Effects of a thin liquefiable foundation layer on deformations of a rockfill dam subjected to earthquake shaking

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ABSTRACT
Seismic safety of embankment dams is affected by dam crest displacements and the intactness of the seepage control system. This paper describes the results of dynamic analyses carried out for a 85-m high zoned rockfill dam retaining tailings founded on liquefiable alluvial soils underlain by stiff residual soils and bedrock. The assessment was carried out for dam closure conditions and MDE ground motions (i.e. PGA = 0.33g) using a coupled stress-flow method of analysis. The constitutive model *UBCSAND* was used to model the liquefiable soils and *UBCHYST* was used to model the nonlinear behaviour of non-liquefiable materials within the dam-foundation system. The results show that the presence of a thin liquefied alluvium leads to a rigid-block type deformation of the dam with insignificant loss of freeboard. Seismic instability is not predicted due to the high shear strength offered by the U/S and D/S rockfill shells.

RESUME
La sécurité sismique des barrages en matériaux meubles est influencée par les déplacements à la crête du barrage ainsi que par le caractère intact des systèmes de contrôle d'infiltration. Cet article décrit les résultats des analyses dynamiques effectués sur une digue à rejet minier, en enrochement, de 85m de haut, fondée sur des sols alluviaux liquéfiables, sous-jacents à un sol résiduel ferme ou sur un substratum rocheux. Cette évaluation a été modélisée lors de la fermeture et avec des paramètres de mouvement de sol (AMS = 0.33g), en utilisant la méthode d'analyse d'écoulement des lignes de force. Le programme numérique UBCSAND a été utilisé pour modéliser les sols alluviaux liquéfiables et le programme numérique UBCHYST, pour modéliser le comportement non linéaire des sols non liquéfiables dans les fondations du barrage. Les résultats démontrent que la présence d'une mince couche de sol alluvial liquéfiable conduit à une déformation de type « bloc rigide » du barrage avec des pertes de revanche négligeables. L'instabilité sismique ne peut pas être prédite due à la résistance élevée au cisaillement qu'offrent les parois en enrochement, en aval et en amont du barrage.

1. INTRODUCTION

The seismic safety of embankment dams is controlled by the permanent deformations induced as a result of strong shaking and the intactness of the seepage control system. The performance of well-engineered rockfill dams during past earthquakes indicates that liquefaction of both foundation soils and soils comprising the dam play a key role in the stability of the dam (Seed 1979; Marcuson et al. 1996 and 2007). The failure of the Sheffield dam during the 1925 Santa Barbara Earthquake (Seed et al. 1969) and the near catastrophic failure of the San Fernando Dams during the M6.5 earthquake in February 1971 (Seed et al. 1975 and Castro et al. 1985) due to soil liquefaction are classic examples of water storage dam failures. The latter is the most documented failure (e.g. Seed 1979; Seed et al. 1988; Castro et al 1990, Beaty and Byrne, 2001 and Li and Ming, 2004 among others) in the history of earth dams, in which the crest of the Upper San Fernando dam moved downstream about 1.5m along with a flow failure of the upstream slope of the Lower San Fernando dam, about 1 minute after strong shaking had stopped (Seed et al. 1975).

The incidents in tailings dams and impoundments are more frequent (ICOLD 2001). Data shown in Figure 1 illustrates that earthquakes are one of the major causes of failure of tailings dams (USCOLD, 1994). In most cases, damage has occurred as a result of a large drop in the stiffness and strength of soil resulting from liquefaction. Two tailings dams within the Mochikoshi gold mining complex in Japan failed due to liquefaction of tailings from the 1978 Izu-Ohshim-Kinkai earthquake. One dam failed at the end of earthquake whereas the other one failed about 24 hours later. Information on the failure of Mochikoshi tailings dams can be found in a number of publications (e.g. Ishihara 1984).

In general, two types of factors control the response of an earth structure to earthquake excitation, namely:

- Mechanical conditions
- Flow conditions

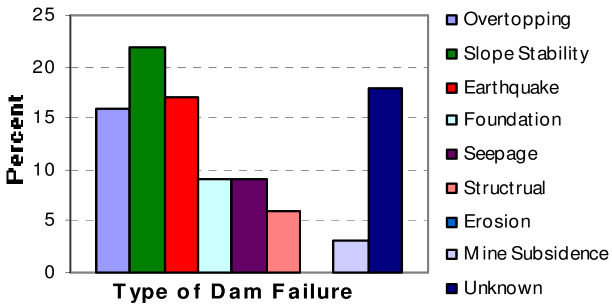

Figure1. Statistics of failure causes of tailings (data from USCOLD 1994).

Mechanical conditions include soil density, stiffness and strength, static stress state, and earthquake characteristics (amplitude, duration, etc.) and are responsible for the generation of excess pore pressures during cyclic loading. Flow conditions include drainage path, soil hydraulic conductivity (permeability) and its spatial variation within the soil medium (permeability contrast) and are responsible for the distribution of excess pore pressures both during and after earthquake shaking. Detailed discussions regarding these factors may be found in Seid-Karbasi and Byrne (2006a and 2007). Although, notable advancements have been made over the past several decades in understanding the mechanism of soil liquefaction, much of the progress was limited to assessing the likelihood of triggering liquefaction focusing primarily on the mechanical conditions.

From an engineering point of view, the earthquake-induced deformations are often the prime concern if significant zones of foundation soils and/or soils comprising the dam body are prone to liquefaction. To accurately predict the induced deformations, it is necessary to employ a coupled stress-flow analysis procedure. This paper describes the results of a coupled stress-flow dynamic analysis procedure used to predict seismic deformations and stability of an 85 m high earth dam. The procedure captures the sand element behaviour observed in cyclic laboratory tests, and has been verified by comparison with physical model tests and field experience. The dam is to be constructed to retain tailings during mining and also following closure. The fully coupled stress-flow analysis has been carried out using the constitutive model UBCSAND that captures the liquefiable soil response. The nonlinear behaviour of non-liquefiable materials was accounted for by employing the model UBCHYST. These user-defined models are incorporated into the commercially available computer code FLAC (Itasca, 2005). The dam specifications, foundation characteristics and predicted behaviour are discussed in the following sections.

2. DAM DESCRIPTION AND SITE CONDITIONS

The Nui Phao complex is a tungsten poly-metallic mine located in the Dai Tu District approximately 80 km northwest of Hanoi, the capital of Vietnam. The open pit mine consists of several impoundments and dams for tailings (e.g. sulphide and oxide) and water retention/storage. According to Canadian Dam Association Guidelines (CDA 2007), the Oxide dam is classified as a “High Consequence” dam that requires analysis of stability following closure using MDE ground motions.

The 85 m high Oxide dam with a crest length of 720 m is a zoned rockfill dam founded on 17 m thick layers of residual and alluvial soils. The configuration of the dam is shown in Figure 2. As may be seen from Figure 2, the dam has an inclined central core constructed out of clayey soil and protected by natural and fabric filters, respectively at the downstream and the upstream sides. The dam has a relatively wide crest (26 m) and is comprised of rockfill shells quarried from open pit mining with slopes of 1.3H:1V and 2H:1V at upstream and downstream, respectively. The dam is raised in seven stages to reach the closure stage elevation (at 140 m). The upstream slope of the dam at the first stage (up to elevation 75 m) is 2H:1V (see Figure 2). The downstream rockfill shell in the outer parts is mainly constructed out of non-acid generating rock (NAG) in order to meet the

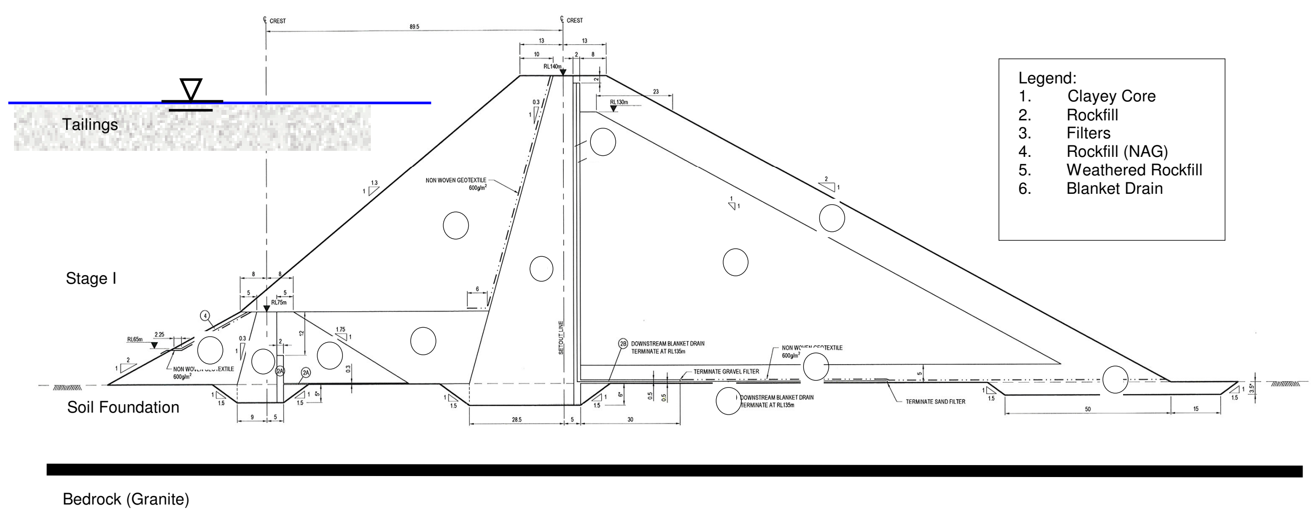

Figure 2. Typical maximum cross-section of Oxide tailings dam with different zones at closure stage.

environmental requirements of the project.

The clayey core is comprised of fine-grained soil obtained from an open pit mine developed in residual soil (Saprolite) and consists mainly of a low to medium plastic clay classified as CL to ML according to the Unified Soil Classification System. The rockfill shells are constructed out of waste rock from mining works and consist of relatively strong Granite rock.

The impoundment of Oxide tailings primarily consists of silt size materials (50 to 60%) and fine sands (≈ 40%) which will be raised up to elevation of 135 m providing a 5-m freeboard.

3. SUBSURFACE FOUNDATION CONDITIONS

The subsurface conditions at the Oxide dam site have been established based on geotechnical investigations including boreholes in the valley beneath the dam body and test pits. The investigations included penetration resistance measurements (SPT) and permeability tests and also collection of disturbed and undisturbed samples for laboratory testing; i.e. gradation, plasticity, consolidation and undrained isotropic consolidated triaxial (UIC) testing. The data indicate that the Oxide dam foundation is underlain by three distinct soil stratigraphic units (Golder 2006a):

Unit 1: 3 to 4 m thick layer of alluvial soils primarily consisting of silty sand.

Unit 2: 14 m thick layer of fine-grained residual soil (Saprolite).

Unit 3: Granite bedrock.

The characteristic behaviour and interpreted material parameters of soils comprising the different stratigraphic units are described below.

3.1 Unit 1: Alluvial Soil

This soil unit (silty sand) is a coarse-grained material with variable fines content that can be described as very loose to compact (or medium dense) based on SPT data. These soils are susceptible to pore pressure generation and/or liquefaction during earthquake loading. This layer has been assigned a characteristic normalized SPT N-value of $(N_1)_{60\text{-}cs}$ = 8 blows/0.3 m in accordance with the recommendations given in NCEER97 (Youd et al. 2001) accounting for overburden (K_σ) and fines content corrections. As can be seen from Figure 2, this material will be sub-excavated from the toe area of the dam over a horizontal distance of about 65 m prior to dam construction to form a shear key.

3.2 Unit 2: Fine-Grained Residual Soil (Saprolite)

Unit 2 is a residual soil (Saprolite) formed as a result of weathering of bedrock. This unit consists of fine-grained soil and is about 14 m thick at the center of the valley beneath the maximum dam section. The soils comprising this unit are classified as silty clay (CL) to clayey silt (ML) with variable sand portions. According to field N-values, the in-situ consistency is inferred to increase with depth from stiff to hard.

The Atterberg limits of soil samples taken from this layer indicate a liquid limit varying from 26 to 64% and a plastic limit ranging from 16 to 35%. The plasticity index varies from 8% to 31% with an average of 22% (Golder, 2006). With a PI > 7%, this layer is expected to behave as a clay-like material when subjected to strong shaking (ref. Boulanger and Idriss, 2006). The natural water content rages from 12 to 56% that is below the liquid limit indicating a low to moderate sensitivity. According to the liquefaction susceptibility criteria recently suggested by Boulanger and Idriss (2006) and Bray and Sancio (2006) this layer is not expected to generate excess pore pressure and is therefore not susceptible to liquefaction.

3.3 Unit 3: Granite Bedrock

Bedrock was encountered at an average depth of about 17 m below the current ground surface. The bedrock is moderately fractured with an RQD ranging from 20% to in excess of 90%. The unconfined compressive strength of the rock samples tested ranged from 65 MPa to 180 MPa (Golder, 2006a). It is understood that a material similar to Unit 3, sourced from the open pit mine operation, will be one of the sources of rockfill during dam construction.

4. SITE SEISMICITY AND EARTHQUAKE MOTIONS

Deterministic seismic hazard analyses have been carried out to establish the MDE ground motions. MDE motions corresponding to the 84th percentile attenuation relations were established for the seismic analysis of dam closure. The MDE scenario was determined to be an M7.2 earthquake producing a peak firm-ground acceleration of 0.33 g.

Five spectrally-matched ground motions were derived for the deterministic response spectra established for the MDE (Golder 2006b). Figure 3 shows the target response spectrum and a typical earthquake record used in the analysis.

5. SAND LIQUEFACTION AND DRAINAGE

Earthquake-induced soil liquefaction results in a sudden loss of shear strength and stiffness due to seismic shaking. The loss arises from the tendency of granular soils to undergo volume change when subjected to cyclic loading. When the volume change is contractive and prevented by the presence of pore water that cannot escape in time, the pore water pressure will increase and the effective stress will decrease. If the effective stress drops to zero (100% pore water pressure rise), the shear strength and stiffness will also drop to zero and the soil will behave like a heavy liquid and is said to have liquefied.

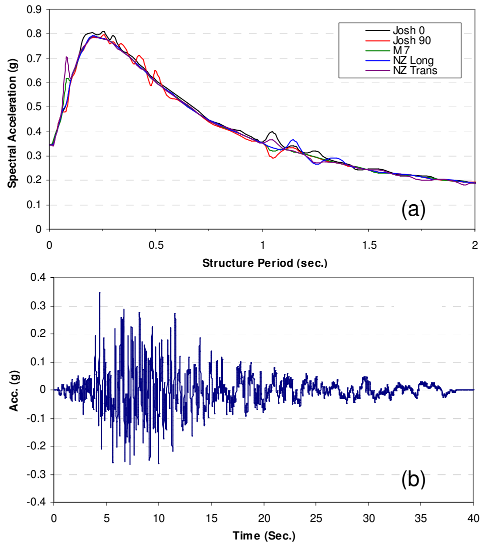

Figure 3. Earthquake motions, (a) target firm-ground response spectrum (5% damping), (b) typical acceleration time history used

The stability of a saturated slope under seismic loads will depend on whether soil liquefaction will be triggered and what level of shear strength and stiffness loss would occur, which in turn depends on the rate of pore pressure generation due to seismic shaking and pore pressure dissipation due to drainage. The potential for large lateral displacements or flow slides will be high if a low permeability layer (e.g. a silt or clay layer) within a soil deposit forms a hydraulic barrier and impedes drainage. This may result in the formation of a thin layer of soil with near-zero shear strength as shown by Seid-Karbasi and Byrne (2007).

6 ANALYSIS PROCEDURE

The response of the dam-foundation and impounded tailings to earthquake shaking has been analyzed in two stages. In the first stage, the dam body and its foundation has been analyzed under gravity loads (static mode) with drained conditions to establish the pre-earthquake stress state. Thereafter, the analysis was switched to the dynamic mode with undrained properties for fine-grained soils (i.e. dam core and Saprolite foundation).

The analysis outlined above has been conducted using the computer program *FLAC* (*Version 5.0*, *ITASTCA,* 2005). This is a commercial, finite difference analysis code capable of coupled stress-flow analysis under static and dynamic loading conditions.

The earthquake motions were applied as a time history of excitation at the model boundaries. As the earthquake motions were "outcrop" records (i.e. without any overburden effects) and the dam is not founded directly on bed rock, the earthquake motions were converted to "within" motion. This process was undertaken by site response analysis employing the 1D computer code *ProShake* (*Civil Systems*, 2001) and taking into consideration the modulus reduction and damping behaviour of the foundation soils as documented in the literature (i.e. Seed and Idriss 1986; Vucetic and Dobry 1991 and Yasuda et al. 1993 & 2003).

To model sandy soil behavior and account for shear induced (excess) pore pressures and the effects of pore pressure redistribution and dissipation during and after seismic shaking, a dynamic coupled stress-flow analysis was employed. In such an analysis, the volumetric strains are controlled by the compressibility of the pore fluid and flow of water through the soil elements. An effective stress-based elasto-plastic constitutive model (UBCSAND) was used in the coupled dynamic analysis. The model has been calibrated using laboratory test data as well as centrifuge data and is described below.

6.1 Constitutive Model for Sands

The *UBCSAND* constitutive model is based on the elasto-plastic stress–strain model proposed by Byrne et al. (1995), and has been further developed by Beaty and Byrne (1998) and Puebla (1999). The model has been successfully used in analyzing the CANLEX liquefaction embankments (Puebla et al., 1997) and predicting the failure of Mochikoshi tailings dam (Seid-Karbasi and Byrne 2004). It has also been used to examine partial saturation conditions on liquefiable soil response (Seid-Karbasi and Byrne, 2006b) and to predict centrifuge model tests under dynamic loading (e.g. Byrne et al., 2004 and Seid-Karbasi et al., 2005). It is an incremental elasto-plastic model in which the yield loci are lines of constant stress ratio ($\eta = \tau / \sigma'$). Plastic strain increments occur whenever the stress ratio increases. The flow rule relating the plastic strain increment direction is non-associated and leads to a plastic potential defined in terms of dilation angle. Plastic contraction occurs for stress ratios below the constant volume friction angle, and dilation above as shown in Figure 4. A detailed description of the model may be found elsewhere (e.g. Byrne et al., 2004 and Puebla et al., 1997).

The constitutive behaviour of sand is controlled by the soil skeleton. The pore fluid (e.g. water) within the soil mass acts as a volumetric constraint on the skeleton if drainage is fully or partially curtailed. This model has been incorporated into the *FLAC* code as a user-defined model.

The key elastic and plastic parameters for the *UBCSAND* constitutive model were developed based on empirical correlations between the parameters and relative density, D_r, or normalized Standard Penetration Test values, $(N_1)_{60}$. The model parameters were chosen so as to simulate the behavior of sand under monotonic and cyclic loading conditions as observed in the laboratory (e.g. Byrne et al., 2004 and Seid-Karbasi et al. 2005). Currently, the constitutive model can be used to simulate the behavior of soils with a range of relative density or N values. The model has also been calibrated to reproduce the NCEER 97 (Youd et al. 2001) liquefaction-resistance chart, which in turn is based on field data collected from past earthquakes and is expressed in terms of Standard Penetration Test, $(N_1)_{60}$ values.

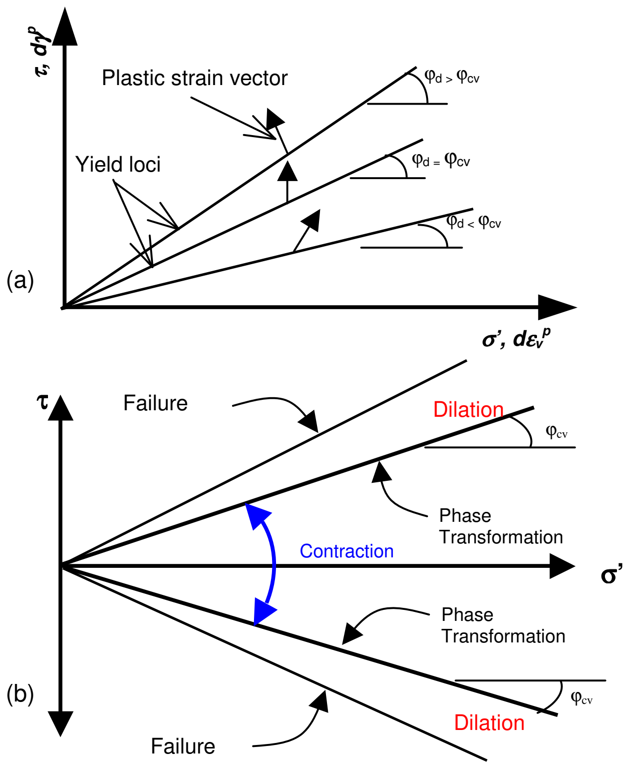

Figure 4. (a) moving yield loci and plastic strain increment vectors, (b) dilation and contraction regions.

6.2 2-D Model and Input Parameters

The 85 m high dam and 17.5 m of foundation soils overlying bedrock were discretized using a finite difference grid comprising 70 x 185 zones in the vertical and horizontal directions, respectively. The nominal thickness of the zones was 1.5 m. The model extended about 200 m from the upstream and downstream toe areas of the dam over a total length of 700 m.

Figure 5 shows the *FLAC* finite difference model along with different material zones considered in the analysis. The analysis was initiated with an elastic model as a first step to define the stress-level dependent parameters, and a Mohr-Coulomb model in the second step. For the elastic analysis, the small-strain modulus, G_0, of the clay core was approximated using correlations with void ratio developed by Hardin and Drnevich (1972). The Poisson's ratio,μ, was taken as 0.33.

The strength parameters of the rockfill shells were estimated using the method proposed by Barton and Kjaersnly (1981). The estimated parameters are summarized in Table 1. The parameters are comparable to those of Leps (1971) and Saboya and Byrne (1993) reported for similar rock-fill dams.

The small-strain shear modulus, G_0 and friction angle, φ were estimated based on correlations found in the literature (i.e. Seed and Idriss 1970; Leps 1971; Seed et al. 1985; Uddin and Gazetas 1997 and Barton and Kjaersnly 1981) expressed as $G_0 = 21.7\ (K_{2\text{-max}})\ P_a\ (P'/P_a)^{0.5}$ and $\varphi = \varphi_1 - \Delta\varphi \log(P'/P_a)$. Where, $K_{2\text{-max}}$, P_a, P', φ_1 and $\Delta\varphi$ are: stiffness parameter, atmospheric pressure (100 kPa), effective mean stress, reference friction angle (at $P' = P_a$), and friction reduction for every log cycle of stress level increase, respectively.

As noted earlier, the fine-grained soils were modeled with drained parameters under static loading conditions. Their strength was switched to undrained strength parameters for earthquake loading.

The undrained strength, S_u of the (undisturbed) samples of Saprolite soil in laboratory triaxial tests (CIU) showed a strength ratio (S_u/σ_v') of 0.7 that is much greater than for a normally consolidated clay. Therefore, this layer was modelled as an over-consolidated clay. The design line for S_u as a function of the overburden stress and over-consolidation was established using Ladd and DeGroot (2004) suggestion that is based on the *SHANSEP* concept (Ladd and Foott, 1974) as: $S_u/\sigma_v' = 0.25\ (OCR)^{0.9}$

The SHANSEP concept was employed to define S_u for the Saprolite foundation with respect to the static overburden pressure both in the free-field and after dam construction.

The hydraulic conductivity (permeability) of the various soil units were assigned based on in-situ and laboratory test data for different zones of the dam-foundation model and the values used are listed in Figure 5.

UBCSAND was used to model the constitutive behavior of the sandy soils (i.e. alluvium and tailings) and the Mohr-Coulomb model was applied to the clayey soils (i.e. cores and Saprolite foundation) and rockfill shells. The nonlinearity and energy dissipation mechanism of the clayey soils and other non-liquefiable materials during dynamic excitation was modeled using *UBCHYST* model that is developed by Byrne (2006) to simulate the hysteretic material damping in non-liquefiable soils.

7. RESULTS AND DISCUSSION

The dynamic analysis was carried out following establishing the equilibrium conditions under gravity loading (static condition) and steady-state flow conditions in order to establish the pre-earthquake stress state within the dam and foundation. The earthquake motions were applied at the base of the model thereafter to simulate seismic loading conditions. For purposes of discussion, the results of analyses carried out with one earthquake time-history are presented herein.

The response of the dam-foundation system in terms of (maximum) excess pore pressure ratio, $R_{u\text{-}max}$ generated in the model over the excitation time is depicted in Figure 6. The results indicate that the tailings and downstream free-field alluvial soil develop high excess pore water pressures and liquefy ($R_{u\text{-max}} \approx 100\%$)

during strong shaking. The alluvial soils beneath the dam body in the downstream exhibited a different response from those upstream. The alluvial soils underlying the downstream shell generated limited excess pore pressures (equivalent to an $R_{u\text{-}max} < 0.4$). This difference is due to higher vertical effective stress (σ'_{v0}) in the downstream dam (as a results of phreatic surface drop) and also greater shear stress bias in the downstream

The results also suggest that permanent settlements of the dam crest are low with respect to the available freeboard. Therefore, it is considered unlikely that the tailings will be released through the crest by overtopping.

Based on the block-type behaviour predicted for the MDE ground motions, it is possible that the core-abutment interface may be subjected to some straining and possible separation endangering the water tightness of the interfaces.

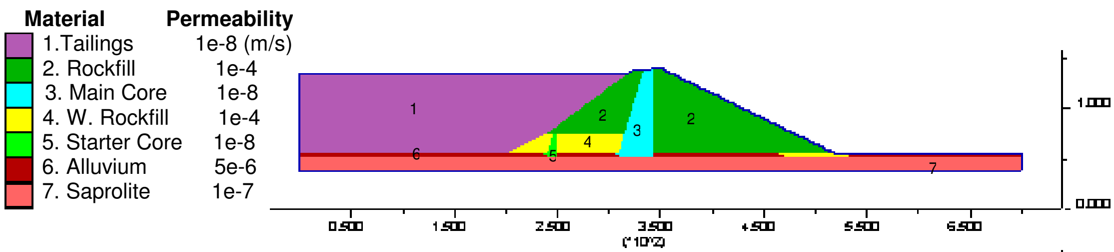

Figure 5. Various soil types and permeability values used in modeling.

Table 1: Material Properties Used in FLAC Analyses.

Soil Type	Saturated Unit weight (kN/m^3)	φ' (deg.)	Δφ[1] (deg.)	G[2&3] (kPa)	B (kPa)	$(N_1)_{60}$[4]	S_u/σ'_v[5]
TAILINGS	2000	---	---	---	---	5	---
Shells, ROCKFILL	2200	45	5.5	3.6e4	9.6e4	---	---
Dam CORE	2000	32	---	(1000Su)/9	2.67G	---	0.25
Weathered ROCKFILL	2200	42	5.5	3.0e4	8.0e4	---	---
ALLUVIUM	2000	---	---	---	---	8	---
Foundation SAPROLITE	2200	34	---	(750Su)/9	2.67G	---	Var.

1) Rock fill strength parameters were estimated based on Barton & Kjaernsli, (1981).
2) Stiffness at σ'_m=100 kPa, G_0, of rockfill was estimated based on $G_0 = 21.7(k_2)_{max}$. Pa . (σ'_m/Pa)^0.5 with $(k_2)_{max}$ = 150 and 125 for zones 2 and 4, respectively.
3) For fine-grained soils i.e. dam cores and Saprolite soil the small-strain shear modulus was estimated upon G_0/S_u = 1000 and 750 for core and foundation soil, respectively.
4) Alluvial foundation and tailings that are susceptible to seismic pore pressure generation were modelled with the user-defined constitutive model *UBCSAND*.
5) Applicable to fine-grained soils in dynamic condition, also used to estimate G_0

slope (stress reversal and dilation effects). The latter is consistent with the observations from centrifuge model tests and numerical modeling results reported by several investigators (e.g. Taboada and Dobry, 1995; Taboada et al. 2002 and Seid-Karbasi and Byrne 2004). Taboda et al. 2002 reported significantly lower R_u values beneath sloping ground compared to level ground conditions.

Figure 7 shows the horizontal displacement pattern of the FLAC model at the end of shaking. The results indicate that the dam body undergoes a permanent displacement of about 1.2 m. As may be seen from Figure 9b, the permanent displacements mainly occur through the loose alluvial soils.

Although extensive liquefaction of the tailings and alluvial soils outside the downstream toe of the dam were predicted, the results indicate that the seismic performance of the Oxide dam is satisfactory. Large permanent displacements are not predicted due to the high strength rockfill shells. The dam fulfills the expectations commonly considered for an MDE ground shaking level corresponding to closure conditions.

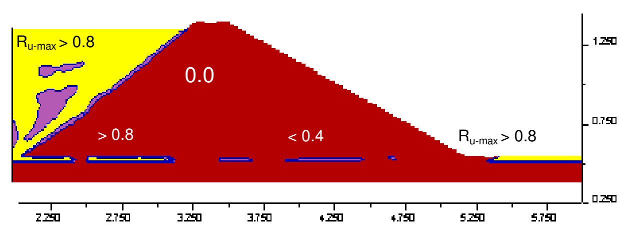

Figure 6. Distribution of $R_{u\text{-}max}$ in dam-foundation.

Therefore, with the alluvium present directly below the dam in most areas, it is prudent to use materials with higher plasticity to develop the areas near the interface of the core. Flaring the core close the abutments may also be considered to maintain the integrity of the drainage system.

8. SUMMARY AND CONCLUSIONS

The seismic stability of an 85-m high, zoned rockfill dam underlain by a thin liquefiable alluvial soil layer overlying a stiff to very stiff residual soil was investigated using an effective stress based, coupled mechanical-flow, dynamic analysis. The following is a summary of the key findings of the analyses:

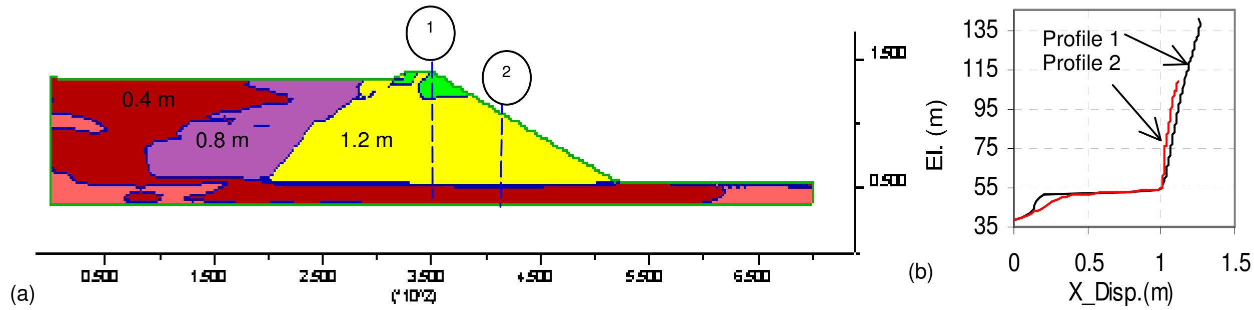

Figure 7. Dam-foundation deformation pattern, (a) x-disp. profile, (b) x-disp. profile at positions 1 and 2.

1) The presence of a thin liquefiable foundation layer is the primary cause resulting in block-type deformation of the Oxide dam under earthquake loading.
2) Rockfill shells with high shear strength and permeability enhance the dam performance to limit the permanent deformations.
3) The over-topping of tailings through the dam crest is unlikely leading to a low risk of failure via this mechanism.
4) The width of the downstream toe shear key is sufficient to reduce the displacements to an acceptable level.

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