SEISMIC RESPONSE OF OVERPRESSURED SUBMARINE SLOPES

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ABSTRACT

Fluid overpressures are an important pre-conditioning factor for submarine slope failures, which occur on very mild slopes. Such overpressures can arise from rapid sedimentation if the deposition of new sediments outpaces the consolidation of the underlying soil. This process can weaken slopes over time, making them more susceptible to seismic loading. In this paper the sediment accumulation process on slopes with subsequent dynamic earthquake loading is modelled in a two-step procedure. The procedure is benchmarked and applied in an example case, investigating the seismic response of slopes at different stages in the sedimentation process. It is shown that in normally consolidated slopes earthquakes tend to cause strain accumulation close to the seafloor. In overpressured slopes, on the other hand, the pre-conditioning due to rapid sedimentation can lead to strain localization at a defined depth during seismic loading, leading to the formation of potentially larger, non-shallow mass movements. The presented procedure can be applied for earthquake vulnerability assessments of submarine slopes at different stages during the sediment deposition process. This could help predicting likely failure mechanisms of submarine slopes, which is a key factor for their hazard potential.

Keywords: Submarine landslides; Sedimentation; Strain localization; Seismic slope stability; Overpressures

1. INTRODUCTION

Submarine landslides can have devastating consequences due to their large extent and tsunami generating potential. Enormous scars of ancient underwater mass movements up to tens of kilometres in length and width are found on slopes with very mild gradients of a few degrees. These gradients are well below the sediments angle of internal friction, suggesting that overpressures (pore-pressures in excess of hydrostatic) play an important role in the failure initiation. These overpressures can arise in short term from seismic loading or in long term from sedimentation, if the deposition of new sediments occurs faster than the consolidation of the underlying soil. This long-term process can weaken slopes over time until eventually a sufficiently strong earthquake provides the final trigger and causes instability of the slope.

The sediment deposition imposes a loading of the underlying sediments, which leads to mechanical compaction of the soil and fluid flow towards the seafloor. If the sediments are deposited at a slow rate, this process leads to the build-up of normally consolidated slopes. In cases of high sedimentation rates and low permeability of the sediments, however, the sediment loading occurs faster than the dissipation of the pore fluid, leading to overpressured and weakened slopes (e.g. Dugan and Sheahan, 2012). Particularly large submarine landslides have been found in regions where sedimentation rates are high with larger areas affected by landsliding (McAdoo et al., 2000; ten Brink et al., 2016). In contrast, areas with high seismicity appear to show numerous but smaller, shallower slope failures (Urgeles and Camerlenghi, 2013). Stoecklin et al. (2017) showed that pre-conditioning through rapid sedimentation can set the conditions for the development of large, slope-parallel slope failures, using a simplified, quasi-static representation for earthquake loading. The purpose of this paper is to explore...
how overpressured slopes behave during dynamic loading. Dynamic analyses of normally consolidated slopes have shown that shear strains tend to accumulate near the surface when subjected to seismic loading (Biscontin et al., 2004). In contrast, with the presence of a weaker layer it was shown that shear strains tend to concentrate within the weak layer, determining the failure mode of such slopes (Biscontin and Pestana, 2006; Nadim et al., 2007). The origin of such weak layers, however, is not well understood (Masson et al., 2010).

In this paper both the sedimentation process and the dynamic earthquake loading is modelled in a two-step procedure. Using this procedure, the effect of overpressures on the seismic response of submarine slopes and its effect on the formation of a shear zones is studied and discussed for an example case.

2. METHODOLOGY

To analyse the seismic response of overpressured slopes, a two-step methodology is developed within the ABAQUS computing environment; (i) In a first step the sedimentation process is modelled by simulating the deposition of sediments over time on an inclined ground using a coupled hydro-mechanical finite-element approach. This step allows simulating the stress evolution within the sediments and their shear strength during the sediment deposition process. (ii) Subsequently, a dynamic, undrained, total stress finite element analysis is carried out to simulate earthquake events, using the stress-, strength and stiffness profiles from the sedimentation analysis as initial conditions. Both the sedimentation analysis and the seismic analysis are carried out for infinite slope conditions, meaning that lateral boundary effects and changes in slope curvature are neglected.

2.1 Sedimentation model

We consider a homogeneous sediment layer on an inclined, impermeable, rigid base. The thickness of the layer (T) is increasing with time due to the deposition of new sediments on the seafloor. This nonlinear moving boundary value problem is modelled using a coupled hydro-mechanical updated Lagrangian finite element approach, in which elements are activated and stacked in a sequence of steps (Figure 1a). The procedure is described in detail by Stoecklin et al. (2017). It allows modelling the fluid flow towards the seafloor and the evolution of overpressures and effective stresses with time. From this simulation the sediments strength-, density and stiffness profile can be derived, which serves as initial conditions for the subsequent seismic analysis.
2.2 Seismic model

Using the stiffness, strength and density profile from the sedimentation analysis, the seismic model is built for the given geometry within the ABAQUS computing environment (Dassault Systèmes Simulia, 2013). A multi-yield-surface constitutive model, described in the following sections, is implemented as a user subroutine and used for the dynamic total stress analysis. It allows matching the deviatoric stress-strain response to the backbone curve of an undrained test and accounting for the non-linear hysteretic un- and reloading behaviour during undrained cyclic loading of clays.

2.2.1 Description of the FE-model

A dynamic, implicit total stress finite element (FE) analysis procedure is employed for the seismic analysis. The geometry consists of a uniform sediment layer on an inclined, elastic base. The soil column is only subjected to shear waves, which are assumed to propagate in the direction perpendicular to the slope. Minimizing the computational cost, only a single column of stacked elements is modelled (Figure 1b). To enforce infinite slope conditions, the rotational degrees of freedom are constrained at all nodes and the translational degrees of freedom of the nodes on one side are tied to the corresponding nodes on the other side (Stoecklin et al., 2017).

In many cases sediments are deposited on scars of previous mass movements, consisting of stiffer, overconsolidated soil. The impedance contrast between the shear wave velocities of the overconsolidated base and the deposited sediments, however, is not necessarily large enough to justify the use of a rigid base boundary condition. Therefore, a compliant base boundary condition was used, absorbing downwards propagating waves and preventing their reflection back into the model. This is done using infinite elements (Figure 1b). Since the quiet boundary must be able to move freely to absorb incoming waves, the ground motion acceleration cannot be prescribed directly. Instead the acceleration time history is transformed into a stress-time history (Equation 1), which is then applied to the quiet boundary (Mejia and Dawson, 2006):

\[ c_s(t) = 2v_{su}(t)\rho V \approx 2v_{su}(t)\sqrt{\rho G} \quad (1) \]
where $\rho$ is the bulk density, $G$ the shear modulus, $V_s$ the shear wave velocity of the base material and $v_{su}$ is the particle velocity of the upward propagating wave, i.e. half the ‘outcrop’ motion (Mejia and Dawson, 2006).

Properties varying with depth such as the small strain shear modulus $G_0$, the bulk density $\rho$ or the undrained shear strength $q_{fu}$ as well as the static stress conditions are derived from the sedimentation analysis results and prescribed as initial conditions.

2.2.2 A benchmark case

In order to verify the FE modelling procedure described above a benchmark analysis was performed. The output is compared to results obtained using DEEPSOIL – a software for one-dimensional site response analyses in the time domain (Hashash et al., 2016). Although the DEEPSOIL software is not applicable for seismic slope stability analyses, it allows benchmarking the developed FE procedure and implemented constitutive model for a special case of a horizontal soil deposit.

A normally consolidated submerged soil layer on an elastic, horizontal base is considered (Figure 2a). The variation of the density $\rho$ and vertical stress is defined using the logarithmic relation between stress and void ratio, typically found from oedometer tests on clay samples (Figure 2b, c). The small-strain shear modulus increases with depth proportionally to the vertical effective stress (Figure 2d). Furthermore, the shear modulus of the elastic base is assumed to be twice the value at the bottom of the soil layer and the bulk density is 5% larger than the bottom value. The backbone curve of the multi-surface constitutive model used in the FE analysis is matched to the DEEPSOIL built-in hyperbolic model (MKZ model) (Hashash et al., 2010).

![Figure 2: Input parameters for the benchmark case: (a) geometry, (b) vertical effective stress profile, (c) bulk density profile and (d) shear stiffness profile.](image)

The soil profile is subjected to the Northridge ground motion (Table 1) and the results are shown in Figure 3. Comparing the output from the FE model with the DEEPSOIL results shows a good agreement (Figure 3b, c), thus validating the FE modelling procedure.
2.2.3 Constitutive model

The undrained shear strength of clays mainly depends on their intrinsic properties and the consolidation stress. Undrained simple shear tests on normally consolidated marine clay samples typically show an increase in shear stress up to a peak value during shearing, followed by a decrease in shear resistance to a residual value (Figure 4b). While the peak strength increases with increasing consolidation shear stress, the residual strength at large strains seems to be unaffected by the consolidation shear stress (Pestana et al., 2000). The stress-strain curve during loading and unloading shows significant nonlinearity, resulting in strain level dependent damping during cyclic loading.

One possible way of capturing this nonlinear hysteretic stress-strain behaviour during irregular cyclic loading is by using multi-surface kinematic hardening models (Mróz et al., 1978; Prévost, 1977; Puzrin et al., 1995). In this study a combination of kinematic hardening and linear, isotropic softening is used to capture both the nonlinear hysteretic stress-strain behaviour during cyclic loading and the post-peak strain softening response. By specifying the hardening moduli of each yield surface, the shape of the backbone curve is specified in a piecewise linear approximation. To capture the influence of the consolidation shear stress on the backbone curve, the yield surfaces are initially arranged around the consolidation stress and linearly distributed up to the peak strength (Figure 4a). The latter is calculated as a function of the consolidation stress according to the Modified Cam Clay (MCC) model (Roscoe and Burland, 1970).

The strain softening response of the employed constitutive model can lead to mesh dependency of the finite element solution (Pietruszczak and Mróz, 1981). To avoid a significant mesh dependency of the solution, a simplified scaling method is employed, assuming that localized softening failure develops within a single element (e.g., Anastasopoulos et al. 2007). With this approach, the post-peak softening branch of the stress-strain curve is scaled as function of the element size used in the FE analysis. The resulting model response for an element under monotonic simple shear loading conditions is shown in Figure 4b. As a result of the kinematic hardening the model response follows Masing rules during unloading-reloading cycles. The described constitutive model allows accounting for the anisotropic, non-linear hysteretic un- and reloading behaviour during irregular, undrained cyclic loading as well as post-peak strain softening. Although the model does not account for pre-peak strength degradation during cyclic loading, for this study, focusing on the principal effects of pre-conditioning on the seismic response of slopes, this is considered acceptable.
3. ANALYSIS OF AN EXAMPLE CASE

Using the two-step methodology described above, the seismic response of an example case is analysed by subjecting an infinite slope profile to a number of ground motions at two different stages in the sediment deposition process. The slope considered in this example analysis is inclined by $\alpha = 6^\circ$ and built of homogeneous sediments with constant intrinsic properties. The angle of internal friction of the soil is $\phi' = 30^\circ$, further analysis parameters can be found in the appendix (Table 2). To highlight the influence of the overpressures, the seismic response at each stage is simulated for two cases; in case A) the sediments were deposited over a very short time interval (fast sedimentation rate) whereas in case B) the same mass of sediments is deposited over a hundred times longer time interval.

3.1 Initial conditions

The simulation of the sedimentation process provides information about the thickness of the accumulated sediment layer and the consolidation state of the slope at the two different stages. At the early stage the thickness of the accumulated sediments is about $T_1 \approx 8m$ and the overpressures are relatively small, resulting in nearly normally consolidated soil profiles for both case A) as well as case B) (Figure 5b). At the second stage, however, the sediments deposited at a fast rate (Case A) are significantly overpressured, resulting in lower effective stresses and a relative weakening of the slope. The sediment layer deposited at a slow rate (Case B), on the other hand, is still nearly normally consolidated. The sediment thickness at this stage is about $T_2 \approx 23m$. Note that due to the different degrees of consolidation of case A) and B) the thickness for both cases is not exactly the same. Knowing the variation of effective stresses with depth, the undrained shear strength profile is computed according to the MCC model. For the normally consolidated soil profile (Case B) the undrained shear strength $q_{u}$ increases linearly with depth, whereas for the overpressured slope (Case A) the relative weakening leads to a lower strength at greater depths (Figure 5c). Furthermore, the sedimentation analysis provides the void ratio and density distribution within the sediment layer (Figure 5d).
Using these results, the initial conditions for the seismic analysis are derived. The sedimentation simulation is based on a coupled, static effective stress analysis, whereas the seismic analysis employs a dynamic total stress analysis procedure. Consequently, the initial, static stress components for the seismic model are calculated according to Equations 2 and 3

\[ \sigma_z = \sigma'_z + \Delta u + u_{hyd} \]  

\[ \sigma_{xx} = \sigma'_{xx} \]  

where \( \sigma'_z \) is the effective stress perpendicular to the slope, \( \Delta u \) the overpressure, \( u_{hyd} \) the hydrostatic water pressure and \( \sigma_{xx} \) the shear stress in direction of the slope. The peak undrained shear strength is taken directly from the sedimentation analysis (Figure 5c) and for the residual strength ratio a value of \( \eta_r = 0.2 \) was assumed. The small strain shear modulus \( G_{max} \) is calculated as a function of the effective normal stress and the void ratio, using a relation for clays derived by Hardin and Black (1968) (Equation 4)

\[ G_0 = 1230 \cdot \frac{(2.973 - e)^2}{1 + e} \sqrt{\bar{s}_o} \]  

where \( \bar{s}_o \) is the mean effective consolidation stress in psi and \( e \) is the void ratio. The stiffness of the elastic base was chosen as twice the value at the bottom of the sediment layer and the density as 1.05 times the bottom value.

### 3.2 Analysis of seismic response

For the analysis three ground motions were selected with similar magnitudes and different frequency contents (Table 1, Figure 6). The time histories were retrieved from the PEER Strong Motion Database (PEER, 2013).
Figure 6: Response spectra of selected ground motions (damping = 5%).

Table 1: Summary of input ground motions

<table>
<thead>
<tr>
<th>Event Name</th>
<th>Year</th>
<th>Record Sequence Number</th>
<th>Magnitude</th>
<th>PGA (g)</th>
<th>Arias Intensity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Northridge - 01</td>
<td>1994</td>
<td>1020.H2</td>
<td>6.7</td>
<td>0.26</td>
<td>0.47</td>
</tr>
<tr>
<td>Loma Prieta</td>
<td>1989</td>
<td>769.H2</td>
<td>6.9</td>
<td>0.17</td>
<td>0.45</td>
</tr>
<tr>
<td>Imperial Valley - 06</td>
<td>1979</td>
<td>165.H2</td>
<td>6.5</td>
<td>0.25</td>
<td>1.20</td>
</tr>
</tbody>
</table>

Figure 7 shows the results of the seismic analysis of the slopes at the first stage ($T_1 \approx 8m$). At this stage overpressures are small for both Case A) and Case B), leading to a nearly constant static shear stress ratio of approximately $\eta_c \approx \tan \alpha$ over the entire sediment layer (Figure 7b). The accumulated shear strains due to the different earthquakes are comparable for both cases, with the largest values occurring near the seafloor (Figure 7c, d). The stress-strain curves at the locations where the largest accumulated shear strains occur also show a similar behaviour for both cases (Figure 7e, f). Even though the shear stress reaches peak strength, the resulting shear strains are not large enough to cause a complete softening of the soil.
Figure 7: Results from seismic analysis at first stage ($T_1 \approx 8\text{m}$): (a) Illustration of geometry, (b) variation of consolidation shear stress ratio and accumulated shear strains due to seismic loading for (c) fast sedimentation case (Case A) and the slow reference case (Case B). The stress-strain curves for the Imperial Valley ground motion are shown for the locations where the maximum strain was reached in (e) (Case A) and (f) (Case B).

Running the seismic analysis for the same two cases at the second sedimentation stage leads to very different results. Whereas for the normally consolidated slope (Case B) the static shear stress ratio remains nearly constant, the same ratio is significantly larger for the overpressured slope (Case A), showing a distinct maximum at a defined depth and illustrating the weakening by the overpressures (Figure 8b). Furthermore, for case B) this maximum lies above the residual shear stress ratio $\eta_r = 0.2$, meaning that the slope is in a quasi-stable state (Puzrin et al., 2016). Applying the selected ground motions to this slope profile leads to accumulation of large shear strains during seismic loading close to the depth of maximum mobilized shear strength (Figure 8c). The stress-strain curve for the location of maximum accumulated strain is shown for the Loma Prieta input motion (Figure 8e). The seismic loading causes an exceedance of the peak shear strength and a strength reduction due to softening, which is, however, not substantial enough to reduce the shear strength below the static shear stress. For the Imperial Valley input motion, on the other hand, the degradation of the shear strength was large enough to bring the slope into an unstable state under static conditions, resulting in a catastrophic failure of the sediments above this localized failure zone. Similar to the Loma Prieta input motion the location of failure is close to the maximum static stress ratio. For the normally consolidated slope profile (Case B), on the other hand, the largest shear strains are accumulated near the seafloor, comparable to the response at the first stage and are much smaller than in Case A) (Figure 8d). Comparing the stress-strain curve at the location where strain localization occurred in Case A) shows that the shear stress does not exceed peak strength and that the shear strains remain within a small range (Figure 8f).
3.3 Discussion of the results

The analysis highlights that excess porewater pressures due to rapid sedimentation can have a significant effect on the seismic response of submarine slopes. For the investigated normally consolidated slope profiles the largest shear strains are accumulated near the seafloor, where the shear strength and stiffness is very low. A similar behaviour was also observed in previous studies (Biscontin et al., 2004; Biscontin and Pestana, 2006; Puzrin et al., 1997). Due to the lower shear strength near the seafloor and the nonlinear stress-strain behaviour, a significant amount of energy is dissipated, limiting the dynamic impact on the sediments at greater depth. For the overpressured slope profile, on the other hand, ground motion can cause strain localization in proximity to the depth where the mobilized shear stress ratio is maximal. This can lead to the nucleation of a slope-parallel shear zone and formation of large, potentially devastating submarine mass movements (Puzrin et al., 2016; Stoecklin et al., 2017). The depth of failure localization corresponds well with the depths suggested by Stoecklin et al. (2017), based on a simplified quasi-static earthquake representation. Among the three ground motions of similar magnitude, the one with the highest Arias intensity and highest energy content at low frequencies appears to have the strongest impact on the stability of the investigated slopes. For a generalized statement, however, a more comprehensive study is required.
The presented approach has a number of shortcomings, which need further development. Stiffness- and strength degradation during pre-peak cyclic loading is neglected in the seismic analysis as well as rate effects on stiffness- and strength properties of the soil. Furthermore, the analysis in this study is limited to infinite slope conditions. While this idealization allows assessing the principle influences of overpressures on the seismic response of submerged slopes, it does not allow computing the total release volume of a potential mass movement.

4. CONCLUSIONS

A two-step procedure for modelling the build-up of submarine slopes over time through sedimentation followed by seismic loading is presented. Using this procedure, the effect of fluid overpressures on the seismic response of slopes was studied by comparing the response of overpressured and normally consolidated slopes. Slow deposition of sediments leads to normally consolidated soil profiles, in which case the shear resistance increases nearly linearly with depth. In contrast, where sediments are deposited at a fast rate, the loading imposed by the sediments can outpace the drainage of the pore-fluid, resulting in overpressured and weakened slopes where the shear resistance increases at a slower rate with depth.

The results of the seismic example analyses show that subjecting the normally consolidated slope profiles to seismic loading results in an accumulation of permanent shear strains near the seafloor where the stiffness and undrained shear strength is the lowest. In the overpressured slope, however, strains no longer accumulated near the seafloor during earthquake loading, but localized at a specific depth below the seafloor, leading to the nucleation of a narrow failure zone. If the ground motion is sufficiently strong to cause a significant dynamic shearing of the soil, it can trigger a catastrophic failure of the slope along this failure zone and lead to the emergence of large, non-shallow mass movements with potentially devastating consequences.

The presented model does not include rate effects on the stiffness and strength properties of the soil and stress- and strength degradation during pre-peak cyclic loading is not accounted for. These potential shortcomings remain to be refined in future work. Furthermore, assessing failure geometries and release volumes of submarine mass movements requires analysis of the full 2D or 3D slope geometry. While this is not possible with the idealized infinite slope analysis presented here, the described methodology does allow investigating the basic dynamic behaviour of overpressured submarine slopes. Furthermore, the presented FE procedure can in principle also be applied for 3D analysis if the free-field movement at the lateral boundaries is accounted for.

5. ACKNOWLEDGMENTS

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6. REFERENCES


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**APPENDIX**

Table 2: Input parameters for example analysis

<table>
<thead>
<tr>
<th>Parameters sedimentation analysis</th>
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<tbody>
<tr>
<td>Slope inclination</td>
<td>$\alpha$</td>
<td>6°</td>
</tr>
<tr>
<td>Angle of internal friction</td>
<td>$\varphi'$</td>
<td>30°</td>
</tr>
<tr>
<td>Initial void ratio</td>
<td>$e_0$</td>
<td>2.5</td>
</tr>
<tr>
<td>Compression index</td>
<td>$C_c$</td>
<td>0.6</td>
</tr>
<tr>
<td>Permeability index</td>
<td>$C_k$</td>
<td>0.4</td>
</tr>
<tr>
<td>Swelling index</td>
<td>$C_s$</td>
<td>0.12</td>
</tr>
<tr>
<td>Sedimentation ratio* (Case A – fast sedimentation)</td>
<td>$\Lambda_0$</td>
<td>10</td>
</tr>
<tr>
<td>Sedimentation ratio* (Case B – slow sedimentation)</td>
<td>$\Lambda_0$</td>
<td>1000</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameters seismic analysis</th>
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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Residual stress ratio</td>
<td>$\eta_r$</td>
<td>0.2</td>
</tr>
<tr>
<td>Residual shear strain</td>
<td>$\gamma_{xx,r}$</td>
<td>20.0 %</td>
</tr>
<tr>
<td>Peak shear strain (for $\eta_r$ = 0.0)</td>
<td>$\gamma_{xx,p}$</td>
<td>7.0 %</td>
</tr>
</tbody>
</table>

*The sedimentation ratio $\Lambda_0 = \frac{k_0}{m_0}$ is defined as the ratio of initial hydraulic conductivity $k_0$ of the sediments near the seafloor and the uncompacted sedimentation rate $m_0$. 