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Modeling of large deformation behaviour of marine sensitive clays and its application to submarine slope stability analysis

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Abstract:

Post-slide investigations suggest that many large-scale submarine landslides occur through marine sensitive clay layers. A nonlinear mathematical model for post-peak degradation of undrained shear strength of sensitive clay is proposed based on experimental results. A method for estimation of model parameters is presented. Incorporating the model, an analytical solution is developed to examine possible mechanisms of large-scale submarine landslides. Analyses are performed for mild infinite slopes where the failure initiates from a “fully weakened zone” of soil having undrained shear strength lower than the shear stress acting parallel to the slope. The driving force, in excess of resistance, generated from the fully weakened zone is then transferred to the surrounding soil elements resulting in shear band formation due to strain-softening behaviour of sensitive clays. When the length of the fully weakened zone is greater than a critical length, catastrophic shear band propagation (self-driven without any additional external force) occurs that could result in large-scale offshore landslides. A simple design chart is developed to calculate the critical length. Compared with a previous study based on a linear post-peak shear strength degradation model (Puzrin and Germanovich 2005), the present study gives a conservative estimation of critical length for catastrophic shear band propagation.

Key words: sensitive clay; large deformation; submarine landslides; shear band; undrained shear strength

Introduction

Submarine landslides near the base of the continental slope are a major threat to many offshore facilities such as pipelines, foundation systems and wellheads. The occurrence of numerous submarine landslides has been reported in the literature—some of them are small while some are

very large such as the Storegga slide in the Norwegian Sea or the Grand Banks slide in offshore Newfoundland. Using advanced technologies, some slide morphologies and potential causes have been explained (Hampton et al. 1996; McAdoo 1999; Locat and Lee 2000, 2002; Canals et al. 2004; Sultan et al. 2004; Gee et al. 2005; Solheim et al. 2005; Krastel et al. 2006; Masson et al. 2006; Locat et al. 2009; Twichell et al. 2009; Conway et al. 2012) and some risk frameworks have been proposed (Lee and Edwards 1986; Lee et al. 1991; Angell et al. 2003; Nadim et al. 2003, 2005; Nadim and Locat 2005; Niedoroda et al. 2003; Masson et al. 2006; Mosher 2009; Laccase and Nadim 2009). Offshore slopes are typically very mild ($<10^\circ$, Hadj-Hamou and Kavazanjian 1985). Therefore, the question is why such large-scale landslides occur in mild slopes. Post-slide field investigations show that the existence of a marine sensitive clay layer could be one of the main causal factors of many offshore landslides (Dan et al. 2007; Sultan et al. 2010). Dan et al. (2007) identified the existence of sensitive clay layers between 30 and 45 m below the seabed near the 1979 Nice harbour catastrophe area. Their analysis shows that the failure of the slope was progressive in nature through the sensitive clay layers. Thin layers of interbedded marine/glacimarine clay layer were also encountered in many other places (e.g. Laberg et al. 2003; Yang et al. 2006; Bryn et al. 2005; Solheim et al. 2005). As an example, Fig. 1 shows the stratigraphic sub-division of the layers involved in Storegga slide inferred from a field investigation and geotechnical laboratory testing (Bryn et al. 2005). The failure occurred through the marine/glacimarine sensitive clay layers. Over the last decades the focus on mapping and characterization of weak layer has increased significantly (Lastras et al. 2004; Kvalstad et al. 2005a; L'Heureux et al. 2010; L'Heureux et al. 2012; Locat et al. 2014). Locat et al. (2014) discussed the formation and role of weak layers on submarine landslides and highlighted that the strain-softening behaviour of a weak layer could be the cause of many large-scale progressive

failures in mild submarine slopes. Many studies also reported that the failure initiates locally in a section of the weak zone and then propagates. Therefore, the role of a weak layer in triggering a landslide is twofold. The shear strength of a section of the weak layer might be decreased due to various reasons such as geological activities, pore pressure generation, earthquakes and plastic shear deformation (Lastras et al. 2004; Locat et al. 2014), which could create a “fully weakened zone” (termed as “discontinuity” in Puzrin and Germanovich (2005)). If the shear stress along the fully weakened zone is greater than the reduced shear strength, the stress will be transferred to the surrounding soil elements through which shear band propagation might occur if the soil has strain-softening behaviour. If the length of the fully weakened zone is greater than a critical length, the propagation of shear band will continue under existing driving forces without any additional external forces, termed as “catastrophic” shear band propagation (Puzrin and Germanovich 2005), which is the focus of the present study.

Limit equilibrium analyses has been used for stability assessment of submarine slopes (e.g. Karlsrud and Edgers 1982; Lee and Edwards 1986; Hampton et al. 1996; Lee and Baraza 1999; Brink et al. 2009; Leynaud et al. 2004; Kvalstad et al. 2005a). While some of the researchers used computer programs, such as Slope/W, in order to accommodate complex geometry and soil profiles, a large number of studies simply assume infinite slope conditions (Almagor and Wiseman 1991; Mello and Pratson 1999; Dimakis et al. 2000; Leynaud et al. 2004; Kvalstad et al. 2005b; Nixon and Grozic 2007). One of the major limitations of the limit equilibrium method is that it cannot explain the progressive development of a failure surface due to post-peak softening of sensitive clays. The formation of large inelastic shear deformation zones (shear bands) in the sensitive clay layer and its propagation could explain the development of a failure surface and many other mechanisms involved in large landslides, as discussed in this paper.

The mechanism of progressive failure of onshore slopes with sensitive clays has been described by a number of authors (e.g. Christain and Whitman 1969; Bernander 1978, 2000; Locat et al. 2008, 2011, 2013; Quinn et al. 2011). Using the concepts of linear elastic fracture mechanics (LEFM) (Palmer and Rice 1973), Quinn et al. (2011) proposed an analytical model for development of a shear band in an infinite slope with a vertical step cut. Quinn et al. (2012) conducted FE analysis to demonstrate the effects of toe erosion on propagation of a shear band through a weak predefined shear zone. Locat et al. (2011) demonstrated the mechanisms of shear band formation for an idealized condition in an infinite slope by applying an external force parallel to the ground surface near the toe above a predefined shear zone. Locat et al. (2013) extended their work (Locat et al. 2011) through numerical modeling using PLAXIS 2D (PLAXIS 2011) and BIFURC (Jostad and Andresen 2002) FE programs. Jostad et al. (2014) conducted FE simulation for placement of fill on a gently inclined slope. Moreover, some improved numerical approaches have been recently developed for modeling large deformation in landslides (Zabala and Alonso 2011; Cuomo et al. 2013; Prime et al. 2014). Soga et al. (2015) presented a review of available techniques for large deformation analysis of landslides. The authors of the present study also conducted large-deformation FE analyses to model spread due to toe erosion as observed in onshore sensitive clay slopes (Dey et al. 2015).

The failure of an onshore sensitive clay slope might occur due to toe erosion and upslope loading that result in upslope and downslope propagation of shear bands, respectively. However, in a long offshore slope, the existence of a strain-softening clay layer and a fully weakened zone might result in propagation of a shear band both in the upslope and downslope directions at the same time (Puzrin and Germanovich 2005). Puzrin and Germanovich (2005) solved this problem for a long infinite slope using three different approaches: LEFM, energy balance and process

zone; which has been further extended to account for nonlinear slope geometry in Puzrin et al. (2015). They showed that, if the length of the fully weakened zone exceeds a critical length, the shear band propagation will continue leading to large-scale catastrophic global failure.

The aim of this study is to present a mathematical model for post-peak nonlinear stress–strain behaviour of sensitive clays based on experimental results available in the literature. This model is then used to calculate catastrophic shear band propagation and it is shown that a relatively small section of weak zone could cause large-scale submarine landslides, which cannot be explained using the traditional limit equilibrium methods. Detailed mathematical formulations, parametric study and comparison with previous studies are also presented.

Problem definition and assumptions

Figure 2(a) shows the geometry of a typical offshore slope inclined at an angle β . In most real cases, the soil is not homogeneous and may have different soil layers similar to the one shown in Fig. 1. The slope might contain strain-softening clay layer(s) such as marine clay. In Fig. 2(a), the upper soil layer represents a relatively strong soil such as glacial clay. Below the upper soil layer of thickness h , a strain-softening layer, such as a marine clay layer, is assumed. The thickness of the softening layer could be very thin to a couple of meters. In this study, a thin marine clay layer is considered. Multiple strain-softening clay layers might be encountered in the field (e.g. Fig. 1); however, for simplicity, this study considers only one strain-softening clay layer. It is also assumed that the lower soil layer below the strain-softening clay layer is stronger than the strain-softening layer. The geometry of the problem shown in Fig. 2(a) is similar to Puzrin et al. (2004) and Puzrin and Germanovich (2005). Similar to those studies, the following three assumptions are made: (i) a fully weakened zone parallel to the seabed (solid thick line CD in Fig. 2(a)) has been formed in the strain-softening clay layer (Puzrin and Germanovich 2005);

(ii) the undrained shear strength in the fully weakened zone is reduced because of significant plastic deformation along this sliding plane; and (iii) the length of the fully weakened zone is considerably larger than the depth of this layer. Figure 2(b) shows schematically the variation of mobilized shear strength (s_u) in the shear band. Plastic shear deformation occurs only in the shear band AB. In the fully weakened zone (CD), $s_u = s_{uR}$. Further discussion on s_{uR} is provided in the following sections. In the end zones (BD and CA), s_u increases from s_{uR} to the peak shear strength s_{up} . Outside AB, the shear strength of the strain-softening layer is not mobilized to s_{up} .

If limit equilibrium analysis is performed on this scenario, failure of the slope may not be predicted as discussed later. Moreover, submarine slope failures occur over a large distance and in some cases several kilometers length (Masson et al. 2006). Now the question is whether this large failure plane is formed by an extension of the fully weakened zone in the form of shear band propagation, and if so, what is the minimum length of the fully weakened zone required to initiate self-driven shear band propagation such that it extends over a sufficiently large distance to cause a global failure of the slope. In order to answer these questions, the strain-softening behaviour of the weak sensitive clay layer is investigated first.

Modeling of post-peak shear strength degradation of sensitive clay

Based on high resolution seismic profiles and close examination of the Storegga Slide morphology, previous studies (e.g. Bryn et al. 2005; Gauer et al. 2005) showed that the failure pattern of this submarine slope is very similar to onshore large-scale landslides through sensitive clays commonly encountered in Eastern Canada and Scandinavia. However, unlike quick clay which could be highly sensitive, offshore marine/glacimarine clays are generally moderately sensitive (DeGroot et al. 2007). The post-peak degradation of undrained shear strength of sensitive clay occurs with plastic shear strain/displacement. Generally a large displacement/strain

is required to reach the remoulded condition; however, in most typical laboratory tests such large strains cannot be applied. For example, in the comprehensive experimental study during the Ormen Lange development project (Kvalstad et al. 2005a) and also in other studies (e.g. Lunne et al. 2006), mostly direct simple shear (DSS) and triaxial tests were conducted where shear strains of the order of 20% could be applied. However, in submarine landslides, shear strains in the failure plane could be significantly higher than 20%, and s_u of marine/glaciomarine clays generally decreases with strain even after 20%. In other words, triaxial and DSS tests do not cover the whole range of strains for s_u degradation from the peak to residual.

Tavenas et al. (1983) reported that retrogression of landslides in sensitive clay slopes depends on energy required to remould the clay. Conducting four different types of tests—impact on a rigid surface, impact from falling objects, extrusion through a narrowing tube; and shear reversals in a large shear box—they showed that the normalized shear strength (defined in terms of remoulding index, I_r) decreases with strain energy (w_N). Based on some realistic assumptions, Quinn et al. (2011) converted the I_r – w_N curves of Tavenas et al. (1983) to stress–displacement relationships as shown by the symbols in Fig. 3. Interested readers are referred to Quinn et al. (2011) for further details. The authors understand that there are some uncertainties both in experimental methods and interpretation techniques; however, this data is used to validate the post-peak softening models because these test results cover the strength degradation behaviour from small to large strains for sensitive clays collected from a number of sites. The authors also suggest that the model parameters need to be calibrated further if a new set of data becomes available.

The following are the key features of s_u/s_{up} vs δ plots in Fig. 3: (i) s_u decreases nonlinearly with δ and the rate of decrease is high at the beginning and very low at large values of δ ; (ii) the rate of decrease of s_u with δ is not the same for all clays tested, (iii) the percentage of reduction from s_{up}

is not same for all clays, which depends upon sensitivity (S_t). In order to capture these features, the following exponential function is proposed for the post-peak s_u variation, as shown in Fig. 4.

$$[1] \quad s_u = (1 + (S_t - 1)e^{-3\delta/\delta_{95}})s_{uR}$$

where s_u is the mobilized undrained shear strength at δ ; δ_{95} is the value of δ at which the undrained shear strength is reduced by 95% of $(s_{up} - s_{uR})$ and $S_t = s_{up}/s_{uR}$. Equation (1) is similar to the strength degradation equation proposed by Einav and Randolph (2005), but in terms of δ not in plastic shear strain (γ^p). In the post-peak region, the shear strain tends to localize in a very small zone along the shear band. Therefore, it is preferred to use δ instead of γ^p , although they are interrelated if the thickness of the shear band is known. Further discussion on this issue is available in Dey et al. (2015) and Quinn et al. (2011). Equation (1) and its similar form have been used previously for numerical and analytical modeling of sensitive clay slope failure (Dey et al. 2013, 2015, 2016; Zhang et al. 2015). However, calibration of this model against laboratory test data is required, which is presented in the following sections.

Quinn et al. (2011) showed that the residual shear strength is mobilized at very large displacements (δ_r). In most cases, this value cannot be estimated properly because the rate of decrease of s_u at large δ is very small (Fig. 3). For example, they reported $\delta_r > 3.0$ m for Saint-Hilaire clay and > 5.0 m for Saint-Léon clay from 4.8 m depth. At this stage one important question needs to be answered—does the value of δ_r need to be defined properly in progressive failure analysis and if not, what displacement and corresponding mobilized s_u could be used for an acceptable analysis.

The mechanism of post-peak softening of sensitive clay is complex. Based on previous studies, Thakur (2014) divided the softening process into two phases. The first phase is viewed as post-rupture stage that might be reached by shear strain levels of 10–20%. On the other hand, the

second phase is referred as a residual phase which is attained at larger strains. A number of experimental studies conducted (Gylland 2014; Gylland et al. 2014; Thakur et al. 2014) show that the post-peak softening behaviour of soft sensitive Norwegian clays in the first phase is significantly influenced by the rate of shearing—more specifically, shear induced pore pressure and localized drainage in the shear band influence the strength degradation rate and shear band thickness. These observations are consistent with previous studies on soft Swedish clay (Bernander and Svensk 1982; Bernander et al. 1985). In the second phase, cohesion and friction softening might occur (Thakur et al. 2014). A complete residual (remolded) state might occur if a specimen is sheared to several hundred millimeters, which is equivalent to several hundred percent shear strain (Stark and Contreas 1996). However, Bernander (2000) argued that post-slide investigations does not show relevance of completely remoulded shear strength (s_{ur}) and therefore recommended an undrained shear strength s_{uR} ($>s_{ur}$) for progressive failure analysis that mobilizes in the shear band as a result of considerable shear displacement. Therefore, s_{uR} is used in this study.

The following method is proposed to estimate the two model parameters (δ_{95} and S_t) in Eq. (1).

To estimate S_t , find the value of s_u/s_{up} when the slope of s_u/s_{up} vs δ curve is very small, as shown by large open circles in Fig. 3. The vertical coordinates of these points represent s_{uR}/s_{up} . The inverse of s_{uR}/s_{up} gives an approximate value of S_t . Table 1 shows the values of estimated S_t for the marine sensitive clays presented in Fig. 3. The sensitivity of these clays would be higher if the completely remoulded shear strength (s_{ur}) is considered (Tavenas et al. 1983; Quinn et al. 2011); however, that may not have significant effects on the initiation of slope failure as discussed below (also see Bernendar 2000).

Although reasonable estimation of s_{uR}/s_{up} is possible, estimation of δ at this s_{uR}/s_{up} is not easy because a slight change in s_{uR}/s_{up} results in a large variation of δ as the curve is very flat in this range of δ . In the proposed model (Eq. 1), δ_{95} is required instead of δ at s_{uR}/s_{up} . To estimate δ_{95} , the following steps are followed:

- (i) Calculate s_u/s_{up} for 50% reduction of $(s_{up}-s_{uR})$, denoted as s_{u50}/s_{up} (Fig. 4), as

$$[2] \quad \frac{s_{u50}}{s_{up}} = (1 + 1/S_t)/2$$

which is shown by large open triangles in Fig. 3.

- (ii) From test results find the value of δ required to mobilize s_{u50}/s_{up} , which is denoted as δ_{50} (Fig. 4). As an example, δ_{50} for Louiseville clay (183 mm) is shown in Fig. 3 by a vertical arrow.

- (iii) Rearranging Eq. (1) and inserting $s_u=s_{u50}$ and $\delta=\delta_{50}$, the value of δ_{95} can be calculated as

$$[3] \quad \delta_{95} = -\frac{3\delta_{50}}{\ln(1 - S_t(s_{u50}/s_{up}))(S_t - 1)}$$

Table 1 shows calculated values of δ_{95} for these soils. Now, using the values of S_t and δ_{95} , the post-peak variation of s_u are calculated using Eq. (1), which are shown by the solid lines in Fig. 3. The calculated s_u/s_{up} matches reasonably well with the test results for a wide range of δ .

In above calculations, the point at 50% reduction of $(s_{up}-s_{uR})$ is considered because at lower displacements a slight change in δ results in a significant change in s_u while at large displacements a small change in s_u significantly changes the value of δ . Both δ and s_u are required in the calculation of δ_{95} as shown in the right side of Eq. (3). Eq. (1) fits better with test data for a wide range of δ if the point at 50% reduction of $(s_{up}-s_{uR})$ is considered as discussed above.

A question still remains—what is the effect of S_t as it has been estimated from the flat segment of the s_u/s_{up} vs δ curve. To show this effect, assume that there are $\pm 10\%$ error in estimation of S_t . Now using 110% and 90% sensitivity values of column 2 in Table 1 and following steps (i)–(iii) the values of δ_{50} and δ_{95} are recalculated and are shown in parentheses in columns 3 and 4 in Table 1. The dashed and dotted lines in Fig. 3 show the variation of s_u/s_{up} for 110% and 90% of S_t , respectively. Small difference between these curves suggests that $\pm 10\%$ error in estimation of S_t does not change the post-peak strength degradation curves significantly.

Eq. (1) is rearranged in a form similar to Skempton's "Residual Factor" (Skempton 1964) as

$$[4] \quad (s_{up} - s_u)/(s_{up} - s_{uR}) = 1 - e^{-3\delta/\delta_{95}}$$

Figure 5 shows that Eq. (4) can model the trend of test data very well in this normalized form. The proposed post-peak softening model (Eq. 1 or 4) has the following advantages. It can model the nonlinear degradation of s_u with δ . The model parameter S_t is obtained from the lower part of the s_u/s_{up} vs δ curve where a small variation of s_u occurs with δ and therefore potential error in estimation of s_{uR} is low. However, the other key model parameter δ_{95} is not estimated from the lower part, because potential error in estimation could be high. The linear models (L1–L3) shown in this figure are discussed later.

In the following section, this model (Eq. 1) is used to examine shear band propagation in a submarine slope having a layer of sensitive clay (Fig. 2(a)).

Shear Band Propagation in Submarine Slopes

As Eq. (1) is an exponential curve, a very large δ is required to mobilize s_{uR} . Therefore, in this analytical formulation the strength degradation from s_{u95} to s_{uR} is defined by a step decrease at δ_{95} , which means that the curve $bcc'd$ is used instead of bcd (Fig. 4). If the shear strength in the

fully weakened zone is less than the shear stress in the x direction, the downward driving force will be transferred to the adjacent soil elements near both ends of the fully weakened zone (Fig. 2(a)), which creates two end zones in the softening layer (BD and CA) where s_u gradually increases from s_{u95} (at D and C) to s_{up} (at A and B) as a function of relative displacement between the upper soil and the strain-softening clay layers (δ). These transition zones BD and CA are referred to as “end zones” and are shown by thick dashed lines in Fig. 2(a). In other words, $s_u(x)=s_{uR}$ in the fully weakened zone and $s_{u95}\leq s_u(x)\leq s_{up}$ in the end zones. The effect of this assumption is also shown later in this paper by a step decrease of s_u at $2\delta_{95}$.

The sum of the lengths of the fully weakened zone and end zones (ω_1, ω_2) defines the total length of shear band, l (AB in Fig. 2(a)). In other words, $s_u\leq s_{up}$ in the shear band. In the absence of any additional triggering effects, two possible scenarios might be encountered: (i) a short fully weakened zone could generate two end zones but the slope is globally stable; (ii) if the length of fully weakened zone exceeds a certain length, self-driven shear band propagation continues over a large distance which might cause a catastrophic global failure.

Referring to Fig. 2, as the vertical stress increases with depth, the normal stress acting on a plane perpendicular to the slope (σ_x) also increases with depth. Assume that the average σ_x over depth h is $\bar{\sigma}_x$, and $\bar{\sigma}_x = p_0$ at the locations far from the fully weakened zone (large x) in intact slope or the location where $\delta=0$. Above the shear band $\bar{\sigma}_x \neq p_0$ at all locations because the shear stress caused by gravitational force (τ_g) is greater than shear resistance in the shear band. Therefore, the redistribution of forces results in $\bar{\sigma}_x < p_0$ in the right (unloading) and $\bar{\sigma}_x > p_0$ in the left side (loading) of the shear band.

Somewhere along the shear band $\bar{\sigma}_x = p_0$ and that plane (point O in Fig. 2) is chosen as the origin (see also Puzrin and Germanovich 2005). With respect to the origin, the segment of the shear band and the end zones are defined by x_i and ω_i , respectively, where the subscript $i=1$ and 2 represent the values for the left ($x<0$) and right ($x>0$) sides of the origin, respectively. Also it is assumed that E_1 and E_2 are the undrained Young's modulus of the upper soil layer in plane strain condition for loading ($x<0$) and unloading ($x>0$), respectively.

Mathematical Formulations

For a mild submarine slope ($\beta < 10^\circ$), shear stress along the plane AB is

$$[5] \quad \tau_g \approx \gamma' h \sin \beta$$

where γ' is the submerged unit weight. Using the equilibrium conditions of a small slice of depth h above the shear band, the variation of $\bar{\sigma}_x$ in the upper soil layer can be written as

$$[6] \quad h \frac{\partial \bar{\sigma}_x}{\partial x} = s_u(x) - \tau_g$$

where the mobilized $s_u(x)$ in the end zones can be found from Eq. (1), and in the fully weakened zone $s_u(x) = s_{uR}$.

Considering only the right side of the origin and assuming uniform elastic deformation, the change in $\bar{\sigma}_x$ in the upper soil layer above the end zone CA (Fig. 2) can be expressed as:

$$[7] \quad \frac{\partial \bar{\sigma}_x}{\partial x} = E_2 \frac{\partial \bar{\epsilon}_x}{\partial x} = E_2 \frac{\partial^2 \delta}{\partial x^2}$$

where $\bar{\epsilon}_x$ is the average normal strain in the upper soil layer in the x direction, which is related to δ as $\delta(x) = \int \bar{\epsilon}_x dx$.

Substituting Eqs. (1) and (7) into Eq. (6), the following equation can be obtained for the end zone.

$$[8] \quad hE_2 \frac{\partial^2 \delta}{\partial x^2} = (s_{up} - s_{uR})e^{-3\delta/\delta_{95}} - (\tau_g - s_{uR})$$

Equation (8) can be written in terms of normalized displacement and coordinate as:

$$[9] \quad \frac{\partial^2 \hat{\delta}}{\partial \hat{x}^2} = e^{-3\hat{\delta}} - r$$

where $\hat{\delta} = \delta/\delta_{95}$ and $\hat{x} = x/l'_u$. The characteristics length (l'_u) and stress ratio (r) are defined as

$$[10] \quad l'_u = \sqrt{hE_2\delta_{95}/(s_{up} - s_{uR})}$$

$$[11] \quad r = (\tau_g - s_{uR})/(s_{up} - s_{uR})$$

The displacement boundary conditions at the tips of the upper end zone CA (Fig. 2(a)) are

$$[12a] \quad \hat{\delta}(\hat{x}_2) = 0 \text{ at point A}$$

$$[12b] \quad \hat{\delta}(\hat{x}_2 - \hat{\omega}_2) = 1 \text{ at point C}$$

where $\hat{x}_2 = x_2/l'_u$ and $\hat{\omega}_2 = \omega_2/l'_u$.

Now, consider a point on the fully weakened zone just left side of point C. The average normal stress in the upper soil layer above this point is

$$[13] \quad \bar{\sigma}_x(x_2 - \omega_2) = p_0 - (\tau_g - s_{uR})(x_2 - \omega_2)/h$$

On the other hand, just on right side of point C, the average normal stress is

$$[14] \quad \bar{\sigma}_x(x_2 - \omega_2) = p_0 + E_2 \bar{\epsilon}_x = p_0 + E_2 \frac{\partial \delta}{\partial x}$$

Equating Eqs. (13) and (14) by satisfying the overall stress equilibrium at Point C, the following equation is obtained in normalized form,

$$[15] \quad \frac{\partial \hat{\delta}}{\partial \hat{x}} = -r(\hat{x}_2 - \hat{\omega}_2)$$

which actually denotes the slope of the displacement curve at point C where $x=x_2-\omega_2$. The negative sign indicates the reduction of displacement with distance from this point.

Assuming $\partial \hat{\delta} / \partial \hat{x} = \hat{Y}$, Eq. 9 can be written as a first order differential equation as

$$[16] \quad \frac{\partial \hat{Y}}{\partial \hat{x}} = e^{-3\hat{\delta}} - r$$

which can be rearranged further as

$$[17] \quad \hat{Y} \cdot \partial \hat{Y} = (e^{-3\hat{\delta}} - r) \partial \hat{\delta}$$

Now, integrating equation (17) with boundary conditions at point C and then equating it with the gradient of the displacement curve given by Eq. (15), the following expression can be found after some algebraic manipulation.

$$[18] \quad \frac{\partial \hat{\delta}}{\partial \hat{x}} = \sqrt{\Omega}$$

where

$$[19] \quad \Omega = -(2/3) \cdot e^{-3\hat{\delta}} - 2 \cdot r \cdot \hat{\delta} + r^2(\hat{x}_2 - \hat{\omega}_2)^2 + (2/3) \cdot e^{-3} + 2 \cdot r.$$

Equation (18) shows the gradient of $\hat{\delta}$ vs $\hat{x}$ curve in the end zone. A viable solution is possible only if $\Omega \geq 0$. Inserting $\hat{\delta}=0$ in Eq. (19), the gradient of this curve at the tip of the shear band (point A in Fig. 2) can be obtained.

$$[20] \quad \left[\frac{\partial \hat{\delta}}{\partial \hat{x}} \right]_{\hat{\delta}=0} = \sqrt{2r - 0.634 + r^2(\hat{x}_2 - \hat{\omega}_2)^2}$$

Equation (20) shows that the gradient of $\hat{\delta}$ vs $\hat{x}$ curve at the tip of the end zone increases with r and normalized length of the fully weakened zone $(\hat{x}_2 - \hat{\omega}_2)$. For a small seabed slope (i.e. low

r), a large fully weakened zone is required for a viable solution (+ve value in the square root); however, for a steep seabed slope a solution is possible even with zero fully weakened zone, although solutions for $(\hat{x}_2 - \hat{\omega}_2) > 0$ are of the interest of this study.

Similarly, the gradient of $\hat{\delta}$ vs $\hat{x}$ at point C (Fig. 2(a)) can be calculated as

$$[21] \quad \left[\frac{\partial \hat{\delta}}{\partial \hat{x}} \right]_{\hat{\delta}=1} = -r(\hat{x}_2 - \hat{\omega}_2)$$

which shows that displacement gradient at this point also increases with r and the length of the fully weakened zone $(\hat{x}_2 - \hat{\omega}_2)$. In summary, the shape of $\hat{\delta}$ vs $\hat{x}$ curve in the end zone depends on r and the length of the fully weakened zone, which also influences the length of the end zone because $\hat{\delta}$ varies between 0 and 1 in this zone.

Similar to shear crack propagation in fracture mechanics, self-driven propagation of shear band under existing gravity load will occur only if

$$[22] \quad d\hat{\omega}_2 \leq d\hat{x}_2 \text{ for } d\hat{x}_2 \geq 0,$$

In other words, the end zone length decreases with increase in shear band length, which can be shown by calculating $\hat{\omega}_2$ for incremental lengths of $\hat{x}_2$.

In order to find $\hat{\omega}_2$ for a given $\hat{x}_2$, Eq. (18) is integrated numerically, as it cannot be done analytically, using Newton-Raphson method, writing a code in MATLAB. Integration is performed for the end zone with limits of $\hat{\delta} = 0$ to 1 and $\hat{x} = \hat{x}_2$ to $(\hat{x}_2 - \hat{\omega}_2)$ from point A to C in Fig. 2. Once the values of $\hat{\omega}_2$ and $\hat{x}_2$ in Eq. (19) are known for a specific value of r , variation of $\hat{\delta}$ in the end zone (from point A to C) is calculated integrating Eq. (18) numerically, which is then used to calculate the values of δ ($=\hat{\delta} \times \delta_{95}$) along the end zone.

Rearranging Eqs. (6) and (7) for the fully weakened zone, where $s_{u(x)}=s_{uR}$, the following equation can be obtained.

$$[23] \quad hE_2 \frac{\partial^2 \delta}{\partial x^2} = s_{uR} - \tau_g$$

Now integrating Eq. (23) twice and using the boundary conditions (at $x=0$, $\bar{\epsilon}_x = \partial\delta/\partial x = 0$ and at $x=x_2-\omega_2$, $\delta=\delta_{95}$) the following equation can be obtained for the fully weakened zone.

$$[24] \quad \delta = \frac{s_{uR} - \tau_g}{hE_2} \frac{x^2 - (x_2 - \omega_2)^2}{2} + \delta_{95}$$

Once δ is known, s_u can be calculated using Eq. (1), which represents the shear stress along the shear band.

So far the calculation has been shown only for the right side of the origin. For the left side, similar equations can be obtained except for Eq. (10) where E_2 should be replaced by E_1 .

Therefore, in normalized term $(\hat{x}, \hat{\delta})$, a similar solution can be obtained, which means that $\hat{x}_1 = \hat{x}_2$, $\hat{\omega}_1 = \hat{\omega}_2$; however, $x_1/x_2 = \omega_1/\omega_2 = \sqrt{E_1/E_2}$. For further details of this derivation readers are referred to Puzrin et al. (2004).

Therefore, the total length of the shear band ($l=x_1+x_2$) can be calculated as

$$[25] \quad l = \hat{x}_2 \left(1 + \sqrt{E_1/E_2} \right) l_u''$$

At critical length, $l^{cr} = x_1^{cr} + x_2^{cr}$, where x_1^{cr} and x_2^{cr} are the minimum length of the shear band segments in the left and right side of the origin, respectively, for initiation of self-driven shear band propagation.

The calculation process can be summarized as follows. For a given geometry and soil properties calculate the value of r using Eq. (11). To calculate the normalized end zone lengths ($\hat{\omega}_i$) in a

given shear band length ($\hat{x}_i$), solve Eq. (18) using the boundary conditions shown in Eqs. (12a and 12b). Similarly, for the critical length, numerically find the minimum value of $\hat{x}_i$ that gives a viable solution of $\hat{\omega}_i$ (i.e. without any imaginary term). Once the values in normalized term are known, multiply them by l_u'' (Eq. 10) to calculate actual lengths.

Geometry and Soil parameters

Table 2 shows the geometry and soil parameters used in the “base case” analysis. The depth of the sliding plane is $h=20$ m and the slope angle is varied from 3 degrees to 10 degrees. An extensive site investigation program for the Ormen Lange gas field development project in the Norwegian Sea provided a wide range of geotechnical properties of offshore sediments. Kvalstad et al. (2005a,b) summarized the soil conditions and geotechnical properties obtained from different phases of site investigations, where geoborings were performed to a maximum depth of 230 m below the seabed. Based on cone penetration, unconsolidated undrained triaxial, direct simple shear, ring shear, Atterberg limits, and fall cone tests, they showed the variation of undrained shear strength and sensitivity in the different soil layers. They divided the soils mainly into two categories: (i) marine and/or glaciomarine clay and (ii) glacial clay. Significant strain-softening occurs in marine clay, while glacial clays show little or no strain-softening. The sensitivity of the marine clays varied between 3 and 6.

Kvalstad et al. (2005a, b) also showed that s_{up} increases with vertical effective stress (σ'_v) and overconsolidation ratio (OCR) as

$$[26] \quad \left(\frac{s_{up}}{\sigma'_v} \right)_{oc} = \left(\frac{s_{up}}{\sigma'_v} \right)_{NC} (OCR)^m$$

Based on triaxial test results, they reported $m=0.7$, and $(s_{up}/\sigma'_v)_{NC}=0.27$ for these marine clays. Moreover, the ratio between s_{up} in DSS and triaxial compression tests is about 0.8 (Kvalstad et al., 2005b). Now, using γ' of the top soil layer, the value of σ'_v at the depth of sensitive clay layer ($h=20$ m) can be calculated. The DSS condition better represents the failure condition through the sensitive clay layer shown in Fig. 2(a). As the marine clays were found to be normally to lightly overconsolidated (Biscontin et al. 2003; Leynaud et al. 2004; Kvalstad et al. 2005b; Yang et al. 2006; Lunne and Andersen 2007), an OCR of 1 is used for simplicity. Based on the above information, $s_{up}=40$ kPa is used for the marine clay layer. For the upper glacial clay, Gauer et al. (2005) assumed $s_u=25$ kPa at the seabed increasing linearly at 2.4 kPa per metre depth. An average $\bar{s}_u=50$ kPa is therefore assumed for the upper clay layer. The undrained Young's modulus $E_u=600s_u$ is used. As glacial clay is not very sensitive (Kvalstad et al. 2005a), $S_r=1$ is used for the upper glacial clay layer.

Results

In the discussion of results, the symbols $\hat{x}_i(=\hat{x}_1, \hat{x}_2)$, $\hat{\omega}_i(=\hat{\omega}_1, \hat{\omega}_2)$ are used for brevity. Moreover, to explain the mechanisms, mainly the right side of the origin is considered (left side is similar). Therefore, the symbols x_2 , ω_2 and $(x_2-\omega_2)$ are used for the length of shear band, end zone and the fully weakened zone, respectively.

Shear Band and End Zone

Figure 6 shows the relationship between the length of the fully weakened zone $(x_i-\omega_i)$ where $s_{u(x)}=s_{uR}$ and shear band x_i where $s_u \leq s_{up}$. The vertical distance of any point on these lines from the line AB (1:1 slope) represents the end zone (ω_i) where $s_{u95} \leq s_u \leq s_{up}$. As an example, for $\beta=5^\circ$, when the fully weakened zone is 200 m (e.g. $x_i-\omega_i=100$ m) the end zone length in each side (ω_i)

is 50 m. Note that in this case $x_1 = x_2$ and $\omega_1 = \omega_2$ because $E_1 = E_2$, as discussed above. In other words, 33% of the shear band length is the end zone. The ratio between the length of the end zone and fully weakened zone ($\omega_i/(x_i - \omega_i)$) decreases with an increase in shear band length. Figure 6 also shows that, for a given length of the fully weakened zone, the end zone length is larger for a smaller slope angle. Therefore, for mild offshore slopes, the end zones cannot be neglected or assumed to be zero especially at low x_i where ($\omega_i/(x_i - \omega_i)$) is high.

The normalized length of the fully weakened zone and the shear band are also shown in the top horizontal and right vertical axes, respectively, in Fig. 6.

The solid lines in Fig. 7 show the variation of normalized end zone length $\hat{\omega}_i$ with shear band length $\hat{x}_i$ for different values of slope angles. The following key features are noted from Figs. 6 and 7:

- i) The very left point of each curve, shown by a solid circle, represents the minimum length of the fully weakened zone required for a self-driven catastrophic shear band propagation. As an example, for $\beta = 4^\circ$, a fully weakened zone of at least 200 m (i.e. $x_i - \omega_i = 100$ m) is required for catastrophic shear band propagation (Fig. 6). If it is less than 200 m, self-driven shear band propagation will not occur under the existing driving force. The value of shear band length $\hat{x}_i$ at this point represents the minimum normalized length required for shear band propagation ($\hat{x}_i^{cr}$). At this point the length of the end zone ($\hat{\omega}_i^{cr}$) is the maximum.
- ii) The length of the end zone decreases with an increase in the shear band length, which means that the relative size of the residual shear strength zone in the shear band increases with shear band length $\hat{x}_i$. Figure 7 also shows that the required condition, $d\hat{\omega}_i \leq d\hat{x}_i$ (Eq. 22), for shear band propagation.

iii) Unlike J-integral based propagation criterion (Palmer and Rice 1973), a complete solution for a wide range of shear band length $\hat{x}_i$, including the solution for low values of x_i , is obtained in the present analysis. Palmer and Rice (1973) presented the solutions only for the cases in which shear band length is very large compared to the end zone. Figures 6 and 7 show that this condition is approximately valid only for large $\hat{x}_i$ when $\hat{\omega}_i/\hat{x}_i$ is small. However, the current analysis shows that propagation could initiate at lower values of shear band lengths when the end zone length is not negligible. As an example, for $\beta=4^\circ$ in Fig. 6, continuous shear band propagation could be initiated at a minimum length of shear band of 366 m ($=2 \times 183$ m) when the end zone length is 166 m ($=2 \times 83$ m) which is 45% of the shear band length.

iv) $\hat{x}_i^{cr}$ increases with a decrease in β and therefore $\hat{x}_i$ vs $\hat{\omega}_i$ curves shift to the right for low β (Fig. 7). In other words, the minimum length required for shear band propagation is higher for lower β . Moreover, for a given shear band length $\hat{x}_i$ the end zone length $\hat{\omega}_i$ decreases with increasing β .

The inset of Fig. 7 shows the ratio $\hat{\omega}_i/\hat{x}_i$ for different values of stress ratio r at the initiation of shear band propagation, which is related to the slope angle β . As shown, $\hat{\omega}_i/\hat{x}_i$ increases almost linearly with r and at $r \approx 0.53$ (i.e. $\beta = 7.0^\circ$) $\hat{\omega}_i/\hat{x}_i \approx 1$. This implies that the end zone length is almost the same as the shear band length for $r \geq 0.53$; however, for $r < 0.53$ a finite length of the fully weakened zone is required to initiate shear band propagation.

Displacement and Shear Stress along Shear Band

Figures 8(a) and 9(a) show the variation of δ along the shear band for $\beta=4^\circ$ and 7° , respectively, for different values of shear band length $\hat{x}_i$ shown by point P₁-P₁₀ in Fig. 7. The solid red lines represent the end zone while the dashed blue lines represent the fully weakened zone where $s_u=s_{uR}$. For clarity, only the right side of $x=0$ (Fig. 2(a)) is shown. The leftmost lines in Figs. 8 and 9 show the variation of δ for the minimum length of the shear band that could cause catastrophic shear band propagation (Fig. 7). Again, as shown in Fig. 8(a), for $\beta=4^\circ$, a 100 m fully weakened zone is required for initiation of shear band propagation. This will result in total 183 m long shear band in one side of origin with 83 m end zone.

For $\beta=7^\circ$, almost the whole length of the shear band is the end zone (≈ 87 m) (Fig. 9a). In other words, shear band propagation might be initiated in a steep slope just after formation of a fully weakened zone of $s_u=s_{uR}$. The shape of δ - x curves is also different from the $\beta=4^\circ$ case. At the end of the shear band (point A in Fig. 2(a)), the slope of δ - x curve is almost zero for $\beta=4^\circ$ while it is greater than zero for $\beta=7^\circ$. This can be explained by putting $\hat{\delta}=0$ in Eq. (19). With an increase in shear band length, the end zone length decreases and the slope of δ - x curves increase for both $\beta=4^\circ$ and 7° . A shorter end zone means more stress concentration that makes propagation of the shear band easier.

Figures 8(b) and 9(b) show the variation of shear stress along the shear band. The shear stress in the fully weakened zone is s_{uR} (dashed blue line); however, in the end zone it increases gradually from s_{u95} to s_{up} . The pattern of shear stress variation is very similar to numerical calculations presented by the authors (Dey et al. 2013) for upward progressive failure.

As mentioned, a step decrease of s_u to s_{uR} at δ_{95} is used in the previous calculations. In order to show the effect of this assumption, a calculation is also performed with a step decreases of s_u at

$2\delta_{95}$, which implies that in the end zone $0 \leq \delta \leq 2\delta_{95}$ while in the previous cases it was $0 \leq \delta \leq \delta_{95}$. The variation of δ and shear stress in the shear band for $\beta=4^\circ$ at the initiation of shear band propagation are also shown by the red dotted line in Fig. 8. The shear band propagation initiates at a slightly higher x_2 ($=196$ m) than the previous analysis as shown by two vertical arrows on the horizontal axis in Fig. 8(a). As expected, the length of the end zone is higher; and, in this particular case, almost the whole length of the shear band is the end zone. Figure 8(b) shows that the shear stress gradually decreases almost to s_{uR} while in the previous case the step decrease at s_{u95} is noticeable. In summary, depending upon the strain level at the step decrease of s_u (cc' in Fig. 4), the length of the end zone might vary; however, the length of the shear band required for the initiation of shear band propagation does not vary significantly, which is the main concern in stability analysis. Therefore, the step decrease of s_u at δ_{95} is used for further analysis.

Comparison with linear post-peak softening model

A linear variation of s_u with δ has been used for progressive failure analysis (Locat et al. 2011, 2013; Quinn et al. 2011); although the problems analyzed in those studies are different from the problem considered in the present study (Fig. 2(a)). Linear strength variation has been also used for stability analysis of submerged offshore slopes of normally consolidated clay and loose sand sediments (Puzrin et al. 2004; Puzrin and Germanovich 2005). Similar to these studies, define the linear variation of s_u as

$$[27] \quad \frac{s_{up} - s_u}{s_{up} - s_{uld}} = \frac{\delta}{\delta_{ld}} \quad \text{for } \delta \leq \delta_{ld}$$

$$[28] \quad s_u = s_{uld} \quad \text{for } \delta > \delta_{ld}$$

where s_{uld} is the undrained shear strength at large deformation δ_{ld} .

When a linear model is adopted, one of the key questions is how δ_{ld} could be determined. A wide range of δ_{ld} , between 20 mm and 2,000 mm, has been reported in previous studies for sensitive clays (Locat et al. 2011; Quinn et al. 2011). As the present nonlinear model (Eq. (1)) reasonably fits tests data (Figs. 3 and 5), first assume $s_{uld}=s_{uR}$ and $\delta_{ld}=\delta_{95}$ and named as model L1, which gives the rightmost dashed linear line in Fig. 5. As most of the test data lies in the left side far from this line, also assume two more linear variations with $\delta_{ld}=0.7\delta_{95}$ (Model L2) and $0.5\delta_{95}$ (Model L3) which are also shown by two dashed linear lines in Fig. 5.

Now using Eqs. (27) and (28), instead of Eq. (1), Eqs. (8)–(19) are reformulated and solved. The lengths of the shear band and end zones are normalized by another characteristic length defined as $l'_u = \sqrt{hE_u\delta_{ld}/(s_{up} - s_{uR})}$ in which, instead of δ_{95} in Eq. (10) for nonlinear model, δ_{ld} is used for these linear models as Puzrin et al. (2004). The dashed lines in Fig. 7 show the variation of $\hat{\omega}_i$ with $\hat{x}_i$ for linear models L1-L3. Although non-normalized end zone length ω_i and shear band length x_i depend on δ_{ld} , the normalized $\hat{\omega}_i$ vs $\hat{x}_i$ curves are independent of δ_{ld} (see Eq. 19). The following are the key features of these curves.

- i) The initiation of catastrophic shear band propagation with a linear model occurs at a higher normalized shear band length than that with the nonlinear model (compare diamonds and circles); and their difference increases with a decrease in slope angle.
- ii) For a given β , the difference between the curves obtained with linear and nonlinear models are negligible at large normalized shear band lengths.
- iii) The normalized length of the end zone at the initiation of shear band propagation is higher in a nonlinear model than that with linear models.

The above features of $\hat{\omega}_i$ vs $\hat{x}_i$ curves could be further explained using the non-normalized lengths of the shear band ($x_i = \hat{x}_i l'_u$) and end zone ($\omega_i = \hat{\omega}_i l'_u$), as shown in Fig. 10(a) for $\beta=4^\circ$. x_i required to initiate shear band propagation for linear model L1 is 220 m which is significantly higher than that required from the nonlinear model (183 m). The reason behind this can be explained using Fig. 5, which shows that s_u degradation is faster in a nonlinear model than that of L1. This quick degradation results in the initiation of catastrophic shear band propagation at low x_i . The calculated length of the end zone is higher with the nonlinear model (83 m) than with L1 (67 m). In other words, the required fully weakened zone ($x_2 - \omega_2$) for the initiation of catastrophic shear band propagation is 100 m with the nonlinear model and 153 m with L1. The x_2 values required to initiate shear band propagation for linear models L2 and L3 are 184 and 155 m, respectively, with end zone lengths of 56 and 47 m, respectively.

The initiation of catastrophic shear band propagation is one of the important aspects of submarine slope stability analysis. Figure 10(b) shows the variation of δ in the shear band for nonlinear and linear models for $\beta=4^\circ$ at the initiation of catastrophic shear band propagation. When $\delta_{ld} = \delta_{95}$ is used (i.e. Model L1), the required displacement δ in the fully weakened zone is significantly higher than that from the nonlinear model. The δ vs x curves with the nonlinear model and linear model $\delta_{ld} = 0.7\delta_{95}$ (i.e. Model L2) are comparable; however, the required length of the fully weakened zone is 100 m for the nonlinear but 132 m for Model 2. Calculated δ and shear band lengths are small with Model L3 because a small δ_{ld} is used.

In summary, as Eq. (27) shows that for a given s_{up} and s_{uld} , only one input parameter (δ_{ld}) can be varied to obtain different linear curves. The above comparison shows that it is difficult to match the results (lengths of shear band, end zone and fully weakened zone, and displacement) using

the nonlinear and linear models simply by varying δ_{ld} in the linear model. As the nonlinear model better fits test results (Figs. 3 and 5), it is expected that calculated results using this model better represent the response. Although mathematical formulation and calculations using a linear model are relatively simple, one of the key input parameters δ_{ld} is very difficult to estimate which might give very different results.

Global Stability

In the previous sections, it is shown that a relatively small length of the fully weakened zone could cause the catastrophic propagation of a shear band. It is assumed in this study that the sensitive clay layer parallel to the slope in Fig. 2(a) is sufficiently long. If the soil properties and geometry meet the conditions for initiation, as discussed before, the shear band propagation will continue and at one stage global failure will occur through the upper glacial clay as shown by the inclined lines in Fig. 2(a). For undrained ($\phi=0$) condition, the failure plane should be inclined at 45° (note that this figure is not drawn to scale). Now, using the concept of limit equilibrium and balancing the forces on the soil block above the shear band acting in the x direction, the factor of safety (F_s) can be calculated as

$$[29] \quad F_s = \frac{P_p + s_{ur}l}{P_a + \tau_g l}$$

where P_a and P_p are the active and passive forces, respectively, acting at the upper and lower ends. The minimum length (l_f) required to cause the global failure of a slope is then calculated by inserting $F_s=1$, $l=l_f$ and $P_p-P_a=4\bar{s}_u h$. Note that this calculation does not account for dynamic forces caused by the catastrophic shear band propagation.

$$[30] \quad l_f = \frac{4\bar{s}_u h}{\tau_g - s_{uR}}$$

For the parameters listed in Table 2, $l_f=580$ m for $\beta=4^\circ$. In other words, if the limit equilibrium method is used, the failure of the slope will be predicted only if the length of the fully weakened zone is at least 580 m. On the other hand, according to the proposed method, the fully weakened zone required for shear band propagation can be calculated using Eq. (25) as 200 m. Shear band propagation will continue and once the shear band length becomes 580 m global failure will occur. This implies that the limit equilibrium method significantly overestimates the factor of safety because it does not consider the shear band propagation. If the sensitive clay layer is not sufficiently long (<580 m), shear band propagation will stop, and global failure will not occur. However, the slope might be marginally stable and any additional triggering factor can cause a large-scale failure. It is also to be noted here that after global failure of the soil block shown in Fig. 2(a), retrogressive failure might occur, which can be simulated using large deformation finite element modeling as presented by the authors in Dey et al. (2016).

Parametric Study

The slope angle and soil parameters influence shear band propagation and failure of a slope. A parametric study is conducted using the nonlinear model described above in which only one parameter is varied while the other parameters are kept constant as listed in Table 2, unless otherwise mentioned.

Effects of δ_{95}

Table 1 shows a wide variation of δ_{95} for different soils. The effects of δ_{95} on total shear band length at which catastrophic shear band propagation initiates are shown in Fig. 11. Each value of

β , $\hat{x}_i$ and $\hat{\omega}_i$ are calculated as shown in Fig. 7. Multiplying the normalized shear band length $\hat{x}_i$ by the characteristic length l'_u , the values of x_i are calculated which are then used to calculate l^{cr} using Eq. (25). Again, l^{cr} represents the minimum length of fully weakened zone required for catastrophic shear band propagation. For example, for $\beta=4^\circ$ and $\delta_{95}=70$ mm, catastrophic shear band propagation will initiate only if the length of the shear band is greater than 300 m which is shown by a triangle in Fig. 11. Figure 11 also shows that l^{cr} increases with δ_{95} because, for a large δ_{95} , the shear strength degradation occurs slowly over a large δ and therefore requires more fracture energy for shear band propagation than that with a small δ_{95} . Moreover, l^{cr} decreases with an increase in β because τ_g increases with β . In other words, high brittleness (i.e. rapid decrease in s_u with δ) and steeper slopes are more prone to failure.

Effects of s_{up} and Sensitivity

Figure 12 shows the $\hat{x}_i$ vs $\hat{\omega}_i$ curves for three values of peak shear strength (s_{up}). With an increase in s_{up} the $\hat{x}_i$ vs $\hat{\omega}_i$ curves shift to the right. As shown by the solid circles, $\hat{x}_i$ required to initiate catastrophic shear band propagation increases with s_{up} . Figure 13 shows the influence of sensitivity on normalized lengths of shear band and corresponding end zone. With increase in S_t , the $\hat{x}_i$ vs $\hat{\omega}_i$ curves shift towards the left.

As the length of the shear band is proportional to $\hat{x}_i$ (Eq. (25)), l^{cr} decreases with decreasing s_{up} and increasing S_t . This implies that a submarine slope that has a weak sensitive clay layer parallel to the slope is more susceptible to failure due to shear band propagation if the weak layer is highly sensitive and has low undrained shear strength.

Effects of stress ratio r and Worked example

In order to provide a simple chart for practical application, a set of analyses are performed for different values of r . Figure 14 shows the normalized x_i at the initiation of catastrophic shear band propagation (i.e. at critical length) with r . The normalized form of this curve is independent of slope angle, depth of sensitive clay layer and geotechnical properties of the sensitive clay layer (s_{up} , s_{uR} and δ_{95}), because Eq. (19) is a function of r and $\hat{\delta}$ only. If the values of s_{up} , s_{uR} and τ_g are known, r can be calculated using Eq. (11). Now, using Fig. 14 one can easily obtain $\hat{x}_i$ (as shown by arrows) which can be then used to calculate l^{cr} (Eq. 25) required for the initiation of catastrophic shear band propagation. The methodology developed in this study could be used to estimate the possibility of failure of an offshore slope with a sensitive clay layer which cannot be done properly using the traditional limit equilibrium methods.

As a worked example, consider an infinite submarine slope inclined at $\beta=4^\circ$. A slide-prone weak sensitive clay layer is at depth $h=30$ m below the seabed. The shear strength properties of the sensitive clay layer are: $s_{up}=50$ kPa, $S_t=6$ and $\delta_{95}=0.1$ m. The geotechnical properties of the upper glacial clay are: $\bar{s}_u=60$ kPa, $\gamma'=10$ kN/m³ and $E_1=20$ MPa, $E_2=40$ MPa. In order to calculate the minimum length of the shear band that initiates catastrophic shear band propagation, the shear stress on the sensitive clay layer is calculated as $\tau_g=20.93$ kPa using Eq. (5). Inserting the value of τ_g in Eq. (11), the stress ratio $r=0.302$ is evaluated. Now from Fig. 14, $\hat{x}_i^{cr}=3.25$ can be obtained for $r=0.302$. The characteristics length l'_u for this problem is 53.67 m (Eq. 10). Finally, using Eq. (25) the critical length of the shear band of 298 m can be calculated, which implies that if the length of the shear band exceeds 298 m it will propagate through the sensitive clay layer without any additional driving force and could cause a global failure.

However, if one wants to use the limit equilibrium method (Eq. 30), the global failure of the slope will be occurred only if the length of the fully weakened zone is 572 m. In other words, the limit equilibrium methods significantly underestimate the possibility of failure of the slopes with strain-softening clay layer.

Conclusions

This paper presents a method to evaluate potential failure mechanisms of submarine slopes having a sensitive clay layer. A fully weakened zone in the sensitive clay layer results in stress concentrations at both ends of the shear band and creates two end zones. When the length of the fully weakened zone exceeds a critical length the catastrophic shear band propagation occurs through the sensitive clay layer resulting in a large-scale landslide. Significant plastic deformation/strain develops in the shear band that reduces the undrained shear strength. Post-peak degradation of undrained shear strength of sensitive clay can be better modeled using the proposed nonlinear equation for a wide range of plastic deformations as compared to linear model. If the mechanisms of shear band propagation are considered, a much shorter fully weakened zone (several hundred metres for the cases analyzed in this study) is required for global failure of the slope as compared to that required for failure using the limit equilibrium method. The initiation of catastrophic shear band propagation depends on slope angle, depth of the sensitive clay layer and undrained shear strength; however, it can be presented in a normalized form using two parameters namely stress ratio r and characteristics length l'_u that combine all these effects. At the initiation of shear band propagation, the end zone length is considerably large as compared to shear band length, and therefore the end zone cannot be neglected as assumed in some previous studies based on the concept of fracture mechanics.

A normalized design chart (Fig. 14) is developed which can be easily used to calculate the minimum length of the fully weakened zone required for shear band propagation that might cause a large-scale landslide. The input parameters required for the analysis are slope angle, location of the weak zone and shear strength properties of soils. Therefore, the proposed solution could be easily used for practical application.

A limitation of this study is that the analytical solution presented above cannot model the retrogressive process after global failure of a soil mass as reported from post-slide investigations. Moreover, the effects of varying slope angle, initial stress condition and soil stiffness need to be investigated. Numerical modeling, such as large deformation FE modeling, is required for that as presented in Dey et al. (2016).

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List of symbols

The following symbols are used in this paper:

β	slope angle
h	depth from seabed
τ_g	gravitational shear stress
σ_x	normal stress acting parallel to the slope
$\bar{\sigma}_x$	average normal stress in upper soil layer
σ'_v	vertical effective stress

$\bar{\epsilon}_x$	average linear strain in upper soil layer
δ	post-peak (plastic) shear displacement
δ_{95}	δ at which s_u reduced by 95% of $(s_{up}-s_{uR})$
δ_r	displacement required to obtain residual strength
δ_{50}	δ at which s_u reduced by 50% of $(s_{up}-s_{uR})$
γ^p	plastic shear strain
δ_{ld}	large displacement for linear model
$\hat{\delta}$	normalized displacement ($=\delta/\delta_{95}$)
γ'	submerged unit weight
E_1	undrained Young's modulus for loading ($x<0$)
E_2	undrained Young's modulus for unloading ($x>0$)
S_t	sensitivity (s_{up}/s_{uR})
s_u	mobilized undrained shear strength
s_{u50}	s_u mobilized at δ_{50}
s_{up}	peak undrained shear strength
s_{uR}	s_u mobilized at considerable shear displacement
s_{ur}	s_u at completely remoulded state
s_{u95}	s_u mobilized at δ_{95}
s_{uld}	s_u mobilized at δ_{ld}
r	stress ratio $(\tau_g - s_{uR})/(s_{up} - s_{uR})$
l_u''	characteristics length $(\sqrt{hE_2\delta_{95}/(s_{up} - s_{uR})})$
$x_i (i=1,2)$	length of shear band

$\omega_{i(i=1,2)}$	length of end zone
$x_i - \omega_i$	length of fully weakened zone
$\hat{x}_i$	normalized length of shear band (x_i / l'_{ui})
$\hat{\omega}_i$	normalized length of end zone (ω_i / l'_{ui})
l	total length of shear band
l^{cr}	critical length
l_f	global failure length
OCR	overconsolidation ratio
P_a	active force acting at the upper end
P_p	passive force acting at the lower end
F_s	factor of safety

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Table 1. Calculated values of S_t and δ_{95}

Soil Type & depth (m)	Sensitivity, S_t	δ_{50} (+10%, -10%) (mm)	δ_{95} (+10%, -10%) (mm)
Saint-Leon 4.8m	3.3	210 (220, 198)	909 (952, 857)
Saint-Leon 9.3m	9.25	130 (136, 125)	563 (589, 541)
Lousiseville 6m	7.6	183 (187, 178)	792 (809, 770)
Saint Hilaire 5.6m	3.5	104 (109, 98)	450 (471, 424)
Saint-Thuribe 6m	9.5	47.5 (49, 46)	206 (212, 199)
Saint-Thuribe 12m	6.75	76 (79, 73)	323 (342, 316)
Mascouche 9m	9	15 (16, 13.5)	65 (69, 58)
Saint Alban 6.6m	4	66 (73, 59)	286 (316, 255)
Saint-Jean-Vianney 30m	20	4 (4.25, 3.75)	17 (18, 16)

Table 2. Parameters used in base case analysis

Parameter	Value
Slope angle, β ($^{\circ}$)	4
Depth of sensitive clay layer, h (m)	20
<u>Top layer/Glacial clay</u>	
Undrained Young's modulus, $E_1 = E_2$ (MPa)	30
Submerged unit weight of soil, γ' (kN/m ³)	10
Undrained shear strength, $\bar{s}_u$ (kPa)	50
<u>Marine clay layer</u>	
Peak undrained shear strength, s_{up} (kPa)	40
Undrained shear strength, s_{uR} (kPa)	6.67
Sensitivity, $S_t = s_{up}/s_{uR}$	6
δ_{95} (m)	0.1

Figure Captions

Figure 1. Seabed stratigraphy in the Storegga slide area (after Bryn et al., 2005) (Note: red arrows show three main marine sensitive clay layers, i.e. potential failure planes)

Figure 2. Shear band formation in typical submarine slope with a strain softening clay layer: (a) geometry, (b) shear stress and mobilized undrained shear strength along failure plane

Figure 3. Comparison between test results and model predictions of post-peak shear strength

Figure 4. Post-peak shear strength degradation model

Figure 5. Post-peak softening in normalized form

Figure 6. Length of shear band and discontinuity for different slope angle

Figure 7. Normalized lengths of shear band ($\hat{x}_2$) and end zone ($\hat{\omega}_2$)

Figure 8. Variation of displacement and shear strength along the shear band for 4° slope angle

Figure 9. Variation of displacement and shear strength along the shear band for 7° slope angle

Figure 10(a). Variation of discontinuity and shear band lengths for linear and nonlinear models for $\beta=4^\circ$

Figure 10(b). Comparison of displacement in shear band for linear and nonlinear models at initiation of shear band propagation for $\beta=4^\circ$

Figure 11. Effects of δ_{95} on initiation of shear band propagation

Figure 12. Normalized lengths of shear band and end zone for different peak shear strength

Figure 13. Normalized lengths of shear band and end zone for different sensitivity

Figure 14. Normalized lengths of shear band and end zone for different stress ratio

Figure 1. Seabed stratigraphy in the Storegga slide area (after Bryn et al., 2005) (thick arrows show three main marine sensitive clay layers, i.e. potential failure planes)

geometry, (b) shear stress and mobilized undrained shear strength along failure plane

Figure 3. Comparison between test results and model predictions of post-peak shear strength

Figure 4. Post-peak shear strength degradation model

Figure 5. Post-peak softening in normalized form

Fig. 6 Length of shear band and discontinuity for different slope angle

Figure 7. Normalized lengths of shear band ($\hat{x}_2$) and end zone ($\hat{w}_2$)

Figure 8. Variation of displacement and shear strength along the shear band for 4° slope angle

Figure 9. Variation of displacement and shear strength along the shear band for 7° slope angle

Figure 10(a). Variation of discontinuity and shear band lengths for linear and nonlinear models for $\beta=4^\circ$

Figure 10(b). Comparison of displacement in shear band for linear and nonlinear models at initiation of shear band propagation for $\beta=4^\circ$

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