to construct beyond available experience, for instance when new construction techniques are being used. After finishing such a pioneering construction a new validated design method can be derived, ready to be incorporated directly into other projects. Physical models are often helpful to understand the measurements and to adapt and verify the new design model.

Van et al. (2003) show a successful example of using the innovation cycle in the Bergambacht-project.. In Van et al. (2009) another example of using the frame work of the innovation cycle, which includes the development of a monitoring system, is described; the next paragraphs 4.1 to 4.7 are based on this article.

4.1 *IJKDIJK tests*

Dutch lowlands are protected by many kilometers of dikes. Despite the fact that building dikes started in the late Middle Ages, today designing, constructing and maintaining dikes still involves a lot of empiricism. During high water conditions information on the actual strength of a dike is usually obtained by visual inspection. Questions about the time to failure or the maximum load increase a specific dike location can still withstand are hard to answer. For other technical applications modern sensor technology is used to obtain (sub)soil information.

After a dike failure at Wilnis in 2003 (Bezuijen et al., 2005), the question was raised if modern sensor technology could be used to assess extra information on dike conditions. At best, sensor technology could be used as an early warning system, by which, when a monitored parameter would reach a certain value, people are warned and action can be taken.

When using modern sensor technology for an early warning system, it should be known which parameter is monitored at which interval in time and space and at which location in the cross-section, but also at what point action will be taken and what time frame is available. In order to answer these questions, the IJkdijk project was initiated. The aim of this project is to study the applicability of modern sensor technology as an early warning system for dike failure. This aim will be reached by bringing instrumented embankments to failure at full scale.

Dikes might fail according to different failure mechanisms, each implying different conditions for a possible early warning system. In the early stages of the project, three failure mechanisms were chosen: piping, wave overtopping and full-scale stability. With full-scale stability the occurrence of a sliding plane through both the embankment and the subsoil is meant. The following paragraphs describe the stability test including a preliminary field test and centrifuge tests to reduce the huge uncertainties in the test design. The first analysis results are discussed. The benefits of modern sensor technology as an early warning system are illustrated by examining one of the many measured parameters: the horizontal deformation measured in the subsoil at the toe of the dike.

4.2 *Subsoils*

In total 33 Cone Penetration Tests, CPT's and 22 continuous Begemann borings were conducted. An interpretation of the CPTs and borings is used to construct a geotechnical profile along the test field. Figure 9 shows a typical CPT and its interpretation found at the middle cross-section of the test embankment. The depth shown by Figure 9 on the vertical axis is related to the reference datum NAP, which is approximately mean sea level. The subsoil consists of a thin, 0.5 to 1.0 m thick clay layer followed by a 1 to 2 m thick peat layer and a Pleistocene sand layer. The CPTs conducted at the site, some up to a depth of 20 metres, did not reach the bottom of the Pleistocene sand layer. The peat has a volume weight of 9.8 to 10.8 kN/m^3 and a water content ranging from 2.0 at the top to 3.0 to 4.8 at the bottom. The volume weight of the top clay is 16.6 kN/m^3 with a water-content of 0.3 to 0.6.

The water table faces a seasonal influence being at ground level during wintertime and at ground level minus 0.5 to 1.0 m during summertime.

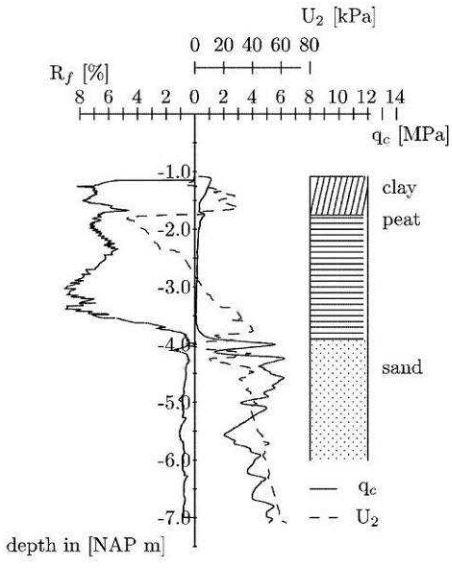


Figure 9. Typical CPT at test embankment

4.3 *Test set-up*

The full-scale stability test consisted of constructing an embankment and make it fail in a controlled manner. Figure 10 shows the dimensions of the test embankment. The length of the test dike is 100m. The test embankment is constructed parallel to an existing canal dike, see Figure 11. Filling the area between both dikes with water simulated free water at the river or seaside of the test dike, see Figure 17a. The area between both dikes is further referred to as the bathtub. The crest height is 6 m. The slope at the outer side, the sea or river side, is 1:2.5 (V:H). At the inner side the slope is 1:1.5, i.e. the slope that is planned to fail. The available measures for bringing the dike to failure are excavation of a ditch at the toe, filling the sand core with water and application of load on the crest of the dike.

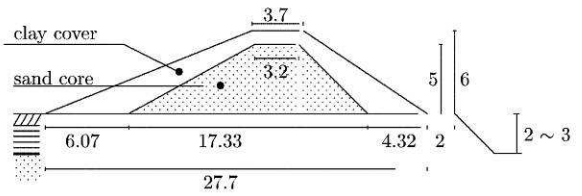


Figure 10. Test set-up, cross section dimensions in [m]

Figure 11. Arial view of test and test site.

The ditch at the toe of the dike is excavated in two steps. First the top clay layer is removed. Second, when no continuous deformation is found, the ditch is further excavated on to the sand layer.

Filling the sandy core with water requires a heavy and watertight clay cover on the sand core to prevent superficial sliding planes or local leakage and erosion problems. Figure 10 shows the dimensions of the clay cover. The free water, present at the outer side of the dike, increases the safety against sliding of the top clay cover at that outer side of the dike.

To be able to imply a load on the crest of the dike during the test, two rows of containers were placed on top. By filling these containers with water, a load could be applied on the crest during the test.

The embankment could now be brought to failure by the following steps: 1) filling bathtub at the front of the dike, 2) excavating the top clay layer at the toe of the dike, 3) excavating the ditch at the toe to the sand layer, 4) filling the sand core to 2/3 of its height, 5) filling the containers at the top 6) filling the sand core completely.

4.4 *Instrumentation*

The applied instrumentation is divided into two groups. First, the reference monitoring. This group of instrumentation is used to assist the construction of the embankment, to safeguard the canal dike and to provide reference data to calibrate the new sensor technology with. This group of instruments is also used to guide the experiment. The second type of instrumentation consists of the new sensor technology which usefulness as an early warning system is to be tested. Figure 12 gives an overview of the applied instrumentation.

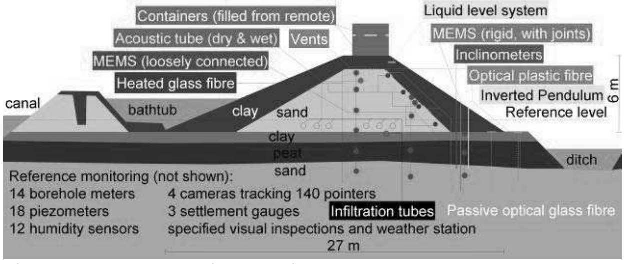


Figure 12. Instrumentation overview

Figure 12 indicates the large number of participants, each testing their (newly developed) equipment. Not indicated in this figure are thermographic cameras and LIDAR.

4.5 *preliminary test*

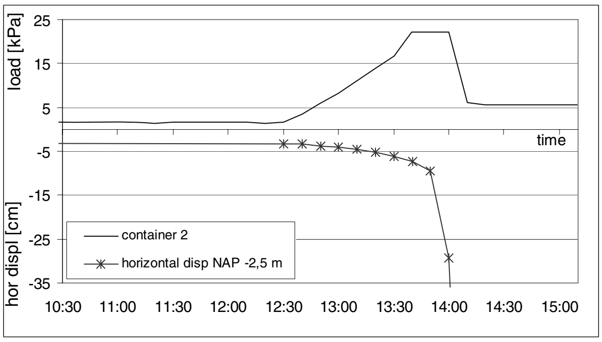
Constructing an embankment, avoiding failure during construction and with the intention to bring it to failure shortly after construction, requires an accurate knowledge of subsoil strength. Due to the construction of the embankment, excess pore pressures will be present in the subsoil during the test. The excess pore pressures may strongly influence the subsoil strength. The exact level of excess pore pressures depends on the drainage capacity of the soil. Among others, den Haan & Kruse (2006) show the difficulty in parameter assessment for peat. Also for the case at hand, problems were encountered, leading to serious uncertainties in the design of the test embankment. To improve subsoil knowledge a preliminary field test and some centrifuge tests have been executed before the large scale test itself.

Figure 13. a) Overview preliminary test b) Measurement row at the centre of the container row c) Filling of the containers d) Failure

In the preliminary field test two rows of four containers each represent an embankment. By filling the containers with water a load could be activated within one hour. During a period of a week the decrease in excess pore pressure is measured. Next, the containers were emptied and at a distance of one meter from the front container row a ditch is excavated. During excavation the ditch was filled with water. After draining the ditch, the containers were filled again. For the first 25 minutes, no deformation was observed. Then, horizontal displacement of the slope of the ditch was observed, leading to progressive failure of the subsoil 2 to 3 minutes later. Figure 13 shows an impression of the stages of the preliminary test.

After failure, the containers were removed and the failure plane was examined by excavating an observation pit. The active part of the sliding plane was found to be very steep, followed by a horizontal part through the peat layer, leading to the ditch bottom. It should be noted that for the location of the preliminary test a 2.5 m thick organic clay layer was present on top of the peat layer.

The instrumentation consisted of eight pore pressure transducers placed in the peat and clay layer, two open stand pipes for measuring the hydraulic head in the sand layer, a settlement tube placed underneath the container row, and an inclinometer to measure the horizontal deformations in the subsoil. Figure 14 briefly shows an impression of some of the measurements. The top part of the figure shows the measured filling of the containers. The lower part shows the horizontal subsoil deformation at a depth of NAP –2.5 m, approximately 1.5 m below ground level, at the front of the first container row and close to the ditch. The figure shows that horizontal deformations were activated directly at the start of filling the containers. This was long before deformation could be observed visually at the surface. The observation that horizontal deformations can be measured before failure is visible is also described in Crabb & Atkinson (1991)



4.6 *Figure 14. Horizontal deformation of the subsoil at ground level –1.5 m during filling of the containerscentrifuge tests*

To get more insight in the observed failure mechanism of the preliminary field test, centrifuge tests were conducted. The test represented the failure of the subsoil underneath the containers during filling. The model is built with a scale of 1:50. The test set-up included the following steps; after reaching the proper g-level a consolidation phase was applied. When the excess pore pressures had disappeared a model in the shape of the ditch was lifted, representing the excavation of the ditch. Next, the containers were filled and the water table in the ditch was lowered. To represent the proper sub soil stress conditions the initial water table was placed above the ground level. When emptying the ditch, the water table is also lowered.

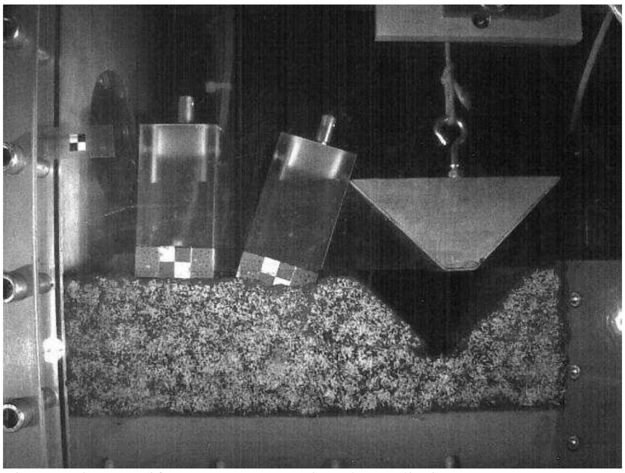


Figure 15. Centrifuge test representing the preliminary test

Figure 15 shows the failure plane observed in the centrifuge tests. Horizontal sliding dominates the failure mechanism. Figure 16 shows the horizontal deformation found in the centrifuge test at three different levels below ground level at the front of the containers. The horizontal displacements are found after creation of vector plots from images like the one shown by Figure 15. The vector plots and the displacement graphs are made using particle image velocimetry (PIV) for use in geotechnical testing(White, 2003). The PIV-analyses was carried out with the PIV software tool developed by White (2002). In comparing centrifuge and the full scale tests, a depth of –18 mm in the centrifuge test corresponds to -0.9 m in the full scale test, likewise -28 mm corresponds to -1.4 m and -38 mm to -1.9 m. In the centrifuge test the slip plane lays between -28mm and -38mm. Figure 7 shows that directly when the filling of the containers start, t =0, a continuous horizontal deformation starts.

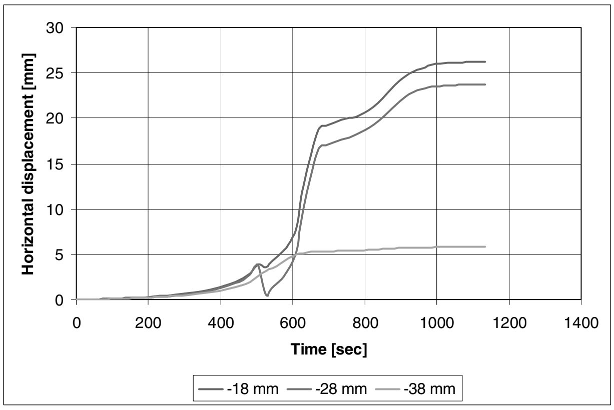


Figure 16. Horizontal deformation at 3 different ground levels in centrifuge test

4.7 *full scale test*

The construction phase of the full scale test embankment started on August 13th 2008 and was finished on September 19th 2008. The test started on September 25th by filling the bathtub in front of the dike, see Figure 17a. Later that day the top clay layer was excavated, see Figure 17b, at the toe of the dike. On September 26th, the ditch was fully excavated, on to the top of the sand layer, see Figure 17c. On September 27th, the sand core was filled with water. The filling started at 12:07 hr, at 16:00 hr deformation could be observed visually, failure was found at 16:02 hr, see Figure 17d.

Figure 17 a) River side of the dike, b) Inner side of the dike during excavation of the ditch, c) Inner side of the dike after excavation of the ditch, d) Failure

Failure was reached during filling of the sand core; the containers on top of the dike were not filled. The observed failure plane had a width of 40 m. Equivalent to the preliminary test, horizontal deformation dominated the occurred sliding. After failure was reached, an observation pit was excavated to examine the sliding plane. The active part of the sliding plane could not be recovered. Probably, the active part is present under the centre of the embankment that could not be reached in the excavation. A long horizontal failure plane was found on the transition of the peat layer and the sand layer.Figure 18 shows an impression of the measurements. The horizontal deformation was measured at 1/3 of the inner slope. The initial horizontal deformation, approximately 20 mm, was found during the construction of the dike.

Figure 18 shows that, although not visually observed, the horizontal deformations started to develop as soon as the excavation at the toe started. On the morning of September 26[th] the deformations seemed to slow down until further excavation started around noon. Then, the horizontal deformation accelerated. The next day, the horizontal deformation further accelerated when filling of the sand core started at noon, until failure occurred at 16:02h. The measurements show that continuous and consequent horizontal deformation occurs in the subsoil long before it can be observed visually at the surface.

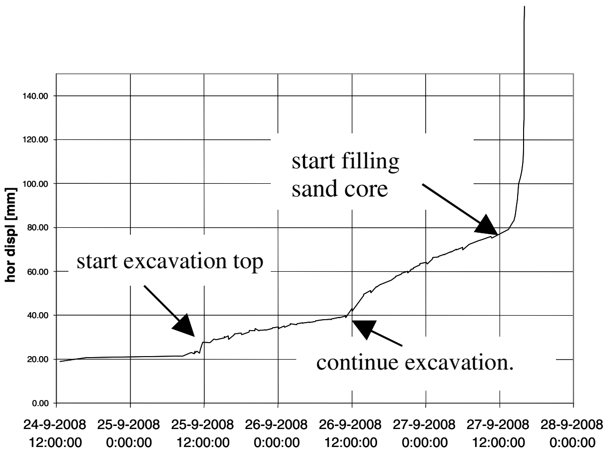


Figure 18. Horizontal displacement at 2 m below ground level

The horizontal deformation measurements illustrate the possibility for modern sensor technology to act as an early warning system. The preliminary test as well as the full-scale test demonstrate that the measured horizontal deformation shows a continuous deformation long before deformations can be observed visually in the field at the surface. Deformations could only be observed by naked eye a few minutes before failure occurred. The unseen horizontal deformations seem to indicate a trend, hours before failure occurred. The centrifuge tests and the preliminary test were essential in understanding the mechanism and decreasing the huge uncertainties in the design of the full scale test.

5 SUMMARY AND CONCLUSIONS

Most geotechnical behavior is unseen under the ground surface. Monitoring, physical modeling, numerical modeling and evaluation of performance in hind casts are valuable options to measure and understand the real behavior of the structure and the subsoil. Unsafe or too expensive structures often resulting from simple design rules are not well recognized. Therefore, monitoring and physical modeling should be a part in a broader research and design framework as well as in order to be able to communicate about the real uncertainties in the geotechnical structure to the stakeholders and decision makers outside the geotechnical field.

Almost all monitoring results in the papers of session 3B (see appendix A) are hind cast studies by means of Finite Element Methods. First class predictions according to Peck are rare. Besides the more complex FEM, a single paper includes an analytical or neural network approach in the evaluation. If these FEM hind casts results, which are the best our profession can do to postdict, are within lets say a band width of 10% to 50% of the measurement the authors (including the authors of this report) are satisfied by the results. And probably in a first class prediction, most authors would be glad to be within the 50% range.

This article further elaborates on the uncertainty our profession has to deal with and describes a framework todetermine and explain the geotechnical uncertainties engineers are confronted with, and a framework to decrease the uncertainties by better understanding the behavior of geotechnical structures.

For sound engineering judgment, as proclaimed by Peck, engineers have to be trained and experienced. If at least three engineers are asked to make an independent prediction an engineering factor can be quantified. Furthermore by weighing probabilities of considering several soil stratifications or several constitutive models from a number of independent experts uncertainties can be quantified. Then the resulting variation is less out of range with regard to other building materials, the value of eventually obtaining additional information becomes emphasized, investments and efforts towards intrinsic prediction model improvements do make sense, and our image outside the profession will improve.

The innovation cycle of calculation model, physical model and field test is an effective way to reduce the design uncertainties. Modern sensor technology can be used to gain extra information on geotechnical structure behavior. Measurements can be used to indicate failure in an early stage. Physical modeling seems to play an important role in understanding the mechanisms and reducing uncertainties due to subsoil heterogeneity and uncertain boundary conditions and validating design rules.

APPENDIX A PAPER OVERVIEW OF SESSION 3B MONITORING - MONITORING, PERFORMANCE AND EVALUATION

	Settle-ments	horizontal displace-ments	Sw ell	test proce dure	dynamic behavior	Stres-ses	water pressures, qualitity and flow
Road / Rail	1,7,26, 39,53				14		
Dam / slopes	3,11,49	29,5					24,36
Tunnel	4,6,16, 19,21, 30,52	6	4	38		17	
Retaining wall / deep excavation	23	2,5,47,10, 13,20,22, 23,31,42	10	8	41		46
Pile / raft foundation	27,28, 34,37, 48,51	45		9,25, 32	25,33,36, 40,44		
Landfill / dredge material	12,15, 43						18

Papers of session 3B Monitoring, Performance and Evaluation are published in the XVIIth International Conference on Soil Mechanica and Geotechnical Engineering, Alexandria, 5 to 9 oct 2009, published in this proceedings.

1, Asiri Karnuawardena, Construction of a trial embankment on peaty ground using vacuum consolidation method for a highway construction project in Sri Lanka

2, A. Kullingsjö, Effects of deep excavations of soft clay on the immediate surroundings

3, A. Gurbuz, T. Dincergok, Long term behavior of staged construction of a dam on soft clay

4, A. Ramon, S. Olivella, E.E. Alonso, Swelling of a gypsiferous claystone and its modelling

5, C.H. Chen, Y.C. Tsai & T.R. Wen, Back analysis for a deep excavation in Taipei MRT Underground Station

6, N. Phienwej, Combined cut and cover and New Austrian Tunnelling methods for MRT station in Bangkok sub soils

7, D.T. Bergado, N. Phienwej & P. Jansawang, Settlement characteristics of full scale test embankment on soft Bangkok clay improved with Thermo-PVD and stiffened deep cement mixing piles

8, S.H. Cheng & D.W. Chang, Modeling and interpretation for ground anchor failure signals under pullout loads

9, Yit Wah Chong, Use of instrumented static pile loca test results as a "crystal ball"

10, K. Stern, Influence of the rate of construction and of rock mass strength reduction on the stability of deep cuts in marl

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13, Antonio Soriano, Monitoring and Evaluation of Movements in Gravity Quay Walls

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