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Seismic triggering of submarine slides in soft cohesive soil deposits

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Abstract

The geological profile of many submerged slopes on the continental shelf consists of normally to lightly overconsolidated clays with depths ranging from a few meters to hundreds of meters. For these soils, earthquake loading can generate significant excess pore water pressures at depth, which can bring the slope to a state of instability during the event or at a later time as a result of pore pressure redistribution within the soil profile. Seismic triggering mechanisms of landslide initiation for these soils are analyzed with the use of a new simplified model for clays which predicts realistic variations of the stress–strain–strength relationships as well as pore pressure generation during dynamic loading in simple shear. The proposed model is implemented in a finite element program to analyze the seismic response of submarine slopes. These analyses provide an assessment of the critical depth and estimated displacements of the mobilized materials and thus are important components for the estimation of submarine landslide-induced tsunamis.

Keywords: slope stability; soft clay; soil deformation; soil dynamics; submarine landslides

1. Introduction

As the economic interest on offshore drilling and the number of related pipe-lines increase, wider attention is focused on the problem of ocean floor stability. Although various mechanisms influence the movement of sediments on the submerged margins of the continental slopes, submarine slides are a major threat to the integrity of offshore engineering structures because of the large displacements and forces developed in such failures. Additional interest has been associated with the potential generation of large tsunamis as a result of seismically induced submarine landslides near highly populated coastal areas such as Los Angeles, CA, USA. These failures can pose a serious danger both in terms of damage and loss of life, because the time lag between the occurrence of the landslide and the tsunami arrival does not allow for warning of the population located in the impacted area. As a result of

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the high risk these phenomena pose, there is a need to develop suitable methods for the identification of potentially unstable areas and the accurate estimation of permanent displacements under seismic and/or cyclic (i.e. storm) loading conditions. This information is essential to estimate slide volumes and displacements which can be used in models to predict tsunami amplitudes (e.g. Watts, 1998).

When the rate of deposition for submarine slopes is faster than the rate of consolidation, the new material can trigger localized gravity failures in the weak, unconsolidated sediments. This is the general case of submarine slopes near river deltas with high sediment load, such as the Mississippi Delta in the Gulf of Mexico. For these cases, the soils are primarily composed of very fine sands and silts in a loose state, which may be susceptible to seismically induced liquefaction. Liquefaction failures have been extensively studied and well established methods of analysis for estimating the triggering potential and evaluating permanent displacements are available (e.g. Seed and Idriss, 1971). In contrast, the rate of deposition for slopes on the continental shelf is sufficiently low, the material deposited is much finer (i.e. fine silts and clays) and the slope inclination is small, with values typically less than 5°. These sediments are allowed to gain sufficient strength and the slopes are theoretically stable under gravity loads. Nonetheless, large-scale submarine slope failures have been observed in these soil and have been attributed to seismic loading (e.g. Frydman et al., 1988; Puzrin et al., 1997) or wave loading (e.g. Schapery and Dunlap, 1978) and they are the focus of the present study. These submarine landslides may occur on very gentle slopes, often with inclinations lower than 4° (e.g. Lewis, 1971; Prior and Coleman, 1978), and are characterized by large dimensions, up to several kilometers both in width and length, and very large runout distances. A compendium of more recent work on submarine slides and their consequences is provided in Locat and Meinert (2003).

In order to obtain accurate predictions of permanent deformations of submerged slopes subjected to a seismic event, the soil must be modeled with a realistic stress–strain–strength relationship that is able to describe irregular cyclic loading. In particular, two fundamental aspects must be captured by the constitutive laws: the non-linearity of soil behavior and the development of pore pressure during cyclic shearing. Current effective stress models used for clays are mainly derived from the theory of elasto-plasticity or developed from direct modeling of the non-linear hysteretic stress–strain response. The Cam-clay model or their derivatives based on the Critical State Soil Mechanics framework (e.g. Schofield and Wroth, 1968; Roscoe and Burland, 1968) fall into the first category, but they cannot simulate realistic excess pore water pressure development and permanent deformations during cyclic loading.

Several approaches have extended the elasto-plastic framework to include plastic strains inside the yield surface and they generally fall into one of the two categories: (a) models based on multiple yielding mechanisms with kinematic hardening (e.g. Koiter, 1960; Prévost, 1978; Mróz et al., 1978) or (b) models based on bounding surface plasticity (e.g. Dafalias and Hermann, 1982). For all of these models, the sophistication of the constitutive laws and the use of a generalized six-dimensional stress space impose significant demands on computation time for the relatively simple problem of one-dimensional shear wave propagation. The second type of effective stress models directly addresses the non-linear stress strain relationship by empirically fitting experimental data. These models usually satisfy Masing rules (Masing, 1926) and provide a continuous (e.g. Ramberg and Osgood, 1943) or a piecewise linear (e.g. Iwan, 1967) expression for the first loading curve. The basic disadvantage with these models is that the stress–strain response is decoupled from the pore pressure generation. Separate models have been introduced to relate stress–strain parameters to the current level of excess pore water pressure through empirical laws, based on the number of cycles (Finn et al., 1977; Ishihara and Towhata, 1982) or the mobilized stress ratio (Puzrin et al., 1995). These simplified models do not implicitly describe the effect of the shear consolidation stress history imposed by the slope. The behavior of slopes can be accounted for by changing the material parameters used in site response analysis of level ground conditions but it requires significant experience and engineering judgement. In addition, these models have been calibrated for shaking in one direction only and therefore additional work is needed to extend capabilities for multi-directional shaking. The following sections briefly describe a new model developed to incorporate the anisotropic stress–strain–strength properties of clayey soil resulting from soil deposition (i.e. consolidation stress history) as well as the evaluation of key factors affecting the response of submarine slopes subjected to earthquake loading.

2. SIMPLE DSS model

A simplified model, referred hereafter as SIMPLE DSS, was developed for the one-dimensional wave propagation analysis of submerged clayey slopes under seismic loading (Pestana et al., 2000; Pestana and Nadim, 2000). When only gravity loads are acting, a generic soil element is subjected to a stress in the direction normal to the slope, represented by the effective normal stress (σ_n), and a stress in the plane of the slope, parallel to the dip, represented by the consolidation shear stress (τ_c) as shown in Fig. 1a. Given the simplicity of the formulation, the earthquake motion is assumed to consist only of shear waves propagating perpendicular to the slope, disregarding those propagating along the plane of the slope. This consideration is analogous to the assumption of vertically propagating 'horizontal' shear waves for level ground conditions. The seismic motion then results in an additional cyclic shear stress acting on the plane of the slope in a direction oriented at some angle with that of the consolidation shear stress (i.e. multidirectional shaking). Although the seismic shear stress changes direction instantaneously, most analyses choose the critical direction to be parallel to the dip of the slope (i.e. the direction of shear shaking and initial shear stress coincide) as shown in Fig. 1. The stress state in this case is the same as that developed in simple shear tests, which have been recognized as good tools for investigating the problem of submerged landslides. In this apparatus, a cylindrical specimen can be consolidated under K_o conditions,

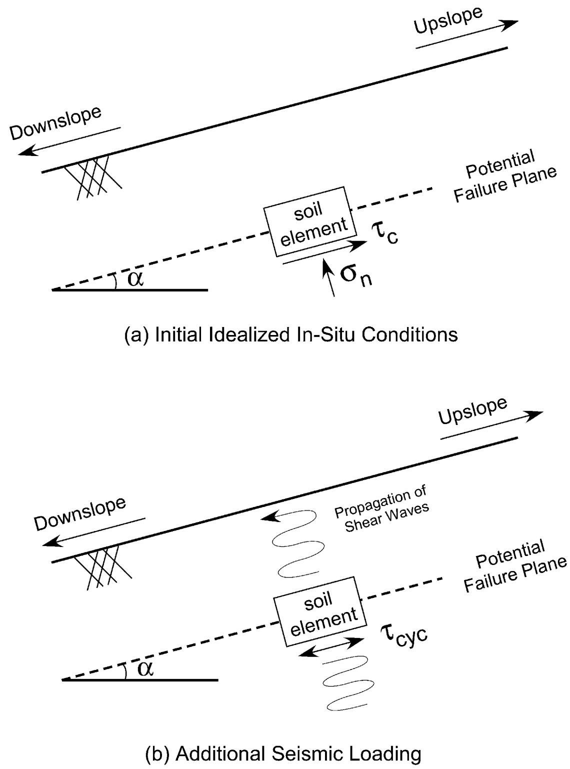

Fig. 1. Infinite slope under one-dimensional seismic excitation.

insured by a wire-reinforced membrane, and then sheared undrained in simple shear conditions (Bjerrum and Landva, 1966; Andresen et al., 1979). The SIMPLE DSS model allows the simulation of simple shear tests on lightly overconsolidated clays. The following discussion focuses on the description of normally consolidated soils sheared in the dip direction. Treatment of overconsolidated soils and multidirectional shaking are outside the scope of this paper.

2.1. Monotonic loading

The proposed effective stress-based model uses the concept of normalized material response at the heart of the Critical State Mechanics Framework (Pestana et al., 2000). The effective stress path for a normally consolidated specimen during monotonic undrained shearing is described by a state surface defined in the normal stress (σ_n) – shear stress space (τ). Modeling monotonic re-

Table 1
Material parameters used in the SIMPLE DSS model formulation

Parameter	Effect on predicted behavior	Parameter determination	Time dependency
β	Controls (primarily) the sensitivity of the material	Excess pore pressure at large strains	negligible
m	Controls (primarily) the undrained shear strength of the material	Determined from measured values of undrained shear strength, s_u	yes
ψ	Describes the effective stress failure envelope	Angle defining the shear stress ratio at large strains ($\gamma \approx 20\%$)	negligible
G_n	Controls the small strain elastic shear modulus	Shear wave measurements (G_{max})	negligible
G_p	Controls the stress–strain curve during the first loading	Calibration with measured stress–strain behavior on NC specimens	negligible
θ	Controls effective stress path for cyclic loading	Calibration with measured pore pressure development during cyclic loading	yes
λ	Controls shear stiffness during cyclic loading	Calibration with measured accumulated shear strains during cyclic loading	yes

sponse requires five parameters, while two additional parameters are needed to model cyclic behavior (cf. Table 1). Special care was taken in the formulation of the constitutive laws to ensure that the material parameters would retain a clear physical meaning to guarantee objective selection. Parameters β and ψ can be determined from the effective stress path conditions at 10–15% shear strain and G_p is selected through a short parametric study to match the stress–strain curve during first loading. Parameter G_n requires some information on the maximum shear modulus (G_{max}), which can be inferred from the shear wave velocity of the sediments. Parameter m controls primarily the undrained strength of the soil. A chart was developed to select m based on the measured strength and parameters β and ψ.

Fig. 2 compares the normalized shear stress–strain–strength response of normally consolidated Boston Blue Clay with simulations with the SIMPLE DSS model for shearing starting from different initial states, $\tau_c/\sigma_p = 0.0$, 0.1, 0.2, and 0.3 (where σ_p is the maximum normal effective stress). Boston Blue Clay is a low plasticity clay extensively documented in the literature (Ladd and Edgers, 1972). Model parameters were derived for the case of no initial shear stress (i.e. $\tau_c/\sigma_p = 0.0$). The same parameters were then used to simulate tests in which a consolidation shear stress (τ_c) was present and no additional adjustment was required. In these tests the specimens are consolidated under a given normal effective stress and an applied shear stress. After consolidation has taken place, the sample is sheared monotonically under undrained conditions. As shown in Fig. 2, the undrained shear strength increases and the strain at failure decreases with increasing consolidation stress ratio for positive shearing cases, while the opposite is true for negative shearing. This means that shearing in the downslope direction the soil displays a higher shear strength, but brittle behavior. Shearing in the uphill direction mobilizes a lower shear strength, but much higher strains are required. As the specimens are sheared, excess pore pressure is generated as evidenced by the decrease in the normal effective stress. At large shear strains, corresponding to $\gamma = 15$–20%, all specimens seem to achieve approximately the same shear strength with nearly constant stress ratio (τ/σ_n). It is very difficult to interpret the results of DSS tests at strains exceeding 20–25% and thus this case is not considered further in the discussion.

2.2. Cyclic loading

One of the key characteristics of cyclic loading of clays is the continued accumulation of plastic strains and shear-induced excess pore water pressure with increasing number of cycles. In the SIMPLE DSS model the pore pressure development is determined by a load state surface controlling the effective stress rate dependent path during cyclic loading, with an approach that re-

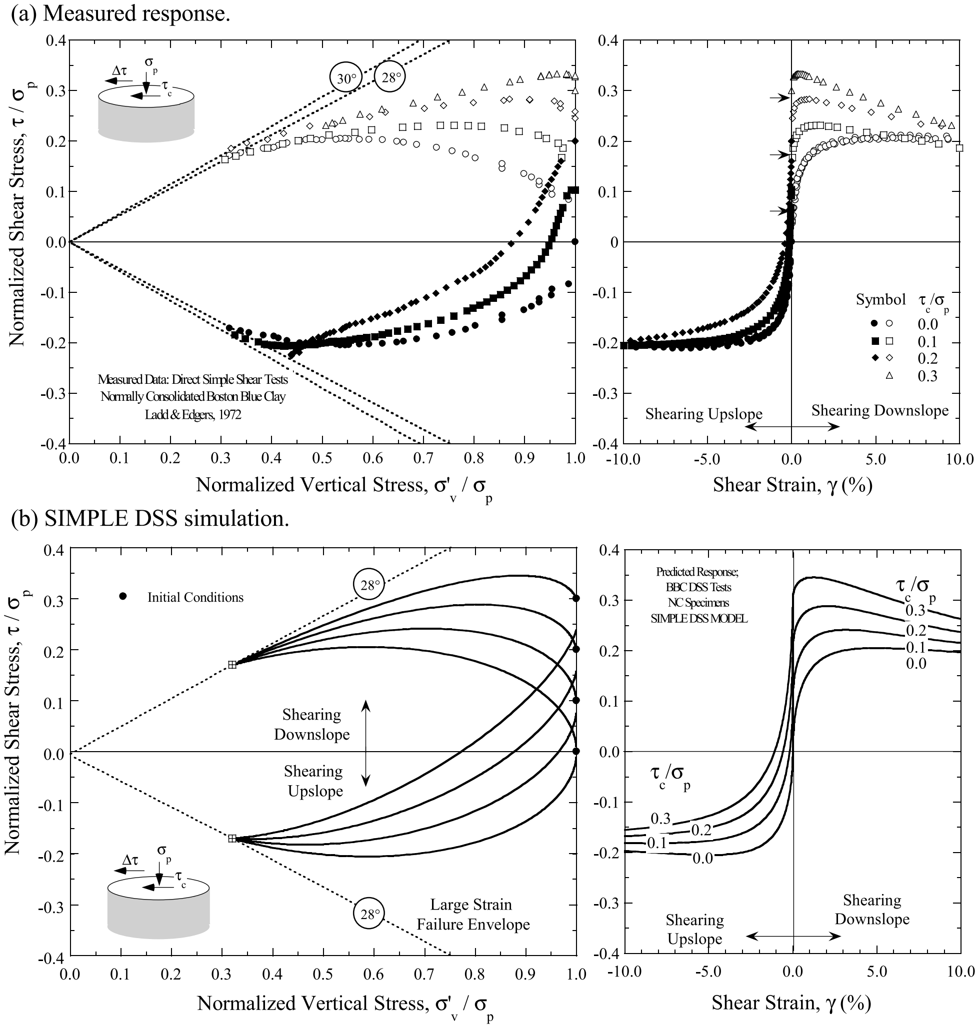

Fig. 2. Comparison of measured and predicted response during monotonic DSS testing for normally consolidated Boston Blue Clay.

sembles that of first loading from NC states. This concept is similar to that of bounding surface plasticity, but it is decoupled from the stress–strain response component to allow for nearly independent determination of material parameters. The proposed formulation assumes a 'transitional state surface' defining the strain rate dependent effective stress path followed by soil specimens for cases in which the plastic state surface is not yet activated. The model produces results similar to those obtained with generalized models based on bounding surface plasticity (e.g. Pestana and Whittle, 1999) but it is computationally much more efficient for the reduced stress space of interest.

The cyclic response is described in SIMPLE DSS by two parameters, θ and λ, which control the generation of pore pressures and the development of plastic strains, respectively. The two parameters can be determined independently from a cyclic DSS test by matching the excess pore pressure and shear strain versus number of cycle curves. Fig. 3 shows the measured response of Boston Blue Clay for one of these tests for a

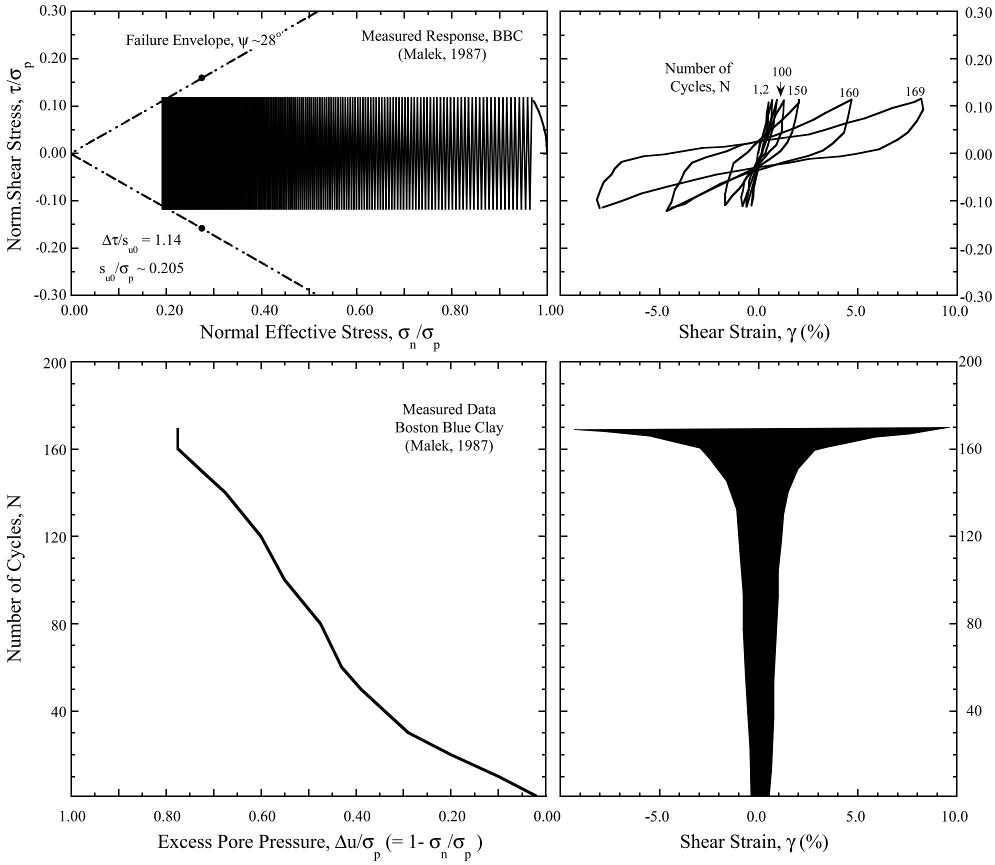

Fig. 3. Measured response of normally consolidated Boston Blue Clay in a cyclic DSS test.

cyclic stress ratio of $\Delta\tau/s_{uo} = 1.14$. The undrained strength normalized by the maximum normal effective stress, s_{uo}/σ_p, is 0.205 at the conventional shearing rate of 5 mm/min. The curves in Fig. 3c,d were used to evaluate the cyclic parameters. The corresponding simulation by the model is shown in Fig. 4. As reported in Table 1, some of the parameters are inherently dependent on the strain rate of shearing and complete discussion is presented by Pestana et al. (2000). A summary of the parameters used to describe the response of BBC is presented in Table 2.

3. Site response analysis

The stability of submerged slopes under seismic load is often evaluated in the framework of limit equilibrium methods using a pseudo-static approach in which the inertial force caused by ground acceleration is applied as a horizontal static load. The factor of safety obtained with these analyses is not always a meaningful measure of slope performance, since values less than one

Table 2
SIMPLE DSS parameters for Boston Blue Clay

Parameter	Boston Blue Clay
β	0.32
m	−0.25 (1.00)[a]
ψ	28
G_n	450
G_p	8.5
θ	30[a]
λ	15[a]

[a] Parameters determined from cyclic tests.

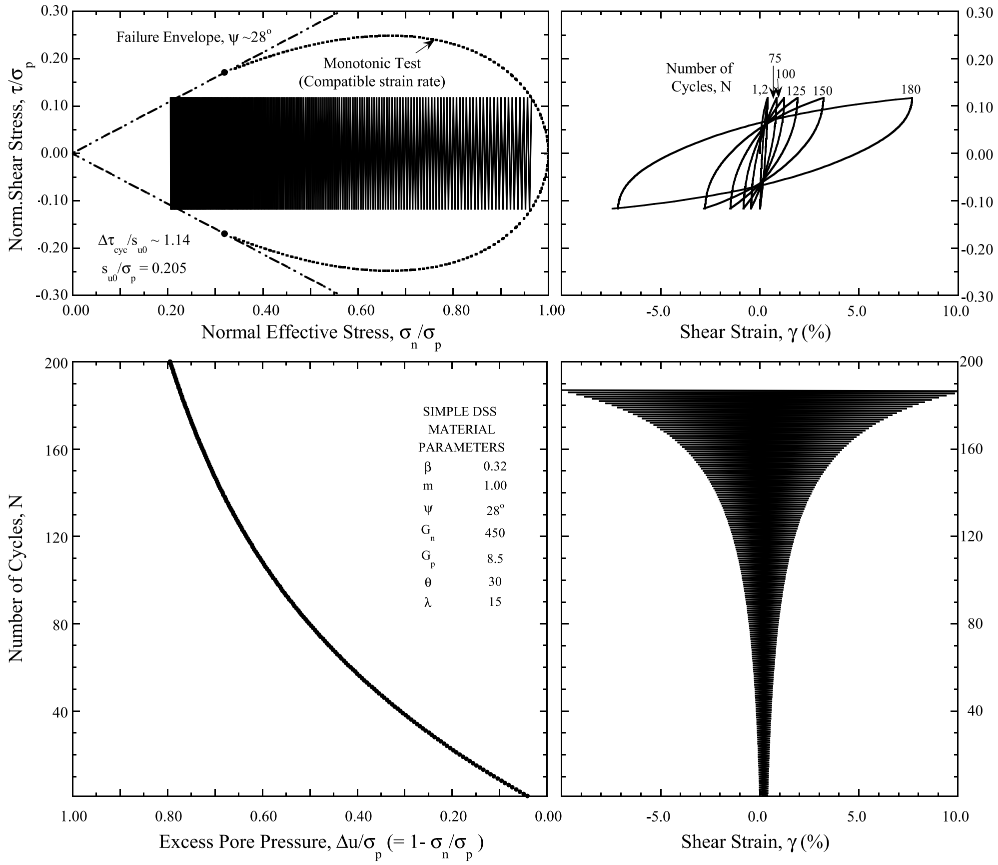

Fig. 4. Model simulation for normally consolidated Boston Blue Clay in a cyclic DSS test.

do not necessarily mean failure and no direct prediction of the amount of expected displacements is given. Since modern design of offshore engineering structures is mostly based on the necessity to limit the amount of movements in order to safeguard operations rather than on safe/fail criteria, methods of analysis able to accurately predict displacements due to seismic loading need to be developed. In the particular case of submarine landslides, the ratio between thickness and length of the sliding mass is so small that side effects can be neglected as a first approximation and the simple infinite slope analysis can be utilized. This problem, commonly referred to as site response analysis, is reduced to the simulation of one-dimensional wave propagation in a layered soft clay deposit and has been extensively treated in the literature (e.g. Lysmer and Kuhlemeyer, 1969; Schnabel et al., 1972; Joyner and Chen, 1975; Lee and Finn, 1978).

In order to perform the seismic site response analysis for sloping ground, the SIMPLE DSS constitutive laws have been implemented in the finite element program AMPLE2000 (Pestana and Nadim, 2000). The soil layers are modeled as non-linear shear beams. The finite element formulation requires the solution of the global dynamic equation of motion in space, while the explicit central difference method is used for the integration in the time domain. The soil profile can be divided into any number of layers, each of them with separate characteristics, including unit height, model input parameters and preconsolidation pressure describing the past consolidation stress history.

The results from the analyses include accelera-

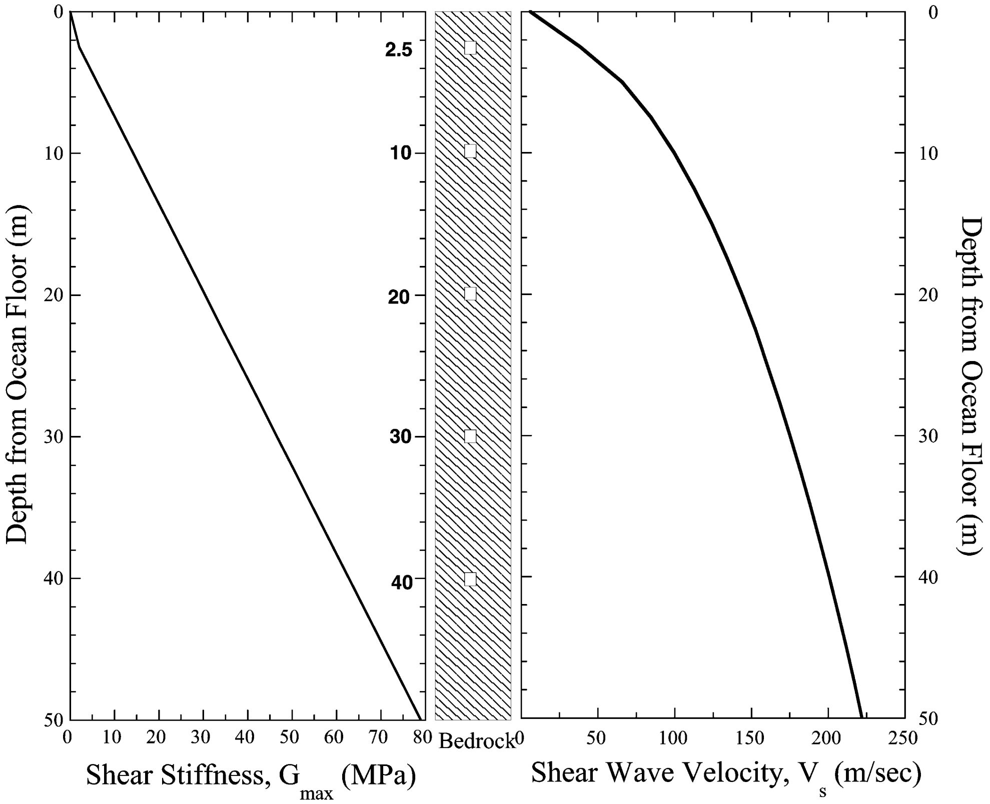

Fig. 5. Simplified uniform soil profile of normally consolidated clay.

tion, displacement, shear strain and shear stress time histories for specified depths as well as maximum and end-of-shaking profiles. It is also possible to obtain spectral accelerations for 5% damping at the same locations. Since the program is not based on limit equilibrium analysis, it will not give a safe/fail type of answer (or factor of safety). For a given scenario, the user will need to define unacceptable performance levels corresponding to 'failure'. The criterion could be specified in terms of displacements or accelerations for some kind of structures, or in terms of a combination of some critical acceleration and displacement which may cause the triggering of a turbidity current. AMPLE2000 in combination with the SIMPLE DSS model allows the predictions of the relevant quantities that will help to characterize the behavior of the slope during shaking.

3.1. Example

A simple example is presented in the following sections to illustrate the site response analysis results obtained with the SIMPLE DSS model. The characteristics and the geometry of the slope were selected as representative of a generic clayey material and slope configuration. The soil profile is composed of a uniform normally consolidated

Table 3
SIMPLE DSS parameters for example slope material

Parameter	β	m	ψ	G_n	G_p	θ	λ
Value	0.35	0.5	28	Increasing with depth	10	25	30

material of approximately 50 m in depth and with a slope inclination of 10°. The density increases nearly linearly with depth, from 1400 kg/m^3 at the surface to 1600 kg/m^3 at 50 m. Similarly, the shear modulus at small strains (G_{max}) is linearly increasing with depth as shown in Fig. 5.

Information of this kind can be obtained by using a variety of geophysical methods that do not require samples to be tested. The parameters for the SIMPLE DSS model are given in Table 3. With the exception of parameter G_n, which is a function of G_{max} and current effective stress, all other parameters are considered constant for a given soil. If some information is available, it can be used to modify some of the parameters to get a better match with the measured quantities. For example, undrained shear strength is often measured in situ with vane shear tests. Some care needs to be exercised in the interpretation of these quantities, especially when there is some dependency on the rate of shearing or the particular method. Knowledge of the geological setting and the deposition process is also valuable in understanding the structure of the slope and whether a uniform profile could be a realistic representation, or more information is needed. It could also provide valuable insight in the variations of the properties of the soil. However, even if the parameters are constant with depth, by letting the small strain shear modulus vary, we ensure that the behavior of each layer will be different. In this analysis, the earthquake ground motion used for the analysis is the Friuli–Tarcento record scaled to 0.35 g and it is applied as a rock outcrop motion (cf. Pestana and Nadim, 2000).

3.1.1. Slope performance and permanent displacements

Fig. 6a shows the predicted shear strain and displacement time histories. Downslope displacements and strains continue to accumulate during the shaking with minor reversals for upslope accelerations. This is due to the effect of the initial shear strain acting downslope, which reduces the available strength and stiffness for shearing in the downslope (i.e. positive) direction. Displacements and accelerations are reported at node locations whereas stresses and strains are reported at the center of the layers. Fig. 6b shows the profile with depth of maximum and end-of-shaking values of displacement and shear strain. In contrast with level ground conditions, there is only a very small difference between the maximum and end-of-shaking values because the displacements continue to accumulate until the earthquake is over. The largest shear strains are experienced by the top 10 m of soil, which may be indicative of the critical depth for a potential slide plane. The profile of pore pressures generated by the earthquake loading is shown in Fig. 7. The highest pore pressures are developed at the bottom of the slope. However, the ratio of pore pressure to initial vertical effective stress is highest at the top and remains nearly constant below 20 m.

4. Post-earthquake pore pressure dissipation

The excess pore pressure generated in soil by the shaking may also play an important role in the post-earthquake stability of the slope. For normally consolidated or lightly overconsolidated clays, earthquake shaking causes the generation of excess pore pressures which will dissipate after the seismic event. In the special case in which an upward gradient is established and stratification of the soil deposit is present, it is possible for the upward migrating pore water to cause an increase in the pore pressure beneath a layer characterized by a slower dissipation of the excess pore pressure. This increase in pore water pressure results in a decrease of the effective stress and may bring the material to a state on instability. A similar mechanism for delayed failure is demonstrated by Kokusho and Kojima (2002) for stratified sandy, silty deposits. Since this paper only deals with fine-grained soils, the time frame involved in the full dissipation of the seismically generated excess pore pressure may be very long, in the order of several years or decades. Similarly, the increase of pore water pressure at the interface may require days or even years to reach its peak and therefore it may be difficult to relate a specific seismic event to the failure. Given the inherent difficulty in estimating the rate of excess pore pressure dissipation in natural soil deposits, and

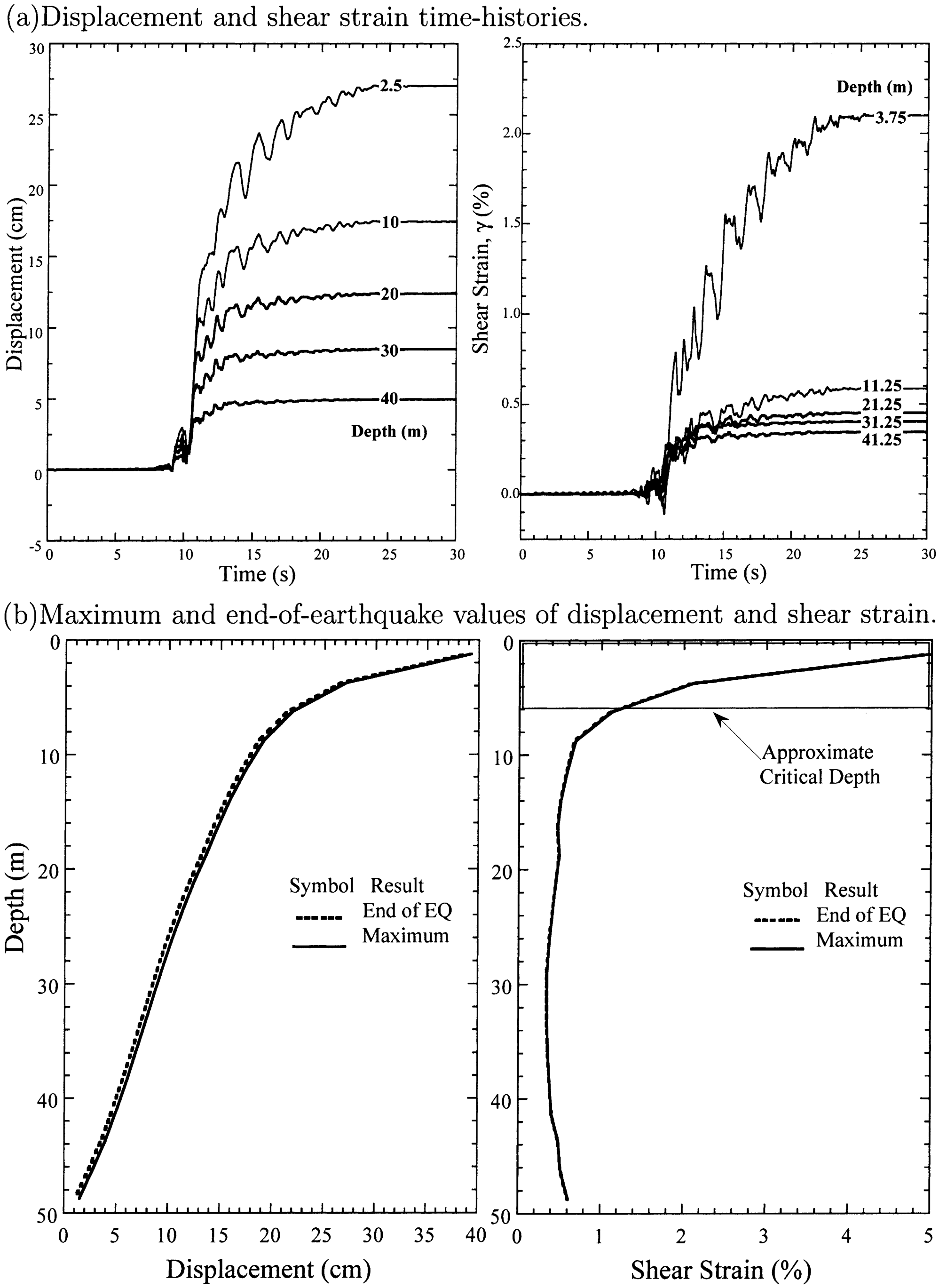

Fig. 6. Seismic response for the selected soil profile.

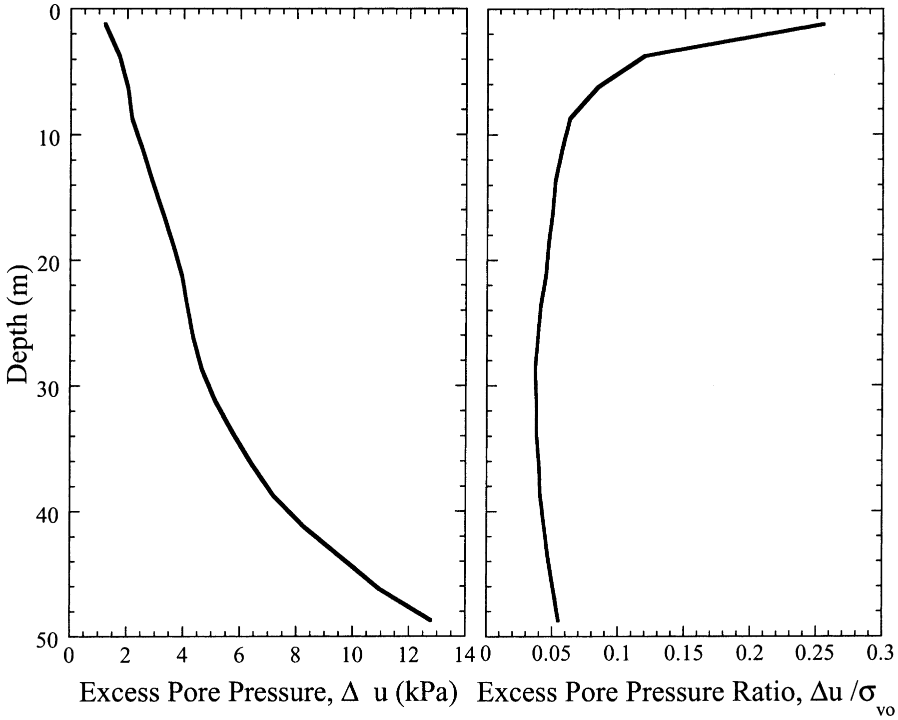

Fig. 7. Excess pore pressure and excess pore pressure ratio at the end of the earthquake.

the lack of site-specific information, this failure mechanism is illustrated here in general terms and proposed as a possible trigger, in addition to the more established causes for instability.

The SIMPLE DSS model predicts excess pore pressures present in the slope profile at the end of shaking. As a result, it is possible to perform a coupled or decoupled one-dimensional consolidation (pore pressure redistribution) analysis to evaluate the dissipation of excess pore pressure after the end of seismic loading. To date, only seepage normal to the slope has been considered. This may be easily extended to a general problem if a three-dimensional geometry is specified.

4.1. Example: redistribution of excess pore pressure

After the end of shaking, excess pore pressures are present in the soil profile examined in Section 3.1 and it is possible to estimate their dissipation as a function of time. In order to obtain meaningful results a representative and realistic value of the coefficient of consolidation (c_v) and its variation as a function of depth are required. However, the difficulty in estimating c_v correctly even in the simplest of cases is well known. Although there are several sources of uncertainty in c_v, namely the compressibility coefficient and the hydraulic conductivity, by and large, the latter is believed to be the dominant source of uncertainty. Two baseline cases for a uniform profile are thus analyzed here: case I – c_v is 5×10^{-6} m^2/s and case II – c_v is 5×10^{-5} m^2/s. Then two hypothetical scenarios have been considered: (A) the coefficient of consolidation is constant with depth; and (B) the coefficient of consolidation for the top layer, from 0 to 2.5 m, is one order of magnitude lower than for the rest of the soil profile. A lower hydraulic conductivity – thus lower c_v – can be the result of gas (decrease in water saturation), presence of gas hydrates, finer soil material or mineral accretion due to biological activity or chemical precipitation. The coefficient of consolidation also depends on the stress level through

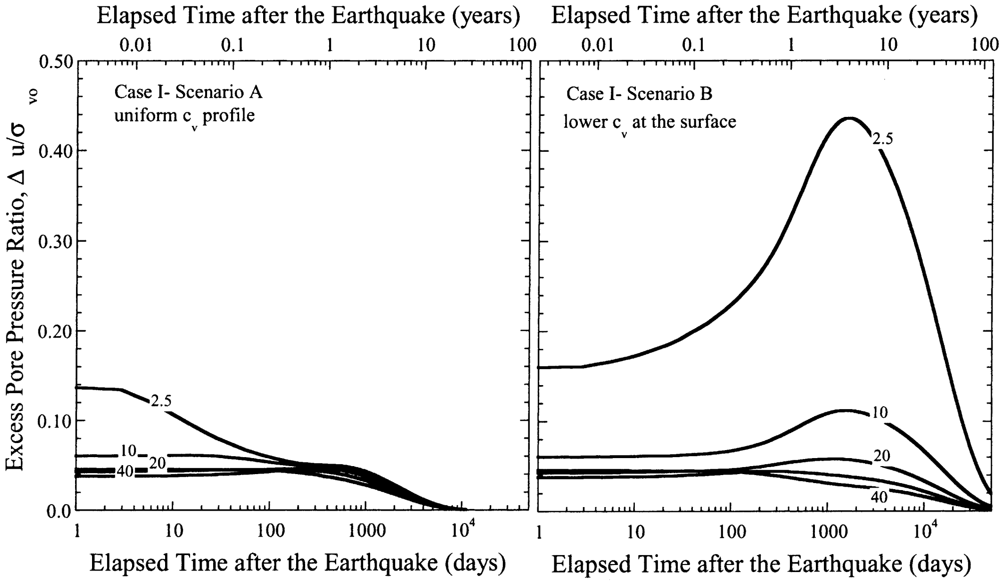

Fig. 8. Pore pressure dissipation for case I.

the coefficient of volume compressibility, m_V. At low stress levels (i.e. at the ground surface) m_V is much greater than it is at the bottom of the slope where the stress level is high. As a consequence, the coefficient of consolidation, c_V, is lower for the top layers of soil than it is for the bottom layers. The analyses presented here are conducted uncoupled, namely there is no consideration of failure/instability or additional shear deformation/displacements as a result of pore pressure redistribution. Thus, these analyses should only be considered as a first evaluation and not as final.

The prediction of the dissipation of pore pressure with time is shown in Figs. 8 and 9 for cases I and II and two scenarios for each baseline case. Fig. 8 shows the results for case I. For the scenario A, the pore pressure monotonically decreases, even if very slowly. In contrast, for scenario B the impervious layer traps the upward flowing water and higher excess pore pressures are accumulated with time at the interface. The increase can be substantial in terms of pore pressure ratio and the peak is predicted approximately 3 to 4 years after the earthquake. In some cases the pore pressures induced by the upward flow trapped by a lower conductivity zone could be enough to produce a situation of instability in the uppermost layers of soil.

For case II in Fig. 9, the results are qualitatively similar to those obtained in case I but the maximum excess pore pressure is smaller and the peak occurs at a much earlier time. As before, in scenario A, the pore pressure dissipates monotonically with time, while in scenario B, the upward flow contributes to an increase in pore pressure ratio with a peak of 0.26 approximately 3 months after the earthquake. An increase in pore pressure causes the effective stress in the soil to decrease. This may drive the effective stress state towards a failure condition, which is often referred to as strength loss, strain softening or fabric instability. For coarser soils such as silty clays or clayey silts, the coefficient of consolidation can be significantly higher (i.e. one to two orders of magnitude) and predicted peak pore pressure ratios can occur at times ranging from several minutes to several hours or days.

5. Summary

The SIMPLE DSS model describes a simplified state of stresses simulating a simple shear condition. The proposed constitutive law incorporates

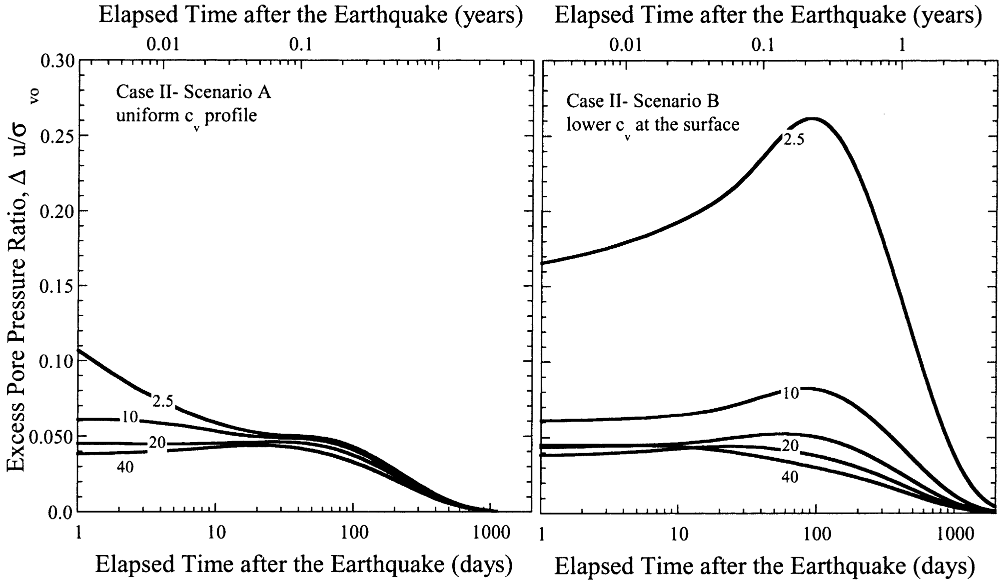

Fig. 9. Pore pressure dissipation for case II.

simplified anisotropic hardening and bounding surface principles to allow the user to simulate different shear strain and stress reversal histories as well as provide realistic descriptions of the accumulation of plastic shear strains and excess pore pressures during successive loading cycles. The model gives excellent predictions of the effect of different consolidation stress histories as well describing qualitatively important trends in the cyclic behavior of clays. These capabilities are key elements in the description of the response of gentle slopes and eliminate the need to estimate or vary material properties according to stress level or inclination of the slope.

The seismic performance of submerged slopes on the continental shelf can be assessed by evaluating the site response analysis for a given earthquake motion (or suite of motions) appropriately scaled to design level. In contrast to limit equilibrium methods, site response analyses provide an estimate of maximum and permanent displacements for a given slope geometry and soil profile. Other measures such as the accumulated shear strain may indicate the progression of damage and its profile with depth may suggest the extent of the potentially unstable domain immediately after the seismic event. The use of an effective stress analysis presents the additional benefit of predicting excess pore pressures at the end of the earthquake as a function of depth. A first order approximation using the uncoupled deformation-drainage analysis provides guidance on the source of a potential – seepage-induced – triggering mechanism that may affect the performance of the slope many days, weeks or months after the seismic event. Identification of this triggering mechanism requires the accurate determination of the coefficient of consolidation profile or at least a measure of the relative magnitude of hydraulic conductivity profile. In general, it is observed that for uniform soil profiles, the critical extent of potentially unstable material can be obtained from 'short-term' conditions corresponding to the end of the seismic event. In contrast, 'long-term' conditions resulting from excess pore pressure redistribution within the soil profile are more critical for profiles exhibiting a smaller coefficient of consolidation near the ocean floor. The time required to achieve critical conditions (instability) for these soil profiles can range from minutes to days or months according to the corresponding coefficient of consolidation profile.

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