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Technical session 2f: Embankments and dams

Séances techniques 2f: Remblais et barrages

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1 SESSION ORGANIZATION

Technical Session 2f, Embankments and Dams, was held on Monday, September 12, 2005. The session was organized as specified below. The Chairperson was Tadatsugu Tanaka (Japan) and the Session Secretary was Shin-ichi Nishimura (Japan).

I. General Report: Andrew Whittle (USA)

II. Panelist Presentations:
Luiz Guilherme de Mello (Brazil)
- Some reflections on design decisions in embankment engineering

Sergei Terzaghi (New Zealand)
- Time dependent soil behavior with respect to dam and embankment design

Henk L. Bakker (Netherlands)
- Calculation of the likelihood of macro instability of dikes

Akira Asaoka (Japan)
- Naturally deposited soils and artificially remolded NC soils

Summaries of these presentations are described in the following sections.

III. Discussion Topics:
1) Time dependent soil behavior and its influence on stability of dams and embankments
2) Dams and embankments on soft soil
3) Use of back analysis in dam designs
4) Safety analysis considering static and dynamic loads

2 GENERAL REPORT

The General Reporter, Professor Andrew Whittle, divided the twenty-eight submitted papers into four categories:

1. Construction Material Behaviour
2. Soft Foundation Problems
3. Stability Problems of Earth Structures
4. Design and Construction

Professor Whittle explained all the papers and then gave a report on the disaster caused by Hurricane Katrina. Finally, he raised the following questions based on the submitted papers:

1) How do advances in the understanding of the material behaviour (weathering, creep, and partially saturated and compacted fills) affect the design, the condition assessment, and the predicted performance of dams and embankments?
2) Ground improvement techniques are well established. Are there examples of well-documented case studies confirming the design assumptions for these techniques?
3) What are the most appropriate methods for evaluating the stability of dams and embankments that account for a) uncertainties in the current state and b) static and dynamic environmental loading conditions?
4) How will recent advances in sensor and communication technologies be used to monitor the construction and the performance of embankments and dams?

3 SOME REFLECTIONS ON DESIGN DECISIONS IN EMBANKMENT ENGINEERING

Four important questions were raised in the context of Technical Session 2f by General Reporter Whittle, as shown in the previous section.

When dealing with embankments in which compacted rockfill is an important part of the structure, as is the case with concrete face rockfill dams and rockfill dams with an impervious compacted core, there is an interesting interplay among recent advances affecting the design, the condition assessment and the prediction of both the long-term and the short-term performance, accounts of the uncertainties in the material properties of aging structures, and difficulties in performing representative back analyses of the monitored stability and/or the deformation behaviour.

Some facts are demonstrated by the monitored performance of rockfill, namely, its excellent long-term behaviour when properly compacted, the difficulties of defining what is failure in a compacted rockfill slope, and its intrinsic relationship with the hardship of adjusting the back analyses due to the non-existence of failure. The very important difference between the failure and the "accommodation" of compacted rockfill has to be acknowledged when the monitoring data from rockfill slopes are interpreted. Large vertical and horizontal displacements are associated with the construction and the impounding of rockfill dams, and these deformations are not associated with any distress of the structure, as demonstrated by the long-term behaviour of numerous embankments.

The difficulty of pursuing knowledge of the behaviour of this type of material in the laboratory, with the intrinsic difficulties of representing samples (dimensions, grains-size distribution, imbrication, etc.) and of reproducing the overall and the contact stress conditions prevailing in the field, would lead to the tentative use of back analyses as an important tool. However, the use of these analyses unfortunately cannot be considered in this case.

Practical solutions have been proposed and successfully implemented in embankment construction. When it was learned that the saturation of compacted rockfill could generate settlements at a constant load (collapse (Nobari and Duncan)), simple

Table 1. Rock fill dam compaction

Dam	Rock Type	Equipment	Roller Weight (tf)	Dynamic Impact (tf)	Vibratory Frequency (Hz)	Equipment Coverage	Roller Speed (km/h)	Water (m3/m3 of rockfill)	Specific Weigth Required (tf/m3)
Foz do Areia	Basalts			18,2	18 a 25	> 4	< 5	> 25%	-
Segredo	Basalts			35,0	23	> 4	< 5	> 25%	-
Salto Santiago	Basalts			-	-	> 4	< 5	no water required	-
Corumbá I	Quartzit	Vibratory Rollers (CA251D)	varying between 8 to 10 tons	-	-	> 4	< 3	25%	2,0
Água Vermelha	Basalts			-	30	-	3 a 5	no water required	-
Tucurui	-			-	18 a 28	3 a 10	< 3	no water required	1,9 a 2,0
Xiaolandgi *	Sandstone			29,0	20 a 30	6	< 3,2	no water required	-

* China

tests were proposed in order to yield index parameters, like the unconfined compressive strength at the natural "water content", and also after saturation of the specimens for long periods, the compaction specifications, associated with design decisions, like that of acceptance of usage of "dirty rockfill", and construction specifications adapted to the specific characteristics under study.

Table 1 below summarizes some specifications of recently compacted rockfill dams in Brazil, using material of different lithologies, into which a specified amount of water is added to the rockfill after placement and during its compaction.

Research has not, to this moment, been able to contribute beyond the practical solutions adopted and tested.

4 TIME DEPENDENT SOIL BEHAVIOR WITH RESPECT TO DAM AND EMBANKMENT DESIGN

4.1 *Types and influences of time dependent behavior*

There a number of considerations with respect to the time dependent soil behaviour with respect to dam and embankment design. These considerations are further modified with respect to whether one is dealing with a dam or an embankment.

If one considers a dam, the major issues with respect to time dependent behaviour are with respect to self weight compaction/consolidation and to particle crushing, and in some types of dams hydro- compression.

With respect to embankments, soft foundations are the usual cause of time dependent deformations, though hydro-compression and self-weight compaction contributed.

Seismic and other forms of cyclic loading (especially traffic) contributed to other aspects of time dependent deformations, particularly on older underdesigned structures. One issue often forgotten is the construction sequencing.

4.2 *Design issues*

Design issues to consider is the trigger mechanism, duration of movement with respect to the design life of the structure, and implications of the movement for the structure. This includes the changes in strength, usability of the structure due to the movements.

A number of interesting findings have been reported and discussed during the course of this conference on these issues. One of these is the influence of salts and influence on particle crushing. It appears that certain salts may retard the rate of particle crushing. Another feature is the influence of structure within soils and the subsequent changes in rate in deformation and strength as the structure is broken during deviatoric loading.

4.3 *Case histories*

To illustrate some of these issues, a case history was presented that addressed during design the issues of primary consolidation and creep. Hydro-compression was indirectly addressed, but was still found to be an issue as a result of one area being undercompacted for some reason, in conjunction with high intensity rainfall event.

5 CALCULATION OF THE LIKELIHOOD OF MACRO INSTABILITY OF DIKES

5.1 *Introduction*

Why is it necessary to conduct a safety analysis of river dikes strengthened with structural elements? The answer is contained in the four reasons given below.

1) As prescribed by law in the Netherlands, dikes must satisfy a certain yearly maximum failure probability.
2) Due to a lack of space and to save objects along the dikes, many dikes are strengthened with structural elements such as sheet-pile walls.
3) After being assigned by the Government, a project is carried out to determine the safety against flooding of all the so-called dike rings. This project is called "Veiligheid Nederland in Kaart" (Safety Netherlands Mapped).
4) The project requires the calculation of the failure probabilities of all dikes, including those strengthened with structural elements.

5.2 *Parameters and uncertainties*

Parameters which describe the behavior of dikes with structural elements include:

- Shear strength of soil
- Strength of structural elements
- Geometry of dike and structural elements
- Ground water tables
- Weight of soil and structural elements
- High water level and other external loads
- Environmental influences

Not only can all of the above-mentioned parameters be uncertain, but changes in time due to alterations in the environment or utilization (for example, corrosion and fatigue) can also be uncertain. Uncertain parameters are described by the statistical distribution functions, for example, the normal (Gaussian) distribution with mean value (μ) and standard deviation (σ).

Combinations of parameters at the border of failure and non-failure are so-called failure functions or Z-functions. From the distribution functions of the parameters and the Z-functions, the trigger of failure can be determined.

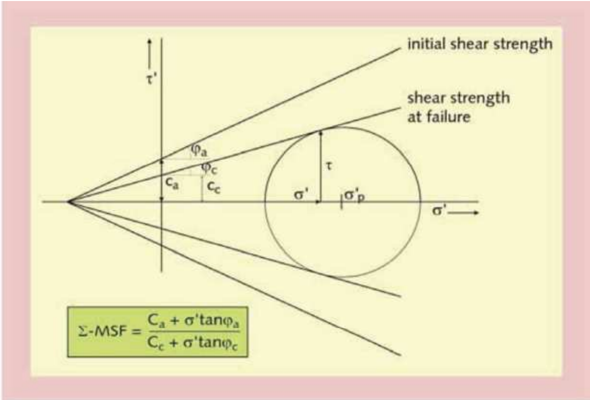

Fig.1 Phi-C reduction method

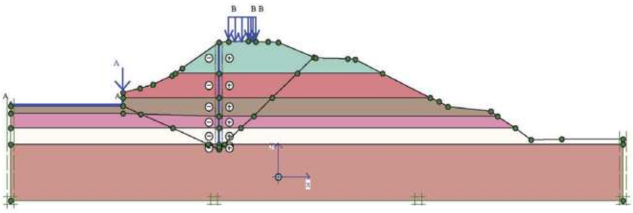

Fig.2 River dike strengthened with sheet pile

5.3 *Analytical models*

Two types of models can be considered for the failure probability analyses of constructions:

1) Structural models for the evaluation of stress and deformation which can be divided into graphics and tables, classic (soil) mechanics calculations, and finite element calculations
2) Probabilistic models for safety analyses

Generally speaking, probabilistic models can be divided into three methods:

Level 1 methods: These methods are used to assure safety against failure by applying partial factors to uncertain parameters.

Level 2 methods: These methods imply the calculation of the failure probability with the linearization of the Z-function, for example, a design point analysis or a FORM analysis.

Level 3 methods: These methods imply the precise calculation of the failure probability, for example, the Monte Carlo simulation with the random drawing of 10,000 parameter sets, and the calculation of each set to determine whether failure has occurred or not.

5.4 *Probabilistic model for river dikes strengthened with structural elements*

Firstly, the random fields of the strength parameters were modeled. As the constitutive model, the conventional Mohr-Coulomb model was employed with the Phi-C reduction method (Figure 1). Phi/C reduction is an artificial process, very different from real loading. The Phi/C reduction is used to find the minimum shear strength necessary for stability and not to describe pore pressure increases due to shearing. The Plaxis was employed as the FEM code here. For the clayey materials , the undrained shear strength should be employed, and for sandy materials, the drained strength parameters are used.

The method is generally a Level 2 method, although Level 3 principles are applied, and the method is composed of five steps. The method can be explained by a practical example (Figure 2).

Step 1: Calculations of soil failure mechanism and a safety factor Σ-MSF with Plaxis for fixed normative high water level (MHW) and determination of the number of statistically independent soil layers (n) in the failure mechanism.
It was found Σ-MSF = 1.28 and 5 soil layers.

Step 2: Calculate the probability of soil failure for the fixed normative high water (MHW). A reliability index for the soil failure of the fixed normative high water level (MHW) was found, namely, $\beta_g|_{MHW} = 3.34$.

Step 3: Determine safety factors Σ-MSF from a variation in the calculations for other uncertainties, for example, the high water level in the river, the ground-water tables and geometry, etc. From the results of variability calculations, the reliability index for soil failure that is inclusive of some uncertainties, but exclusive of other uncertainties in a high water level, can be determined. For the reliability index for soil failure that is inclusive of some uncertainties, but exclusive of uncertainties in a high water level, $\beta_g|_{MHW} = 2.99$ was found.

Step 4: Include the likelihood of sheet-pile failure in the failure probability of the construction. In this case, it was found that the likelihood of sheet-pile failure can be disregarded, thus, $\beta_c|_{MHW} = \beta_g|_{MHW} = 2.99$.

Step 5: Include uncertainties of the high water level in the calculation of the reliability index. Uncertainties in the high water level can be described by a Gumbel distribution function. The reliability index is inclusive of uncertainties in the high water level and follows from the integration over all the values for H of the failure probability for a fixed value for H. The reliability index, which is inclusive of uncertainties in the high water level, is $\beta_c = 4.08$. It corresponds to a yearly failure probability of 2.25×10^{-5}.

5.5 *Concluding remarks*

1) A probabilistic method was proposed to calculate estimates of the reliability index of dikes strengthened with structural elements.
2) A practical example of the proposed method was presented.
3) The method can be used for other geotechnical structures.
4) The method requires only a small number of geotechnical calculations and only input that can be acquired easily and is understandable to practical geotechnical engineers.
5) The method is implemented on a number of spreadsheets.

6 NATURALLY DEPOSITTED SOILS AND ARTIFICIALLY REMOLDED NC SOILS

Naturally deposited clays/sands are mostly found in the structured states. In addition, these soils are usually in overconsolidated states. Thus, the following two problems arise:

- What is structure?
- What is overconsolidation?

As shown in Figure.3, structured soil occupies a greater bulk than remolded soil. Since structured soil can take its mechanical state in the "impossible region" in the sense of the conventional critical state theory, the superloading yield surface lying outside the Roscoe surface is proposed. Decay/collapse of soil skeleton structure yields the plastic deformation which always acts in the direction of plastic volume compression, and consequently, it can lead the delayed compression/secondary consolidation of structured clay.

In overconsolidated soil, the loss of overconcolidation (from OC to NC) acts in the direction of the plastic volume expansion. At the beginning of the expansion, plastic hardening occurs above the critical state line with volume expansion (Figure 4). In order to model the behavior of OC soil, it is necessary to allow for elasto-plastic responses inside the initial yield surface. This problem can be completely solved by introducing a subloading yield surface which is assumed to lie inside the initial yield surface.

In order to describe the loss of overconsolidation and the decay of the soil skeleton structure simultaneously, the super/subloading yield surface model is proposed. The model can clearly simulate the difference between the elasto-plastic behavior of overconsolidated sand and that of structured clay.

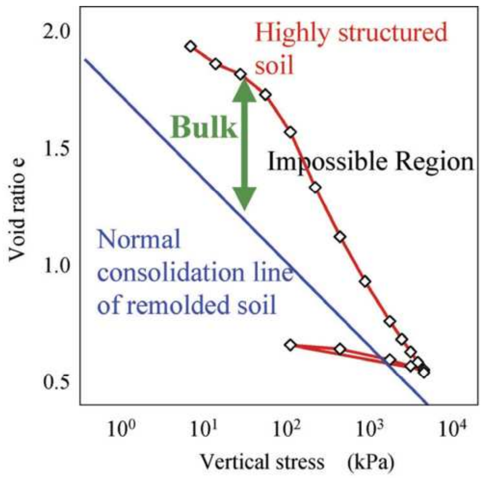

Fig.3 Compression curve of structured soil

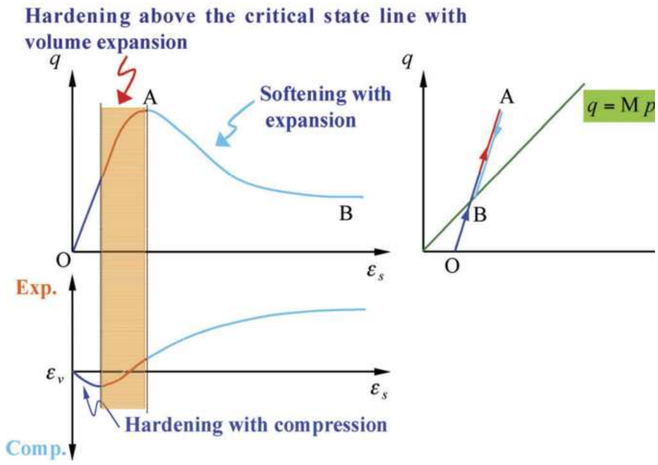

Fig.4 Behavior of overconsolidated soil

7 FLOOR DISCUSSIONS

The floor discussion was proceeded by Professor Marte S. Gutierrez (USA) as a discussion leader, and the following is a record from the floor:

Comment and question from P. Brenner (Switzerland) "Back-analysis" can only tell us how good our models is, and it appears to be only useful if applied to projects that are similar (to the ones models that have been calibrated or previously applied to). It may only be useful to verify assumptions and may not be useful outside of previous applications and experience in the use of the model.

L. de Mello (Panelist) agreed that back-analysis, although it provides very useful results, cannot be universally applied. For instance, back-analysis is difficult for rock-fill materials and may not account for aging of materials. It is only applicable, so far, to static analysis.

M. Gutierrez commented that strictly the model and not just the parameters should be part of the back-analysis.

A gentleman from Delft, Netherlands pointed out that one of the roles of the back-analysis is to find out the cause of failure. He pointed out that from his experience back-analysis often provides a reliable picture of response which is in agreement with observed field behavior. However, there were also cases when the back-analyzed failure mode was quite different actual mode of failure.

S. Terzaghi (Panelist) commented on back analysis and role in design:

Back analysis is a powerful tool in the armoury of the geotechnical engineer, however, like all tools, the purpose and expected outcomes need to be carefully considered during the execution of the analysis. The parameters, mechanisms and outcomes resulting from the back analysis must be carefully matched against the field results, and the differences discussed. Equally the limitations of the models used must also be described, along with the reasons(advantages) why the particular model was used.

V. Griffith (USA) inquired about the role of spatial correlations to back analysis. He noted that materials may have the same mean values and standard deviations but may be spatially correlated and that for the same material the spatial correlation might vary.

H.L. Bakker (Panelist) responded that spatial variability should indeed be accounted for in back-analysis and that spatial variability can be expressed in term of correlation lengths. He mentioned that in, soils horizontal variability is typically less than vertical variability (for example spatial correlation lengths are typically about 30-40 m horizontally and but only a few cms vertically).

K.R. Massarch (Sweden) commented that a major issue in major embankments and dams is the difficulty of conveying the importance of geotechnical engineering to the public. He noted that increasing public awareness of the role of geotechnical engineers should be an important aspect of geotechnical engineering and he alluded to a current EU-funded project along this direction.

An additional discussion from the floor mentioned that the insurance should be an important part of the project and that like medical doctors engineers should be aware of potential liabilities and risks associated with the project. Engineers should continuously inform the public of the nature of uncertainties in geotechnical engineering. Engineers should also be aware of the political aspects of their project and that they should be part of the politics related to the decision process.