Assessment of the Risk of Failure of Pit Slopes in China Clay Deposits of South West England.

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ABSTRACT

The development of a slope stability assessment and management procedure is described for the kaolinised granites of S.W. England from which china clay is produced. The area has relatively high rainfall and the impact of high groundwater levels on slope stability is important. A comprehensive programme of site investigation was conducted to obtain representative soil samples for shear strength and clay content and to monitor piezometric levels. An EDM monitoring system was installed to detect possible slope movement and to confirm existing stability conditions.

The initial design studies involved the matching of measured piezometric levels for existing slopes with a computer model of the groundwater and then extrapolating to predicted levels for deepened slopes at the same site. This was followed by slope stability analysis including the evidence from EDM that the existing slopes were stable overall. The deepened slopes were then designed to provide an equivalent degree of stability.

It was considered desirable also to investigate the possible risks of failure on a probabilistic basis. A procedure was developed using the FOSM method to determine initial estimates and a Bayesian scheme to determine updated estimates based on observed successful slope performance. Some ideas are suggested for future transient groundwater modelling to further define piezometric level variations. This is one of the most important sources of uncertainty accounted for in the risk assessment.

INTRODUCTION

The exploitation of China clay resources in Cornwall is leading to deeper pits in very mixed ground derived from alteration in the host rock granite. Typically, pits are of the order 50 to 100 m deep but some will eventually be as much as 150 m deep. Some of the more critical slopes for stability are within fully or nearly fully kaolinised granite (FKG or NFKG), a sand-like material with some clay. Groundwater conditions within the material are both complex and critical.

Deterministic stability analyses have been undertaken to assess the potential for stability problems, and ways of managing this. Whilst the analyses have given some guidance for suitable slope angles, it is recognised that there is considerable variability in conditions. The use of probabilistic methods of stability analyses is being examined to provide guidance on the risk of failure. Of particular significance are the variability of shear strength parameters and piezometric pressure distributions.

Some preliminary analyses incorporating probabilistic approaches are described and some other possible methods of analysis and approaches to risk management considered. The work described is based on an investigation and design project conducted by CSM Associates (CSMA) for ECCI and a University of Exeter - Camborne School of Mines (CSM) Ph.D research programme funded by SERC.

CHINA CLAY PIT GEOLOGICAL AND PRODUCTION SETTING

The annual production capacity of ECC International, from the UK operations, is 3.0 million tonnes of kaolin. In addition to the main producing area, centred on the St. Austell granite, the Company has mining operations on the Dartmoor and the Bodmin Moor granites.

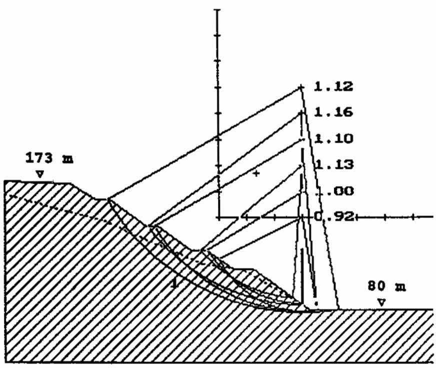

Figure 1: Typical slope profile including measured piezometric profile and critical failure surface from the program SLIDE.

The St. Austell Granite consists of a series of both magmatic and metasomatically altered granites [1] all of which are subject to varying degrees of kaolinisation which is thought to be a low temperature, possibly supergene phenomenon [2]. The kaolinisation is clearly structurally controlled and generally occurs in association with sheeted greisen, tourmaline and quartz veining.

In recent years a more systematic approach to short and long term mine planning has been adopted. Because of the depth of reserve in some areas and the restriction on lateral development it is becoming increasingly necessary to consider steeper slopes.

The deposits are exploited from open pit mines using a combination of hydraulic mining, mobile plant and conventional quarry drilling and blasting. Typical slopes are cut in 15 m deep faces with 40° lie back angles and 12 m wide benches. Where there is particular concern then the slope is considered in more detail and engineered appropriately. A typical slope profile is shown in Figure 1.

EXAMPLE SLOPE STABILITY INVESTIGATION / DESIGN

One slope of particular interest is currently about 90 m deep, has a crest length of about 300 m and NFKG predominates. It is intended to deepen the pit to at least 120 m. The slope crest is in its final position.

Site investigation and laboratory testing

Site investigation and testing included the following main components:

o A grid of 15 boreholes was drilled from benches throughout the existing slope at five main slope cross-sections. The holes were drilled by ECCI with a Hydreq Gryphon rig using a triple tube sampling barrel to obtain 100 mm diameter samples. A total length of 262 m of core was obtained. The depths of the holes were between 20 and 50 metres corresponding to the maximum depths of theoretically critical failure surfaces.

o Soil / rock cores were logged and samples selected for shear strength testing and mineralogical analysis. Twenty nine samples were prepared for triaxial shear tests, and a further 23 samples tested in a 60 mm shear box. The triaxial tests were mainly undrained with pore pressure measurements but with a few drained tests to check the results.

The triaxial tests were conducted over a range of effective confining pressures (σ_3') of 30 to 500 kPa, similar to those expected at the locations of critical failure surfaces. The data points are plotted as σ_1' vs σ_3' in Figure 2. (The values were taken at maximum σ_1'-σ_3'in each test). This form of plot is particularly useful for combining all test data. Each test result can be represented by a single point rather than the more common circles in the Mohr-Coulomb space (τ vs σ_n'). Regression analysis can be easily applied to the individual points to provide a rigorous measure of test result variance. Values of the Mohr-Coulomb effective strength parameters, cohesion (c') and friction (ϕ') can be derived from the slope and intercept of the plot:

the slope of σ_1' vs σ_3' is

$$\frac{1+\sin\phi'}{1-\sin\phi'} \tag{1}$$

the intercept is

$$\frac{2c'\cos\phi'}{1-\sin\phi'} \tag{2}$$

Linear regression was applied to the data in Figure 2 resulting in c'= 5 kPa and ϕ' = 37°. The use of all the data (over a wide stress range) was considered appropriate for the slope as a whole (90 to 120 m deep). The effective cohesion in this case is quite small and if only one high outlier point is removed the cohesion becomes slightly negative, which is inadmissible. For later risk analysis purposes, for the overall slope stability, cohesion was taken to be zero and the regression line forced to pass through the origin (shown in Figure 2). The 95% confidence limits for the mean line are also shown. This resulted in tan ϕ' = 0.795 (ϕ' = 38.5°) with a standard deviation of 0.025.

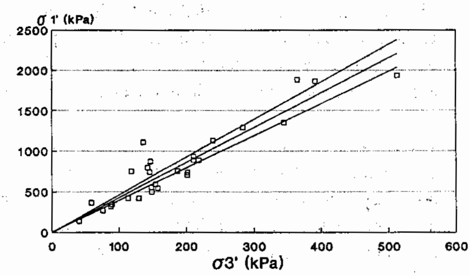

Figure 2: Triaxial shear strength test data for kaolinised granite.

The coefficient of variation of about 3% is considered low and probably does not reflect the true in-situ variability of shear strength.

It is evident from observation of the soil core samples and the actual slope material that the cohesion is in fact non-zero and this is probably important on the scale of individual bench stability. In the current research programme, further nearly undisturbed specimens are being obtained using the Leeds shear box [3]. These are being tested at low confining stresses to determine effective cohesion values. The orientation of the samples are such that the shear plane in the test is sub-parallel to potential shear planes in the field. This is not possible with the triaxial samples from vertical boreholes.

o Casagrande-type piezometers were installed in every hole with double installations in some. These have been read on a regular basis since installation. Falling head tests were conducted soon after installation to determine permeabilities. The results were highly variable and probably reflected variations in kaolinisation and the effects of lodes which intersect the slope. This clearly has implications for variations in and rate of response of piezometric pressures to rainfall.

o Daily rainfall records are taken by ECCI staff at three locations within the St Austell granite area and these provided a good database for correlation with piezometric levels and for groundwater modelling.

o An electronic distance measuring (EDM) system was installed by ECCI / CSMA to monitor any overall slope movements. The system can provide measurements to an accuracy of about ± 3 mm. Some example results for the existing slope are shown in Figure 3. The readings showed that the slope was stable overall, but there have been local bench failures at the toe of the slope.

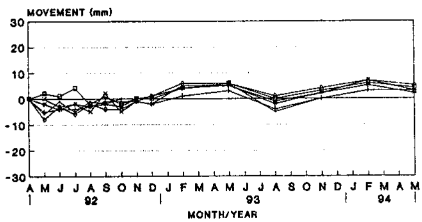

Figure 3: Example EDM readings from 6 stations, measured in the down-slope direction.

Groundwater modelling

The piezometric data provided point values for the current slope. For stability analysis a full piezometric surface was needed which could also be extrapolated for the new boundary conditions which will apply for the deeper slope. Initially the piezometric surface was modelled with the analytical solution due to Strack et al [4] and then with the finite difference model FLAC [5] run to provide a static equilibrium.

The piezometric boundary conditions at the toe and crest boundaries were assumed to be hydrostatic. The piezometric level at the toe boundary was coincident with the current toe elevation. The level at the crest boundary was adjusted until a good match was obtained with the piezometer readings. The FLAC model treats the determination of the phreatic surface (top flow line) rigorously and allows piezometric pressures to be determined throughout the grid. In this way a true piezometric surface for the most critical potential failure surfaces was determined.

The permeability was modelled as uniform, a considerable simplification, but no systematic pattern of variation could be determined from the in-situ tests. It is planned to investigate the effect of random permeability variations within the model on the piezometric pressures, constrained by the in-situ values. If significant, this will be incorporated in the risk analysis as a factor increasing the variance of the piezometric surface elevations.

The model was then changed to reflect the fully excavated slope and the phreatic and piezometric surfaces determined with similar boundary condition assumptions. The results of this model are used to check the development of actual piezometric pressures as the slope is excavated. If excessive pressures occur this may lead to a decision to reduce the excavation rate locally or change the lower slope profiles.

Slope stability analysis

Slope stability was assessed using the Bishop method of circular failure analysis incorporated in the program SLIDE [6] and specific results were checked with the program JACOB, written by the lead author. Typical SLIDE results are shown in Figure 1. For critical failure surfaces, involving most of the existing slope, the factor of safety was about 1.12 and for the fully deepened slope this reduced to about 1.08. The reduction is due to changes in the relative position of the piezometric surface.

It was decided to adjust the final slope geometry to achieve the same, or better factors of safety as the present values. This will be undertaken by a combination of reduced slope angles in the lower slope and possibly the use of free-draining coarse rock backfill to depress the piezometric surface. Short bays of the toe benches will be over-excavated to give a double bench and replaced with the backfill. The feasibility of the approach will be controlled by the short term stability of the over-excavated faces. The actual (non-zero) cohesion values will be important in this respect. The limited proposed bay width should be such that a true 2D failure surface of any significance cannot develop. It was considered that the use of horizontal drainage holes would be much less effective than the free-draining fill material because of the nature of the slope material, NFKG (sand with clay) and the lack of a persistent joint structure to intercept.

It is recognised that the factors of safety reported above may appear low, but there are certain aspects which suggest actual factors of safety will be higher. These include the presence of lodes of more competent rock running through the slope. Although narrow (a few metres) they are sufficiently continuous that their effect is locally significant. The soil-like kaolinised material of which the critical parts of the slope are composed is buttressed at each end by relatively unkaolinised material. This will have a 3D stabilising effect near the ends. The combined effect of this and the lodes, although of uncertain magnitude, could be very significant.

RISK ASSESSMENT APPROACH

The deterministic analyses described above have been made with some simplifying, mainly conservative, assumptions to provide a safe design. It was also of considerable interest to attempt a "best estimate" analysis of stability with the associated risk of failure due to data variability. It must be emphasised that this is complementary to the deterministic analysis which is "calibrated" by the monitored stability of the existing slope. It is not possible to calibrate a probabilistic analysis in the same way unless there have been some actual failures, but Bayesian statistical analysis can be useful. It is hoped that the methodology developed will give better understanding of the deterministic analysis limitations and provide the basis for further similar risk-based analysis and design.

The failure risk associated with variability of the unit weight, shear strength and piezometric pressures was investigated. The slope height and slope angle were controlled parameters.

First Order Second Moment method (FOSM)

The initial analysis employed was the First Order Second Moment method [7];[8], in which the variance of the output (in this case factor of safety, F) can be expressed as a function of the variance of the input parameters (σ_p) and the sensitivities of the output to the inputs ($\partial F/\partial p$):

$$\sigma_F^2=\sum_p \sigma_p^2(\frac{\partial F}{\partial p})^2 \quad (3)$$

If the partial derivatives cannot be determined analytically, an alternative is to use numerical values based on the slopes of sensitivity plots:

$$(\sigma_F)^2=\sum_p \sigma_p^2(\frac{\Delta F}{\Delta p})^2 \quad (4)$$

This method is useful where the solution for the output (F) is too complicated for repeated simulation by Monte-Carlo or similar methods. The method is accurate if the sensitivities are linear, and usable otherwise provided that the extreme ranges of the output distributions are not required. The form of the output distribution must be assumed. From previous simulations using Monte-Carlo or similar methods [9] an approximately log-normal distribution is commonly expected for complicated combinations of input variables as occur in limit state slope stability analysis. Note that the Bishop method requires sub-division of the failure mass into slices and an iterative solution.

Log-normal distribution parameters can be obtained from normal distribution parameters. If $y = \ln(F)$ is normally distributed, then F is log-normally distributed (specified by μ_F and σ_F). The mean and standard deviation of y is given by:

$$\mu_y=\ln(\frac{\mu_F}{\sqrt{1+(\sigma_F/\mu_F)^2}}) \quad (5)$$

$$\sigma_y=\sqrt{\ln(1+(\sigma_F/\mu_F)^2)} \quad (6)$$

The parameters μ_y and σ_y can be used with the standard normal distribution to predict probabilities of certain values of F not being achieved, eg $p(F<1.0)$ or $p(F<1.5)$. Examples are given later.

Variability of shear strength

The variability of the shear strength can be treated adequately in terms of mean and standard deviation values for the effective friction angle ($\tan \phi'$). The values were determined from the data in Figure 2 as described above.

Variability of piezometric pressures

The variability of the piezometric pressures was evaluated directly from the measured values. At each slope cross section mean and extreme high and low piezometric profiles were identified. Each profile included three individual piezometer readings. The profiles used were as observed on a particular day so that, for example, the extreme highs at each piezometer did not all occur simultaneously. The sensitivity $\Delta F/\Delta w$ (where parameter w signifies the piezometric elevation) was evaluated by re-running the slope stability analysis using the three profiles. The standard deviation σ_w was based on the judgement that the extreme profile values were at ± 1.5 standard deviations from the mean. When the piezometric record is longer it will be possible to assume that (updated) extreme values are at wider limits (say ± 2 or so standard deviations from the mean, encompassing about 95 % of all possible values). It is proposed to examine this further by using the rainfall record and correlation with piezometric profiles with a transient groundwater computer model.

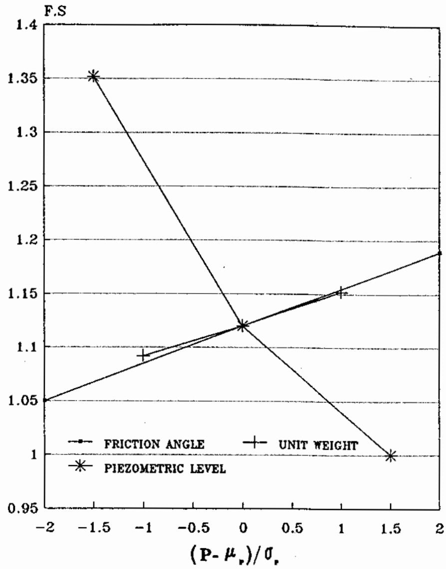

Figure 4: Sensitivity of slope stability factor of safety to variations in separate input variables.

An alternative approach would be to characterise the piezometric pressure profiles by a single parameter such as average r_u, the average ratio of pore pressure to total normal stress acting on the failure surface, or by factoring the unit weight of water γ_w to simulate higher or lower piezometric pressures generally. This is attractive for use in initial studies with stability charts which use r_u or an equivalent [10];[11], but less so where a Bishop-type slices analysis is being used and measured pore pressures are available, as here.

Example result

Results of a sensitivity analysis are shown in Figure 4. The mean value of F, μ_F, was 1.12. The x axis is scaled to the numbers of standard deviations of the individual inputs and therefore the individual slopes are equal to $\sigma_p\ \Delta F/\Delta p$. Using the subscripts s,w and g to denote input parameters shear strength, piezometric elevation and unit weight respectively gave:

$\sigma_s\ \Delta F/\Delta s = 0.035$: $\sigma_w\ \Delta F/\Delta w = 0.117$: $\sigma_g\ \Delta F/\Delta g = 0.030$

Hence from equation 2:

$$\sigma_F = \sqrt{(0.035^2 + 0.117^2 + 0.030^2)} = 0.13$$

Using equations 5 and 6 gave the values

$\mu_y = 0.107$ and $\sigma_y = 0.116$

Hence the probability that (F<1) or (ln F < 0) was given by the statistic Z = 0.922, the number of standard deviations, σ_y, that μ_y was above 0. From the standard normal distribution p(F<1) = 17.8%. Similarly the probability that (F< 1.3) or (ln F < 0.262) was given by Z = -1.34, and p(F<1.3) = 91%.

Similar results can be obtained for critical potential failure surfaces of different scales (single bench up to whole slope). In general the smaller scale failures are considered less critical and relatively easily managed.

Bayesian update

Clearly if the slope has not failed, the above analysis was too conservative. What can be done to provide a better estimate of the factor of safety and probability of failure in similar or deeper slopes in the same material ? A potentially useful approach is to use a Bayesian statistical update. The analysis used was a modified version of that suggested by Chowdhury and Zhang [12]. The approach is a preliminary one and requires further theoretical consideration / validation.

Table 1 shows the updated results for the existing slope. In the table the initial stage is before the slope is excavated and is based on the FOSM analysis (in practice the excavation preceded the analysis). The subsequent stages assume a sequential successful excavation of 3 similar cells, each of which led to an update of the stability indicators (μ_F, p(F<1)). The columns headed F_n/F_1 and F_n/F_{n-1} show how the factor of safety was increased stage by stage relative to the original mean factor of safety and to the previous stage mean factor of safety. The latter show a progressive decrease and convergence towards 1.0. For the theoretical successful excavation of 10 cells the last incremental improvement was a factor of only 1.005. (Final convergence will occur when effectively p(F<1) = 0, after a large number of successfully excavated cells).

Stage	Mean F	P(F<1) %	F_n/F_1	F_n/F_{n-1}
Initial	1.120	17.8		
1 cell	1.162	10.4	1.038	1.038
2 cells	1.190	7.2	1.063	1.024
3 cells	1.209	5.5	1.080	1.016
10 cells	1.283	1.7	1.145	1.005

Table 1: Updating of existing slope stability indicators using a Bayesian approach.

After three successful stages, which occurred with the existing slope, the indicated mean factor of safety improved from 1.12 to 1.21 and the probability of failure reduced from 17.8 % to 5.5 %. The scale-up in mean factors of safety based on successful excavation and the original probabilistic analysis gave the basis for adjusting stability indicators for future slopes. Two examples are given in Table 2.

Case 1 is for the deepened slope, with a similar size failure surface daylighting at the toe as in the analysis for the existing slope. The relatively higher piezometric levels led to an initially lower factor of safety and higher probability of failure. By scaling up the factor of safety by 1.08 (after 3 successful

Case	Original F	Original p(F<1) %	Revised F	Revised p(F<1) %
1	1.080	28.0	1.166	9.3
2	1.220	3.5	1.318	0.3

Table 2: Revised stability indicators for new slopes based on updated indicators in Table 1.

cells per Table 1) and retaining the same σ_F, the revised stability indicators are improved. Case 2 is for a deepened slope with the same overall geometry but with some natural material replaced with free draining fill and the piezometric levels correspondingly reduced. There is a substantial reduction in the probability of failure.

Risk management

With the approach described above it is possible to perform a cost-benefit analysis based on the probability of failure, the cost of failure, the value of china clay material recovered and the cost of cut and fill operations for different overall slope angles. The procedure can be refined to account for possible failures on small and whole slope scales.

DISCUSSION

The main initial uncertainties in the stability analysis were the shear strength of the slope material, and the piezometric levels. It is not possible to resolve these uncertainties separately for future slopes from existing stable slopes, but by updating the stability indicators using the Bayesian approach a useful combined update appears possible.

A key assumption is that the variance of F remains constant for all analyses and this is considered reasonable for similar size potential failure masses in similar slopes. Clearly if the slopes are very heterogeneous such an assumption is invalid. It could be argued instead that the coefficient of variation of F (σ_F/μ_F) is a constant and then both μ_F and σ_F should be updated. This requires further investigation.

Another assumption is that the distribution of F is log-normal and this is considered reasonable based on previous risk analyses and simulations. However, for the current case, if a normal distribution for F is assumed the results are very similar. For the data in Table 1 the probabilities of failure are within 1 % for the two alternative distributions. Where the initial stability analysis is performed by Monte-Carlo or other simulation methods the distribution of F can be processed non-parametrically in the Bayesian update, which is performed numerically.

The groundwater modelling has concentrated on static models initially, matching observed piezometric levels and extrapolating for new slopes. Although the piezometric data record is now about 3 years long it is very unlikely that the long term extremes in levels have been seen. A possible development under consideration is to model the groundwater transiently, including rainfall, and to again match the observed piezometric levels. Then extreme rainfall scenarios from the rainfall record may be used to predict extreme piezometric levels. Another possible benefit from a transient model would be to determine the effects of excavation rate on piezometric level equilibration.

The shear strength variability determined from the laboratory tests is almost certainly too limited compared with real in-situ variability. In particular the effects of the adjacent rock in the lodes and abutments have not been directly accounted for, but it is difficult to make the necessary adjustments rationally. However the updated stability indicators have taken some account of such effects.

CONCLUSIONS

In the context of the excavation of china clay pit slopes down to 140 m deep in the FKG and NFKG material some specific and general conclusions can be drawn.

The ground water conditions are critical for slope stability and it is essential to take proper account of the variability of piezometric levels. This can be effective with a combination of piezometric measurements in existing slopes, static groundwater modelling and risk analysis. An approach using Bayesian statistical updating of the slope factors of safety has been developed and described with example calculations.

The updating scheme can also take account of uncertainty in shear strength values and other (usually less uncertain) parameters.

The use of EDM measurements is a useful approach for both confirming existing slope stability and monitoring future performance. Routine piezometric measurements are also useful to confirm design assumptions and to give warning of potential instabilities.

Quantifying the risk of slope failure provides management with a useful tool for decision making and prioritising.

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