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Back-analyses of soft soil failure with "strain-softening" behavior by the "equivalent sensitivity" concept

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Article

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Equivalent sensitivity

Abstract

Stability analyses of slopes in soft soils are usually affected by strain-softening, resulting in unrealistic (unconservative) safety factors. The loss of post peak strength cannot be accounted for by classic limit equilibrium analyses. In practice, however, the overall loss of soil strength is generally approximated by Bjerrum correction factor $\mu \leq 1$, which is believed to account for the different failure velocities during the field tests (usually vane tests) and the actual failure in the field, in addition to anisotropy (Schnaid & Odebrecht, 2012). The objective of this work is to demonstrate that strain-softening reduces the overall safety factor to a value nearly equivalent to the application of Bjerrum's correction factor. To accomplish this, a simple constitutive model (Mohr-Coulomb with residual stress) is used for total stress finite element analyses by means of the concept of "equivalent sensitivity" proposed by Pereira Pinto (2017). The results showed that equivalent sensitivity can be a great instrument to simulate the strain-softening behavior of soft soils.

1. Introduction

Low permeability soils mobilize the undrained strength when the load is applied at a rate greater than the ability of the soil to allow for dissipation of pore pressure excess. Existing techniques for pore pressure predictions in effective stress finite element analyses estimate the pore pressure by the product of the elastic volumetric strains by the bulk modulus of the water. This procedure seems to produce acceptable values mostly for stress paths under dominant volumetric strains, such as in prediction of settlements for embankments on soft clays. Unfortunately, under dominant shear distortions (as in slope stability analyses) the prediction of pore pressures is generally unreliable, as it depends strongly on the small (positive or negative) volumetric strains produced, which are generally difficult to predict (Boyce, 1978; Duncan & Wright, 2005).

In addition to difficulties to correctly predicting pore pressures, neglecting strain-softening can severely overestimate safety factors either in limiting equilibrium or FE analyses if the post-peak loss of strength is disregarded. This is demonstrated by the case history presented in this paper.

A relevant difficulty to model strain-softening is the negative stiffness produced after peak, which causes the results to be affected by the discretization of the FE mesh in the most common geotechnical software, which are based on the classic continuum of Cauchy. This means that different mesh discretizations will produce different results. More up to date software based on the generalized continuum of Coserat can address adequately negative stiffness. Unfortunately, despite the ability to model strain-softening with no dependence of mesh discretizations, these modern softwares are still prone to numerical instabilities, thus making it difficult the use on routine basis.

Strain-softening can be accommodated by correcting the undrained shear strength of the soil (S_u), determined in field or laboratory tests, by a reduction factor called Bjerrum's correction factor $\mu \leq 1$ (Bjerrum, 1972), which is correlated to the plasticity index *PI* (Cheng & Lau, 2014; Chowdhury et al., 2010). Bjerrum´s correction factor was conceived by comparing the measured undrained strength in field or laboratory tests to the strength needed to produce failure in the field, considering that the strength measured was generally higher than the true strength in the field. Therefore, according to Bjerrum (1972), the higher *PI*, the smaller the true strength. Soft soils holding high *PI* values are more susceptible to strain-softening. However, the correlation between μ and *PI* produces such a high scatter that estimating the reduction factor is prone to a great deal of uncertainty. Experience with Brazilian soft clays indicates that $\mu = 0.6$

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to 0.8 produces generally better results regardless of the correlation with *PI*. Furthermore, in the case history presented in this paper, *PI* is very low, suggesting $\mu \cong 0$, whereas the ratio *FS* (no strain-softening)/*FS* (strain-softening) $\cong \mu$ is the range above, as typical of Brazilian clays.

In this paper, a simplified procedure to model strain-softening in undrained FE analyses is presented. As a result of strain-softening, the undrained shear strength decays to the residual value individually, in each element, as the maximum peak stress is exceeded, gradually propagating to neighboring elements by progressive rupture.

2. The equivalent sensitivity concept

Both RS2 and RS3 geotechnical software provide the option of Mohr-Coulomb constitutive model with residual stress to simulate the instantaneous loss of strength shown in Figure 1. To overcome well-known numerical limitations to handle the post peak negative stiffness shown by the dashed curve in Figure 1, Pereira Pinto (2017) proposed the concept of equivalent sensitivity (S_t^*), according to Equation 1.

$$S_t^* = \frac{S_{u,p}}{S_{u,r}^*} \tag{1}$$

Similarly to the classical definition of soil sensitivity, the equivalent sensitivity S_t^* is given by the ratio of the peak undrained stress $S_{u,p}$ to the equivalent undrained residual stress $S_{u,r}^*$. As shown in Figure 1, strain-softening depends on the limiting strain $\in_{u,p}$ above which the material softens and, therefore, the concept cannot be extended to limit equilibrium analyses. As show in this work, the equivalent residual stress $S_{u,r}^*$ is determined by back analyses of previous failures (*FS* = 1), provided that the peak strength $S_{u,p}$ and the undrained stiffness E_u are known. Accordingly, $S_{u,r}^*$ is increased gradually from $S_{u,r}^* = 1$ until failure (*FS* = 1) is achieved numerically.

For the soft soils found on the banks of the Amazon River, Pereira Pinto (2017) found an equivalent sensitivity of approximately 1.4 in 2-D finite element analyses. More recently, Silva (2021) obtained an equivalent sensitivity of 1.6 in 3-D finite element simulations, which is compatible to the value obtained in 2-D simulations. From the Authors experience in other slope stability analyses in soft soils, an equivalent sensitivity of 1.5 accommodates satisfactorily for both 2-D and 3-D analyses in most Brazilian soft clays.

3. Case study

The case study is a slope on the banks of the Amazon River, in the municipality of Santana, in the State of Amapá, Brazil. In January 2013, a catastrophic failure occurred in the river port in the vicinity of the city of Santana, resulting in human and material losses, in addition to extensive environmental damage due the spill of a large amount of iron ore into de river. After the failure, an extensive site investigation was carried out to determine the most likely mechanisms of failure.

The port activities initiated in the 50’s where, at the time, the site investigation (only SPT tests) identified a deposit of quaternary soft soil on the banks of the river, as well as a more resistant tertiary material upstream (Barreiras Formation). A safety strip 140 m wide was then delimited between the riverbank and the limiting resistant soil, for which there could be no storage of ore. The original port structure was composed of a floating pier connected to the site by two steel trusses, which allowed the pier to oscillate as the tide swayed. The trusses, being close to the extremities, were called East and West. In the central portion there was a conveyor belt, which assisted in transporting the ore from the storage area (on resistant soil) to the floating pier.

In 1993 a first landslide occurred in the vicinity of the east truss, in the submerged portion of the slope, which displaced about 30,000 m^3 of mass towards the bed of the

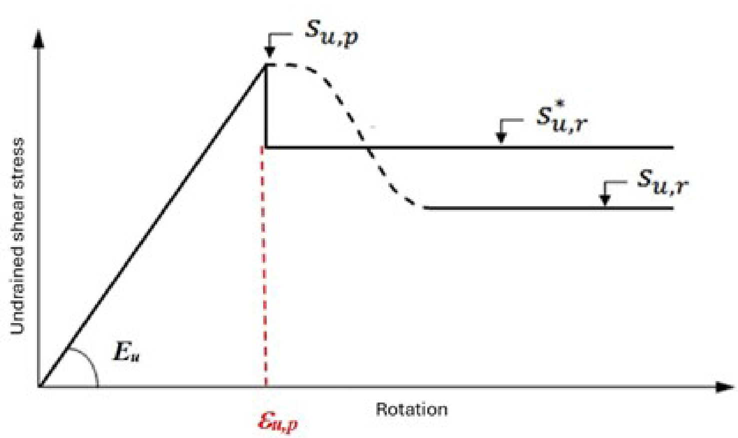


Figure 1. Equivalence between stress vs. rotation curve of the abrupt loss model and the real behavior in vane test (Pereira Pinto, 2017).

Amazon River. Figure 2 shows the scenario of the riverbank after the failure of 1993.

In 2008, a new soil investigation was carried out, which consisted of SPT, CPTu and vane tests, in addition to Atterberg limits and grain size analyses. The alluvial soft soil strip was confirmed, and the storage restrictions were strictly enforced.

In January 2013, the stack reclaimer failed, resulting in unplanned changes in ship loading. The ore began to be transported to the ships in trucks and loaders directly to the ship holds. During the period between January and March 2013, the storage position of the stacks changed continuously, sometimes advancing inadvertently the safety range of 140 m, thus allowing the placement of stockpiles on the weak soft soil. Figure 3 shows the overlapping of all identified stacks by satellite images, after failure of the stack reclaimer and before the slope failure, according to the forensic report published and discussed in Public Audience (Brasil, 2014).

In March 2013 the slope failed suddenly, with no previous warning, as typical of progressive failure triggered by

Figure 2. Scenario of the 1993 failure: 1) west truss; 2) stack reclaimer; 3) conveyor belt; and 4) east truss (Pacheco et al., 2014).

Figure 3. Overlapping of stacks between January and March 2013. The boxes in red indicate the boundary between the soft soil and the resistant soil, according to forensic studies (Pacheco, 2017).

strain-softening. The movement generated a wave about 5 to 6 meters high, comprising a collapsed area of approximately 16,000 m^2 and a detached mass volume of approximately 750,000 m^3, followed by 20,000 tons of ore that slid towards the bed of the Amazon River. The post-failed scenario is shown in Figure 4.

Figures 5 to 7 show records of the post-failure scenario and the presence of stockpiles on the strip of weak alluvial soil. Such evidence corroborates and complements what was presented in Figure 3.

A new site investigation was carried out after the accident, consisting of SPT, CPTu and vane tests. Grain size analysis and Atterberg Limit tests were also performed. Grain size analysis indicated the soft soil with about 55% silt and 25% clay particles. The Atterberg limits indicated a material of medium to high plasticity, with a medium

Figure 4. Post-failure scenario, highlight for the remaining stockpile (modified from Brasil, 2014).

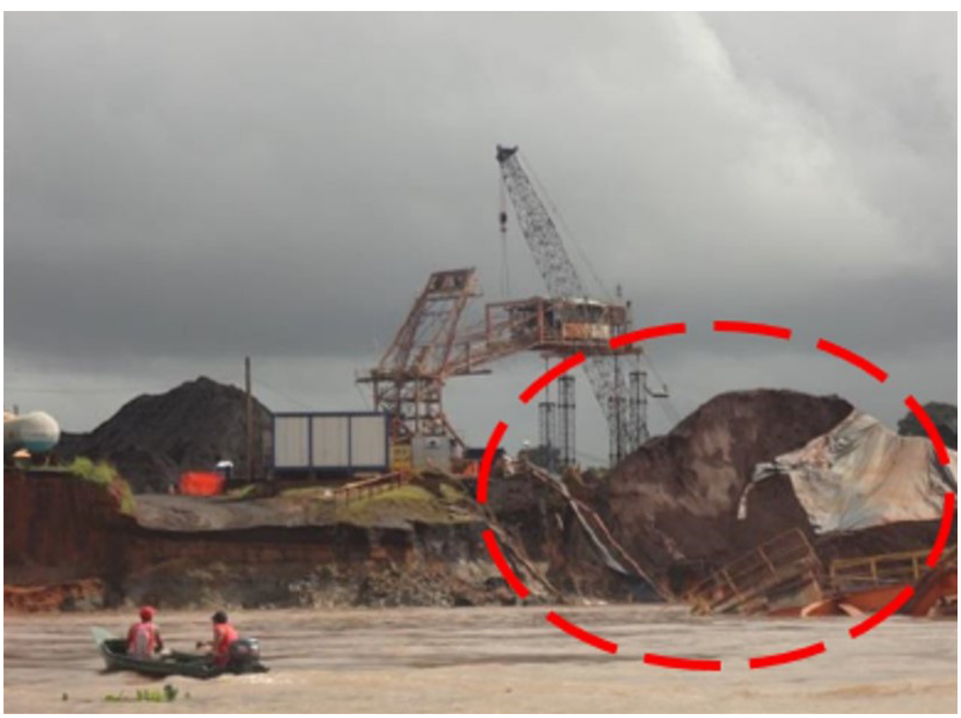


Figure 5. View of the post-failure scenario, highlight for the remaining stockpile, East side (modified from Brazil, 2014).

Figure 6. Post-failure scenario, both West and East sides. Highlight for the remaining ore Stack (adapted from Pacheco, 2017).

Figure 7. Detail of the remaining stockpile at the eastern side, indicating that the failure occurred for a stack height of 6 m (Pacheco et al., 2014).

value of plasticity index (*PI*) about 25%, and with liquidity index values higher than unit at some. Figure 8 presents the distribution of the Plasticity Index with depth. Figure 9 shows the result of one vane test from the site that reproduces the strain-softening behavior, with a sensitivity value about 2.4, and Figure 10 shows the sensitivity values obtained from CPTu tests with the additional data from vane tests. CPTu soil sensitivity values calibrated from vane tests ranged between 1.5 and 4, classifying the soil sensitivity as medium to low (Pacheco et al., 2014; Pereira Pinto, 2017). Figure 11 shows the basic CPTu data (q_c, f_s and u) for one borehole that is representative of the entire deposit.

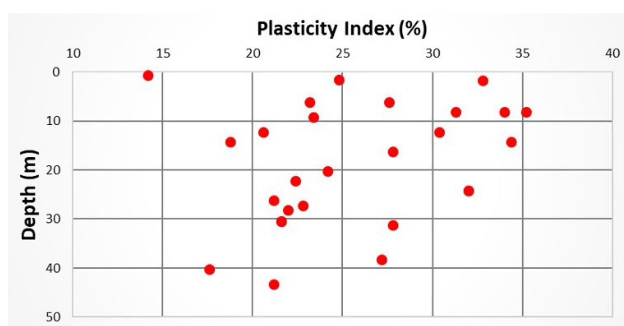


Figure 8. Spatial distribution of the Plasticity Index (*PI*) with depth.

For the sake of stability analysis, the undrained response of the material was confirmed by CPTu classification index values shown in, Figures 12 and 13. Table 1 presents the strength parameters and unit weight values used in the stability analyses. The undrained model was assumed as $E_u = 50.S_u$ for all soft soil layers.

Table 1. Strength parameters and unit weight values for stability analyses (Pereira Pinto, 2017, apud Silva, 2021).

Material	c (kPa)	$\phi(°)$	$\gamma\left(\frac{kN}{m}\right)$
Fill	10	35	19
Soft soil 1	46	0	16
Soft soil 2	36	0	16
Soft soil 3	46	0	16
Soft soil 4	56	0	16
Soft soil 5	64	0	16
Resistent soil	10	35	17

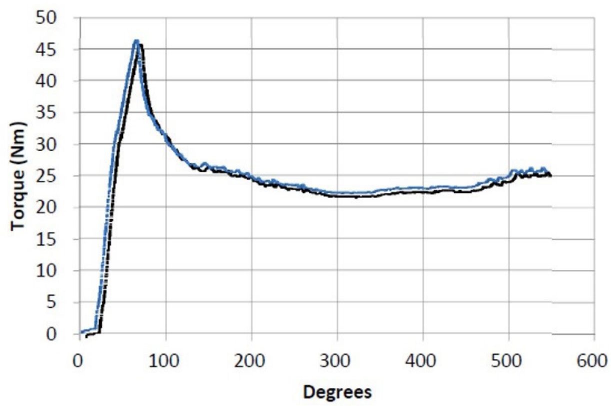


Figure 9. Typical vane test plot from the site (Pacheco et al., 2014).

4. Bidimensional stability analyses

Pacheco et al. (2014) processed 2-D stability analyses by the limit equilibrium method, with the aid of the software Slide 2. The analyses were processed using two main cross sections, also called East and West. Strain-softening was clearly identified by vane tests, according to the sensitivity values shown in Figures 9 and 10.

Pacheco et al. (2014) performed undrained analyses considering mean strength parameters reduced by Bjerrum’s correction factor $\mu = 0{,}7$. The value of Bjerrum’s correction factor used by Pacheco et al. (2014) was adopted considering the Brazilian experience from the literature. As can be seen in Figure 8, the spatial distribution of the plasticity index can result in several values of correction factor. However, the higher values of plasticity index seem to justify the use of the correction factor. Figure 14 shows a comparison of Brazilian values (Almeida et al., 2008; Azzouz et al, 1983; Bello, 2004; Magnani, 2006; Massad, 1999; Oliveira, 2000; Oliveira & Coutinho, 2000; Sandroni, 1993; Silva et al., 2022; Tabajara, 2021) for Bjerrum’s correction factor. Considering only the medium value of the plasticity index of the deposit (PI = 25%), the correspondent value of correction factor is about 0.95, what would result in no change in the undrained shear strength. Pacheco et al. (2014) concluded that, for the Brazilian soils, the correction factor should be studied

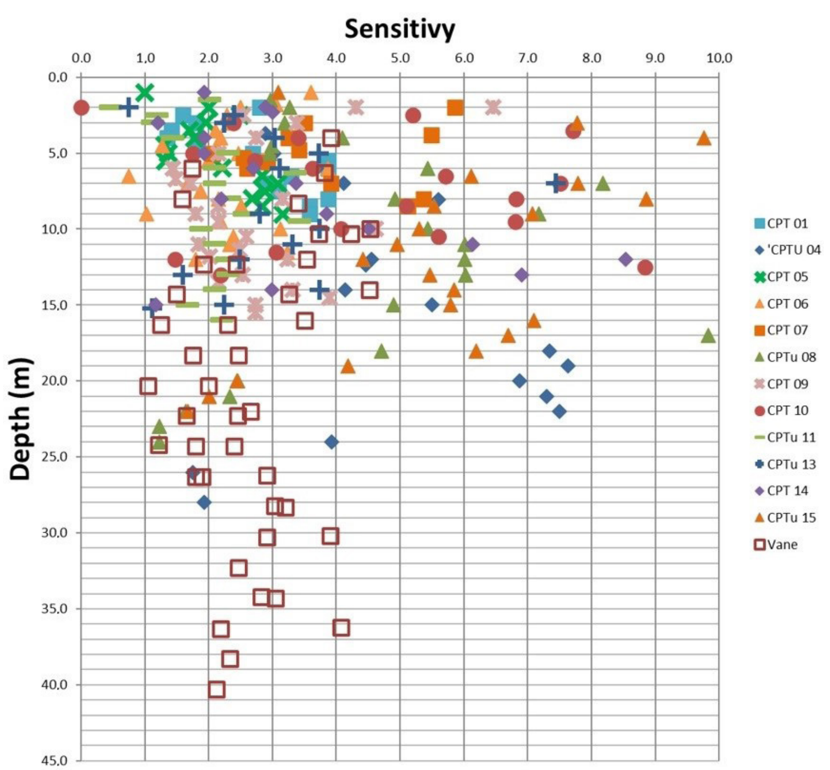


Figure 10. Soil sensitivity obtained by vane tests and correlation with CPTu tests (adapted from Pacheco et al., 2014).

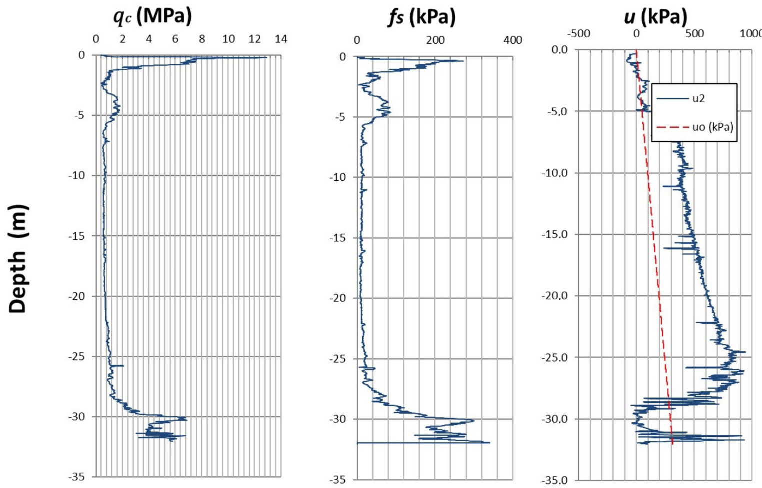


Figure 11. Basic CPTu data for one borehole - q_c, f_s and u.

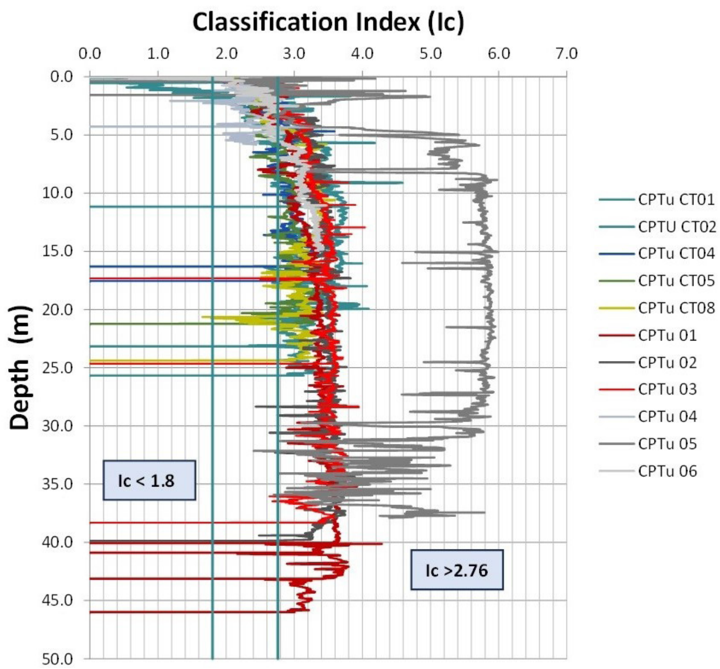


Figure 12. Classification index, West side (Pereira Pinto, 2017).

carefully and individually. Stability scenarios were also verified with and without the presence of ore stacks.

Pereira Pinto (2017), with the aid of software RS2 conducted 2-D finite element stability analyses. The East and West cross sections were again used, in addition to the same scenarios without loading and with loading simulating the ore stacks. Pereira Pinto (2017) used his proposed method of equivalent sensitivity to simulate strain-softening and found nearly the same safety factors as Pacheco et al. (2014) for undrained strength corrected by $\mu = 0{,}7$: *(i)* $FS \cong 1{,}4$ (no stockpiles) and *(ii)* $FS = 0.98$ (with stockpiles).

5. Three-dimensional stability analyses

Three-dimensional analyses were performed with the aid of the RS3 software, aiming at verifying the influence and applicability of the concept of equivalent sensitivity (Pereira Pinto, 2017) to 3D modelling, as well as to cross checking the most likely failure scenario.

The analyses were processed in two modalities, *(i)* analyses without loading, with the objective of verifying only the influence of strain-softening and *(ii)* analyses simulating the loads from the ore stacks, to check the joint interaction of the ore stacks and strain-softening.

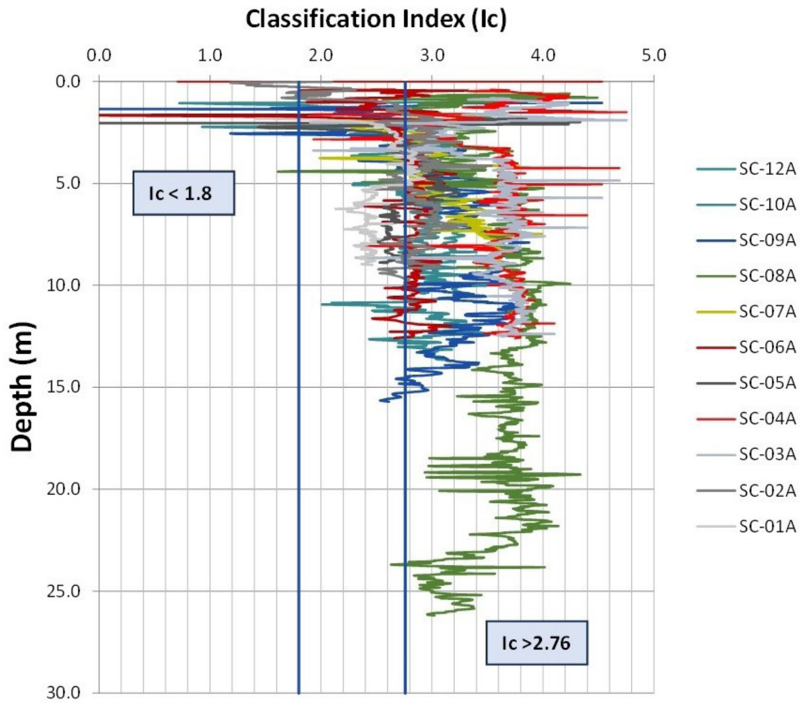


Figure 13. Classification index, East side (Pereira Pinto, 2017).

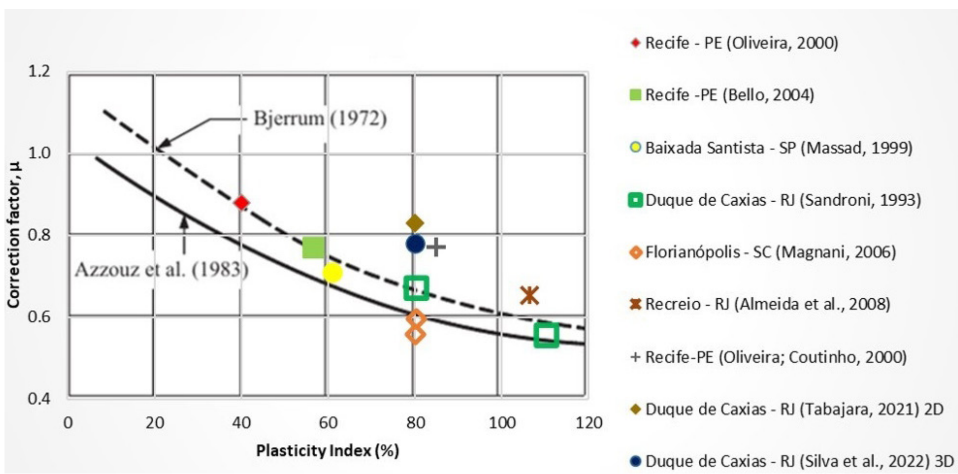


Figure 14. Undrained shear strength correction factor and plasticity index considering international and Brazilian experience (adapted from Schnaid & Odebrecht, 2012).

The three-dimensional model (Silva, 2021) used twenty-four different cross sections representing the entire area of interest, instead of only two cross sections (East and West) used by Pacheco et al. (2014) and Pereira Pinto (2017). The cross sections were drawn from the bathymetric survey of 2007, the only one available before the 2013 accident, thus representing the geometric shape of the slope before the 2013 failure and accounting for the remaining scar of the landslide of 1993. The three-dimensional model is shown in Figure 15.

The East and West cross sections shown in Figures 16 and 17 are the same used by Pacheco et al. (2014) and Pereira Pinto (2017).

Regarding the stockpile loading, a circular uniform load of 150 kPa was adopted, corresponding to the 6 m high ore stack shown in Figure 7. The distributed load was applied in three different loading stages, with increasing radius. The variation in the size of the loaded area had the objective of simulating the increasing variation of the ore stacks, considering part of the stack stored in resistant soil and part inadvertently stored within the safety range of soft soil.

The 3-D finite element analyses processed without ore stacks and without strain-softening, indicated a safety factor close to 1.8 (Silva, 2021), whereas the corresponding 2-D analyses by Pereira Pinto (2017) indicated a safety factor of the order of 1.4. For slopes with geometrically uniform cross-section, the difference between 2-D and 3-D safety factors is generally smaller than 10%, where 3-D safety factors are usually higher than 2-D. In the present analyses, however, this difference is about 28%, indicating the tridimensional effect of the concave scar produced by the 1993 failure. In Table 2, for $S_t^* = 1$ (stockpile loading and no strain-softening), FS = 1.54.

Table 2 shows safety factors obtained for each equivalent sensitivity considered in the back-analysis. According to Figure 1, the equivalent sensitivity is lower than the medium sensitivity values obtained by field tests.

The results above seem on first thought to indicate reasonably safe stability conditions. However, when strain-softening is incorporated into the analyses, the results processed with stack loading indicated FS = 1 for $S_t^* = 1.6$, according to the back-analysis shown in Table 2, thus simulating the failure scenario in Figure 18.

Validation of the critical failure surface is provided by observation of the contours of maximum shear strains (Bradley & Vandenberge, 2015). Accordingly, the maximum shear strain contours in Figure 18 indicate a deep failure throughout the soft soil region, propagating progressively from the riverbank towards the upstream resistant soil, as verified in the field, denoting the triggering scenario of the failure.

For comparison, Figures 19 and 20 present the contours of maximum shear strains for an auxiliary cross-section located under the ore stacks. The failure scenario is shown in Figure 20.

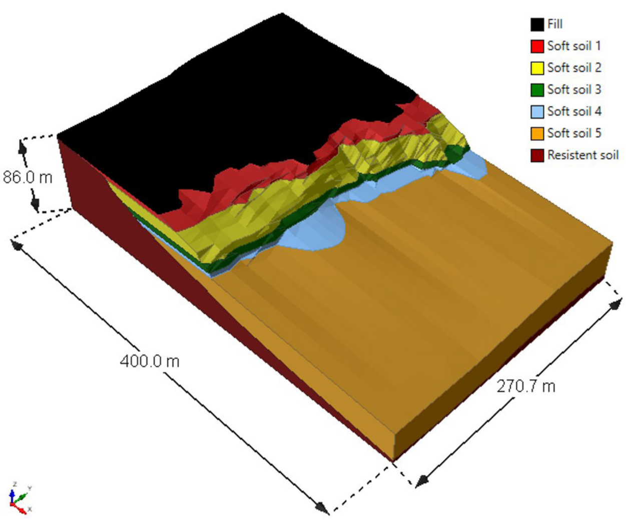


Figure 15. Three-dimensional slope model (adapted from Silva, 2021).

Table 2. Safety factors for back analyses with load and post-peak strength loss.

Equivalent sensitivity (S_t^*)	FS
1.0*	1.54
1.4	1.13
1.5	1.08
1.6	1.0

*No strain-softening.

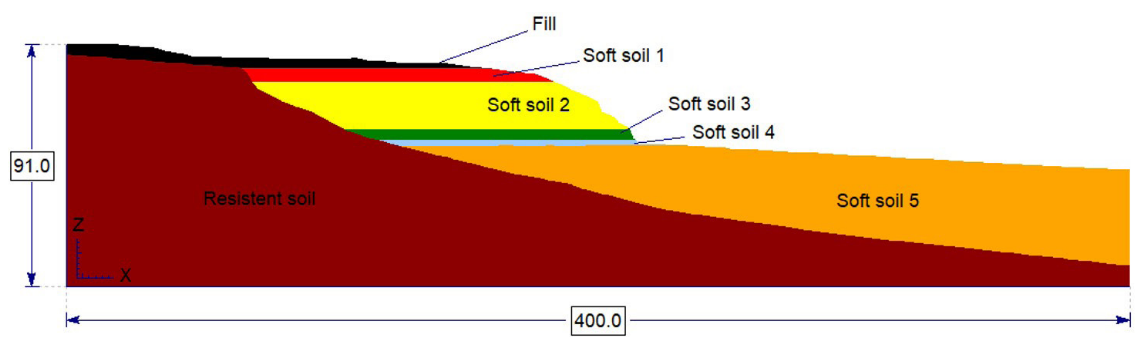


Figure 16. East cross section, drawn from the three-dimensional model.

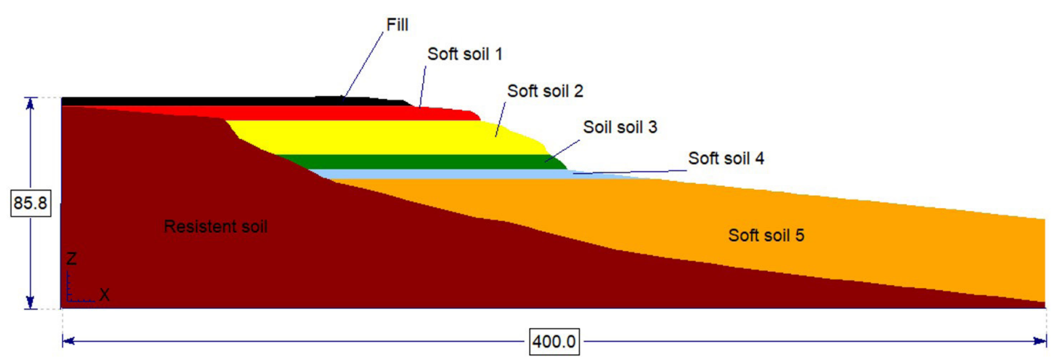


Figure 17. West cross section, drawn from the three-dimensional model.

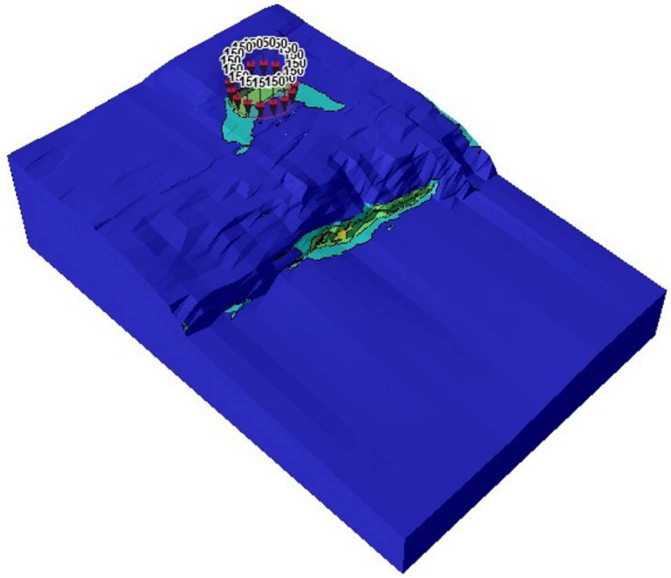


Figure 18. Contours of maximum shear strains at failure, for the analysis with stockpile loading and strain-softening (Silva, 2021).

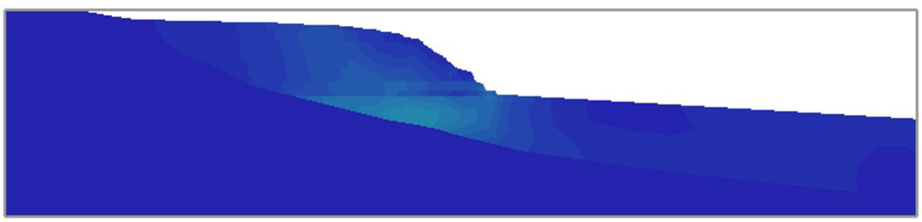
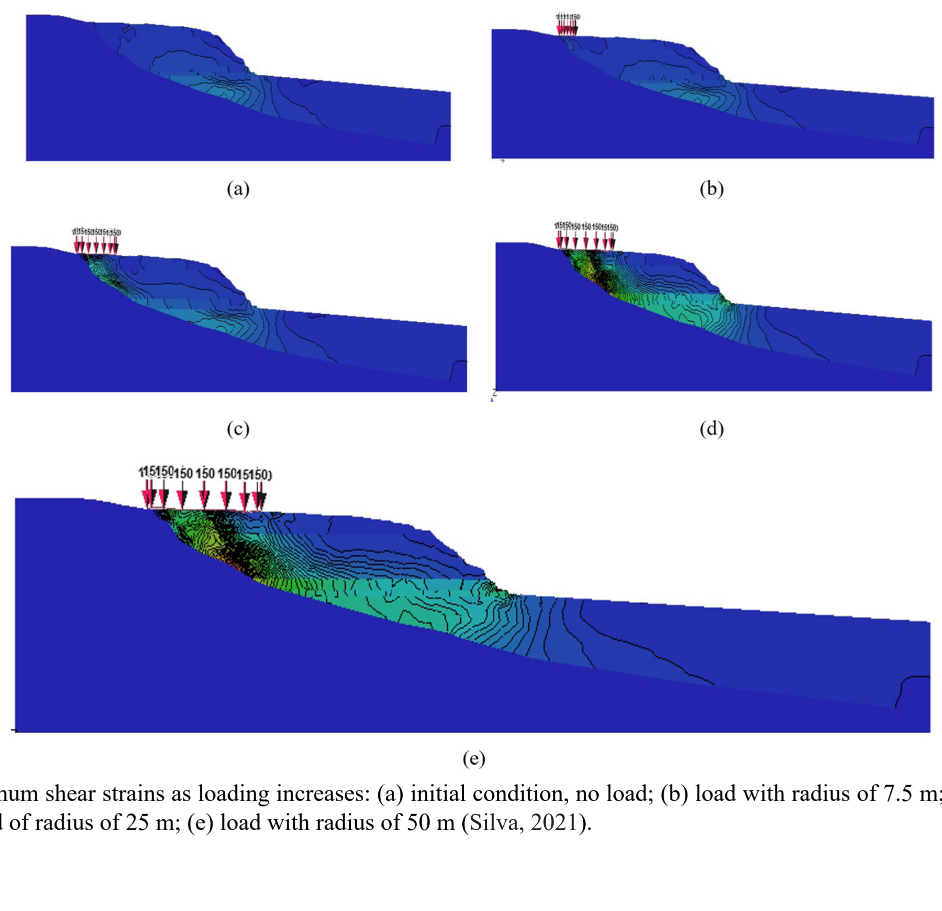


Figure 19. Maximum shear strains - no loading and no strain-softening, *FS* = 1.8 (Silva, 2021).



(a) (b) (c) (d) (e)

Figure 20. Maximum shear strains as loading increases: (a) initial condition, no load; (b) load with radius of 7.5 m; (c) load of radius of 17.5m; (d) load of radius of 25 m; (e) load with radius of 50 m (Silva, 2021).

6. Conclusions

The three-dimensional analyses presented by Silva (2021) validate the concept of equivalent sensitivity for Brazilian soft soils of low to medium sensitivity proposed by Pereira Pinto (2017) in bidimensional analyses. The equivalent sensitivity obtained by Silva (2021), in the three-dimensional model (S_{t^*}= 1.6), is compatible with the value obtained by Pereira Pinto (2017), in the two-dimensional model (S_{t^*} = 1.4). For practical purposes, an equivalent sensitivity (S_{t^*}= 1.5) seems satisfactory in both the 3-D and 2-D models.

Both 2-D and 3-D analyses show that significant unconservative errors are obtained in stability analyses that neglect strain-softening on medium to low sensitivity clays commonly found in Brazil, leading to unrealistic and excessively high safety factors.

For the 3-D analyses presented in this work, it is concluded that ratio between *FS* (strain-softening)/*FS* (no strain-softening) = 1/1.54 = 0.65 is approximately the stress reduction corresponding to Bjerrum correction factor. Further research is needed to determine the corresponding contribution of strain-softening, difference of shear velocity and anisotropy on the shear stress reduction in the field.

Realistic safety factor values can be obtained from conventional limit equilibrium analysis with application of the Bjerrum's correction factor (1972) to correct (reduce) the mean undrained resistance of the soft soils obtained in laboratory and field tests. However, Bjerrum's correction factor should be selected with good engineering judgment, due to the high uncertainty in selecting appropriate μ values as a function of the Plasticity Index (*PI*). From the Authors experience, values of the correction factor should be in the range of 0.6 to 0.8 regardless of *PI*, for Brazilian soft soils of medium to low sensitivity.

The equivalent sensitivity model proposed by Pereira Pinto (2017) coupled to the Mohr-Coulomb constitutive model with residual stresses provides a simple and stable numerical processing, independent of the discretization of the finite element mesh.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Lennon de Souza Marcos da Silva: M. Sc. Dissertation (conceptualization, methodology, writing – original draft). Marcus Peigas Pacheco: Dissertation Advisor (supervision, writing – reviewing and editing).

Data availability

The datasets generated analyzed during the current study are available from the corresponding author upon request. All data produced or examined during the current study are included in this article.

List of symbols

c	Cohesion
CPT_u	Cone Penetration Test with pore pressure measurement
E_u	Undrained stiffness
FS	Safety Factor
PI	Plasticity Index
SPT	Standard Penetration Test
SRF	Strength Reduction Factor
S_u	Undrained Shear Strength
$S_{u,p}$	Peak undrained shear strength
$S_{u,r}$	Residual undrained shear strength
$S^*_{u,r}$	Equivalent residual shear strength
S_{t^*}	Equivalent sensitivity
γ	Specific weight of soil
$\epsilon_{u,p}$	Undrained limiting strain
μ	Bjerrum's correction factor
ϕ	Angle of internal friction of soil
ϕ_u	Undrained angle of internal friction of soil

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