Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering

doi:10.3233/978-1-61499-656-9-3071

General report on technical session 2f: Embankments and dams

Séances techniques 2f: Remblais et barrages

Andrew J. Whittle
Massachusetts Institute of Technology, USA

1 INTRODUCTION

The topic of dams and embankments has featured in all prior ICSMFE/ICSMGE Conferences. It is therefore interesting and insightful to begin this review by considering the 28 papers submitted for this session and asking what they reveal about the state of research and practice for these types of earth structures:

Perhaps the most noticeable observation is the widespread/ubiquitous use of numerical analyses, particularly in design and back-analysis applications. Of the 10 papers that deal extensively with numerical analyses, only 2 actually relate to the prediction of performance. There has been much recent progress in understanding the behavior of partially saturated soils. The session includes 4 papers that illustrate the role of partially saturated soil behavior in deformation and stability problems, but ironically only one that deals directly with properties of compacted fills. However, there is one paper presenting an intelligent compaction system for controlling the placement of rockfill. Another group of 4 papers focuses on deformation properties of rockfills and presents interesting new results on the effects of weathering and pore fluid composition on shear strength and compression properties.

The session includes some interesting contributions dealing with case studies including a cluster of three papers relating to the failure of peat dikes in the Netherlands, and reports on the field performance of two rockfill dams in Iran. There are also two useful reviews on i) the long term performance of Portuguese rockfill dams with impervious upstream facing, and ii) performance of rockfill dams under strong seismic shaking.

It is also notable that certain 'classical' topics are absent. There is only one contribution dealing specifically with seepage/flow problems or hydraulic properties of fill materials. Similarly, there are only a couple of papers dealing with consolidation and creep of soft clays. There are 3-4 papers that deal with case studies involving ground improvement techniques for soft foundations, but none reporting novel developments in this field.

In proceeding with a detailed review I have attempted to cluster papers within four general areas, i) construction material behavior, ii) soft foundations, iii) stability problems and iv) construction and performance.

2 CONSTRUCTION MATERIAL BEHAVIOR

The time dependent deformation behavior of rockfill is of great importance in the design and performance of dams. There were several landmark laboratory studies in the 1970's (Marachi et al., 1969; Marsal, 1973; Penman & Charles, 1976) that established particle breakage or fracture as the underlying cause of these deformations. These micro-mechanisms were also clearly enhanced by the presence of water, although the underlying mechanisms were not fully defined.

Sayao et al. present results from a program of large scale laboratory direct shear and drained triaxial shear tests on specimens of weathered basalt rockfill (from the upstream slope of the Marimbondo dam in Brazil) and fresh basalt (from the same nearby source quarry). After 25 years in the dam slope, there is a measurable change in the surface texture of the basalt that is attributed to fluctuations in temperature and reservoir water level (inundation cycles). Their data, presented in the form of non-linear strength envelopes, show a significant reduction in the shear strength of the weathered basalt compared to fresh specimens with the same particle size distribution and relative density.

The Authors have investigated laboratory methods for simulating the natural weathering process of the basalt over a reduced time scale using a process of 'continuous lixiviation'. This is accomplished using a novel extractor system (Fig. 1) that simulates the natural cycles of inundation and temperature change. Maia et al. (2003) report that this artificial weathering process extends existing micro-fractures in the vesicular basalt particles but produces only minor chemical alteration (surface iron oxidation). The shear strength of specimens that have been artificially weathered for 450 hrs is in close agreement with the results from the Marimbondo dam specimens and hence, considered to be equivalent to 25 yrs of natural weathering. The Authors then postulate upper and lower limits on the expected shear strength of the rockfill after 75 yrs. The basis for this extrapolation is not provided. However, according to their best estimate, the low stress friction angle of the basalt reduces from an initial/fresh value, $\phi' \approx 53°$ to $37°$ at 75 yrs.

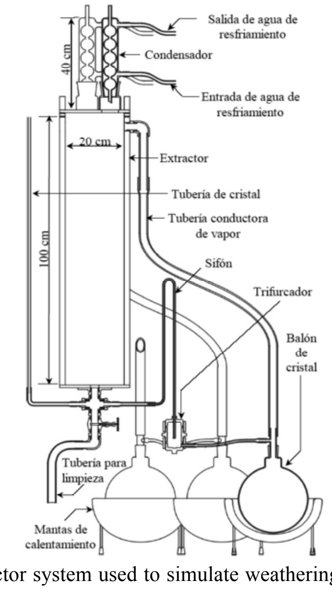


Figure 1. Extractor system used to simulate weathering of basalt (Maia et al., 2003)

The paper by Romero et al. considers the effects of pore fluid composition on compression behavior of a crushed slate rockfill material. This work builds on the basic hypothesis that the pore fluid can enhance particle breakage and fracture through a process of stress corrosion (Atkinson & Meredith, 1987) that enables subcritical crack growth. Oldecop and Alonso (2003) have recently proposed total suction, ψ, as a key parameter for representing this influence of pore water on the mechanical response of rockfill, Figure 2. This has been supported by experimental measurements of compression response in 1-D oedometer tests with controlled Relative Humidity (for equilibrium conditions, $\ln[RH] \sim -\psi$) presented by Oldecop and Alonso (2001, 2003). Romero et al. describe 1-D compression tests where compacted specimens of rockfill (with initial ψ = 100MPa) are imbibed/flooded with fluids of different composition (such that there is no matric suction and hence, $\psi = \pi$, where π is the osmotic suction). Further controlled changes in total suction are then brought about by replacing the composition of the pore fluid.

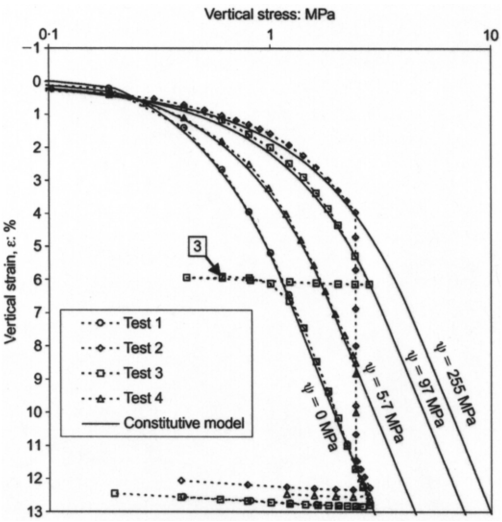


Figure 2. Effect of total suction on compressibility of rockfill (Oldecop & Alonso, 2003)

The results show that the compressibility of the saturated rockfill depends on the total suction. Saline pore fluids with high osmotic suction cause less damage to the rockfill particles and correspondingly smaller compressive strains than tests performed with distilled water (ψ = 0MPa). This behavior conforms with prior expectations and is consistent with measurements of the post-test particle size distribution and splitting tests that show tensile strength increasing with total suction (Romero - Figs. 5, 6, respectively).

Other aspects of the test behavior relating to the volume changes occurring during flooding and fluid replacement stages are more difficult to explain. The Authors assert that the measured time dependent strains in these tests (Romero - Figs. 2, 4) can be explained if there are two fluid potentials in the test specimen. At any given time, the potential of the fluid in the interparticle void space need not be in equilibrium with the fluid potential inside pores within the rock particles themselves. The initial collapse on flooding reflects the step change of matric suction in the interparticle void and damage of particle asperities; while creep strains occur due to the much longer timeframe required for fluid transport and equilibration of potentials between the particle pores and interparticle void space. This is certainly an intriguing concept that deserves further investigation.

The paper by Athanasiu et al. also deals with creep strains and settlements in rock fills. The Authors use empirical equations to describe diffusion and linear phases of creep processes measured in laboratory tests. The paper is discussed at more length in section 5.

Guerpillon and Virollet discuss the degradation of fill materials in more general terms. They contrast the behavior of two designated types of compacted fills used in highway embankments in France, i) 'non-evolving' fills comprising low and high plasticity clays (classified by GTR as A3 and A4, respectively) which can undergo post-construction swelling and shrinkage due to water content changes; and ii) 'evolving' chalk and marl rockfills (R1 and R3, respectively) which deform due to changes in their granulometry. The Authors argue that the clay fills should be compacted on the wet side of optimum to reduce in-service deformations, but recognize practical constructability issues. Other investigators such as Lawton et al. (1989) have found volume changes occurring in clay fills can be minimized by controlling the relative compaction ($RC = \gamma_d/\gamma_{dmax}$), Figure 3.

According to Guerpillon and Virollet the chalk and marl fills consist of larger blocks of material embedded in a fine matrix. Volume changes during imbibition are due to fragmentation of the blocky fraction. Using simple phase relations, they assert that these volume changes can be minimized if the fill has a fine fraction greater than 30% (by weight). Given the practical limitations in fill availability, the Authors then provide recommendations on zoning of earth fills for large embankment structures.

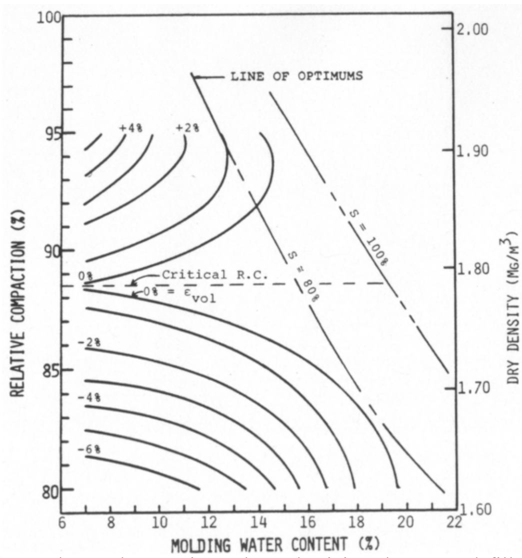


Fig. 3. Volume changes in a low plasticity clayey-sand fill (from Lawton et al., 1989)

Kulathilaka and Muhunthan consider the use of lightweight fills in controlling the settlement of an embankment on an 8m deep layer of peaty clay (for the Colombo-Katunayake expressway). The preferred fills comprise low cost mixtures of the local laterite with tire chips, sawdust or paddy husks. Although the mixtures have very low density (e.g., γ_d = 432-508kg/m^3 with paddy husks), they are also much more compressible than the native laterite. The Authors present numerical simulations of settlement-time response for staged construction of the embankment (including pre-loading using laterite), with properties of the underlying peaty clay described by the Modified Cam Clay model. The results show very large settlements (2.0 – 2.5m), compared to solutions using more expensive polystyrene blocks (1.0m). The lightweight fills produce only a relatively modest reduction in the long term settlement (by up to 20% for the sawdust mix) compare to the laterite but are more effective in reducing the timeframe required for embankment construction. The mixtures are much less effective than the more expensive use of polystyrene blocks. The Authors do not discuss the long term properties of their proposed fill mixtures (i.e., potential degradation mechanisms), or creep properties of the underlying peaty-clay.

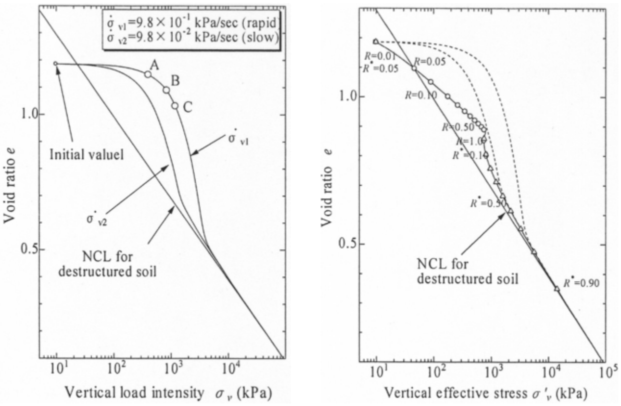
There is only one paper in the current session that deals directly with flow and transport in dams. Park et al. describe the design of a laboratory device for evaluating the selection of 'critical filter' materials that are able to prevent internal erosion and piping within fine grained cohesive soil once cracks have developed within the core of a zoned earth dam. The Authors cite field evidence of cracks that have developed either due to post-construction settlement or strong ground shaking. Their principal goal is to establish whether existing gradation criteria (USBR, USACoE, NRCS) for granular filters are adequate for such extreme events. They have designed a laboratory flow cell with an open flow channel/'crack' up to 2.3mm wide across adjacent layers of base cohesive soil and granular filter materials. The Authors measure the upstream pressure, flowrate and turbidity of the effluent downstream and film directly the state of the flow channel during the test (through a transparent window). Successful filter materials slump to fill the crack and then collect/trap sufficient fine grained particles to prevent further erosion of the base cohesive material, yet do not become fully clogged such that there is a long term build up of pressure (to maintain the flow rate). The Authors illustrate typical results of a test and report that filters meeting current gradation specifications also function successfully in their 'critical filter' test apparatus. It would be interesting to use the experimental measurements as a basis for evaluating current analysis capabilities for modeling internal erosion in earth fills.

3 SOFT FOUNDATION PROBLEMS

The prediction and interpretation of ground movements beneath embankments on soft clay remains one of the most challenging problems in geotechnical engineering. Asaoka et al. report on the field performance of an embankment (Kanda site, Joban expressway) overlying a soil profile that includes 15m of lightly overconsolidated alluvial clay. The embankment has settled more than 2m since the end of construction 20 years ago (with small load increments due to 5 overlays), while there has been no dissipation, and even a small increase, in the pore pressures within the clay during the period 1993-2002. This type of behavior defies conventional wisdom regarding consolidation behavior.

The Authors believe that this behavior can be explained by modeling the progressive breakdown in the soil structure. They introduce a relatively complex, generalized effective stress soil model that can describe the anisotropic stress-strain-strength of clay and simulate progressive destructuring. Figure 4a illustrates the typical 1-D compression behavior considered in the model formulation. The initial stiffness of the structured soil is broken down as the soil is loaded above the apparent preconsolidation pressure (A), but is only fully 'destructured' by loading to high stress levels. Figure 4b shows the proposed model simulation of a lab-scale consolidation test with $\dot{\varepsilon} \rightarrow 0$. The net effect is that the proposed formulation is very similar to the S-shaped consolidation curves used in 1-D settlement models (e.g., ILLICON, Mesri, 1994). However, it should be noted that Asaoka et al. do not model creep strains and (erroneously) assume constant hydraulic conductivity in their analyses.

Asaoka et al. present simulations for the Kanda embankment. They find that the middle sub-unit of the clay (at depths 11-20m) has a much higher in-situ void ratio and therefore more highly structured than the remainder of the clay. By modeling this profile, the are able to replicate, qualitatively, the unusual pore pressure response from the Kanda site. It appears that there is only limited data from this site (and no information on lateral spreading). Hence, the current paper provides only a partial validation for the proposed model.



(a) (b)

Figure 4. Breakdown of soil structure a) observed in 1-D experiments loaded as constant strain rate; b) numerical simulation of 1-D laboratory consolidation using Asaoka model (Asaoka et al., 2000)

Boutonnier and Virollet also consider the interpretation of pore pressures generated by embankment loading on clay. They propose a new theory for estimating the pore pressure coefficient, B, for clays which are not fully saturated. They define three domains of interest in the matric suction-saturation relations: D2, with suctions less than the air entry pressure; D3 with zero matric suction (and pore water pressure, $u_{wsat} \geq u_w \geq 0$) but incomplete saturation; and D4 fully saturated ($u_w > u_{wsat}$). Details of the proposed formulation are not presented, but some of the assumptions made by the Authors deserve further scrutiny. For example, they assume that the hydraulic conductivity is a unique function of the void ratio in D2, D3 and D4. Similarly, eqn. (8) does not generate B = 1 for $u_w = u_{wsat}$ (boundary of zones D3 and D4). The Authors make a brief comparison of their computed B values with field measurements from the Cubzac-les-Ponts site (test embankment 'B'; Magnan et al., 1983). It is not clear in these comparisons how the Authors have estimated changes in the horizontal total stresses needed to interpret B?

Alonso et al. simulate the potential settlements of an earth fill aqueduct/canal caused by volume changes induced by wetting of the underlying, partially saturated, clayey-silt materials. The paper outlines the selection of input parameters for the Barcelona Basic Model (BBM; Alonso et al., 1990) which is used to characterize the mechanical properties of the natural silts. The Authors show that large volumetric strains (up to 12%) can occur upon wetting this material. This response is well described by the Loading-Collapse yield surface in the BBM model at relatively low confining pressures, $\sigma_v \leq 200$kPa (but overestimates the collapse at higher pressures). Similar calibrations have also been carried out for specimens of valley silts that were compacted at optimum moisture content (Standard Proctor). This is considered a candidate technique for improving the foundation performance. It is not clear how well the BBM actually describes the shear behavior of the compacted silts or how the authors are able to estimate the initial yield state parameters for this material (p_0*, s_i).

The paper illustrates FE predictions (using CODE_BRIGHT, Olivella et al., 1996) of the aqueduct settlement due to infiltration from seasonal rainfall and leakage from the canal itself. The results show relatively small movements (ca. 2.5cm) that stabilize after one year if the aqueduct lining remains impermeable. However, much larger settlements (up to 0.3m) can develop if there is a major leak in the lining. The Authors show that this risk is eliminated if the foundation soils are compacted at the optimum moisture content. The paper gives an excellent illustration of the advances that have been achieved in the understanding and modeling of partially saturated soils.

The paper by Sternik involves the back-analysis of settlements measured beneath an 8.1m high motorway embankment built over 25m of clay and sand deposits. The project is located in an area of general mining subsidence. The Author assumes that subsidence occurs uniformly in the project area and can be estimated from settlements measured outside the footprint of the embankment. Numerical simulations of the consolidation settlements are carried out using simple linearly elastic-perfectly plastic soil models (with effective stress strength parameters for both the fill and foundation soils). The Author back-fits the elastic stiffness of the foundation separately during a) the 6 month construction phase and b) the 6-month post-construction monitoring period. The paper shows very good agreement between the computed and measured surface settlement, reaching 15cm one year after the start of construction.

It is difficult to review these results objectively as the Author provides no details of the soil conditions at the site (that would enable an evaluation of the selected soil parameters) and there are no supporting measurements/comparisons with pore pressures in the foundation soils. The backfigured value of foundation soil stiffness during construction and post-construction (E' = 67MPa and 6MPa, respectively) are much higher than I would expect for typical clays and hence, must reflect the presence of stiffer sand layers in the profile.

Logar et al. summarize the design and measured field performance of an 11m high embankment supported by (60cm diameter) stone columns that extend 12m through a clay profile, with 4.0 – 5.5m of soft organic silty clay, down to an underlying dense gravel layer. The design layout of the stone columns was controlled by stability considerations and analyzed using a finite element program (Plaxis; with the c-ϕ' reduction technique; Brinkgreve & Bakker, 1990). The Authors established that the embankment could be safely built using a two stage construction, allowing consolidation of the clay at an initial fill height of 7m. By then including stone columns, the timeframe for consolidation was greatly reduced and additional stability provided by the additional shear resistance of the stone columns. However, I am surprised by the very small difference in the factor of safety reported from 2-D and 3-D analyses.

Logar et al. present measurements of ground surface settlements (approximately 40-50cm) during construction with a further 10cm occurring in a 5 month monitoring period post-construction. The lateral deflection data are more difficult to interpret as the inclinometers were installed during the embankment construction. The Authors also report using the Asaoka (1978) method to interpret consolidation, but give no details on the backfigured consolidation properties. However, piezocone measurements obtained during construction provide direct confirmation of the original stability design assumptions. The current paper gives no details of the method (referenced to Pulko) used to estimate settlements in design.

Bhanderi and Roychowdhury outline the original design for a 5m high embankment to be built on 10m of very soft marine clay ($s_u \approx 6.5$kPa) assisted by Prefabricated Vertical (PV) drains. The Authors give no details of any field instrumentation that could be used to monitor the consolidation of the clay during the 10 week period of construction. They report a deep seated failure through the soft clay and fill at an embankment height of 3.3m which was apparently triggered by further local stockpiling of fill, while local construction guidelines (Indian Roads Congress) would have allowed an initial stage of only 1.25m (with FS = 1.5). The failed section has subsequently been redesigned with additional band drains (reducing the spacing to 1.15m from the original 2m triangular grid), geotextile basal reinforcement, and a 10m wide, 1.33m high berm. The re-design was based on more extensive geotechnical site investigations and the on-going construction performance is being closely monitored.

This paper exemplifies the classic soft ground foundation problems that arise due to inadequate control of staged construction. The Authors also raise several questions regarding the appropriate design assumptions using PV drains, although they conclude that the drains should perform satisfactorily (based on calculations of well resistance). One aspect of PV drain design that deserves further attention is the potential disturbance effects caused by installation. Saye (2001) reports that conventional methods for estimating the equivalent diameter, d_m, based on the cross-sectional area of the mandrel often underestimate the zone of disturbance and overestimate the effective horizontal coefficient of consolidation in the field, $c_h(e)$. Saye (2001) proposes that d_m should be computed from the perimeter, p, of the mandrel and/or any protruding anchor used in installation ($d_m = p/\pi$). He then summarizes the field performance of PV drains from a series of very well documented case studies, Figure 5. These data show that disturbance effects of closely spaced PV drains ($n' = d_e/d_m \leq 7$ - 10) can be detrimental with $c_h(e)/c_v < 1$, while $d_e/d_m > 30$ are needed to achieve ratios $c_h(e)/c_v > 2$. In the current paper, Bhanderi and Rowchowdhury report d_r = 54mm (based on mandrel cross-sectional area), and hence the spacing ratio, n' = 39 (assuming $d_r = d_m$; where, d_e = 2.1m). This is consistent with the design assumption, $c_h/c_v = 2.5$. However, the value of d_m may underestimated significantly. The redesigned embankment uses a much lower spacing ratio and therefore disturbance effects are likely to offset any advantage associated with the reduced drainage path length.

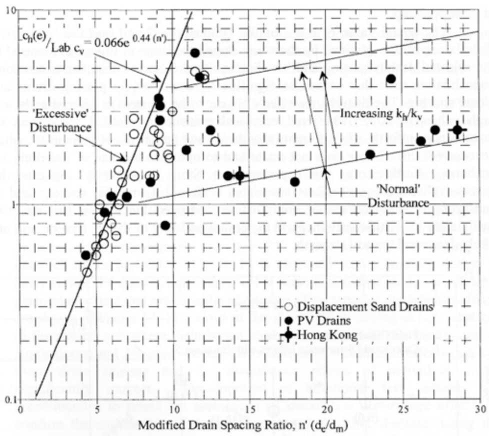


Figure 5. Effect of disturbance due to PV drain installation on consolidation properties of soft clays (Saye, 2001)

The paper by Gusmão et al. describes the collapse of a prefabricated reinforced concrete hangar built on shallow foundations over 6m of poorly compacted sandy and silty clay fill, and 12m deep deposits of soft organic clays. The structure developed cracks immediately after construction due to a combination of self-weight induced vertical settlements and downslope lateral spreading of the fill. These problems were exacerbated by further filling to level the hangar floor surface. Final collapse did not occur until several months later and involved a classic deep seated rotational failure extending through the soft organic clays. The Authors have confirmed that the failure is consistent with circular arc slope stability calculations (using Bishop's method) and the undrained shear strength measured in post-failure field vane tests. No geotechnical information was used in the original design of the fill or hangar foundations.

4 STABILITY PROBLEMS OF EARTH STRUCTURES

A cluster of three papers have been presented by engineers from the Netherlands dealing with the stability of peat dykes. The first by Bezuijen et al. describes the observation and interpretation of the failure of a secondary, containment dike at Wilnis that occurred in 2003. The second contribution by van der Kamp and Rob considers the use of an observational method for dike management and the third by Bakker proposes a method for estimating the probability of failure of dike rings. The Wilnis failure has also been analyzed independently in a recent publication by van Baars (2005).

Bezuijen et al. provide an interesting account of the history of secondary peat dikes. The dike structures are formed of the original peat while the land enclosed by the dikes has been systematically mined (for fuel etc) and later infilled to form polders. Water levels in the canals are artificially controlled. At Wilnis the crest elevation of the dike was at –1.5m below sea level and the canal at –2.2m. The banks were protected by lines of wooden sheet piling extending to –6.5m. The sudden ('enigmatic') failure of a 60m long section of the dike at Wilnis occurred after a very dry summer and involved a horizontal sliding wedge mechanism with displacements up to 6m. Post-failure site investigations by the Authors found that the peat is underlain by relatively permeable Pleistocene sands (at –9m) with piezometric head at –5.5m. The failure mechanism is clearly driven by water pressures from the canal, and assisted by reduction in the overburden pressures due to drying of the peat in the side slope (van Baars reports γ_{sat} = 10kN/m^3 and γ_{unsat} = 5kN/m^3). Bezuijen et al. report that the failure extends along the peat-sand interface and can only be explained if movements of the sheet pile bring about a hydraulic connection (fracture through the peat) between the canal and underlying sand (such that there is a higher piezometric head in the sand below the crest of the dike). They speculate that these sheet pile movements can occur due to shrinkage of the peat in the side slope. This mechanism is then consistent with observations of running water that preceded the actual failure.

In contrast, van Baars (2005) considers a horizontal failure plane extending from the toe of the wall through the peat and asserts that the failure can be explained by the very low sliding resistance in the peat (c' = 5kPa, ϕ' = 25°). While this is a much simpler explanation, the analysis assumes hydrostatic water pressures acting on the wall and does not explain precursor observations of flow.

According to van Baars (2005) there has been a long history of secondary dike failures in the Netherlands, and much delayed maintenance of sections that have been categorized as unsafe. Given the unacceptable cost of strengthening large sections of (both primary and secondary) dikes, van der Kamp and Rob propose a two tier monitoring system using an observational approach. They suggest that the system should include an automated and relatively robust Early Warning System for general deployment. For example, they suggest that simple measurements of water pressures in the underlying sand layers would be a good indicator for assessing the stability of primary dikes (in flood stage conditions). A second Close Monitoring System would be installed only at sites where dike safety is critical. They suggest that this type of system would could potentially include automated surveys of ground deformations.

The Authors provide no insight into the practicalities in deploying such a large-scale (geographically disperse) monitoring system. However, measurements of pore pressures in the sand (beneath the crest of the dike) would certainly have helped to understand the Wilnis failure.

Bakker proposes a method of computing the probability of failure for dikes reinforced with structural walls. The methodology appears to use a First Order Second Moment (FOSM) method for modeling uncertainties in the shear strength, canal water level and structural properties. The calculations of safety factor are based on finite element analyses using c-ϕ' reduction procedures (Brinkgreve & Bakker, 1990) with conventional Mohr-Coulomb effective stress strength parameters. Some details of the methodology are not clear to this reviewer. For example, what is the significance of the weight factors in computing the reliability analysis (eqn. 3.8) and how are the Monte-Carlo simulations performed? There are other problematic technical issues such as the assumed inter-relationship of the effective stress and undrained shear strength parameters using Mohr-Coulomb. Use of conventional (c', ϕ') can lead to serious overestimation of s_u for normally and lightly overconsolidated clays.

Beyond these details, the Author makes two key assumptions which deserve further clarification: i) There is no correlation of shear strength properties in each soil layer, which may be misleading when characterizing sub-units of a single geologic unit. ii) c-ϕ' reduction assumes the same factor of safety operating on apparent cohesion, c', and tanϕ'. This type of assumption should be used with caution. For example, it may be more appropriate to deal with separate factors of safety on cohesive and frictional components of strength as first suggested by Taylor (1948) – this is certainly relevant to the next paper.

Katzenbach and Werner consider the stability of railway embankments up to 10m high that were built using uncompacted sandy fills towards the end of the 19th century. With side slopes of up to 40°, conventional stability calculations would predict factors of safety in the range FS = 0.9 – 1.1. The Authors have also measured the ground deformation due to surcharge loading using a track mounted vehicle (BEFLA at Site A). They find that the measured deformations are an order of magnitude smaller then those computed using moduli based on dynamic probing tests (0.4cm vs 4.0cm) at a maximum surcharge load, 125kPa.

They present an interesting review of literature relating to the effects of vegetation on slope stability, but conclude that it is difficult to quantify these effects due to uncertainties in estimating parameters such as the root zone depth. Instead, they report data from large scale, field direct shear tests on samples with differing root density. Their results show that roots can add a significant cohesive component to the shear strength (up to 10kPa) of root-free fill. However, the data are not agreement with prior empirical equations based on the tensile strength and areal fraction of roots. Finally, the Authors have instrumented one particular slope extensively with tensiometers and TDR probes. They present typical data showing seasonal fluctuations from tensiometers within 2.5m of the ground surface and report that average suction during the winter months is less than 10kPa. The data are to be used in a more comprehensive analysis of slope stability.

Bellezza and Fratalocchi also consider the effects of matric suction in computing slope stability during reservoir drawdown. They propose a simplified model for the shear strength above the water table using conventional Mohr-Coulomb parameters (c', ϕ'), and effective stress as defined by Bishop (1955) with matric suction weighting factor, χ. The weighted soil suction [$\chi(u_a-u_w)$] is then related through the Brooks-Corey characteristic curve to the soil saturation, S_R such that the augmented shear strength is ultimately defined in terms of 4 parameters; i) residual saturation, S_{RES}; ii) pore size distribution index, λ (McCuen et al., 1981); iii) the normalized air entry pressure, $\psi_b/\gamma H$; and iv) saturation at the crest of the slope, S_{R0}.

The Authors use this shear strength criterion in limit equilibrium analyses (using Modified Bishop method) of an idealized, homogeneous slope with a four stage drawdown process (extending an earlier study published by Lane & Griffiths, 2000): 1) rapid drawdown to level L_1; 2) equalization of pore pressures at L_1; 2) slow drawdown to level, L_2; and 3) rapid drawdown to base elevation (L = H). They present a simple parametric study which shows that matric suction has minimal impact on the stability conditions affecting the first phase of drawdown, but reduces significantly the third phase of slow drawdown (reducing the magnitude of L_2-L_1). The Authors caution that practical applications of drawdown stability using the shear strength of partially saturated fill should be based on field measurements of suction.

Arnorsson and Erlingsson describe static and dynamic stability analyses for the design of two 14-15m high rockfill dams in southern Iceland. Seismic loading conditions are critical for both projects and the Authors compare three potential dam designs: i) with a central low permeability loessoidal core, ii) with a central asphaltic concrete core and iii) with an upstream impermeable concrete membrane. The dams overlie recent lava flows (8000 yrs old) which are formed on top of much softer alluvial and marine deposits with potential for significant site amplification.

The side slopes are designed using pseudo-static limit equilibrium methods (Morgenstern-Price method) and a seismic coefficient, k = 0.2, consistent with recorded ground acceleration (up to 0.84g) and shear strength parameters derived from laboratory triaxial tests.

Non-linear time domain dynamic analyses have also been performed using the Plaxis FE program in order to estimate permanent ground deformations and shear stresses in the loessoidal core (i.e., design option i). The analyses use three local time–acceleration records scaled to be consistent with the Eurocode (EC8) elastic response spectrum. The Authors also use simple soil models with linear unload-reload behavior (Mohr-Coulomb and 'Hardening Soil') with reduced shear stiffness ($G = 0.1G_{max}$) of the soil and rockfill and relatively high material damping ratio (ξ = 13.5%). They compare the three designs and conclude that the loessoidal core solution is likely to generate the smallest permanent deformations. They also confirm that the loessoidal core is not vulnerable to liquefaction by comparing computed shear stresses with data from cyclic triaxial shear tests (i.e., cyclic stress ratio, CSR vs number of cycles to failure, N_f) using the method of Seed et al. (1975). The paper gives a very clear demonstration of the state of practice in seismic design of rockfill dams.

5 CONSTRUCTION AND PERFORMANCE

This last group of papers deals with construction issues and the performance of earth structures. The paper by Yanaka et al. describes the use of an 'intelligent system' that enables near real time monitoring of rockfill compaction. The system is based on continuous vertical vibration measurements of a vibratory roller and empirical correlations proposed by the Authors relating the measured 'turbulence factor' to the in situ water content. They report an excellent unique correlation between the water content and peak friction angle for three different types of rockfill materials (limestone, basalt and mixed) used in the Minamiaike dam based on laboratory shear tests performed on specimens prepared with fixed amounts of compaction energy.

Vibration measurements during compaction show a unique acceleration amplitude spectrum component (occurring at the operating frequency, f_0) during the first pass of the roller. As the rockfill becomes more compact (increased ground stiffness), spectrum components occur at harmonic and half sub-harmonic frequencies. The Authors interpret the Fourier amplitude spectrum using a ratio of the amplitude components referred to as the turbulence factor. They find that turbulence factor can be well correlated with the dry density for each of the three classes of rockfills, but produces a unique correlation with water content.

They illustrate results of the proposed system for monitoring compaction of the Minamiaike dam. The location of the roller is established through GPS, and reduced data on the turbulence factor are transmitted via a wireless Local Area Network to the control office. The system represents a major advance on conventional compaction control methods which rely on a small number of field density tests that are expensing and time consuming. In contrast, Yanaka et al. report collecting real time acceleration data at the scale of $0.25m^2$. However, the success of their system relies on the quality of the underlying empirical correlations as documented in this paper. This paper represents a very significant advance in the automation in earthwork construction control.

Barchiesi et al. describe the technical issues in the construction of a cut-off diaphragm wall to be installed beneath a 120m high concrete-faced rockfill dam in a seismically active region. The diaphragm wall extends up to 60m through alluvial deposits in a steep-sided channel and must be sealed into the underlying granitic bedrock. The Authors explain the selection of the excavation method (clam shell and chisel with jet grouting to ensure good connection into the rock), and use of plastic concrete and key-joints to ensure watertight connection between panels. Special design calculations were carried out to investigate potential rotations and loading of the joint between the top of the diaphragm wall and foot-slab of the dam. The Authors report movements and structural stresses during construction and first filling of the dam based on 2D elastic analyses. They propose a simple criterion for evaluating he efficiency of the wall based on measurements of pressure head.

Alexiw presents a brief review of the various methods that have been proposed for the design of geosynthetic basal reinforcements for pile-supported embankments. Although there are some differences in details, most are based on an assumption of an arching mechanism within the fill. The geosynthetic acts as a tensioned membrane which supports the remainder of the fill and surcharge loads that are not transferred directly to the pile caps. Tensile forces are linked directly to strains by assuming a deformed shape for the membrane. The Author gives four examples of (anecdotally) successful recent projects (1998-2003) using geosynthetic-reinforced piled-embankments and highlights some unique components of each design. No data is provided to confirm the actual loads or deformations of the reinforcing layers.

Two papers submitted to this session provide reviews of field performance of dams. The first by de Santayana et al. discusses the performance of seven Portuguese rockfill dams with impermeable upstream facings. Three of the dams were built pre-1965 with dumped rockfill, while the other four were completed post-1992 with thin layer compaction using vibratory rollers. The later group were fully instrumented with inclinometer and extensometer devices to measure internal deformations in accordance with dam safety codes introduced in 1990. Five of the dams were built with reinforced concrete membranes, one with asphaltic concrete and the oldest, Pego do Altar (1948), has a steel membrane. The Authors summarize some of the key observations of vertical deformations and leakage during construction, first filling of the reservoir and subsequent operations. There has been a similar survey of CFRD dams published recently by Hunter and Fell (2003).

Measurements of the central section settlement profiles for the compacted dams at the end of construction are consistent with simple theoretical models (e.g., Pagano et al., 1998), showing maximum movements occurring at an elevation of approximately one-third of the crest height. In three of the four cases, the maximum settlements (5 – 22cm) are in good agreement with design estimates based on stiffness properties for the rockfills. However, measured settlements (72cm) in the Odeleite dam were much larger than predicted. The Authors attribute this to weathering of the meta-greywacke fill material and its greater sensitivity to water. All four post-1992 dams exhibited small settlements during first filling (1.0 – 5.6cm) and continue to settle at a rate ranging from 0.1 – 1.0 cm/yr in subsequent operations (the lower range applying to cases of the Lagoacho and Apartadura and the higher end to Odeleite and Arcossó).

There is minimal data on the construction settlements of the older, pre-1965 dams. However, Pego do Altar underwent significant movements during first filling causing cracking of expansion joints and significant leakage (crest settlements reached more than 30cm). Both of the other two dams continue to undergo significant rates of settlement despite their age. The highest dam, Paradela (108m) has been repaired several times to address leakage problems due to the continuing rockfill deformations. The Authors report more than 12 cm of crest settlement at Paradela since 1980, with settlements continuing at a rate of approximately 0.6cm/yr. They also note that more movement occurs during the rainy winter season.

Okamato presents a summary of the measured permanent/residual displacements of rockfill dams due to strong earthquake shaking. None of the records show problems of overflow due to settlements of the rockfill, however, at least two dams (Cogoti and Minasi) developed significant leakage due to cracking of the upstream concrete membrane. The Author points out that most cases where significant residual deformations occurred involve older dams with high lift dumped rockfill, and almost no cases involving well compacted rockfill in dams built since 1970. He also reports a compilation of data on the construction settlement-crest height ratios for rockfill dams in Japan and elsewhere. The final section gives recommendations on allowable residual settlements to avoid overflow or leakage through rockfill dams.

Okamoto also presents results of centrifuge model tests that simulate the behavior of loose and dense rockfill dam under seismic loading. The Author summarizes the measured crest settlement as functions of the maximum basal acceleration and side slope angle and proposes empirical equations to fit these data. It is difficult to appreciate these results due to lack of information on the experiments such as the selection of scaled rockfill material etc.

The paper by Athanasiu et al. summarizes very extensive experience in modeling creep settlements of rockfills. The Authors propose phenomenological equations to characterize the creep strains observed in laboratory tests. They distinguish between an initial diffusion phase ($t \leq t_d$) and a long term phase where creep strains are proportional to log t ($t > t_d$) (after Marsal, 1973).

The examples presented in the paper all use a simplified modeling framework based on an initial field of elastic stresses and 1-D integration of vertical strains to compute settlements. The creep equations require a total of 5 model parameters. Ranges of these parameters have apparently been evaluated from detailed back-analyses of projects such as the Troll Onshore plant (Kollsnes). The Authors show very good agreement between computed and measured creep settlements from 17 rockfill dams using a single long term creep parameter, β. However, they do not show how well this back-fitted parameter relates to measured creep behavior in laboratory tests. There is no discussion of the effects of pore water on creep properties and no indication how measured settlements in the field cases are related to inundation or other groundwater conditions.

The Authors propose a simple extension of their 1-D settlement model to superimpose creep deformations due to self-weight of the rockfill and stresses caused by foundation loads. These calculations appear to ignore the diffusional component of creep? They present example calculations for the Orman Lange terminal.

The papers by Roosta and Tabibnejad and Pakbaz and Zolfagharian present back analyses to interpret monitoring data from two large rock fill dams in Iran:

Roosta and Tabibnejad analyze data from 170m high Maroon dam over a period of 6.8 yrs (including a construction phase of 2.7 yrs). The dam is founded on hard limestone and has an inclined central clay core. The structure is extensively instrumented with inclinometers in the core and rockfill shoulders as well as piezometers and total earth pressure cells in the core. The data show settlements exceeding 2m during construction and reaching 2.3m at the end of the monitoring period. The maximum settlements occur at an elevation of 90-100m in the clay core.

The Authors use a commercial finite difference program (FLAC4) to simulate flow and deformation in the dam, with linear elastic and Mohr Coulomb yield properties for all soil and rock fill materials. The Authors use measurements of pore pressures to calibrate permeability properties of the clay core and artificially adjust the bulk stiffness of the pore water to approximate conditions of partial saturation during dam construction. The remaining elastic stiffness and shear strength properties are back-fitted to measurements of deformations from a series of 4 inclinometers. They also compare predictions of effective stress paths in the core with measurements from stress cell data, but do not explain what assumptions were used to obtain these data.

The analyses provide a reasonable first order match to the measured settlement distributions at the end of the monitoring period, but do not explain more detailed aspects of performance such as the magnitudes of post-construction movements.

Pakbaz and Zolfagharian describe similar comparison between the measured and computed performance during construction of the 123m high Gavoshan dam. This clay core dam is also very well instrumented with multiple lines of inclinometers, and piezometers and earth pressure cells at multiple locations within the clay core. The Authors use a commercial finite element program (Plaxis) to compute deformations, stresses and pore pressures using Mohr-Coulomb and linear elasticity to describe the clay core and rockfill materials. The Authors do not give any details regarding the fitting of these parameters or the handling of consolidation within the (partially saturated) clay core.

Their results show excellent agreement between the computed and measured settlements during the 4yr construction period (through February 2003), with maximum settlements of approximately 2.1m at mid-height of the core (i.e., very similar to data for Maroon dam). The measurements of pore pressures and total stresses in the core ($r_u = u/\gamma H$, $A_r = \sigma_v/\gamma H$) are consistent with ranges of behavior reported in the literature. The analyses underestimate significantly the measured pore pressures towards the edge of the core, but are in good agreement with the vertical total stress (measured by earth pressure cells). Therefore, it is unclear how useful the back-analyses are in evaluating stress conditions within the core?

Al-Damluji and Ziboun have also used FE analyses to analyze the performance of a large zoned earth dam, Llyn Brianne, that was built in Wales in the 1960's (Penman & Charles, 1972). The Authors introduce more complex models of soil behavior including: i) hyperbolic (Duncan-Chang) model for the rockfill shoulders and foundation bedrock materials; ii) bounding surface critical state plasticity model for the clay core; and iii) an endochronic model for the filter materials. The hyperbolic model has been widely used in modeling compacted fills (e.g., Marachi et al., 1969; Duncan, 1996) and ranges of recommended parameters have been published. However, the Authors give few details of the other two models and do not indicate how the input parameters were obtained. There is also no information on the modeling of the partially saturated conditions in the clay core.

The Authors show very good agreement between computed and measured settlements (up to 1.2m) during construction of the 90m high dam. They also report excellent agreement with lateral displacements (up to 1.0m), which are generally much more difficult to predict. Finally they report computed pore pressures for a series of vertical sections over a period of 3 years (both during and after construction). No data are provided for comparison and no information is given regarding the history of reservoir filling.

The paper by Scheid et al. summarizes the results of a safety inspection and stability assessment for a 20 year old arch dam, Foum Gleita, in Mauritania. The dam has had effectively no maintenance and was clearly in need of some repairs (e.g., irrigation channel damaged by flash floods and local farmers!).

Although the Authors found that almost half of the monitoring instruments were no longer functional, they were able to recover data from a number of key piezometers in the underlying rock foundation and from vertical pressure gauges in both abutments of the dam (recorded on a local TELEMAC data logger). The results showed seasonal fluctuations associated with reservoir operations and long term stability in the pore pressures, with no evidence of potential problems due to uplift. Similarly, there has been little change in the horizontal stresses acting on either abutment and hence, no immediate stability concerns. However, a major crack between the left abutment and rock will need to be monitored.

The Authors highlight the need for much more careful monitoring (and maintenance) of the structure to ensure future safety. They recommend replacing defective instruments and training local personnel on the functioning of these devices and the importance of careful monitoring.

6 CONCLUSIONS

In preparing this General Report, I have tried to provide a complete and succinct summary of the papers presented to Session 2f, highlighting new findings, themes of common interest etc. Inevitably, this process has been influenced by my own personal background and experience. I have attempted to provide objective comments on all contributions, but apologize in advance to the Authors if there are any mis-representations or errors in my interpretation of their work.

In the course of preparing this report, I have identified several items which I hope will be discussed more extensively at the conference session:

1. How do the recent advances in understanding the behavior of soil and rock materials (such as the weathering and creep behavior of rockfill, behavior of partially saturated compacted clays) affect the design, condition assessment and predicted performance of dams and embankments?
2. Although ground improvement technologies are widely used in practice to address problems of soft ground construction. Do any of the contributing panelists have well documented cases confirming the design assumptions of ground improvement 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 material properties of aged structures; and b) design for dynamic and static environmental loading conditions (seismic, transient infiltration etc.) conditions.
4. What is the appropriate role of back-analyses? What lessons are learned from back-analyzing dams and embankments?
5. How are recent advances in sensor and communication technologies to be used in monitoring the construction and performance of embankments and dams? What are the possibilities for large scale automated monitoring such as that proposed for dikes in the Netherlands?

ACKNOWLEDGMENTS

I would like to thank Matthieu Vandamme for help in translating the two papers written in French, Dr John Christian for input on stochastic methods and Prof. Kazuyoshi Tateyama for background information on the dynamics of roller compaction.

REFERENCES

Alonso, E.E., Gens, A. & Josa, A. (1990) "A constitutive model for partially saturated soils," *Géotechnique*, 40(3), 405-430.

Asaoka, A. (1978) "Observational procedures for settlement prediction," *Soils and Foundations*, 18(4), 87-100

Asaoka, A., Nakano, M., Noda, T., Kaneda, K. (2000) "Delayed compression/consolidation of natural clay due to degradation of soil structure," *Soils & Foundations*, 40(3), 75-85.

Atkinson, B.K. & Meredith, P.G. (1987) "The theory of sub-critical crack growth with applications to minerals and rocks," *Fracture Mechanics of Rock*, B.K. Atkinson, Ed., Academic Press, 111-166.

Bishop, A.W. (1955) "The use of slip circle in the stability analysis of slopes," *Géotechnique*, 5(1), 7-17.

Brinkgreve, R.B.J. & Bakker, H.L. (1990) "Non-linear finite element analysis of safety factors," *Proc. 7th Conf. On Computational Methods and Advances in Geomechanics*, Balkema, 1, 117-122.

Duncan, J.M. (1996) "State-of-the-art: Limit equilibrium and finite element analysis of slopes," *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 122(7), 577-596.

Hunter, G. & Fell, R. (2003) "Rockfill modulus and settlement of concrete face rockfill dams," *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 129(10), 909-917.

Lane. P.A. & Griffiths, D.V. (2000) "Assessment of stability of slopes under drawdown conditions," *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 126(5), 443-450.

Lawton, E.C., Fragaszy, R.J. & Hardcastle, J.H. (1989) "Collapse of compacted clayey sand," *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 115(9), 1252-1267.

Magnan, J-P., Mieussens, C. & Queyroi, D. (1983) "Etude d'un remblai sure sols compressibles: Le rembali B du site expérimental de Cubzac-les-Ponts," Rapport de Recherche LPC No. 127.

Maia, P.C.A., Sayão, A.S.F.J., & Nunes, A.L.L.S. (2003) "Evaluación experimental de la alteración de escolleras," *Proc. 12th Pan-American Conference on Soil Mechanics and Geotechnical Engineering*, P.J. Culligan, H.H. Einstein & A.J. Whittle, Eds., Verlag Glückauf, 1, 789-794.

Marachi, N.D., Chan, C.K., Seed, H.B. & Duncan, J.M. (1969) "Strength and deformation properties of rockfill materials," Research Report TE69-5, University of California, Berkeley.

Marsal, R.L. (1973) "Mechanical properties of rockfill," *Embankment Dam Engineering* (Casagrande Volume), R.C. Hirschfeld S.J. Poulos, Eds., Wiley, NY, 109-200.

McCuen, R.H., Rawls, W.J. & Brakensiek, D.L. (1981) "Statistical analysis of the Brooks-Corey and the Green-Ampt parameters across soil textures," *Water Resources Research*, 17(4), 1005-1013.

Mesri, G. 91994) "Settlement of embankments on soft clays," *Vertical and Horizontal Deformations of Foundations and Embankments, ASCE GSP No. 40*, 1, 8-56.

Oldecop, L.A. & Alonso, E.E. (2001) "A model for rockfill compressibility," *Géotechnique*, 51(2), 127-139.

Oldecop, L.A. & Alonso, E.E. (2003) "Suction effects on rockfill compressibility," *Géotechnique*, 53(2), 289-292.

Olivella, S., Gens, A., Carrera, J. & Alonso, E.E. (1996) "Numerical formulation for a simulator (CODE_BRIGHT) for the coupled analysis of saline media, *Engineering Computations*, 13(7), 87-112.

Pagano, L., Desideri, A.& Vinale, F. (1998) "Interpreting settlement profiles of earth dams," *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 124(10), 923-932.

Penman, A.D.M. & Charles, J.A. (1972) "Observed and predicted deformations in large embankment dam during construction," *Proc. Inst. Civil Engineers*, 49, 1-21.

Penman, A.D.M. & Charles, J.A. (1976) "The quality and suitability of rockfill used in dam construction," *Dams and Embankments*, BRE, The Construction Press, 6, 72-85.

Saye, S.R. (2001) "Assessment of soil disturbance by the installation of displacement sand drains and prefabricated vertical drains," *Soil Behavior and Soft Ground Construction (Ladd Volume),* J.T. Germaine, T.C. Sheahan & R.V. Whitman, Eds., ASCE GSP 119, 325-362.

Seed, H.B., Idriss, I.M., Makdisi, F. & Banerjee, N. (1975) "Representation of irregular time histories by equivalent uniform stress series in liquefaction analysis," EERC Report 97-29, University of California, Berkeley.

Taylor, D.W. (1948) *Fundamentals of Soil Mechanics*, Wiley & Sons, NY (p 414-417).

Tateyama, K. (1996) "A study on the vibration behavior of vibratory rollers during compaction," *Proc. Japanese Society of Civil Engineers*, No. 554/II-37, 231-237.

Van Baars, S. (2005) "The horizontal failure mechanism of the Wilnis peat dyke," *Géotechnique*, 55(4), 319-323.