Technical Session 2-B: Slopes and Embankments

Séance Technique 2- B: Pentes et remblais

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ABSTRACT
Sixty one papers presented to the Session are discussed in the Report. They have been classified in the following general topics: Analysis, design and stabilization procedures, rainfall and drainage, seismic analysis, embankment materials and foundation soils, and other topics. The Report highlights a few relevant aspects: the generalization of finite element methods in analysis, design and evaluation of performance, the popularity of (c, ϕ) reduction methods to estimate Safety Factors and the increasing use of three-dimensional models. Despite these trends analytical and semi-analytical methods are still developed and useful in practice in a wide variety of topics. It is stressed that the determination of material parameters remains as the fundamental issue in most of the work reviewed. Specialized topics with their own methodology are also required in practice. Current advances in unsaturated soil mechanics provide also a new insight into classical problems such as slope stability and rainfall or the design and construction of dams and embankments

RÉSUMÉ
Les Soixante et un documents présentés dans cette session sont examinés dans ce rapport. Ils ont été classés dans les thèmes généraux suivants: analyse, conception et méthode de stabilisation, précipitations et drainage, analyse sismique, matériaux de remblai et sol de fondations ainsi que d'autres thèmes. Le Rapport met l'accent sur quelques aspects pertinents: la généralisation de la méthode des éléments finis, la conception et l'évaluation de la performance, la popularité des méthodes de réduction des caractéristiques (c, ϕ) pour évaluer les facteurs de sécurité et l'emploie croissant des modèles en trois dimensions. Malgré ces tendances, on continue à employer des méthodes analytiques et semi analytiques, qui en pratique s'avèrent être utiles dans plusieurs domaines. On doit souligner que la détermination des paramètres des matériaux reste la question fondamentale dans la plupart des articles examinés. Des thèmes spécialisés avec leur propre méthodologie sont également nécessaires dans la pratique. Le progrès actuel en mécanique des sols non saturés a apporté de nouvelles clarifications dans des problèmes classiques tels que la stabilité des pentes et l'analyse des précipitations ou de la conception et la construction de barrages et de digues.

Keywords : slope stability, embankment construction, finite element analysis, stabilization, rainfall, drainage, safety factor, analytical solutions, seismic design, rock slides, probability

1 INTRODUCTION

Sixty one papers are included in the Technical Session. They are evenly distributed in the two main topics. They are listed at the end of the Report. They could be further grouped as follows.

- Slopes:
  - Analysis (9 papers)
  - Rainfall and drainage (7 papers)
  - Stabilization procedures and design (8 papers)
  - Engineering geological aspects (4 papers)
  - Probabilistic analysis (2 papers)
  - Rock slopes (2 papers)
- Embankments:
  - Analysis (6 papers)
  - Design (8 papers)
  - Seismic analysis (7 papers)
  - Properties of embankments materials (4 papers)
  - Foundation soils (2 papers)
  - Other (2 papers)

Given the similarities of the concepts and tools usually employed to perform the analysis of slopes and embankments and the common ground usually found in the engineering design and investigation procedures associated with slopes an embankments, the set of papers presented to the Session will be further grouped into a reduced number of topics as follows:

1 Analysis (15 papers)
2 Design and stabilization procedures (16 papers)
3 Rainfall and drainage (8 papers)
4 Seismic analysis (7 papers)
5 Embankments materials and foundation soils (6 papers)
6 Other topics in slopes and embankments (engineering geology, rock, probability) (9 papers)

2 ANALYSIS

2.1 *Safety factor*

A trend observed in the papers presented to the Session but also in a broader sense, is the increasing use of finite element procedures to estimate the safety of slopes and embankments. The "(c', φ') reduction" option included in some commercial programs is the technique followed to calculate safety factors which, in principle, should be similar to the safety factors calculated with standard limit equilibrium methods. In fact, the notion of safety factor is the same in both cases. In limit equilibrium procedures it is defined as the ratio between the available shear strength along the failure surface and the shear stress in strict equilibrium. The (c', ϕ') reduction procedure implies a similar concept: Once a situation of equilibrium is found for a particular set of soil constitutive parameters, the progressive and uniform reduction of strength parameters is introduced until equilibrium is no longer possible. The reduction coefficient is interpreted as the inverse of safety factor. The procedure has been described in a few references

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(Matsui and San, 1992; Brinkgreve and Bakker, 1991, Griffiths *et al*. 1999).

Differences when comparing limit equilibrium and the (c',φ') reduction procedure should exist for a number of reasons. The equilibrium stresses calculated in FE procedures will not be equal to the stress calculated by LE methods on any of the potential failure surfaces. Also, the critical failure mechanism is not necessarily the same when using the two procedures. However, in simple geometries LE methods are known to approximate with sufficient accuracy the failure surface automatically obtained by a FE (c',ϕ') reduction method.

This is not always the case. Consider, as an example, the failure mechanism calculated by a (c',ϕ') reduction method for a caisson breakwater under wave action (Fig. 2.1). The caisson is founded on a granular embankment sitting on a deep NC clay soil. The construction process (initial excavation of the sea bottom, construction of the granular fill in successive layers and the sinking of caissons) is reproduced in the FE analysis. In this case, given the soft and impervious nature of foundation soils, the critical situation is the undrained stability of the underlying clay under the action of gravity and storm loads (represented by a pseudo-static force).

Figure 2.1 shows an interesting result: the critical failure surface identified by the contours of incremental plastic shear strain, initiates under the caisson as a "deep" failure type but then follows parallel to the granular bed and eventually daylights on the sea bottom at the edge of the fill. In this way a relatively complex failure surface, involving changes in the sign of its curvature, is predicted by the model. Limit equilibrium software based on slice methods are often not prepared to deal with this situation in a systematic manner.

The advantage of the FE procedure is clear in this case. Note also that the construction process involves a parallel consolidation of the soft soils which has a paramount importance to guarantee the stability. The increase in time of undrained strength, c_u , is a natural outcome of the FE analysis if the constitutive model is prepared to predict correctly the current c_u values. "Cap" type models are required. Again, this is an advantage of FE models when compared with LE methods in the presence of a consolidation process.

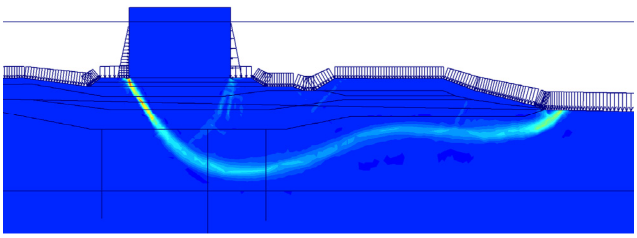

Figure 2.1 Failure surface developed in a (c', ϕ') reduction method.

In the case briefly outlined above there was also an interest in an independent check of the FE results using the conventional LE (methods of slices) procedures. The conventional stability calculation is illustrated in Figure 2.2. The domains indicated in the figure correspond to specific values of the undrained strength which were independently determined for a particular distribution of effective stresses in the foundation soil.

In the case represented, the undrained strength follows the relationship $c_u = 0.25\ \sigma'_v$ which is a good approximation for the normally consolidated low plasticity deltaic soils of the Barcelona Harbour.

Safety factors were compared for the two types of analysis (FEM and LE) for equivalent strength distributions and loading conditions. In general, a good agreement was found. Differences in SF were in the order of 0.1, the FE procedure providing the lower estimations.

The use of the (c',ϕ') reduction method to calculate safety factors via FE (in most of the cases reported here through the PLAXIS program) is illustrated in some of the contributions presented to the Session. The widespread dissemination of flexible and user friendly FE programs facilitates the analysis of complex geometries and construction stages, invites the performance of sensitively analysis, allows the evaluation of codes of practice, facilitates the analysis of alternative stabilization designs and serves to establish the limitations of simplified design or calculation procedures. Examples of all of these possibilities are given in the papers presented to the Session.

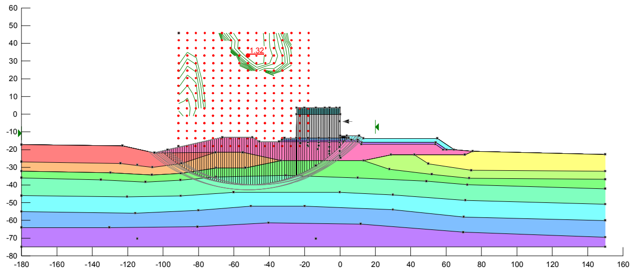

Figure 2.2 LE failure analysis (Morgenstern-Price method, circular failure surface) of caisson under static horizontal wave loading and selfweight.

One of the risks of an easy access to a sophisticated and versatile computational tool is to maintain the analyses within the purely numerical world. Sensitivity analyses, for instance, are often a favourite application of a FE program. It may teach some lessons but there are no alternatives to a sound geotechnical approach. One of the criticisms which may be made to some of papers received is the very limited attention, if any, devoted to the methodology to justify the parameters used in the analysis.

Some of the papers using commercially available FE tools are now briefely reviewed, having in mind the preceding comments.

Totsev and Jellev (2009) compare a few popular limit equilibrium methods with a (c',ϕ') reduction procedure for a steep slope in hard clay. Surprisingly the (c',ϕ') reduction methods provides an unstable slope (SF = 0.74) when the four limit equilibrium methods result in SF values in the range 1.50-1.60. Some inconsistency is probably introduced in the analysis.

Gaszy ski and Posłajko (2009) report the use of rows of piles driven at the toe of both sides of road embankments in Poland. Soil parameters were approximated by simulating the unstable conditions prior to remedial works. Dimitrievski *et al.* (2009) analyze the stability of a tailing's dam in Macedonia. Both a FE program and a conventional limit equilibrium method were used. Varga *et al.* (2009) stress the relevance of capillary effects to increase the apparent cohesion of cohesive soils. They perform an academic comparison, via FE, of undrained and drained analysis of a cut slope having a simple geometry.

Heibaum and Herten (2009) discuss the relative merits of alternative design approaches. They favour the so-called Design Approach 2* introduced in German standards. Under this approach external actions are increased and soil strength parameters are maintained in their realistic values. They stress the advantages of using FE procedures and highlights that they provide also stresses, deformations and pore pressures at intermediate construction stages and therefore they are convenient tools to follow the observational method.

The approach to select an adequate definition of safety factor should be a flexible one. Probably no single method could be selected as the best procedure under all circumstances. The (c',φ') reduction method is probably a good choice when failure is associated with the development of a continuous shearing surface within the soil. It allows a direct comparison of LE and FE methods as illustrated below.

Moreover, the "scale" of the SF determined in this manner is in general well understood. Values of SF in excess of 1.5 are generally seen as safe values. Smaller SF values require a more detailed consideration, typically addressing the important issue of the reliability of the strength parameters used. In the presence of strain softening, which leads to progressive failure phenomena, the (c',ϕ') reduction is not particularly relevant to analyze safety. The concept of strength mobilization between the peak and residual strength values at the moment of failure has been discussed in connection with brittle overconsolidated clays by a number of authors (Skempton, 1964; Potts *et al.*, 1990; Stark and Eid, 1994; Gens and Alonso, 2006) Most of the papers presented to the session refer to soft soils and the issue of progressive failure has not been discussed in them.

Figure 2.3 illustrates two situations which are not properly analyzed within a general methodology of soil strength reduction. The two cases sketched (overturning failure under horizontal load and bottom instability of excavations) require other techniques. In the first case safety may be defined as a ratio between actions leading to failure (external load) and the actual one. Note that soil properties are only marginally involved in this case. The second case is conventionally addressed by calculating the ratio of stabilizing vertical stresses (soil total weight per unit area on the impervious surface) and the un-stabilizing stresses (pore water pressures on the lower boundary of the impervious clay layer). No reference to shear stresses is made here.

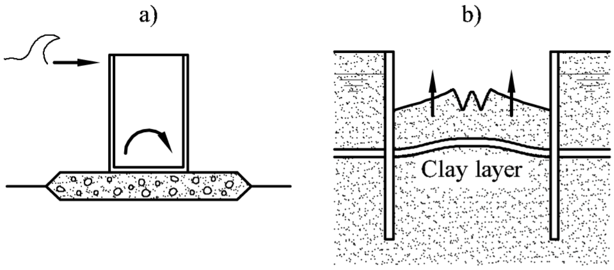

Figure 2.3: Two failure scenarios which cannot be properly handled by a (c',φ') reduction method: (a) Overturning failure of a caisson against external horizontal loading. (b) Bottom instability of an excavation due to non compensated water pressures.

Further discussion on the calculated safety factors of embankments on soft soil are given by Mansikkamäki and Länsivaara (2009) and Salokangas and Vepsalainen (2009). Both refer to real cases in Finland. The first authors confirm that LE and FE methods (c',ϕ' reduction) should provide similar results if correctly applied. The issue of performing the so called undrained "effective stress analysis" is, however, more controversial. It should be made clear that knowledge of pore pressures prior to failure does not provide the required information to perform an "undrained" analysis based on effective stress parameters. The relevant pore pressures are generated during the undrained failure itself and are not accessible to a drained analysis.

A further problem arises in FE calculations because the predicted c_u value depends on constitutive model details. Masikkamäki and Länsivaara (2009) illustrate the effect of stiffness parameters on c_u prediction for a popular model. In general it is recommended, when performing undrained analysis through a FE code, checking the performance of underlying constitutive models and their ability to predict reasonable c_u values. A sensible procedure is to model a well known laboratory test (simple shear, triaxial) and to check FE results against expected c_u behaviour. This check should include overconsolidation conditions, very often found in preloading schemes. One example is given in Figure 2.4, which compares a FE analysis of a simple shear test under different OCR's and the expected result given by the well accepted expression:

$$(c_u)_{OC} = (c_u)_{NC}(OCR)^{0.8} \tag{2.1}$$

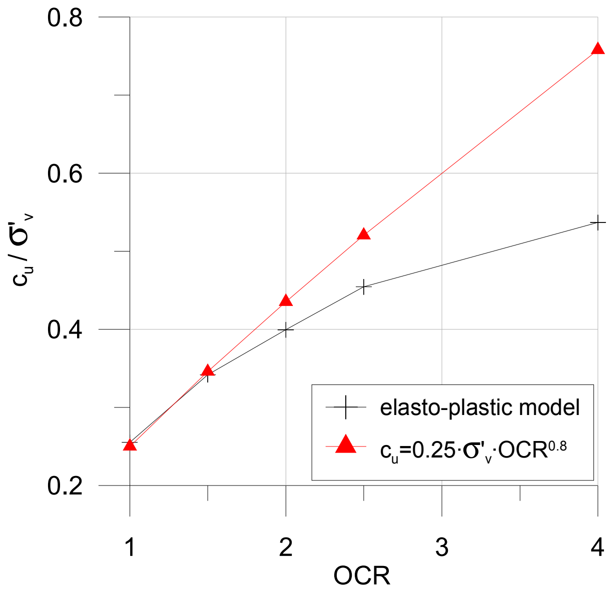

Figure 2.4: Simulation of undrained FE simple shear.

The FE analysis was conducted with a well known cap-type model. The figure shows that the FE analysis tends to under estimate the expected undrained strength of the soil.

An additional point concerns the comparison of safety factors obtained by the (c', φ') reduction technique and the so - called "gravity increase" method. In the second case the SF is defined as the ratio between the vertical required gravity acceleration inducing failure and the actual g value (9.8 ms^{-2}). In the case of undrained analysis and in simple cases the two procedures should provide the same answer. For instance, if the stress applied by an embankment is defined as γh, γ being the natural unit weight of the embankments soil and h its height, there exists a linear relationship between γh and c_u of the foundation soil:

$$\gamma h = N_c c_u \tag{2.2}$$

Therefore SF definitions based on c_u ratios or on γ ratios would be identical. In the case of simple slopes under undrained conditions, stability is defined by means of a stability number N which depends on geometry.

$$\frac{c_u}{\gamma H} = N \tag{2.3}$$

where H is the height of the slope. Again a linear relationship between c_u and γ is obtained. In drained cases this linearity is immediately lost when one compares γ and (c', tan φ') for liming conditions and therefore safety factors based on g increase or (c', φ') reduction methods would be widely different. Of course, in mixed problems (drained and undrained "soils" involved in a given failure mechanism), the comparisons between the numerical values of both measures of safety will be problem dependent.

2.2 *Soil reinforcement*

A few interesting methods to include soil reinforcement into the analysis, which range from purely analytical to FE numerical analysis are described in four papers.

Cuira and Simon (2009) describe a simplified procedure to analyze the response of an element of pile or inclusion and the "structure" (a slab, an embankment) adding weight on its upper end.

The elements of pile and surrounding soil are treated in a similar simplified manner. The formulation of vertical equilibrium on both elements is the starting point for the solution. Pile and soil are assumed to be elastic columns. The interaction between them is resolved by means of a shear stress, which is a nonlinear function of the relative vertical displacements of pile and soil. The method, which requires the solution of a system of first order differential equations, seems to be accurate if compared with numerical solutions (Figs. 2.5 and 2.6).

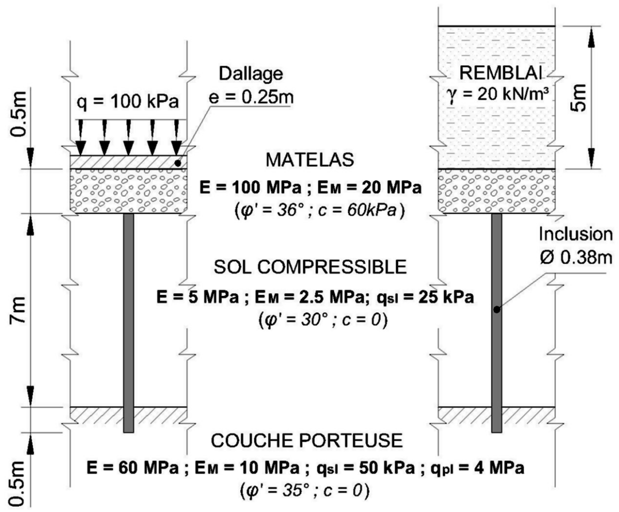

Figure 2.5: Example of reinforcement by means of stiff inclusions (Cuira and Simon, 2009).

The authors describe also the three-dimensional case of slabs on a soil reinforced by inclusions of the previous type. Soil and inclusions are modelled by means of nonlinear springs. Special cases, such as the tensile states that could develop, can be solved by an iterative procedure. Again, a good comparison between the simple model and a 3D FE analysis is reported.

Barvashov *et al*. (2009) develop an analytical procedure to design reinforcing nails in slopes. The main assumption is that the reinforced soil behaves as a Mohr-Coulomb material. Then, the necessary increase in apparent cohesion of the slope to ensure stability is calculated as

$$\Delta c > 0.5\sqrt{K_a}\left(\gamma z + q\right) - c \qquad (2.4)$$

where K_a is the active pressure coefficient, γ is the unit weight, z the vertical coordinate below the surface, q the overburden stress and c the existing soil cohesion.

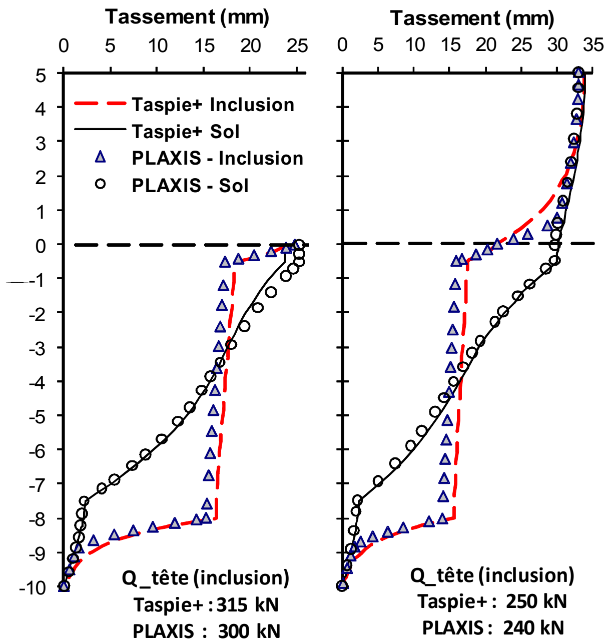

Figure 2.6: Settlement profiles calculated for two configurations: paving (left) and embankment (right). (Cuira and Simon, 2009).

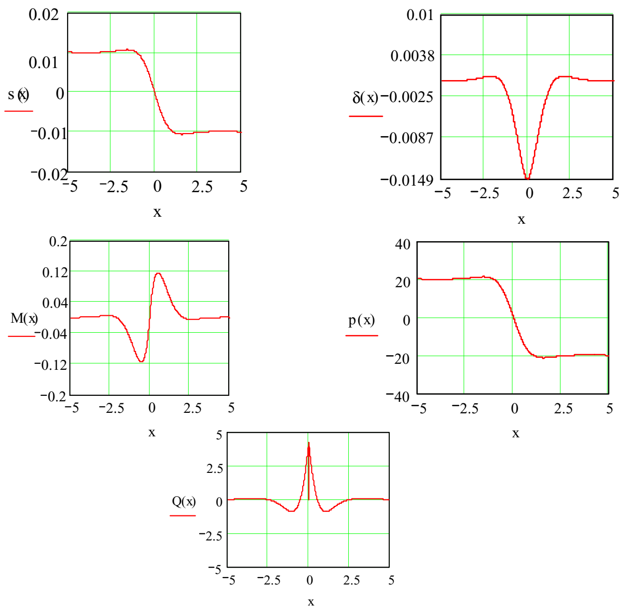

Figure 2.7: Displacements s(x), inclinations δ(x), bending moments M(x), shear forces Q(x), soil reactions p(x) (tons and meters) (Barvashov *et al*., 2009).

The potential failure surface is assumed to cross the nails at a given angle which depends on position z. The paper discusses the conditions leading to nail working conditions (tension, shear, bending moment) and proposes a design procedure. The procedure is based on a theoretical solution of a nail being subjected to a jump of displacements at the position of the nail-slip line cross section. The beam (nail) equation is solved for a Winkler type foundation and solutions are illustrated in Figure 2.7.

A similar problem is addressed by Askari *et al*. (2009). (Fig. 2.8). The problem is solved in three dimensions assuming a given shape for the failure mechanism. The unstable body is discretized into rigid horizontal blocks, each one of them reinforced by a row of nails (Fig. 2.8). Then, the upper bound theorem of plasticity is applied assuming that nails resist only in tension. The method seems to provide similar results to other procedures developed for 2D conditions. The novelty of the procedure is to introduce 3D effects. Figure 2.9 provides the required average reinforcement in a 3D situation compared with the more conventional 2D analysis. Coefficient k_t is defined as

$$k_t = \frac{nT_t}{H} \tag{2.5}$$

where n is the number of reinforcement layers, T_t, the tensile strength of the nail and H the height of the slope. The effects of changing the assumed failure mechanism or indeed, other failure modes of the reinforcement (bending, shear or a combination of them) are not discussed in the paper.

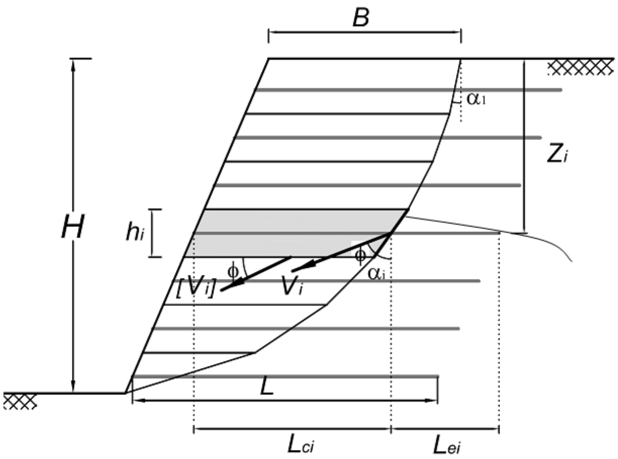

Figure 2.8: Transitional multi-block mechanism (Askari *et al.*, 2009).

In the fourth paper Farias and Durand (2009) describe a procedure to include reinforcement bars into any continuum finite element model. Each bar is discretized by means of nodes which are the cross points of the assumed reinforcement and the existing boundaries of the discretized continuum mesh. At these intersections, connecting springs provide a procedure to define the bar-soil stiffness, which may be nonlinear or based on an elastoplastic model. Fixed (grouted) or free sliding zones may be accommodated through spring constants.

Figure 2.10 illustrates a solved case: a vertical excavation supported by several rows of nails. Figure 2.10b shows the calculated axial forces in reinforcement at the end of excavation. The method is interesting because it may be applied in connection with any existing FE code although algorithms for element - bar intersection are required. It would be interesting to extend the spring stiffness to conditions of strain softening a very common case in applications.

2.3 *Other topics*

Ohtsuka and Isobe (2009) perform stress-controlled ring shear tests to investigate a stress path typically associated with slope failure under the increment of pore pressures: a reduction of effective normal stress at constant shear stress. The reported tests were performed on high plasticity clay. A set of results for a specimen remoulded and reconsolidated in the shear cell are shown in Figure 2.11. The authors found that the actual failure

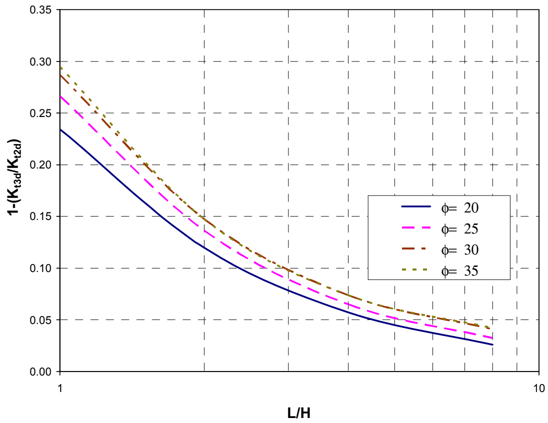

Figure 2.9: Required reinforcement strength (β = 80) (Askari *et al.*, 2009).

conditions stay on the left of the peak strength failure line independently determined through a conventional procedure. Shear displacements seem to increase gradually from an initial yield state (on the peak strength envelope) to a failure state. The case of re-sliding was also investigated by means of similar tests on a pre-sheared specimen. The response in this case was "brittle" in the sense of a sudden increase in shear displacements when the stress path reached the residual strength line. Field "experience" however, often indicates that reactivated slides present a continuous creep-like motion extending over long time periods. The delayed response of slopes is certainly a complex subject.

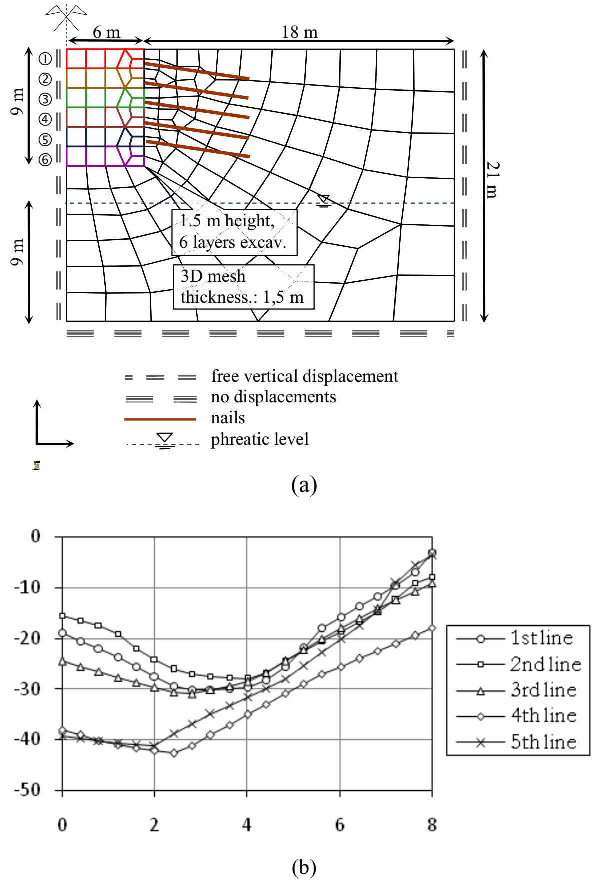

Figure 2.10: (a) Vertical excavation with soil nailing. (b) Axial forces in the reinforcements at the end of excavation (Farias and Durand, 2009).

Debris flows are a class of landslides which require attention to the travelled distance of the moving mass because of the involved risk. The paper by Zhou *et al*. (2009) addresses this issue by means of a number of physical flume tests performed on two different uniform sands (mean grain size 0.1 mm and 0.5 mm respectively) and a well graded natural soil (completely decomposed granite, CDG). They found that the travel angle (the lower the travel angle the longer the distance travelled by the moving mass) was controlled by the initial mass of soil tested, the sand fraction (in mixtures) and the water content. The two first factors were well identified: The travel angle decreases with the initial total mass and increases with the increasing proportion of coarser particles. However, the effect of initial water content (Fig. 2.12) is more difficult to explain. The decrease in travel angle for water contents in excess of 20% – 25% may be explained by the development of positive pore water pressures.

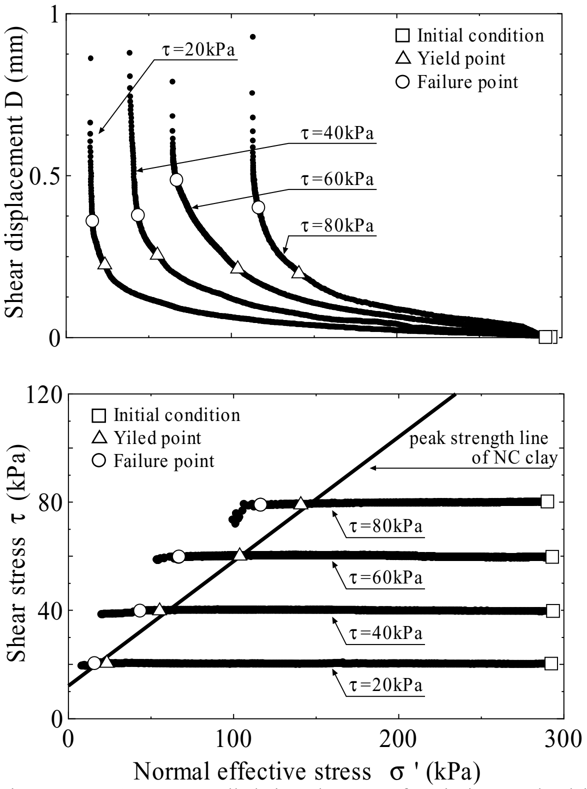

Figure 2.11: Stress controlled ring shear test for virgin type landslide (Ohtsuka and Isobe, 2009).

However, a continuous increase in travel distance with water content was recorded for most of the range of water contents tested. This is not easily explained if one accepts that suction and suction-induced stiffness and strength should increase the lower the water content. However, the contribution of suction to "effective stress" is an open issue not yet well solved. The effective stress increment induced by suction may be written:

$$\Delta\sigma' = \chi(S_r)s \qquad (2.6a)$$

$$\Delta\sigma' = S_r\, s \qquad (2.6b)$$

$$\Delta\sigma' = S_r^{\alpha}\, s \qquad (2.6c)$$

The first equation corresponds to the Bishop (1959) proposal. The second one to a simplified version, $\chi = S_r$, and the third one to a recent proposal which takes into account the soil microstructure (Alonso *et al*., 2009). However, the paper does not provide data on the water retention curve which would be necessary to use Equations (2.6).

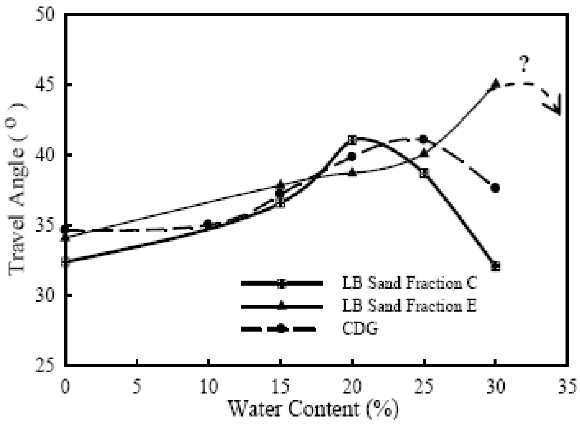

Figure 2.12: Effect of water content on flow mobility (Zhou *et al*., 2009)

A novel procedure to analyze stability and failure of slides is presented by Bui *et al*. (2009). The SPH is a Lagrangian mesh free particle method which has advantages when dealing with failure and large deformations. The method is outlined in the paper. It is applied to analyze the stability of a homogeneous embankment. The safety factor was calculated by means of a (c', φ') reduction technique as in conventional FE methods. Both methods lead to similar results. However, the SPH method allows a substantial distortion of the slope.

The final paper in this Section (Barends, 2009) presents a wider picture of the current state of knowledge of slope and embankment stability analysis. Barends highlights the fundamental role of the human factor (experience, intuition) in engineering decisions and the illusive precision of sophisticated models. After discussing a number of interesting case histories, the paper suggests that predictions should be made considering alternative interpretations of a given set of data or by the joint contribution of several experts.

3 DESIGN AND STABILIZATION PROCEDURES

The sixteen papers assigned to this Chapter are further subdivided into the following topics:

- Piled embankments
- Reinforced slopes
- Other stabilization methods
- Waste landfills
- Other topics

3.1 *Piled embankments*

Embankments on soft soils are sometimes piled in order to reduce settlements and consolidation times. In occasions, a reinforcement of the soil in order to improve safety against overall failure is also sought. The transfer of embankment loads to piles involves arching effects and horizontal loads on pile heads, especially on the slopes of the embankment. The load transfer is facilitated by reinforcing the base of the embankment. Geosynthetic meshes or granular compacted layers are used. Several examples are discussed in the papers summarized below.

Van Eekelen *et al*. (2009) discuss the arching effects of a piled embankment reinforced with a geosynthetic grid. The problem is illustrated in Figure 3.1. The embankment load is transferred in part to piles through arching (A) but some part is also acting on the grid which, in turn, loads the pile caps. Finally, some proportion of the weight loads directly the soft soil (C). The paper discusses the British and German design standards (BS 8006; EBGEO), and uses also a FE calculation to check the capabilities of the standards.

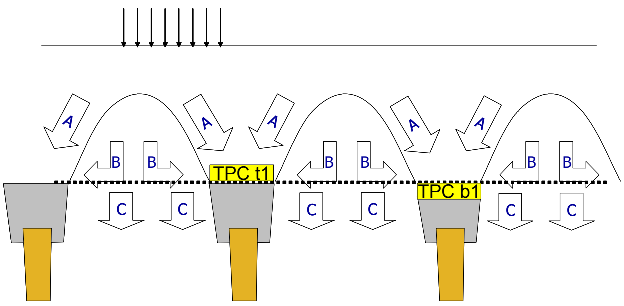

Figure 3.1: Load distribution into load parts A, B and C. Locations of Total Load Pressure Cells (TPC's) in the Kyoto Road field test (Van Eekelen *et al*., 2009).

They found that EBGEO is closer to numerical analysis. In addition, "in situ" measurements in a full scale test are well reproduced by EBGEO. The authors point out some inconsistencies in BS 8006.

Slaats and van der Stoel (2009) examine the basic phenomenon controlling the behaviour of piled foundations, namely the arching effect, the shear interaction between the geogrid and the soil, and the lateral spreading effect, directly related to the behaviour of piles under horizontal loading. The main purpose of the work reported is to check the capabilities of 2D and 3D FE analysis (through Plaxis and Mohr-Coulomb or "soft soil" constitutive models). They have chosen a few reported tests (including centrifuge tests) for the validation of the models developed. In general, they report a higher accuracy of 3D simulations and indicate also some improvements to be made in the numerical code: updating mesh geometry to take into account secondary effects and including anisotropy. In general improvements may also derive from more accurate constitutive models. A difficulty to evaluate the accuracy of the numerical analysis in practice is the lack of information on the methodology followed to select soil parameters.

Figure 3.2: Large-scale Direct Shear test on nails in clay (Lengkeek and Bruijn, 2009).

FE analysis is increasingly used as a design tool and this is exemplified in the work reported by Lengkeek and Bruijn (2009). The paper describes the ongoing project to increase the safety of existing dykes in the Netherlands by means of a soil nailing technique. This is an unusual application of nails since foundation soils are soft materials, unsuited to resist lateral forces applied by nails and exhibiting significant creep effects. However, the authors reckon that suitable design modifications (longer nails, proper orientation) could overcome those inconveniences. The analysis presented combines 2D and 3D FE as well as specialized software. In addition, they report large scale direct shear tests on clay reinforced by nails (Fig. 3.2).

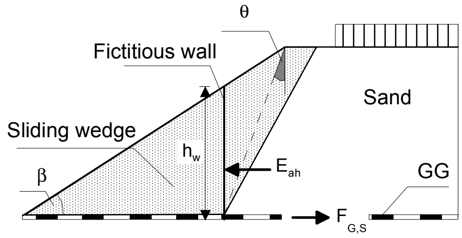

Figure 3.3: Sliding soil wedge to determine the spreading force (Fahmy *et al*., 2009)

The authors conclude that the reinforcement of soils under embankment loading is a feasible and cost effective solution.

The design of the horizontal geosynthetic layer at the base of a piled embankment requires the determination of the maximum horizontal tensile force ($F_{G,S}$ see Fig. 3.3).

Fahmy *et al*. (2009) propose an analytical method which modifies the procedure described in the German standard EBGEO (2007). The modification incorporates the results of FE analysis which were shown to be accurate to reproduce a large-scale model test. The modification proposed is sketched in Figure 3.3. The force $F_{G,S}$ is proposed to be equal to the active force E_{ah} acting on a virtual wall within the embankment defined by angle θ. The paper provides guidelines for the determination of θ in practical situations.

3.2 *Reinforced slopes*

Reinforced earth is a well established procedure to materialize steep slopes in compacted embankments. Design rules are described in several codes, standards and papers quoted in Rodriguez and Freitag (2009). Finite element procedures allow also an examination of construction details which range from the quality of the fill to the characteristics (stiffness, strength, long term behavior) of the reinforcement and the foundation soils. Rodriguez and Freitag (2009) report a 2D FE sensitivity analysis of some of these aspects. They use the (c',ϕ') reduction method to calculate safety factors. Among other findings they conclude that a redistribution of the reinforcing, reducing it at high levels and increasing it in the proximity of the foundation (creating a sort of "footing") may prove convenient in case of soft foundation soils and low quality fill.

Figure 3.4: A general view of the supported shaft walls on completion of the excavation (Taheri *et al*., 2009).

Taheri *et al*. (2009) describe the design and construction of a deep cut (16-18 m) in highly fissured shales and sandstones. Steel piles were first installed in drilled holes and they were later anchored against the rock as excavation proceeded. The resulting wall was later protected by sprayed concrete (Fig. 3.4). The authors use limit equilibrium as well as a Distinct Element code to estimate safety and the expected deformations associated with the construction process.

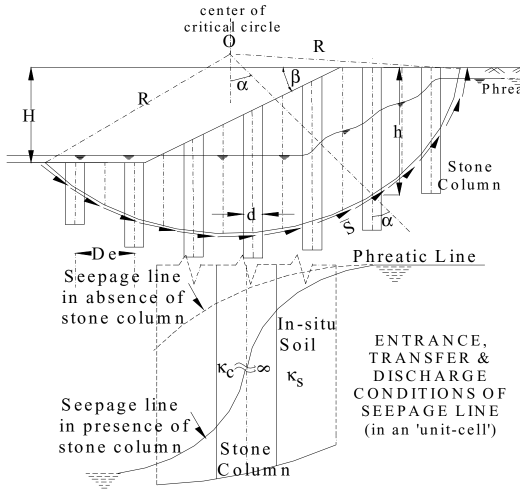

Figure 3.5: The Problem definition in a 2D Plane (Saha, 2009).

Granular columns are proposed by Saha (2009) to improve the stability of slopes. The added overall permeability is also a benefit in cases of rapid drawdown conditions. The problem is sketched in Figure 3.5. The author describes a limit equilibrium method in which columns are “embedded” into the classical method of slices. The search for the minimum safety factor is then performed with the help of genetic algorithms. The paper describes finally an application to stabilize slopes of the lower reach of Ganges river.

3.3 *Other stabilization methods*

Whichman *et al*. (2009) present a comparative study of alternative reinforcing methods for an existing dyke in the Netherlands. Enviromental conditions are quite specific but the paper illustrates a methodology to perform the analysis. The study is, however, quite preliminary since soil conditions were only approximately known.

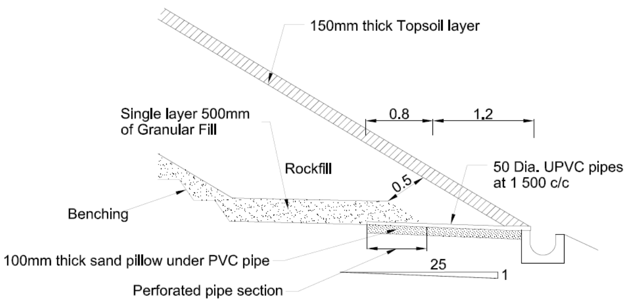

Figure 3.6: Typical design of filter layer and contour drain channel. (Mickovski and Smith, 2009)

When available, rockfill provides a very convenient solution to build embankments or to stabilize slides. Rockfill is a stable, free draining material which allows steep slope angles. It should be compacted and fully wetted when installed to reduce settlements associated with particle breakage. Mickovski and Smith (2009) describe the case of a replacement of an unstable fill slope by a rockfill. The paper describes the design and construction in some detail. Drainage design is reproduced in Figure 3.6. Rockfill was covered by a top soil to allow vegetation growth. The entire area was well drained to avoid water overpressures.

The protection of riverbanks is a specialized topic which requires attention to details and a proper consideration of drainage and soil migration problems. It should consider also wave action and local practices and construction capabilities. All of these aspects are described in the Nile river bank protection described by Rizkallah *et al*. (2009). The design is summarized in Figure 3.7, which provides the cross section of the solution and the basic properties (friction, grain size distribution) of the bank and filter materials.

The paper discusses also the minimum safety factor (1.5) adopted in calculations. Two and a half years after completion of the works the protection is reported to be in a very good condition.

Shallow slides develop in slopes of high plasticity clays due to crack formation and subsequent rainfall action. One example taken from the paper by Puppala *et al*. (2009) concerns the slopes of a long rolled earthfill dam built with high plasticity clay (Fig. 3.8). Crack formation is a risk in arid or semi-arid climates because of the intensity of evapo-transpiration and surface desiccation. The paper describes the performance of four alternative stabilization procedures. The original soil is mixed with:

a) compost (wood fibres) in a proportion of 20%
b) 4% lime addition and 0.3% polypropilene fibres
c) 8% lime and 0.15% fibres
d) 8% lime

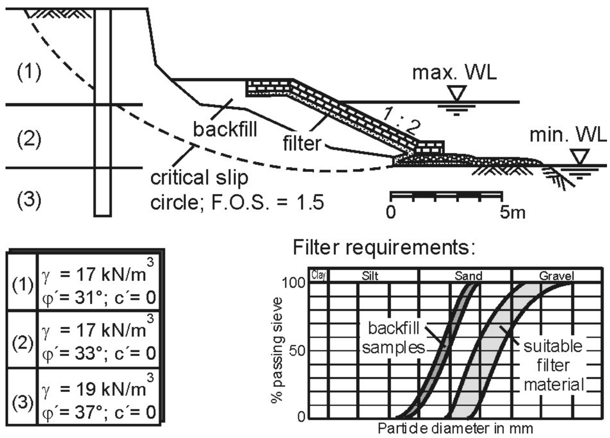

Figure 3.7: Exemplary cross section of the bank protection, designed in accordance with the guidelines. (Rizkallah *et al*, 2009).

The paper describe the behaviour of test sections built on the slopes of the Joe Pool dam and conclude that the 8% addition of line plus fibres results in the best performance. It seems that a 4% lime addition has a limited capability to improve a high plasticity clay (PI= 37%).


Figure 3.8: A typical surficial failure at Joe Pool Dam (Puppala *et al*., 2009).

3.4 *Waste landfills*

Conventional clay liners or geosynthetic clay liners (GCL) are probably the weakest layers when examining the stability conditions of waste landfills protected by multilayer cover systems. GCL uses bentonite, a material exhibiting residual friction angles, when saturated, in the vicinity of 10º. The procedure to avoid sliding in this low friction material is to reinforce GCL's by means of needdle punched or stitch bonded techniques. This is discussed in Datta (2009), who provides an analysis of safety factors of some cover designs. He also points out the need of avoiding bentonite extrusion, for the same reasons. Chen *et al*. (2009) provide a thorough description of the Chinese experience on municipal solid waste landfills. The paper refers to stability issues. Some interesting information given in the paper concerns the evolution of cohesion and friction parameters with time (Fig. 3.9), the shear strength of GCL – geomembrane interfaces, the behaviour of GCL when sheared and the water levels recorded "in situ". Regarding Figure 3.9, note that strength parameters are conventionally defined for a given shear strain (typically 10%). The evolution of (c,ϕ) parameters are conventionally defined for a given shear strain (typically 10%). The change of (c,ϕ) parameters in Figure 3.9 implies an evolution of safety factor with time although this analysis is not reported. When interpreting strength data on MSW, attention should be paid to the composition of MSW and, in particular, to the content of organic matter.

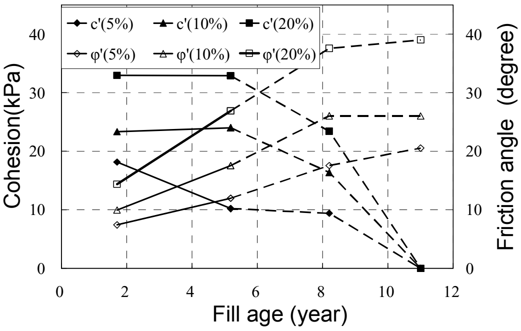

Figure 3.9: Relationships of shear strength parameters to the fill age of MSW (Chen *et al*., 2009).

3.5 *Other topics*

Nossan *et al*. (2009) report the case of bridge piers crossing an active landslide. Among different possibilities, the solution was to resist the sliding thrust by means of diaphragm walls aligned with the direction of the slide motion and founded on stable marls (Fig. 3.10). The design reported in the paper follows a logical sequence of steps. First a backanalysis of the existing slide provide an estimation of strength parameters at the slide failure surface. This is made through a (c',ϕ') reduction method built into a 3D FE model. Then the structural (pier) elements are introduced into the model and a further (c',ϕ') reduction method provides the new safety factor. The FE analysis provides also the forces on the structural elements and helps to design them. The value of the case could be improved if the back-analyzed strength parameters could be compared with field or laboratory data. Also, there exist a number of analytical solutions for the problem of a soil "flowing" among obstacles which could be compared with the numerical results.

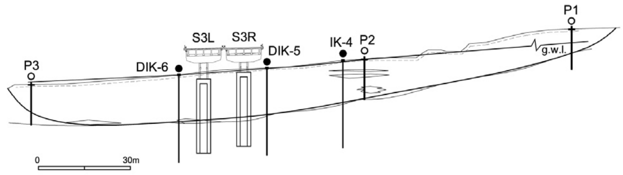

Figure 3.10: Cross-section down the middle of the sliding mass with pier S3. (Nossan *et al*, 2009)

The design of bridge abutments requires a progressive transition from the usually stiff structure to the soft embankment. This is a common situation in road bridges but also in railways. Figure 3.11 shows an embankment design for the transition towards bridges used in the high speed railway lines currently being built in Spain. In this case the transition is solved by wedges of treated or high quality compacted soil which are part of the embankment. Another solution is to place a structural slab on top of the compacted embankment. This is the arrangement analyzed by Ravnikar Turk *et al*. (2009). The paper refers to the situation shown in Figure 3.12. The embankment settlement is essentially induced by the soft foundation soils. However, this is not the sole origin of differential settlements. In many occasions embankments deformations are often relevant and cases of partial embankment collapse under rainfall-induced wetting are often encountered. The paper describes a sensitivity analysis through a FE 2D modelling of the longitudinal section of the embankments. A pattern of vertical displacements is imposed on the lower boundary of the embankment. Under these conditions the pavement settlements are independent on the embankment height. In fact, only one parameter is identified as a relevant one: the rate of settlement of the free end of the slab. The paper concludes with a guide to select the appropriate slab in terms of the estimated settlement and consolidation time.

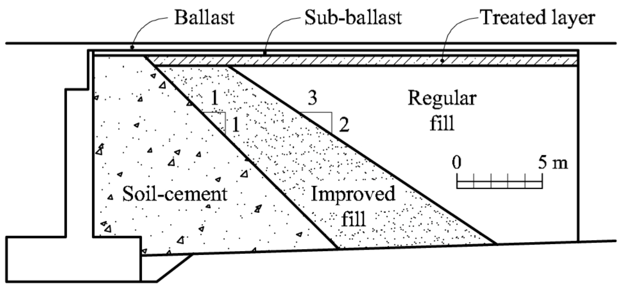

Figure 3.11: Example of transitional wedge to a railway bridge abutment.

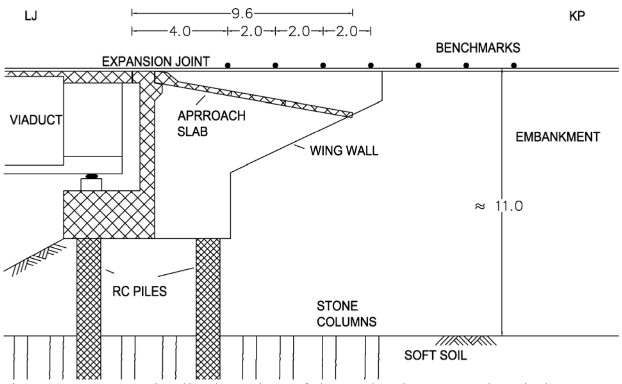

Figure 3.12: Longitudinal section of the embankment at the viaduct.

Ilievski *et al*. (2009) describe the works to divert a river for the purpose of exploiting the coal layers under the riverbed. A key structure was the construction of a 40 m high embankment. It was stabilized by means of geogrid layers. The analysis was performed by means of a 2D FE code.

4 RAINFALL AND DRAINAGE

Slope failures are often associated with extreme rainfall events. The soil in slopes is often unsaturated, even in temperate climates, because of the natural gravity drainage in slopes. The infiltration process from the surface boundary is therefore best analyzed as flow in an unsaturated soil. Zhou *et al*. (2009) consider the case of loose fill slopes and report the modelling performed through a general purpose FE program. They define the mechanical constitutive equation in terms of a Bishop-type effective stress (χ parameter being substituted by the degree of saturation). This approach is, however, not well suited to deal with loose unsaturated soils, because it cannot reproduce collapse effects induced by suction reduction. The authors present some results on the results of a field test subjected to surface as well as subsurface infiltration by means of injection pipes (Fig. 4.1).

The effectiveness of double infiltration is well demonstrated in Figure 4.1. This is a common situation because heavy rainfalls lead to water table rises, which may facilitate capillary induced flow from the base of fills.

Kang *et al*. (2009) focus on the prediction of the time necessary to saturate a simple slope characterized by its height, permeability and water retention curve. They perform a number of numerical analysis and summarize the results in a predictive formula. Unfortunately, the formula, based on regression analysis, is not dimensionally correct and this casts doubts on its general applicability.

Lee *et al* (2009) present also the effect of infiltration on an unsaturated soil slope. Failure conditions are very simplified: 1D infiltration from a horizontal surface and an infinite slope in order to calculate the safety factor. Figure 4.2 illustrates the type of results obtained. Note first the general decrease in safety factor with depth, which is a consequence of the model assumptions: an infinite slope in a soil having a constant cohesion parameter. The effect of surface infiltration results in a jump of SF at some particular depth which is identified as the depth of the infiltration front. Below this point suction maintains its original value. Above it the soil is close to saturation conditions and shear strength is reduced.

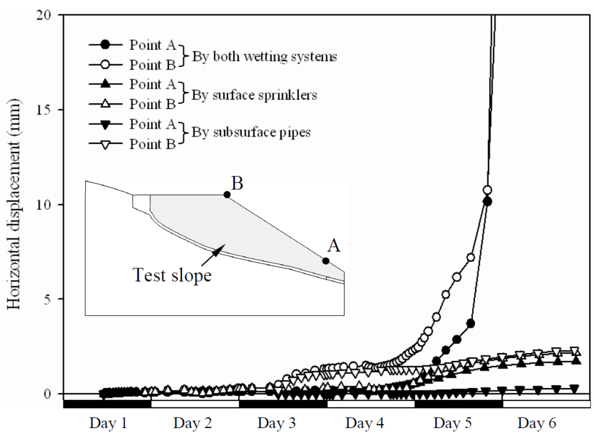

Figure 4.1: Comparison of wetting induced horizontal displacements by different infiltration schemes (Zhou *et al*., 2009)

This paper, as well as the preceding one raises the important issue of the relevance of hydraulic parameters and hydrologic regime on the risk of slope failure associated with rain infiltration.

In a case study reported by Alonso *et al*. (2003) this aspect was discussed in some detail. The slope geology was characterized by three layers (α, β, γ), a result of the natural weathering of the underlying clay substratum. Slope creep motions led also to a reworked upper layer. These situations are common in nature and result in heterogeneous profiles in terms of hydraulic as well as mechanical properties. For a given rainfall sequence, the distribution of permeability in the slope determines the expected pore water pressure response at every point.

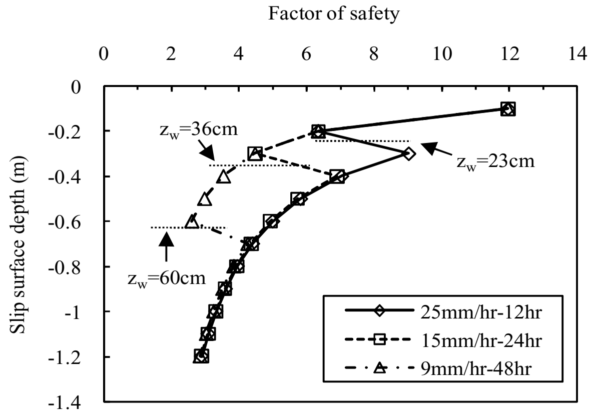

Figure 4.2: Variation of factor of safety along slip surface. (Lee *et al*, 2009)

To illustrate this comment, Figure 4.3 provides the pore pressure rise at a point within the slope, for different combinations of layer permeability. The plot shows that a particular combination of layer permeability leads to the "strongest" response of the slope in term of the pore pressure generated.

The combination $K_\alpha = 10^{-7}$ m/s and $K_\beta = 10^{-8}$ m/s leads to the maximum evaluation of pressure (positive) at the point considered. More or less pervious α and β layers result in lower elevations. It may be concluded that a particular sequence of permeability is a critical one for a given rainfall record. There are no easy rules to predict such a critical profile of permeability because the computed water pressure is the result of several phenomena: infiltration flow, flow transport parallel to the slope and (changing) storage capacity of the soil. In fact, the "critical" situation identified for a particular heterogeneity is probably not an absolute concept since it may change with the initial conditions. But this result stresses the relevance of soil permeability and its distribution within the slope to generate critical stability conditions for a given slope geometry, material properties and a given rainfall record.

A further analysis along these lines of thought is presented by Scotto di Santolo and Evangelista (2009). They perform a hydrologic study of a two layer profile, initially unsaturated, subjected to a given infiltration rate at the surface. They refer to the pyroclastic soils covering large areas of the Campania region in the south of Italy. They try to identify a critical rainfall leading to instability although stability conditions are not described in the paper. A critical rainfall is defined when a change in the infiltration regime is detected in calculations. It is difficult to assess the validity of the approach and the capability of the model developed to represent field conditions. The authors find some agreement between their findings and the empirical relationship between rainfall intensity and rainfall duration triggering instabilities (Fig. 4.4).

However, the paper concludes with a pessimistic evaluation of the capabilities of hydrologic thresholds to establish risk of

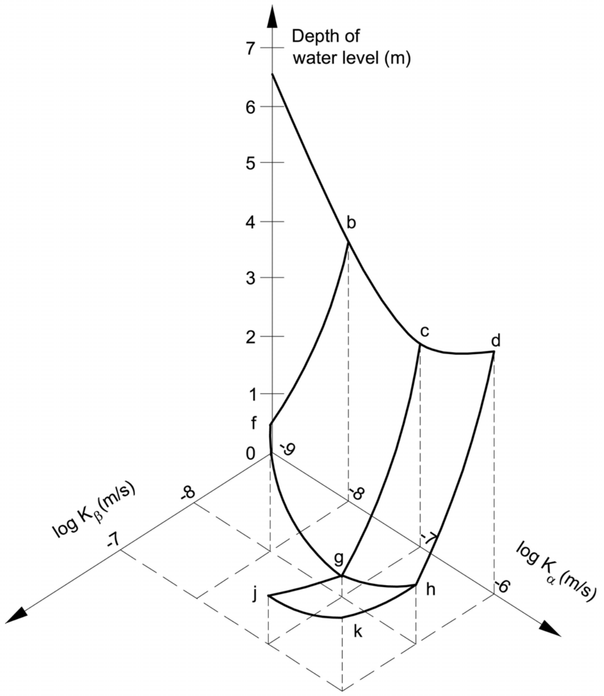

Figure 4.3: Computed depth of water level at piezometer P5699 for the time t = 350 days after the beginning of the prediction exercise (October 1st, 1992). Depth of sensor: 2.80 m (Alonso *et al*., 2003).

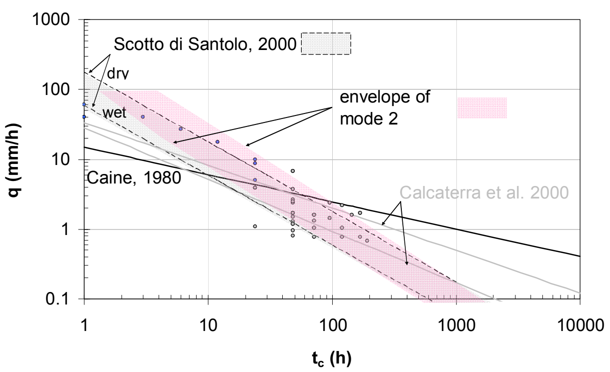

Figure 4.4: Empirical threshold and critical rainfall for flows in the Campania region, Italy (Scotto di Santolo and Evangelista, 2009).

instability. They favour direct "in situ" measurements of variables such as suction or water content.

Bardanis *et al*. (2009) analyze the case of some paleo-landslides which cover pervious levels, possibly connected to a given (low) water level. "In situ" pore water pressure profiles show first an expected increase in pressure which eventually decreases due to the effects of under-drainage. They describe a real case and conclude that, in order to be successful, the analysis should include:

- a decrease in permeability with depth of the upper clay formation (this is an effect of the increasing affective stress with depth which results in a progressive reduction of porosity)
- a further reduction in permeability at the level of the sliding surface
- an appropriate infiltration rate at the soil surface. This is a difficult estimation and the authors admit that this is a matter of backanalyzing pore water pressures in the slope

Figure 4.5 shows a comparison of measured and calculated pwp profile for the Prinotopa landslide. A convenient procedure to stabilize these landslides is to drain them vertically towards the lower pervious level.

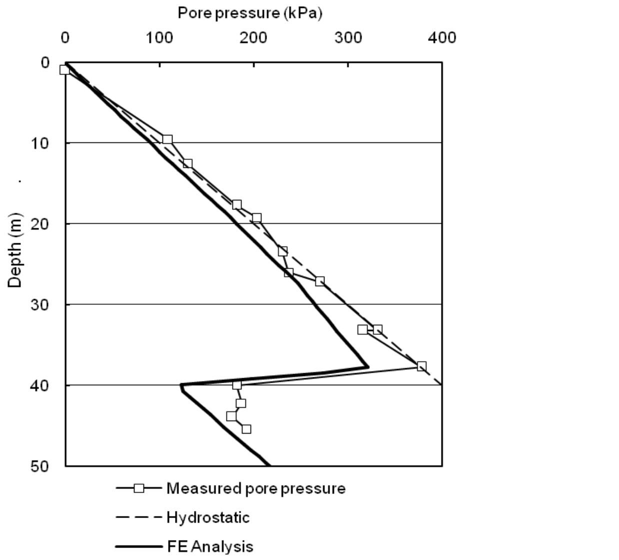

Figure 4.5: Measured and predicted by FEM initial pore-pressure profile (Bardanis *et al*., 2009).

Springman *et al*. (2009) describe the conditions and set-up of a small scale landslide triggering experiment in the Swiss banks of the Rhine river. The results of the experiment are not given in the paper, which describes the soil conditions and field and laboratory experiments conducted to identify the upper soil layers. The idea of the experiment is to simulate heavy rainfall conditions by means of a sprinkling system. The involved soils are unsaturated low plasticity silty sands of high porosity. The paper provides a summary of the tests performed.

Nonveiller (1981) published an interesting paper which provided a solution for the pore pressure distribution in slopes stabilized with horizontally drilled drains. He produced charts ready to use. Gjetvaj *et al*. (2009) follow the work of Nonveiller but this time using available numerical techniques for flow analysis. They perform a 2D flow analysis and import the results into a slope stability program. Then, safety factors could be calculated as a function of the time factor ($T=c_v t/H^2$, where H is the total head difference in the slope) for different spacing and lengths of drains (S,L). Figure 4.6 shows some results. The gain in Safety Factor is defined as

$$F_g = \frac{F_d - F_t}{F_d - F_o} \tag{4.1}$$

where F_o is the safety factor of the slope without drain, F_t is the SF at time t and F_d the SF at the final steady state. Note, however, that the selected length of drains in Figure 4.6 (50 m) is probably unattainable in most practical cases.

Montgomery and Karstunen (2009) report their experience in modelling embankments with vertical drains. They compare several geometrical calculation approaches: an axisymmetric cell centred on a given drain, a plane strain analysis and a 3D "slice" approximation of the real embankment. Their conclusions are in part a consequence of the current development of the 3D FE software they used. They also found that the most relevant property to contribute to a good match of vertical settlements history is the permeability of the smeared zone around drains. But the most significant result was the limited capability of the models used to reproduce the measured horizontal deformations (Fig. 4.7). This is probably linked to

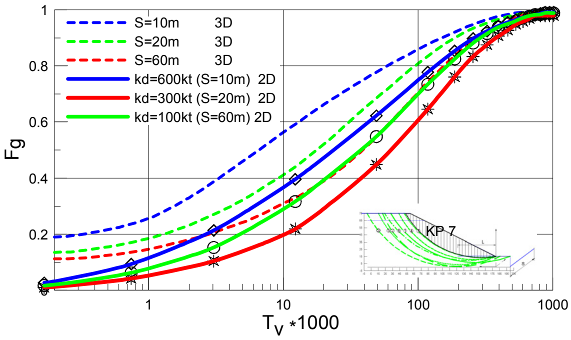

Figure 4.6: The gain in the factor of safety Fg versus the time factor Tv for slip surface 7 (critical slip surface) L=50m, S=10, 20, 60m. (Gjetvaj *et al*, 2009)

constitutive model used as well as to the representativeness of material parameters derived from laboratory testing. The case presented corresponds to an instrumental test embankment. For further details on parameter estimation the authors refer to other publications.

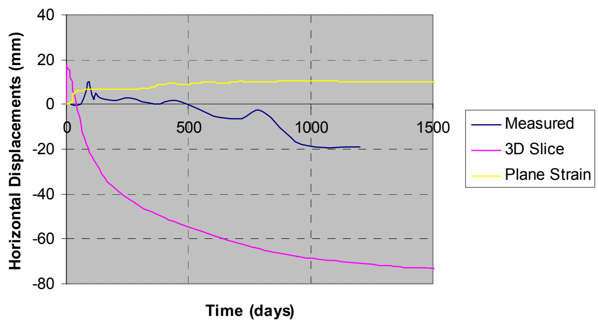

Figure 4.7: Comparison of horizontal deformations (Montgomery and Karstunen, 2009).

5 SEISMIC ANALYSIS AND DESIGN

Six papers dealing with different aspects of seismic analysis of dams and embankments were included in the Session. A seventh paper deals with the measurement of temperature in an embankment in a seismic area and it has been also included here.

Matsumaru *et al*. (2009) report the results of slaking table tests on a sandy slope of very low density, partially submerged. In fact, water was flowing from an upstream level to the slope toe. They were able to measure the water content in several points of the cross section and to determine the variation of degree of saturation throughout the slope. This information and the results of triaxial tests on unsaturated specimens (not reported in the paper) made it possible to assign cohesion intercepts (0, 1, 2 kPa) and friction angles to the soil profile. The slope was subjected to sinusoidal and irregular acceleration records and measurements within the soil include displacements, acceleration and excess pore water pressures. Displacements were interpreted through the Newmark method, assuming a circular failure surface (which could be identified at the end of shaking by calculating and plotting shear strains on the basis of coordinates of reference points). The comparison between measured and computed residual displacements, given in Figure 5.1, is quite satisfactory, perhaps somewhat surprising in view of the simplicity of the Newmark method and, specially, of the fact that the soil was very loose (e= 1.5) and straining associated with other mechanisms (liquefaction) could have developed. The representativeness of these 1g small-scale models remains always a problem.

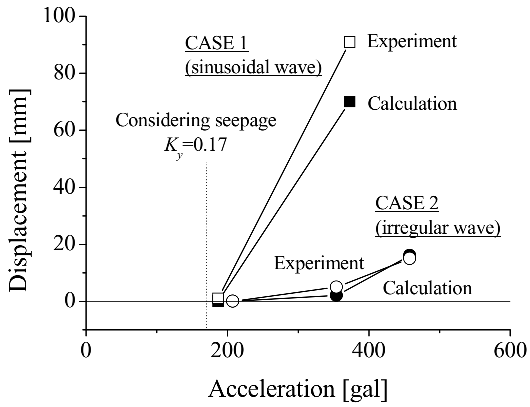

Figure 5.1: Residual displacement of calculation and experiment (Matsumaru *et al*., 2009).

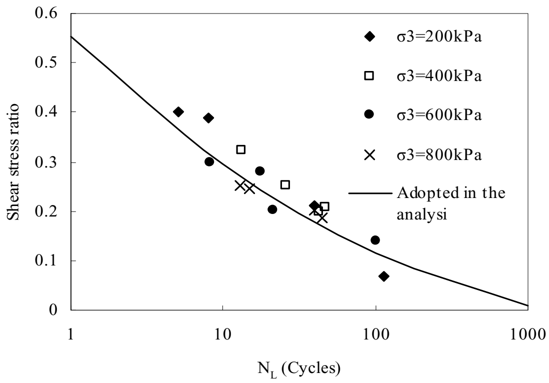

Figure 5.2: Number of cycles to cause liquefaction of filter layer. (Han-long and Kank, 2009)

Han-long and Kank (2009) report the 3D FE viscoelastic calculations performed on a very large dam (293 m high) proposed to be built in the Yaling River, China. The dam is conceived as a zoned earth and rockfill dam with a central clay core. The paper describes the material viscoelastic parameters used, although test results are not given. One of the concerns of the analysis was the risk of liquefaction of the granular filters. They report the results of triaxial cyclic tests (Fig. 5.2). The number of cycles to reach liquefaction, N_L, depends on the applied cyclic stress ratio, CSR = $\sigma_d/2\sigma'_{30}$, where σ_d is the maximum cyclic axial stress and σ'_{30} the initial effective confining pressure. It was found that the confining stress had no effect on the measured relationship CSR vs N_L. They calculate liquefaction conditions through an indirect and simple procedure since they suggest that the pore pressure ratio in the filters ($r_u = u/\sigma'_3$) is related to N/ N_L, N being the number of cycles applied. They conclude that the filters are safe during the design earthquake. Permanent dam deformations during an earthquake are also estimated by imposing empirically proposed residual volumetric and shear strains in the calculation model.

The horizontal slice method (HSM) (Shahgholi *et al*., 2009) is an interesting development which allows the simplified dynamic analysis of slopes and embankments. The paper by Choudhury and Nimbalkar (2009) compares the results of a pseudo-static and a pseudo-dynamic procedure to calculate the safety factor of slopes subjected to vertical and horizontal dynamic accelerations. In the pseudo-dynamic analysis, in particular, the soil accelerations are defined by sinusoidal time functions. The safety factor is defined as in limit equilibrium methods and the solution for a circular sliding surface requires the joint equilibrium of forces on each horizontal slice and the moment equilibrium for the entire discretized slide. Figure 5.3

shows a comparison of the two methods for a particular case. The pseudo-dynamic method may take into account the time length of the motion and the phase differences in waves propagating through the slope.

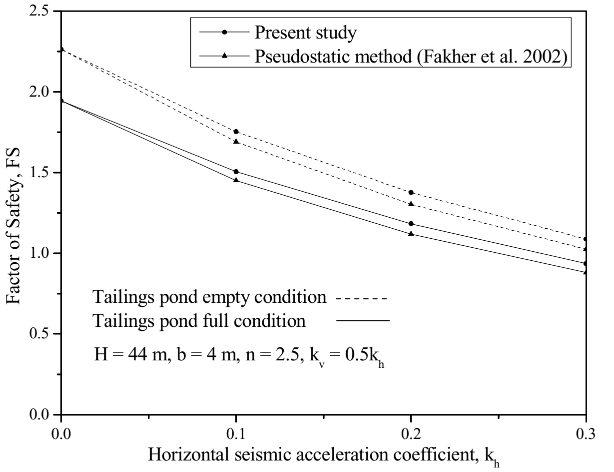

Figure 5.3: Comparison of factor of safety (FS) obtained by pseudo-dynamic results with those by pseudo-static results with k_v = $0.5k_h$ (Choudhury and Nimbalkar, 2009).

Vrettos (2009) provides a step-by-step guide on how to conduct a seismic response analysis of an embankment when limited information is available. His approach is summarized as follows. The first step is to characterize the seismic input for the particular structure being analyzed. Current codes of practice (Eurocode EC8, for instance) provide a target response spectra which includes information on earthquake intensity, soil type and type (risk associated) of structure. Then, an accelerogram is synthetically generated through a simulation code which takes into account local soil conditions. Dynamic soil properties are estimated through some correlations. Then a dynamic FE analysis, which includes the embankment and the foundation, is run. Since the input accelerogram is specified at the base of the model, a procedure to relate surface accelerograms to base (rock) input is derived. The FE model allows the calculation of acceleration histories, which are converted into effective accelerations for use in quasi static stability calculations. The author remarks that the method is applicable to low-rise dams and indicates that more sophisticated methods are required for larger dams.

Erlingsson and Hauksson (2009) discuss the economic advantages of earthdams with a geomembrane covering the upstream face (Fig. 5.4). The paper examines the effect of earthquake effects on such a design. The calculation stages described in the paper are in some respects similar to the steps defined in Vrettos (2009)

However the main purpose of the analysis now is to investigate the possibility of membrane rupturing. Their target is therefore to estimate the displacements of points on both sides of the membrane during the design earthquake (Fig. 5.5). The relative plastic displacement depend critically on the contact friction angles.

Sivakumar and Srivastava (2009) investigate the effect of a horizontal liquefiable layer in the natural soil on the response of a dam subjected to a strong earthquake. The case is based on the effects of the Bhuj earthquake in India, in 2001, which reached a 7.6 magnitude and caused extensive damage. The authors had to approximate soil properties because actual field data was scarce. The liquefiable soil layer was characterized by a pore pressure generation model (Finn and Byrne) included

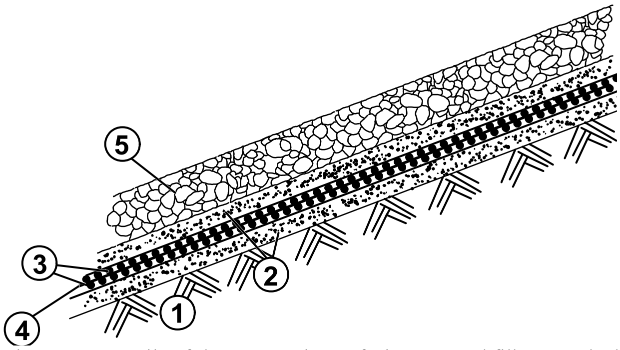

Figure 5.4: Details of the geomembrane facing. 1, Earthfill; 2, crushed gravel; 3, non-woven geotextile; 4, geomembrane; 5, rockfill (after Girard *et al.*, 1990).

into the commercial software used for the analysis. They found that pore pressure generation was significant under the assumed earthquake and a "deep" failure mode through the foundation was calculated (Fig. 5.6). The calculated motions of the dam crest, a downstream displacement of 7.2 m and a similar settlement, was consistent with field observations of the Chang dam after the earthquake.

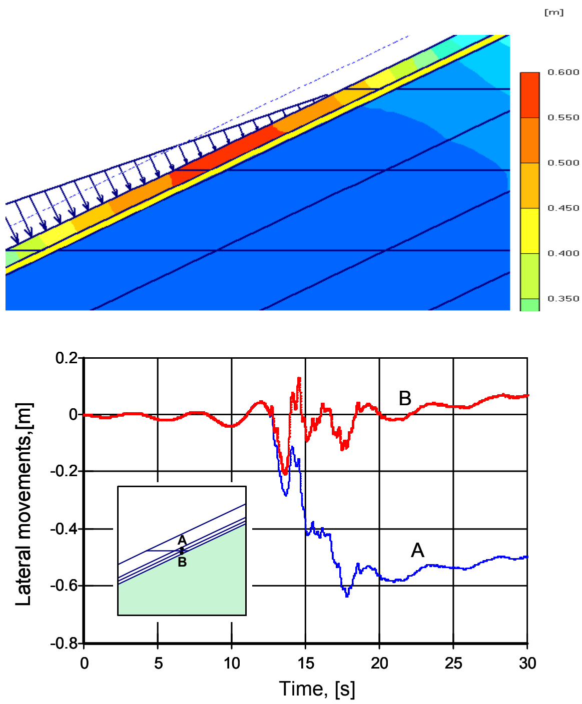

Figure 5.5: (a) Close up of the largest deformation from the time history at Hella. (b) History of the lateral movement of two points A and B on each side of the geomembrane during the earthquake loading. (Erlingsson and Hauksson, 2009)

Moraci *et al*. (2009) present yearly records of temperature measured on a reinforced embankment located in a seismic area. As expected, temperature records inside the embankment are smoothed versions of the outside temperature, although some significant shifts in the position of extreme temperature values are observed. Authors state that the tensile creep behaviour of reinforcements are critically controlled by temperature and this was the reason for the real scale experiment conducted.

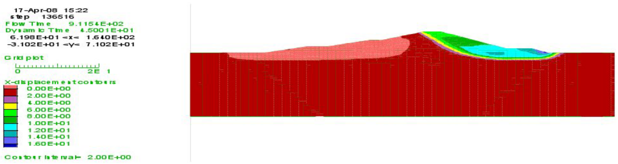

Figure 5.6: *x*-displacement contours within the body of the dam section indicating a base type failure due to presence of liquefiable layer. (Sivakumar and Srivastava, 2009)

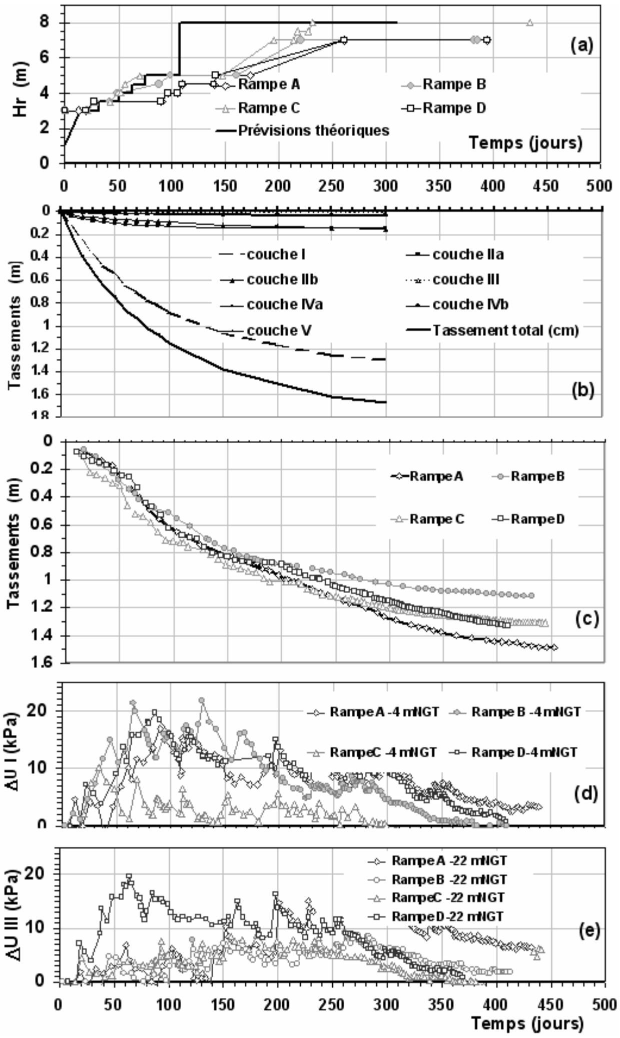

Figure 6.1: Synthesis of the different estimations and measurements after preloading. (Zaghouani *et al*., 2009)

6 EMBANKMENT MATERIALS AND FOUNDATION SOILS

Two papers included in this Section refer to the construction of embankments on very soft soils. This is a common situation in harbour environments.

Zaghouani *et al*. (2009) describe the soil conditions in the Tunis lake and the precautions adopted to build some embankments that reached heights of 7 m. The presence of two soft silty clay layers in the natural soils dictated most of the construction details (loading sequence in time, target safety factors, instrumentation). The gain in undrained strength was taken as $0.37\sigma'_v$, a value that was apparently based on vane tests.This is a value above other recommendations for normally consolidated clays. No details of stability calculations are given. Figure 6.1 provides a summary of settlement and piezometric records measured in the clay layers. Measured surface settlements are in excess of 1-1.4 m and show a similar trend in all parts of the project, despite the large area involved. Excess pore pressures, also shown in the figure, are a good indication of safety if interpreted through a calculation model. The upper silty level, unlike the lower one (at a depth of 20-25 m), was drained by means of prefabricated geosynthetic drains. However, the rates of pore pressure increase and subsequent dissipation are quite similar in the two clay layers. Another interesting feature, found also in other projects, is that the excess pore pressures during loading, are lower than the theoretical values associated with the calculated total stress increase. The figure shows that the subsequent dissipation is rather complete. One year after the beginning of loading pore pressures have essentially dissipated.

Roy and Singh (2009) concentrate on the determination of undrained failure conditions in embankments built in soft clayey soils. A successful construction requires often the gain in undrained strength due to soil consolidation. The paper presents a simplified procedure to estimate this increase in c_u, which may be achieved by simple stress calculations and 1D vertical dissipation of excess pore pressures. Later, they check the stability by limit equilibrium procedures. Three examples are given. The necessary key information in this process is the relationship among undrained strength, effective vertical stress and OCR. The plot in Figure 6.2 provides data in this respect, which seems to confirm early results reported by Ladd *et al*. (1977).

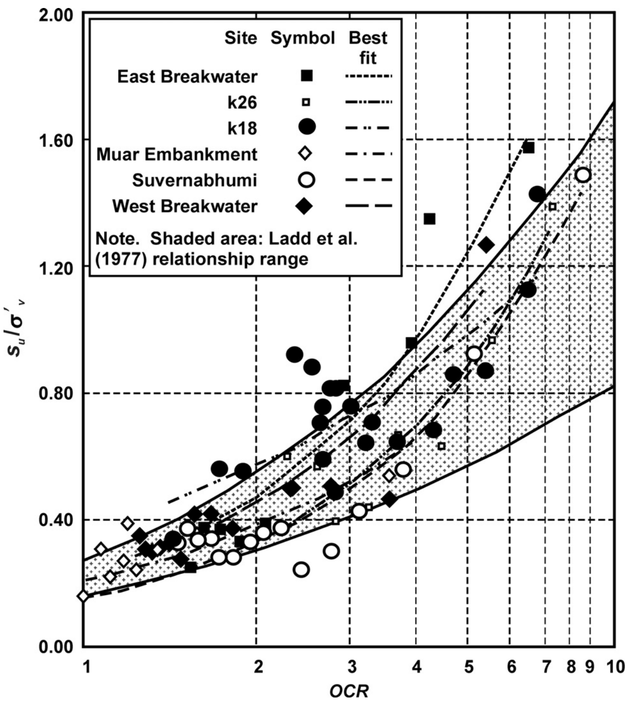

Figure 6.2: s_u/σ'_v – *OCR* relationships. (Roy and Singh, 2009)

Three papers deal with earthhdam materials. Soroush *et al*. (2009) present an academic exercise on the effect of core behaviour in the development of excess pore pressures and deformations during dam construction. Two soils are selected to conduct a comparison: a clay material and a well-graded mixture. Their permeability is one order of magnitude apart and this is probably the main factor contributing to the differences observed in calculations. The authors consider the soils saturated and this is probably the major limitation of the work developed. Both the generation of excess pore pressures during the undrained stage and its subsequent consolidation are fundamentally affected by the initial state of saturation after compaction

Jafarzadeh and Yousefpour (2009) report the results of an extensive program of cyclic triaxial testing on samples of asphalt concrete, a material considered for the construction of impervious cores. An example of their results is given in Figure 6.3. It shows the variation of shear modulus, G, and damping ratio, D, for increasing shearing deformation, γ. Asphalt mixtures tested had bitumen content in the range 5.5%-6.5% and the granular mix had maximum sizes of 25.4 and 12.7 mm.

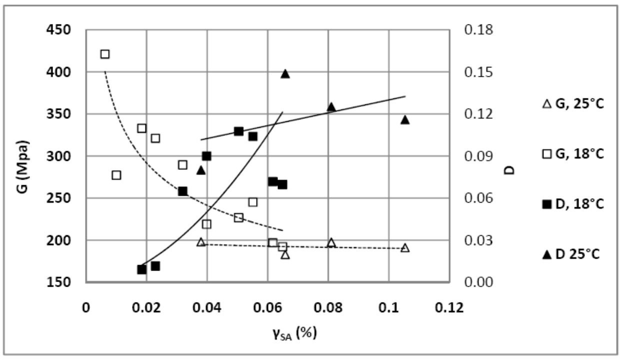

Figure 6.3: Effects of temperature on G and D, M.A.S.=12.7 mm, *b* =5.5% and σ_3=400 kPa (Jafarzadeh and Yousefpour, 2009).

Figure 6.3 refers to a 12.7 mm maximum size aggregate and a bituminous content of 5.5%. The plot shows the decrease of G with γ, the incerase in D and the effect of temperature (T=18ºC or T=25ºC). Despite the trends identified by the authors, the effect of temperature, in the range mentioned, is not that relevant, although it seems that the entire strain range was not properly covered by tests performed at T=25ºC. The authors conclude, on the basis of the reported tests, that no evidence of cracking and degradation was found after 5000 cycles of loading.

Bazaz and Gaghari (2009) describe the effect of treating dispersible natural clays with lime and aluminium sulphate to reduce the risk of dispersivity. The criteria and tests described by Sherard *et al*. (1976) (salt content and pinhole test in particular) are used to estimate the risk of dispersion.

One example is given in Figure 6.4 in the familiar plot of Sodium percentage vs. total dissolved salts. Adding lime by the percentages indicated in a low plasticity clay (CL, w_L=24.8%, w_P=7.7%) results in a progressive migration towards the safe area of the plot. The use of aluminium sulphate, advocated in the paper, is perhaps open to a different kind of risk: the development of expansive salts such as ettringite or thaumasite. In fact, the sulphate and aluminium ions, in the presence of calcium and water may lead to the formation of those minerals that destroy cementation effects and induces swelling.

McCartney and Parks (2009) present an interesting discussion on the difficulties of estimating water retention and permeability functions in unsaturated soils. They point out the errors associated with some predictive equations, which use water retention data to estimate permeability as a function of suction, or volumetric moisture content. Predictions are difficult for clayey soils where porosity distributes usually around two dominant sizes but also for granular soils. In the first case, the predictive equations do not represent properly the physics of Darcy flow. The uncertainties encountered are summarised in Figure 6.5, which compares the α parameters (sometimes λ) of Van Genuchten model found when fitting water retention and permeability data (through Mualem model). The plot shows the inconsistency of a procedure which is sometimes referred to as a "consistent" approach. The authors point out the significant changes associated with the prediction of pressure profiles when two α parameters, fit to WR or K-function data, are used.

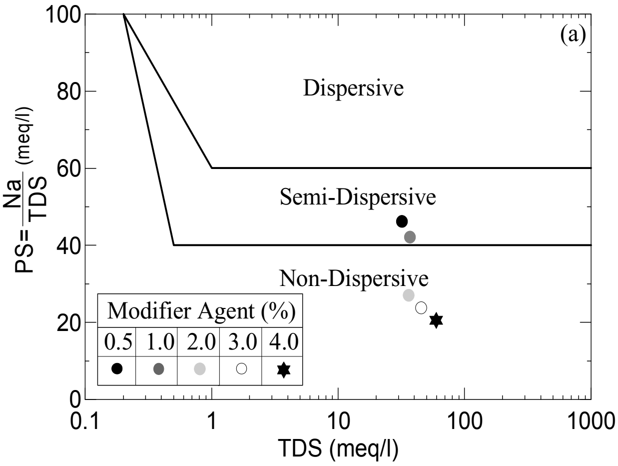

Figure 6.4: Dispersivity classification of sample 3, improved with Lime (Bazaz and Gaghari, 2009).

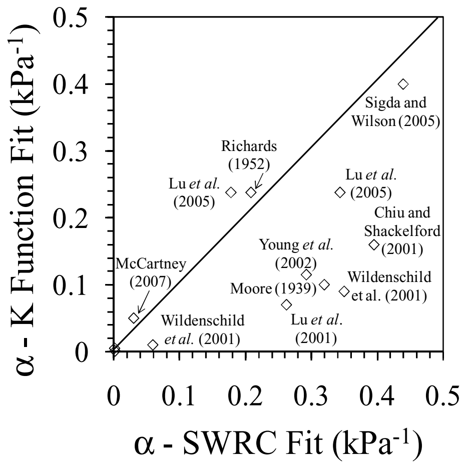

Figure 6.5: Comparison between α parameters for different soils obtained from fitting the van Genuchten (1980) model to the k-function data and to the SWRC data. (McCartney and Parks, 2009)

7 OTHER TOPICS

Two papers address engineering geological aspects and provide an account of slope stability problems in two countries: Bulgaria (Hamova *et al*., 2009) and Albania (Muceku, 2009). In the first case the authors summarise a large variety of conditions and describe some methods that proved to be useful in some cases. One example is given in Figure 7.1 in connection with a road construction on unstable ground. The embankment is stabilised by means of two longitudinal walls tied by a rigid anchor. The outside wall is further stabilised by a counterfort wall or rockfill.

Muceku (2009) stresses the difficult situations found in Albania, where flysch-type rocks are affected by active tectonism, weathering mechanisms and coluvial activity. He describes two large landslide areas, which cause serious damage in two towns (Lehza and Kruja). A representative cross-section of the Lehza town, under the castle, founded on limestones, is given in Figure 7.2. The paper reports some geotechnical properties of the soils involved in the mass movements (low to medium plasticity sandy clays).

Rock slopes are described in two papers. Sumi *et al*. (2009) propose a calculation procedure based on the stability conditions of a pyramid block having a triangular base. The base of the rock slide is discretized by means of triangles. The stability of unit blocks are examined against three failure modes: rockfall, sliding and toppling. Unstable blocks are searched, once the entire slope is discretized. A "fragility" number is defined as the ratio between the total volume of the unstable block and the total volume of all movable blocks in a given slope. The authors claim that all that is required to establish the 3-D configuration of planes is a photographic surveying by means of a digital camera. In the example developed in the paper, the fragility number is plotted against the horizontal seismic stability (Fig. 7.3). Two cases are considered regarding some assumed distribution of pore pressures. Experience with rock masses indicates that identifying the internal geometry is not a simple task, however.

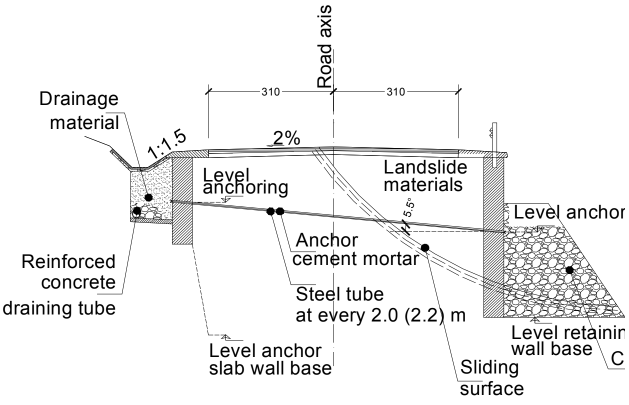

Figure 7.1: Anchored walls- plates. (Hamova *et al.* , 2009)

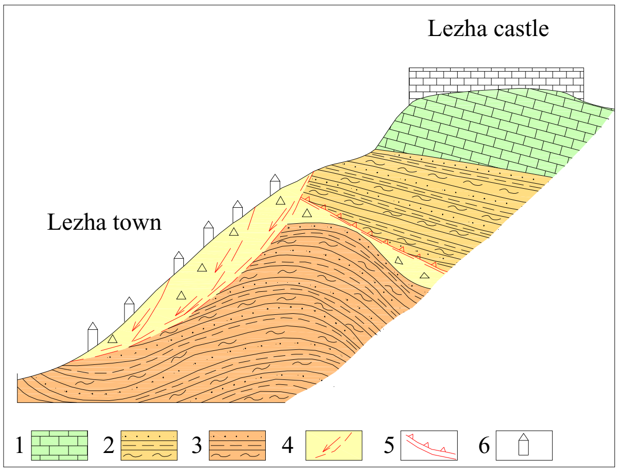

Figure 7.2: Flysch rocks- siltstones and claystones, 3. Flysch rocks- siltstones intercalated with sandstones and marl layers, 4. Landslide, 5. Tectonic, 6. Building. (Muceku, 2009)

Méndez *et al.* (2009) describe the stability analysis of a deep cut (280 m) in a highly fractured and complex rock mass. They develop a 3D model, making use of a commercially available code for rock masses. Block interaction is simulated by means of normal and tangential stiffness coefficients. Some criteria was selected to limit the number of discontinuities introduced into the model. Contacts were assumed to be purely frictional and friction angles were derived from field surveying. Safety factors were determined, presumably by means of a (c,ϕ) reduction technique. Safety conditions for the design earthquake were based on calculated displacements when time acceleration records were applied to the model boundaries.

Figure 7.4 shows the calculated displacements at the slope toe for the maximum credible earthquake. The calculated maximum displacements (40 cm) indicate the need to introduce some kind of reinforcement. The authors rightly identify the need to know accurately the strength parameters as the major difficulty to carry out realistic predictions.

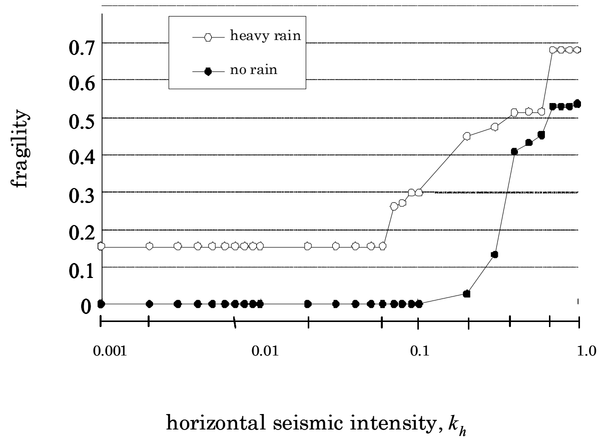

Figure 7.3: Fragility intensity with respect to the seismic intensity. (Sumi *et al.*, 2009)

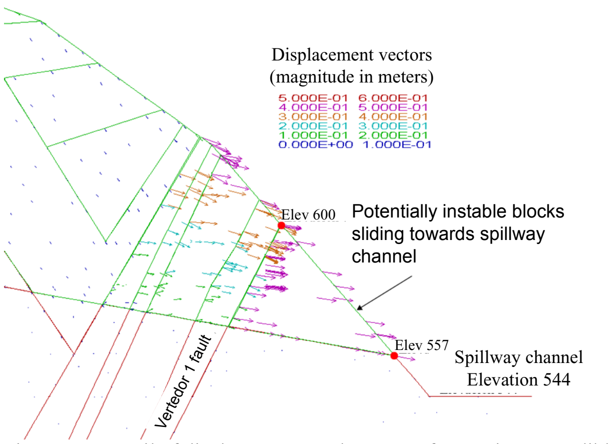

Figure 7.4: Detail of displacements at slope's toe for maximum credible earthquake (Tr10000). (Méndez *et al.*, 2009)

Two interesting thought-provoking papers dealing with probabilistic aspects in slope stability are included in the Session. Griffiths *et al.* (2009) present theoretical results supported by a 3D Random FE analysis of a simple slope configuration. The slope is initially defined in a 2D cross-section and then extended in a normal direction a length, *L*. The two geometrical parameters defining the problem for a fixed slope angle (2H:1V) are the length L and the total height, *H*. The analysis is made undrained and the strength c_u is described by means of a random spatial function, lognormally distributed. The probability of failure, p_f, is determined by a "Monte-Carlo" procedure. The spatial correlation of c_u is defined for the Gaussian log c_u random field. It is a common knowledge in slope stability that 3D effects increase the calculated safety factor because of the additional contribution of the "sides" of the slope. However, the calculated p_f for 3D conditions increases above the 2D value as *L/H* increases. This is shown in Figure 7.5 for a particular value of the coefficient of variation of c_u and for some correlation distance. The result has a simple explanation from the perspective offered by a random spatial variation of c_u. As the length of the slope increases, the probability of finding a "weak spot" increases and this effect does not compensate the deterministic 3D effect, even if the sides are "rough".

There are, however, some remarks to be made. The first one concerns the disturbing high failure probabilities (0.2-0.3) determined for a slope whose conventional safety factor is 1.39. SF = 1.39 would be regarded as a perfectly acceptable and safe SF in an undrained analysis of slope stability. The second remark concerns the fundamental hypothesis of representing the uncertainty in undrained strength by means of a log normally distributed random field. There is a lower limit for c_u values in practice and this is given by the relationship $c_u = \alpha\sigma'_v$ for N.C. consolidated conditions, where α = 0.2-0.3. In addition, the determination of properties in a precise location of the slope modifies immediately the spatial correlations because properties are known with certainty at some locations (see Castillo and Alonso, 1985). This effect obviously becomes increasingly important as the correlation distance increases. In sedimentary soils, horizontally layered, high horizontal correlation distances are expected. Then a vertical boring may extend its influence on long distances and the assumption of a homogeneous random field representation of heterogeneity may be grossly in error.

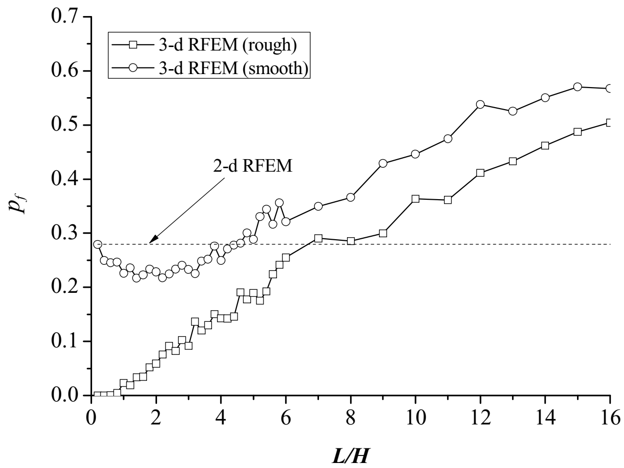

Figure 7.5: Probability of failure versus slope depths (Θ_{Cu} =2.0, v_{Cu} = 0.5) (Griffiths *et al*., 2009).

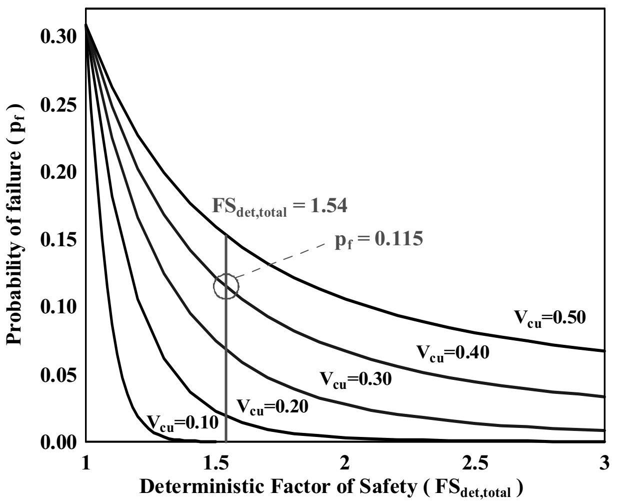

Figure 7.6: Probability of failure as a function of the deterministic factor of safety (FS_{det}) and the variability index of c_u (V_{cu}) for slopes in cohesive soil under undrained conditions. (Kavvadas *et al*., 2009)

Kavvadas *et al*. (2009) explore the recommendation of Eurocode 7 from a probabilistic perspective. They adopt a random variable approach and this implies a very conservative attitude because spatial correlations are disregarded as well as the effect of "site investigation" mentioned before. Adopting a simplified geometry of the slope and a planar failure surface, a relationship between the probability of failure and the deterministic safety factor could be found (Fig. 7.6). The provisions of Eurocode in terms of partial safety factors result in a global safety factor, in case of undrained analysis, of 1.54. For this value and for an accepted coefficient of variation of c_u (0.4), a probability of failure of 11.5% is read in Figure 7.6. This is, again, a high value in view of practice. Safety factors adopted in undrained analysis are often lower than 1.5, especially if some effort is given to the reliable determination of c_u. In those cases Figure 7.6 provides also high probabilities of failure. A remark can also be made to the adopted truncated distribution for c_u. In general, values in excess of c_u for NC conditions are guaranteed in most situations.

Xu *et al*., (2009) discuss the patterns of earthdam distress and the likely causes on the basis of a survey of dam incidents in China. 150 incidents were analysed through a Bayesian network tool. The most frequent distress patterns are foundation leakage/piping, embankment cracking and embankment sliding. These incidents are a consequence of inadequate cut off or filtered drainage at the foundation and inappropriate or faulty embankment material for the second and third incidents. This conclusion comes as a result of the Bayesian procedure followed. The paper describes the advantages of such an approach and the automatic updating when new data becomes available. Obviously, results are dependent on the initial (engineering) assessment of the causes of the observed distress. This is not a straightforward outcome, however, and the lengthy and controversial discussions for the reasons behind a given incident or failure, illustrate the comment. In particular, the assignment of a good proportion of distresses to inappropriate materials is probably too vague and at odds with current sustainability trends.

8 CONCLUDING REMARKS

The availability of commercial software to address general problems in geotechnical engineering is progressively changing the approach and attitude of engineers and researchers. Complex analyses now become a relatively easy task. In a sense, there is a permanent invitation to use those tools in a large variety of circumstances. Both, failure conditions (limit conditions) and deformations (serviceability states) are properly solved. The (c',ϕ') reduction method, in particular, is a popular tool. Certainly its use has advantages over alternative procedures: the failure surface is no longer imposed but a consequence of the analysis; complex geometries and construction sequences may be dealt with and the evolution of SF may be followed. Undrained strength (in the case of undrained analysis) is properly updated if the soil model is well calibrated. It should be added that the method is essentially consistent with the classical limit equilibrium methods if conditions are similar. In fact, limit equilibrium procedures could be used to check the consistency of a more sophisticated approach.

There is also an increasing trend to shift from 2D plane strain and axisymmetric analyses to general 3D conditions, although some of the papers reviewed point out the current difficulties. Others propose desirable features for the next generation of codes.

The review made suggest, however, a complementary set of comments:

- The attitude behind the appropriate methodology to determine the risk of failure (in short, the safety factor) should be flexible and adapted to the problem at hand. This issue has been discussed in some detail.
- Despite the increasing pressure to use FE techniques, there is still room for simplified, ad-hoc, semi-empirical or analytical procedures. A significant number of papers dealing with widely different topics (piled embankments and rafts, reinforced slopes, dynamic analysis of slopes, embankments on soft soils, rock slopes) support this comment.
- The selection of appropriate soil models and the determination of material parameters remain the fundamental issue in most of the work reviewed, irrespective of the method of analysis. However, a sound conceptual representation of the field problem, which often requires the joint consideration of field and laboratory data, as well as a correct theoretical understanding of basic soil (and rock) mechanics, is also of fundamental importance. The two aspects, material behaviour and basic mechanics, remain the cornerstone of geotechnical engineering.
- A number of additional specialized topics are required in practice. A few of them have been described in the papers reviewed: riverbank protection; soil treatment to stabilize, increase strength or enhance some desirable feature; the design of landfill isolation, etc.
- Advances in understanding some phenomena (flow and deformation of unsaturated soils) add a new perspective to classic problems, such as the risk of failure of slopes under rainfall or the construction of embankments and dams. The papers reviewed here adopt relatively simple procedures, which are always of interest. But sophisticated FE tools are already available to examine those problems in more detail. Again, the immediate comment here is that the determination of material properties and models of behaviour would become the fundamental practical issue in the future.

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