BACKANALYSIS OF THE 1929 GRAND BANKS SUBMARINE SLOPE FAILURE

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ABSTRACT
The 1929 Grand Banks Slide was triggered by a major earthquake and resulted in a turbidity current that severed trans-Atlantic telegraph cables. Landslides of this kind can definitely be considered as a serious potential hazard to human life and economical resources along the coastline. Backanalyses of the Grand Banks Slide are performed by: a) conventional method of Limit Equilibrium, and b) numerical modelling utilizing a state-of-the-art finite element program, DYNAFLOW. The finite element program uses a multi-yield constitutive model and solid-fluid coupled field equations to simulate the behaviour of soil materials and consequently capture the build-up and dissipation of the excess pore water pressure. This research is part of COSTA-Canada, a contribution to the study of continental slope stability, aimed at increasing the reliability of economic activities along Canada’s continental margin and coastline.

RÉSUMÉ
La glissière de Grandes Banques en 1929 a été déclenchée par un tremblement de terre et a eu comme conséquence un courant de turbidité qui a divisé les câbles transatlantiques de télégraphe. Des glissières de cette sorte peuvent certainement être considérées comme des risques sérieux à la vie humaine et aux ressources économiques le long du littoral. Backanalyses de la glissière de Grandes Banques sont effectués par: a) méthode conventionnelle d'équilibre limite, et b) numérique modelage en utilisant un programme des éléments finis, DYNAFLOW. Cette recherche fait partie de COSTA-Canada, une contribution à l'étude de la stabilité de pente continentale, destinée pour augmenter la fiabilité des activités économiques le long de la marge continentale et du littoral du Canada.

1. INTRODUCTION

The 1929 “Grand Banks” earthquake on the continental slope (Figure 1) south of Newfoundland, Canada, induced landslides and widespread seabed slumps and initiated a turbidity current that severed trans-Atlantic cables and travelled for up to 1000 km (Piper et al., 1999). It also generated a tsunami that killed 27 people. The earthquake epicentre has been located on the western margin of St. Pierre slope (Piper at al., 1999). The region is still seismically active. According to National Earthquake Database (2000), there have been more than 50 seismic events in the last 30 years. With the current development of off-shore economic activities, especially oil and gas resources in the neighbourhood of the continental slope, a study of continental slope stability, such as COSTA – Canada, is of great importance.

In this study, results of conventional methods of slope stability analysis (i.e. Limit Equilibrium) are compared with finite element predictions using DYNAFLOW (Prevost, 1998). The focus is on capturing seismic liquefaction effects using current state-of-practice simplified procedure of excess pore water pressure (EPWP) estimation, pioneered by Seed (1979) and a state-of-the-art method using finite element analysis.

The analysis procedure and some preliminary results are presented in this paper. Detailed results will be presented at the conference.

2. SITE CHARACTERISTICS

Very few geotechnical data are available for the GB slope failure site; however, several studies have been performed concerning the geological aspects of the region (e.g. Piper at al. 1999 and Hughes Clarke, 1988).

Further processing of geological findings as well as conducting in-situ tests to obtain useful data for geotechnical analysis is necessary. According to aforementioned studies, two types of rotational slumps have occurred during and after 1929 earthquake: a) shallow failure (small scale): 2-5m thick, involving only Holocene sediments, and b) deep failure (larger scale): 5-30m thick, involving proglacial sediments.

The geological setting at one of the slope failure locations is described and summarized in Figure 2 and Table 1. Table 2 shows the soil strata assumed in this study. It should be emphasized that Table 2 is only a rough hypothetical assumption to present proposed method capabilities on the basis of available geological descriptions.

Moreover, the soil properties assumed in this study are taken from real soil data from an on-shore soil deposit with properties close to those described by Piper et al. (1999). A number of confidential laboratory and in-situ soil test results are available for the aforementioned soil deposit.

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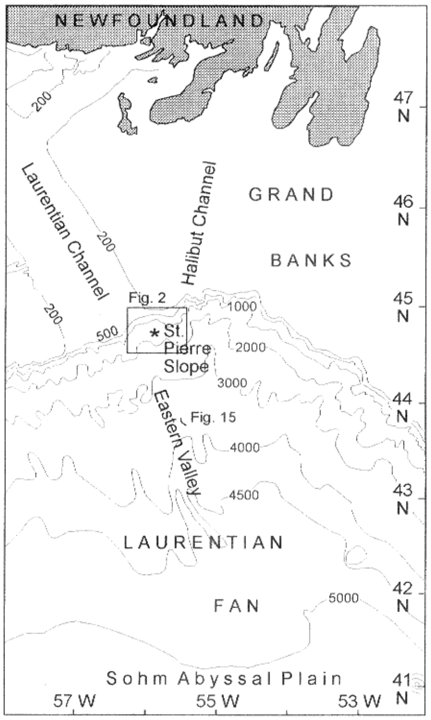

Figure 1. Regional map of 1929 Grand Banks earthquake and its epicentre (*) (Piper et al., 1999).

Generally, glacial deposits are among the most erratic with which the engineers have to deal with (Terzaghi et al., 1996). Glacial deposits may contain clay, silt or sand.
Proglacial refers to the area immediately adjacent to a glacier, often affected by outwash and by ice- or moraine-dammed lakes (Bates and Jackson, 1987). Diamicts are poorly sorted gravel-sand-mud deposits. The Holocene is the name given to the last 11,000 years of the Earth's history - the time since the end of the last major glacial epoch, or "ice age."

Table 1. Soil Strata

Depth (approximate average)	Description
0-5 m	Holocene mud
5-25 m	Proglacial sediments
Below 25 m	Diamicts and sands

Table 2. Presumed Soil Strata

Depth	Description
0-5 m	Clay
5-15 m	Silty Sand
15-30 m	Sand

Owing to the very large length vs. depth ratio of the domain of interest, the "infinite slope" assumption (i.e. 1-D analysis) is considered. This assumption is subsequently checked using 2-D, plane strain finite element analysis (see Figure 4).

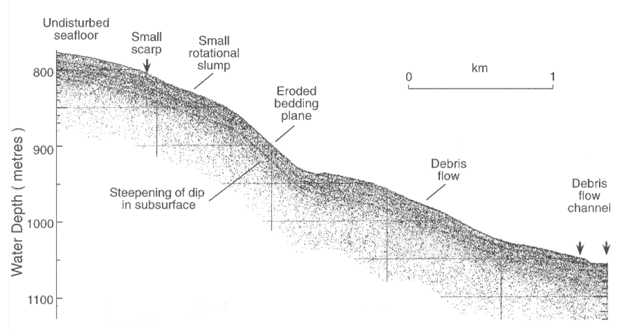

Figure 2. Selected profile, one of which have undergone sediment failure. (Piper et al., 1999)

3. CONVENTIONAL METHOD

Conventional method of slope stability analysis under earthquake loads is basically a pseudo-static analysis. Additional inertial forces are computed based on an assumed maximum seismic acceleration and applied as permanent forces rather than transient forces.

3.1 Limit Equilibrium Method

The extension of the limit equilibrium method, taking into account the effects of excess pore water pressure build-up, leads to the procedure described hereafter (e.g. Hadj-Hamou and Kavazanjian, 1985). In this approach to stability analysis of infinite slopes, the failure surface is assumed to be a plane parallel to the slope and Mohr-Coulomb failure criterion is applied. The Factor of Safety (F_s) is expressed by the ratio of available soil shear strength (τ_f) to the shear stress developed on the failure plane (τ):

$$F_s = \frac{\tau_f}{\tau} \qquad [1]$$

in which soil shear stress at failure is expressed in terms of effective parameters according to Mohr-Coulomb failure criterion:

$$\tau_f = c' + (\sigma - u)\tan\phi' \qquad [2]$$

where c' = soil effective cohesion, ϕ' = effective internal friction angle, σ = total stress (normal to the failure surface) and u = pore water pressure. Denoting the depth of the assumed failure plane by d, the slope inclination by β, the soil saturated unit weight by γ_s and the water unit weight by γ_w, the following expression can be derived for static factor of safety in terms of effective parameters for a totally submerged infinite slope. (see, for example, McCarthy, 1998.)

$$(F_s)_{static} = \frac{c' + \gamma' d \cos^2\beta \tan\phi'}{\gamma' d \sin\beta \cos\beta} \qquad [3]$$

in which, $\gamma' = \gamma_s - \gamma_w$ is the soil buoyant unit weight. This method assumes that the stress conditions are the same at every point on the failure plane.

The effects of seismic excitation are included in a pseudo-static approach, as shown in Equation 4 (e.g. Hadj-Hamou and Kavazanjian, 1985):

$$(F_s)_{pseudo-static,cohesionless} = \frac{(\gamma' d \cos^2 \beta - \Delta u_e) \tan \phi'}{\gamma' d \cos \beta (\sin \beta + K \frac{\gamma}{\gamma'})} \quad [4]$$

in which K = seismic coefficient factor (i.e. acceleration in terms of g), Δu_e = the excess pore water pressure due to earthquake. In the above equation, only the horizontal earthquake acceleration component is taken into account, which is multiplied by the total saturated weight of soil.

The traditional Limit Equilibrium method has several limitations, (e.g. Chen, 1975, Griffiths and Lane, 1999 and Chang et al. 1984), most importantly, that it cannot predict the actual displacement of the slope.

3.2 Estimation of EPWP Build-up

The transformation of a granular material from a solid to liquefied state as a consequence of increased pore water pressure and reduced effective stress is known as soil liquefaction (Marcuson, 1978). In the past 35 years, important advances have been made to develop empirical correlations that are mostly based on laboratory and/or field observations. Youd and Idriss (2001) have summarized the latest recommendations that are used herein as the state-of-practice method for the estimation of EPWP build-up. There is a high likelihood of liquefaction in a submarine slope containing granular soils that are most likely fully saturated. Therefore, the simplified procedure pioneered by Seed (1979) and summarized along with further recommendations by Youd and Idriss (2001) is used in the first part of this study.

The procedure steps are briefly stated hereafter:

Step 1 - Earthquake Equivalent Cyclic Loading: The equivalent number of uniform stress cycles at $0.65\tau_{max}$, N, is calculated using the method proposed by Seed et al. (1975a).

Step 2 – Number of Cycles to Induce Initial Liquefaction (N_L): Using the aforementioned soil test data, the number of cycles of shear stress needed for inducing initial liquefaction (N_L) is derived.

Step 3 - Assessment of EPWP Build-up: Equation 5 is proposed by Seed et al. (1975b) to assess the excess pore water pressure generated in N cycles:

$$\frac{\Delta u_e}{\sigma'_v} = \left(\frac{2}{\pi}\right) \arcsin \left(\frac{N}{N_L}\right)^{1/2a} \quad [5]$$

in which Δu_e is the EPWP, σ'_v is the initial effective vertical stress, N is the number of cycles of shear stress (step 1), N_L is the number of cycles of shear stress needed for initial liquefaction (step 2) and α is a constant approximately equal to 0.7.

3.3 Displacement Analysis

So far, current engineering practice in liquefaction assessment has been discussed, however, the necessity of displacement (or deformation) criteria rather than factor-of-safety criteria should be mentioned and emphasized. The importance of displacement analysis of slopes in case of seismic analysis was first introduced by Newmark (1965). It is possible that the factor of safety becomes less than one several times during an earthquake although it does not lead to slope collapse. Thus, in addition to extending the traditional method of submarine slope stability analysis to consideration of seismic liquefaction effects, displacement analysis according to Newmark's analytical method is envisaged. This is actually the state-of-practice in slope stability analysis.

Newmark's (1965) analytical procedure, is summarized for infinite slope displacement analysis in the following general steps (after Chang et al., 1984):

Step 1 – Yield Acceleration (K_y): The yield acceleration, which initiates soil block movement, now can be computed by equating F_s in Equation 4 to unity.

Step 2 - Block Motion Acceleration (a_i): Equation 6 is used to calculate the acceleration of an infinite block of soil subject to acceleration K_i (Chang et al., 1984):

$$a_i = (K_i - K_y) g \frac{\cos(\phi - \beta)}{\cos^2 \phi} \quad [6]$$

where K_y is yield acceleration calculated by Equation 4.

Step 3 - Down slope displacement is computed by double integrating that portion of the acceleration time history exceeding the yield acceleration.

4. FINITE ELEMENT ANALYSIS

The state-of-the-art in seismic evaluation of earth structures is represented by finite element programs such as TARA-3 (Finn et al. 1986), using effective stress analysis. The stress-strain relations are expressed using nonlinear models such as the hyperbolic model proposed by Duncan and Chang (1970). The EPWP is updated during the analysis based on empirical relations (e.g. Martin et al. 1975), and the reduction of soil shear strength is introduced by: (1) accounting for reduction in effective stress (e.g. Finn 1990b), or (2) using a triggering criterion to switch the strength of any liquefiable element to residual strength at the proper time (as in TARA-3FL, Finn and Yogendrakumar 1989). The direct (empirical) soil constitutive models used require relatively complicated regression analysis procedures for parameter calibration. Moreover, validity of these constitutive models is only guaranteed for the conditions under which experimental observations were made (see e.g. Dafalias, 1994) and therefore they may not capture the plastic dilation behaviour under arbitrary 3D stress states.

As for the post-liquefaction analysis, the focus is on assigning a value of the residual strength. It does not directly provide the actual dynamic response of the structure, including continuous yielding of the material induced by EPWP build-up, and gradual strengthening after the shaking, following the pore water pressure dissipation.

4.1 Proposed Analysis Procedure

Three aspects need improvement in a seismic analysis involving EPWP build-up (see e.g. Popescu, 2001):

a) Coupled analysis: solid and fluid coupled field equations have to be used in a step-by-step dynamic analysis to correctly capture the inertial and dissipative coupling terms.
b) Soil constitutive model: correct simulation of dynamically induced EPWP build-up and continuous softening of the material requires soil models able to reproduce the experimentally observed nonlinear hysteretic behaviour and shear stress-induced anisotropic effects, and to reflect the strong dependency of plastic dilatancy on effective stress ratio (e.g. Byrne and McIntyre 1994). Advanced plasticity models, such as multi-yield or bounding surface plasticity, in combination with kinematic hardening rules, can offer a material representation of considerable power and flexibility.
c) Material properties: the constitutive model parameters have to be represented by traditional soil properties that can be estimated from results of standard in-situ and/or laboratory soil tests using well-defined and robust methodologies.

The multi-yield plasticity soil constitutive model (Prevost, 1985) implemented in DYNAFLOW (Prevost 1998) meets the above requirements for liquefaction potential evaluation. Some of its capabilities, pertinent to simulation of seismic behaviour of soil deposits and earth structures, are outlined below (Popescu, 2001):

a) Effective stress analysis, using solid and fluid coupled field equations (Biot 1962). This formulation, including the extension to the nonlinear regime and its implementation in DYNAFLOW, is presented in detail by Prevost (1993).
b) Soil constitutive models able to reproduce experimentally observed soil behaviour, including accurate simulation of (1) shear induced plastic dilation and subsequent EPWP build-up, (2) Soil softening induced by EPWP, and gradual hardening when pore pressures start dissipating, and (3) hysteretic effects under cyclic loading conditions.
c) A well-defined methodology for calibrating the soil constitutive model parameters based on results of standard in-situ and/or laboratory soil tests.

DYNAFLOW (Prevost 1998) is a finite element analysis program for the static and transient response of linear and nonlinear two- and three-dimensional systems. The finite element analysis is performed in one run consisting of two phases. First, gravity loads are applied and the soil is allowed to fully consolidate. The consolidation phase is calculated dynamically, by setting the Newmark algorithm parameters in the integration scheme as α=1.5 and β=1. After consolidation is completed, the nodal displacements, velocities and accelerations are zeroed, the time is reset to zero and the input acceleration is applied at the mesh boundaries. The Newmark parameters are chosen as α=0.65 and β=(α+0.5)2/4= 0.33, to introduce a slight numerical damping and maximize high frequency numerical dissipation. No additional viscous physical damping is introduced. Post-earthquake analysis, including EPWP dissipation and, where applicable, post-liquefaction deformations, is simulated in the same phase, by continuing the analysis for the desired time period after the end of the seismic motion.

The seismic motion is prescribed as an acceleration time history at the base of the finite element mesh. This is usually selected as a more resistant soil layer, assumed rigid and impervious, and situated at a certain depth, so that those assumptions do not significantly influence the calculation results.

4.2 Multi-yield Plasticity Soil Constitutive Model

The multi-yield plasticity model implemented in DYNAFLOW is a kinematic hardening model based on a relatively simple plasticity theory (Prevost 1985) and is applicable to both cohesive and cohesionless soils. The concept of a “field of work-hardening moduli” (Iwan 1967, Mroz 1967, Prevost 1977) is used by defining a collection of nested yield surfaces in the stress space (Figure 3a). Drucker-Prager type surfaces are employed for frictional materials (sands). The yield surfaces define regions of constant shear moduli in the stress space, and in this manner the model discretizes the smooth elastic—plastic stress-strain curve into a number of linear segments. The outermost surface (called failure surface) corresponds to zero shear modulus.

The plastic flow rule is associative in its deviatoric component. To account for experimental evidence from tests on granular soil materials, a non-associative flow rule is used for the dilatational component (Figure 3b).

The material hysteretic behavior and shear stress-induced anisotropic effects are simulated by a purely kinematic hardening rule. Upon contact, the yield surfaces are translated in the stress space by the stress point, as illustrated in Figure 3c. The direction of translation is selected such that the yield surfaces do not overlap, but remain tangent to each other at the stress point.

The constitutive equations are integrated numerically using a stress relaxation procedure (Figure 3d). The return mapping algorithm proposed by Simo and Ortiz (1985) is modified for the multi-yield plasticity case (Prevost 1993).

4.3 Soil Constitutive Model Parameters

The required constitutive parameters of the multi-yield plasticity soil model are listed in Table 3. The yield and failure parameters are used to describe the initial position, size and plastic modulus corresponding to each yield surface. ϕ, ε_{dev}^{max}, k_0 and G_0, are included in a modified hyperbolic expression proposed by Prevost and Keane (1989) and Griffiths and Prevost (1990) describing a wide range of soil stress-strain relations. Hayashi et al (1992) upgraded this expression using a hyperbola whose shape depends on the characteristics of the grain size distribution through the stress-strain curve coefficient α. They developed their model based on shear stress-strain curves obtained in a simple shear soil testing device, using soil specimens with a wide variety of grain size distributions tested under k_0 condition.

The dilation parameters are used in the plastic flow rule for calculating the dilation (shear-induced plastic volumetric strain). The dilation angle, ψ, is in fact the phase transformation angle, and the dilation parameter, X_{pp}, is a scale coefficient for plastic dilation, basically depending on relative density and soil type (fabric, grain size) – Popescu (1995).

Except for the dilation parameter, all the constitutive model parameters are expressed in terms of traditional soil properties, and can be derived from the results of conventional laboratory (triaxial, simple shear) or in-situ (cone penetration, standard penetration, wave velocity) soil tests. An example of parameter estimation from laboratory soil test data is presented by Popescu and Prevost (1993a). A method for constitutive model calibration based on penetration test results (SPT or CPT) proposed by Popescu (1995) and Prevost and Popescu (1996). The dilation parameter, X_{pp}, which controls the amount of plastic dilation, is evaluated based on the results of liquefaction strength analysis, as shown by Popescu and Prevost (1993a) and Popescu (1995, 2001).

Table 3. The parameters of the multi-yield plasticity model (Popescu, 2001)

Constitutive parameter	Symbol	Type
Mass density – solid Porosity Hydraulic conductivity	ρ_s n^w k	State param-eters
Low strain elastic moduli Reference effective mean normal stress Power exponent	B_0, G_0 p_0' n	Low strain elastic param-eters
Friction angle at failure Maximum deviatoric strain Coefficient of lateral stress Stress-strain curve coefficient	ϕ ε_{dev}^{max} k_0 α	Yield and failure param-eters
Dilation angle Dilation parameter	ψ X_{pp}	Dilation Param-eters

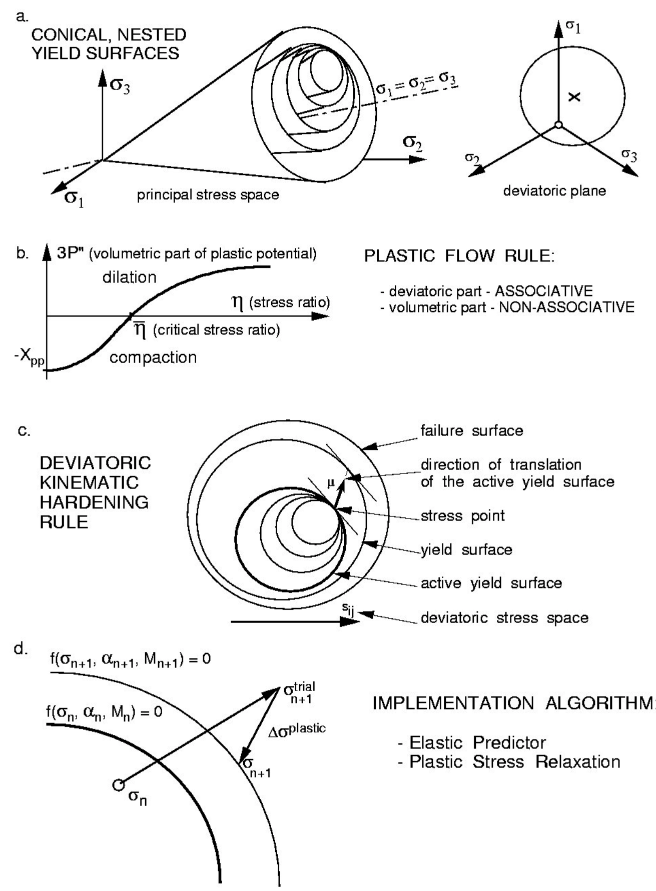

Figure 3. Main features of the multi-yield plasticity soil constitutive model: a) yield surfaces; b. plastic flow rule; c. hardening rule; d. numerical integration (after Popescu, 1995).

4.4 Model Validations

The multi-yield plasticity model, its implementation algorithm in DYNAFLOW, and the methodology to estimate the constitutive parameters have been repeatedly verified and validated in the past for soil liquefaction analysis, using full-scale data (e.g. Keanne and Prevost 1989, Popescu et al.1992) and centrifuge geotechnical models (e.g. Popescu and Prevost 1993a,b, 1995). The most comprehensive validation of the proposed model was carried out during the VELACS (Verification of Liquefaction Analysis by Centrifuge Studies) project (Arulanandan and Scott, 1993, 1994). This study was aimed at better understanding the mechanisms of soil liquefaction and at acquiring data for the verification of various analysis procedures. Nine centrifuge models (horizontal and sloping, homogeneous and non-homogeneous soil deposits, embankments, and structures on liquefiable soil) subjected to seismic motion were tested and duplicated at several centrifuge centers in US and UK. The numerical predictions submitted by 20 groups of researchers were class 'A' predictions, and thus were made before the relevant experiments were performed. Those predictions were based on the results of conventional laboratory soil tests performed on the soil materials to be used in the centrifuge models. Class 'A' predictions using DYNAFLOW were submitted for all the nine centrifuge models. A summary comparison of the performance of all class 'A' predictions is presented by Popescu and Prevost (1995), and a detailed comparison, showing all recorded and predicted pore pressure, displacement and acceleration time histories, has been posted on the www at: http://cee.princeton.edu/~radu/soil/velacs/. It resulted from these comparisons, as well as from studies presented by other authors (Arulanandan and Scott, 1994) that the VELACS project validated the mathematical model and methodology proposed here.

4.5 Preliminary Analysis Results

Two models are analysed: a) an infinite slope (1-D analysis) similar to the one considered for conventional method and b) a more realistic slope (see Figures 2 and 4) with three different inclinations (4^o, 10^o, and 3^o, respectively) to account for the effects of lateral boundaries (2-D analysis). The slope geometry (Figure 4) is based on the profile presented by Piper et al. (1999) and soil strata are mentioned in Table 2. Figure 4 presents the deformed shape of the slope with two milder slopes at each side.

Contours of predicted EPWP ratio with respect to the initial effective vertical stress (r_u) at the end of shaking are also presented. r_u=1 corresponds to liquefaction.

Two different soil properties are assigned to the middle layer of silty sand and the results are shown in Figures 4a and 4b, respectively: 1) medium to dense silty sand with relative density of 65%, and 2) Loose to medium silty sand with relative density of 47%. Case 2 is believed to closer match the in-situ soil properties. The bottom layer is considered as a dense sand.

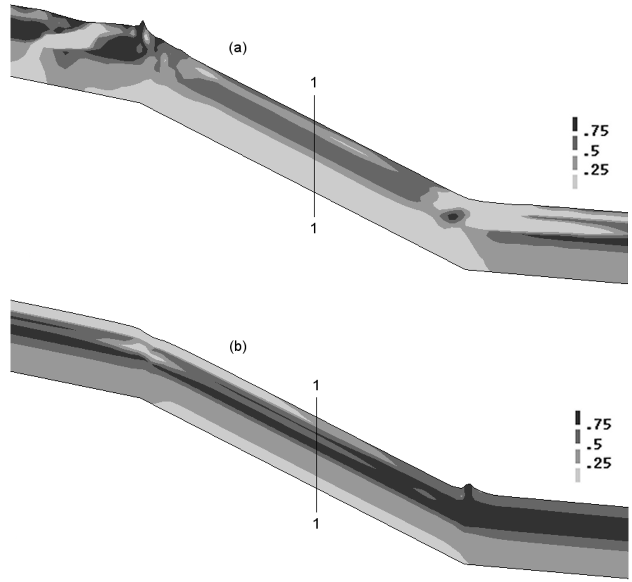

Figure 4. Deformed shape and EPWP ratio contours in: a) slope with medium to dense silty sand, and b) slope with loose to medium silty sand. (Vertical scale is 3 times larger than the horizontal. Displacements are not magnified.)

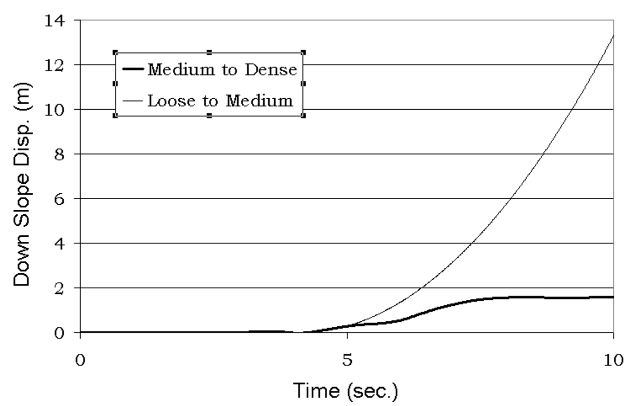

Figure 5. Predicted displacement at the top of the silty sand layer (Section 1-1 in Figure 4).

The preliminary results (Figures 4a and 4b) show that in case of medium to dense sand, which is less susceptible to liquefaction, the milder slopes liquefy much sooner than the steeper slope, in which static shear stress is higher. Accordingly, the steeper slope deformation is relatively small. In the second case, however, in which the middle layer is a medium to loose sand, the steeper slope is predicted to undergo large deformations as well as wide spread liquefaction.

Seismic and post-earthquake analyses were performed for both cases up to time t = 35 sec. As shown in Figure 5, the slope displacements stabilized at the end of shaking to about 1.8 m for case 1. For the soil materials assumed in case 2, the slope displacements are predicted to continue indefinitely after the earthquake.

More comprehensive and comparative results will be presented at the conference.

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