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Beyond limits of FEM calculation methods

Au-delà des limites des méthodes des éléments finis

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ABSTRACT

The accuracy of the simulation of geo-engineering phenomena by finite element calculation methods is limited by the method itself, uncertainty in soil properties and subjective choices and arbitrary assumptions of the engineer. The effect of these limitations can be surprisingly large and may be underestimated. Corresponding results may not comply with the expectations of decision makers and clients. The paper will address these aspects, tries to catch the engineer's role in an engineering factor, and will give some new directions how to cope with these uncertainties, which are in fact inherited from the empirical nature of the geotechnical profession.

RÉSUMÉ

Le précision de la simulation des phénomènes géotechniques par les méthodes des éléments finis est limitée par la méthode elle-même, l'incertitude dans le charactère du sol et par les choix subjectifs et suppositions arbitraires de l'ingénieur. L'emportance de l'effet de ces limitations peut parfois être surprenant et sérieusement sous-estimé. Les résultats qui en découlent peuvent être en conflit avec les attentes des décideurs et des clients. Le but de cet article est de discuter ces problèmes, de définir le rôle de l'ingénieur dans ce qu'on pout appeller le facteur ingénieur, et de donner de nouvelle idées pour contourner ces incertitudes qui dérivent de le nature empirique de la profession géotechnique.

Keywords : geo-engineering, prediction contest, FEM, engineering factor, uncertainties, variation coefficient

1 INTRODUCTION

For large projects, where soil mechanics constitutes a relevant part, cost and time seem not well predicted; they usually overrun seriously, and sometimes it goes wrong entirely. Why? Peck (1980) proclaims that sound engineering judgment is of prime importance above sophisticated approaches. After studying reports over a period of 25 years, Littlejohn (1991) states inadequate soil investigation and lack of awareness of this aspect is to blame. 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.

In this article the role of the engineer is considered with regard to his capability to predict and decide, i.e. to apply quantifying prediction models adequately. 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.

2 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[1] for steel it is not beyond ±5%, for concrete it is about ±15%, for soil it is usually beyond ±50%. Outside our 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 environment 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 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

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

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with regard to groundwater and soil matrix in terms of stresses, fluxes and/or displacements. This also holds to some extent for 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.

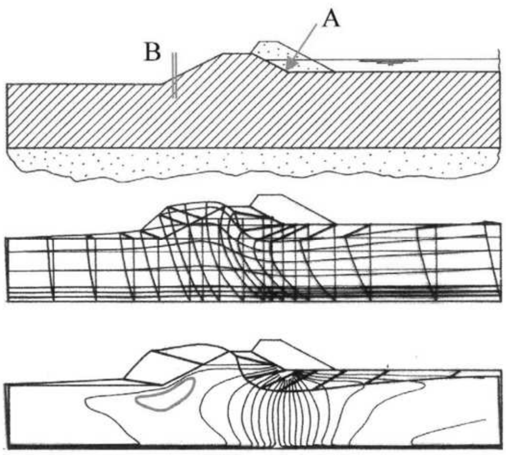

Figure 2. A river embankment subjected to sudden water level rise.

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, which did not comply with the hydrodynamic period of the clay layer (months). The 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[2] 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.

4 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 4). 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.[3] 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.

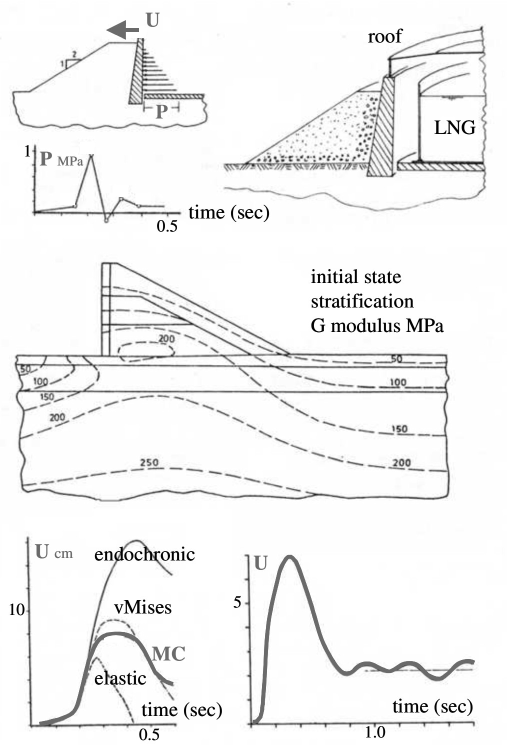

Figure 4. Effect of the choice of a constitutive model

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.

5 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 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

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

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

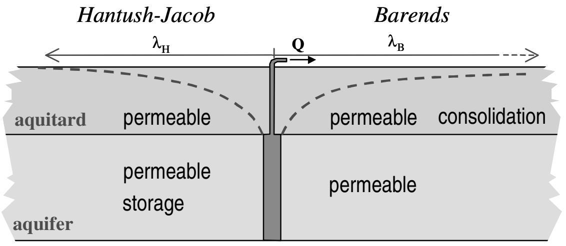

Figure 3. Physical processes in a semi-confined aquifer system

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.

6 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.

6.1 *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).

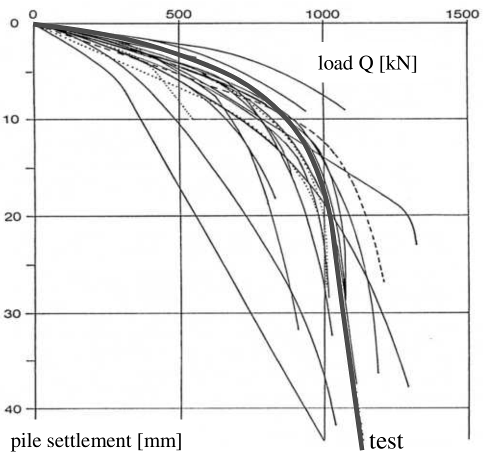

Figure 5. Predicted and actual load-settlement of a concrete pile

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

6.2 *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.

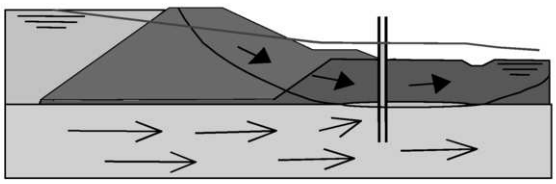

Figure 6. The uplift phenomenon affecting river embankments

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 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)

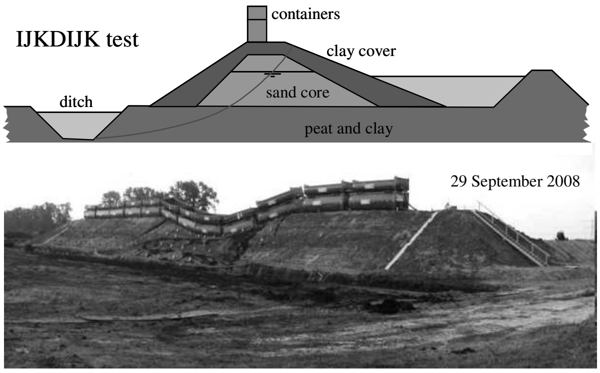

Figure 7. The real scale slope stability test

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

6.3 *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.[5] 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.

7 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 coefficient varies somewhere between 0.20 and 0.45, which is quite large. In this respect, further investment in improvement of prediction models is in fact practically not worthwhile, unless the size of this large variation is drastically reduced.

8 HOW TO DEAL WITH IT?

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, multi-disciplinary cooperation is stimulated, 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.

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

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