Investigation of the stability of a fly ash pond facility using 2D and 3D slope stability analysis

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Abstract

A numerical investigation of the effect of pore pressure regime on the safety factor and the critical failure mechanism is presented for fly ash storage facility. Pore pressures' measurements from standpipe piezometers and pore pressure estimated from seepage analysis are used to compare the factor of safety for a fly ash slope. This was applied for considering static and seismic scenarios. A probabilistic approach was applied to account for the uncertainties resulting from the limited data available and support a qualitative risk assessment evaluation. Slope stability analysis is conducted in two and three dimensions, adopting the limit equilibrium analysis approach, and also a finite element seepage analysis, to assess the stability of the slope. The two-dimensional cross-sections were extruded to three-dimensional models to estimate the factor of safety and associated shear failure. The results from the performed analysis suggest an increase in safety factor values of 5%.

1 Introduction

Fly ash is a coal combustion residue of thermal power plants and has always been considered a challenging waste to manage globally. The coal reserves in South Africa are relatively shallow developed in relatively thick seams, which is making them cost-effective to mine while supporting energy generation for 17 power plants. The annual fly ash production is estimated to be 42 million tons, of which the cement industry currently uses only 7%; 74% is used for effluent treatment, leaving 19% available for other uses (Reynolds-Clausen and Singh, [1]). In South Africa, a hydraulic deposition method is commonly used for fly ash in the form of a slurry, which is subsequently allowed to settle and consolidate, while the excess water is reverted to the plant in order to be re-used.

As an aftermath of the significant tailing dams' failures in Brumandino, Mount Polley, Tennessee Valley Authority's Kingston Fossil Plant, there is a raised awareness amongst the mining industry, and mine waste geotechnical engineers

of the risks related to tailing storage facilities and similarly fly-ash dams.

The failure of the Tennessee Valley Authority's Kingston Fossil Plant containment embankment in 2008 released a volume of approximately 1.9 million m^3 of ponded fly ash. In the aftermath of the event in the USA, there was a wide discourse on the safety of fly ash storage facilities and the preparatory factors that trigger static liquefaction and subsequent flow of ponded fly ash (Bachus et al., [2]). The dam collapse during September 2022, at the abandoned Jagersfontein mine in South Africa, occurred at a time of elevated global concern around the safety of tailings dams, and the necessity for historic monitoring of slope instability. About nine houses were swept away and more than twenty damaged due to the released mudflow. The plume of tailings travelled ~7 km over dry land before reaching the reservoir of the Wolwas dam. Tailings continued to flow along ~56 km of streams and rivers causing some of them to overflow. The estimated released volume of tailings was between 4 and 6 million m^3 (Torres-Cruz, L.A., O'Donovan, 2023).

In the last decade, the frequency of tailing dam failures is on the rise, as per the catalogue of tailings dams (https://worldminetailingsfailures.org), the database hosted by the Centre for Science in Public Participation CSP2 (http://www.csp2.org/tsf-failures-from-1915) [3], by Chambers et al. 2011 [4], Bowker and Chambers [5]. The same data

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suggest that there is an increasing frequency and severity of significant failures as evidenced by cumulative release volumes (102.7 Mm^3 vs 14.1 Mm^3 runout distance (859.6 km vs 292.3 km), and the number of deaths (486 vs 342) which are all dramatically higher in 2010–2019 than in 2000–2009). This trend has raised globally awareness on the necessity for safer design, management and operation of tailing dams. Picuillo et al. (2022) proposed a new look at the statistics of tailings dam failures. According to the findings of the analysis that was carried out using CSP2, the historical trend regarding the number of failures has an average of 2.5 failures per year. The safety of these plants is greatly dependent on the static and cyclic characteristics of the tailing residues, as well as the geological and hydrogeological regime of the selected site for disposal. Clarkson and Williams [6], when the guidelines, acts and regulations were reviewed, reported that the possible cause of the occurred failures is the global misalignment between the standard of practice for monitoring and the installed instrumentation. In addition, the International Commission on Large Dams (ICOLD) after investigating the failure of 221 tailings dams suggested that most of these failures were avoidable (ICOLD, [7]). Santamarina et al. [8] noticed that post-failure investigations often are related to departures from regulation and good practice.

Roberson (2015) reports that based on estimations in each one-third century, the probable risk of the tailings dam and fly ash ponds failure increases by 20-fold. Although coal combustion patterns advise that there may be more than 9000 fly ash ponds worldwide, in-depth analysis of tailing dams and fly ash pond failures shows that there are still gaps in the knowledge and understanding of failure mechanisms, and particularly of time-delayed triggering mechanisms (Santamarina et al., [8]).

Thus, assessing the stability of the structure that contains the coal ash pond is essential to minimise the risk of failure, which can result in catastrophic consequences. Conducting slope stability analyses using an appropriate methodology, strength characterisation and reliable pore pressure distribution data, for various failure modes, is critical for the management of the anticipated failure risks associated with tailings and fly ash pond storage facilities. The key parameters that can influence the stability of an embankment dam retaining fly ash are the geometry of the dam, the material strength parameters, the pore pressure distribution and the seismic activity. Pore pressure distribution has a high spatial and temporal variation, even in relatively short periods of time. Thus, it has a significant impact on the factor of safety (FS) of the structure, in the long term.

The scope of this paper is to illustrate through slope stability analysis, the sensitivity of the FS calculated, based on different assumptions, limited available geotechnical data and phreatic level measurements, which is very common in the case of old or inactive facilities. A probabilistic

slope stability analysis was performed to account for the limited data available, and subsequently, a qualitative risk assessment evaluation was performed. The FS was also estimated using pore pressures calculated from numerical seepage analysis. A three-dimensional (3D) model of the hypothetical cross-section is generated to investigate the effect of modelling, two-dimensional (2D) vs. three-dimensional (3D) limit equilibrium analysis, on the estimated value of FS, and related critical surface.

2 Materials

2.1 Input Parameters

In principle, slope stability assessment is a stepwise procedure, based on the definition of a potential failure mode, the quantification of input parameters and the calculation and evaluation of stability results. To reliably estimate the critical safety factor, it is essential to use representative input parameters. Fourie et al. [9] conducted laboratory tests on fly ash from four different hydraulic fill ash dams in South Africa, to investigate the characteristics of fly ash material. The results showed little variation in basic geotechnical properties, despite differences in grain size distribution. Trends for fly-ash properties from 95 worldwide reported cases were compiled and statistically analysed by (Bachus et al., [2]). The physical parameters and material properties adopted in the current paper were based on the aforementioned published reports, which are summarised in Table 1. Probabilistic analysis was performed to account for the limited amount of data regarding physical and material properties, to establish the most probable material properties.

Table 1 Summary of fly ash material properties

Material	G_s	Unit weight (kN/m^3)	Friction angle (°)	Cohesion (kPa)
Fourie et al. [9] (samples from four fly ash dams)				
Ash daywall	2.21(2.24–2.79)	–	35	0
Ash basin	2.21(2.24–2.79)	–	35	0
Data trends compiled from published reports on fly ash produced around the world (average values) [2]				
Ash daywall	2.27	–	36.7	
Ash basin	2.27	–	36.7	
Current study				
Ash daywall	2.21	14.5	35	0
Ash basin	2.21	13.6	35	0

2.2 Failure Mode

Slope failure of tailing dams and fly ash ponds can develop under different failure modes that mobilise drained or undrained shear strength. In this case study, a drained shear failure was considered under the assumption that failure propagates at a very slow rate, or that the geomaterial exhibits dilation, and as a result no excess pore pressure can generate during shearing.

The geometry of the analysed model and the phreatic lines that were considered in the analysis are presented in Fig. 1 along with the assigned material regions. The water table location W(mean) represents the measured phreatic surface levels from standpipe piezometers. Sensitivity analysis was performed in order to check FS value for the tested scenarios for the range of maximum W(max) and minimum W(min) water table levels. In this case study, the height of the slope is 40 m and the slope angle is 1v:3 h.

2.3 Pore Pressure

The seepage and pore pressure regime are significant factors to consider during the planning, design and operation of coal ash and mine waste tailing facilities. Tailing dams and ash ponds are affected not only by induced surface loads, but also by changes in deposition rates, rainfall, drainage, decanting, consolidation, etc. In addition, the possible movement of fine grains can result into progressive clogging of internal drainage pathways and can result in phreatic line rise within the impoundment. The results of the processes mentioned above might take years to become evident. Thus, there is a need for continuous monitoring of such facilities

to properly assess their advancement in time (Santamarina et al. [8]). Thus, pore pressure measurements with cone penetration testing (CPTu) probing is one of the most popular practices to determine the phreatic line, complementing the multiple standpipe piezometer measurements.

Hawley and Cunning [10] suggested that understanding of the expected groundwater conditions, both within the waste dump like a fly ash facility or stockpile and in the underlying foundation, is key to developing a reliable geotechnical model. In extreme cases, high pore pressures can result in liquefaction failure with potentially catastrophic consequences. The same authors highlight that where the foundation is composed of saturated, fine-grained soils with low hydraulic conductivity, the potential for construction-induced pore pressures and undrained failure must also be considered. In this study, based on the life assessment of the hypothetical facility, the rate of rise for the dam is approximately 1.70 m, the estimated end of life elevation based on the targeted deposition rate is 1650 masl and the rate of rise predicted is 1.65 m per year. The final height of the dam could be reduced if the predicted life of the dam could be extended.

The measurement of pore pressures in a tailing storage facility or a fly ash pond is an important input parameter for slope stability analysis. The pore pressures can be established by conducting (CPTu) probing or using standpipe piezometers which only give isolated point in time values unless they are fitted with electronic 'retro-fit' measuring devices. CPTu tests are increasingly used because they give a vast array of other important information in addition to the phreatic levels (e.g. density, consistency, permeability/ hydraulic conductivity, material character, undrained shear
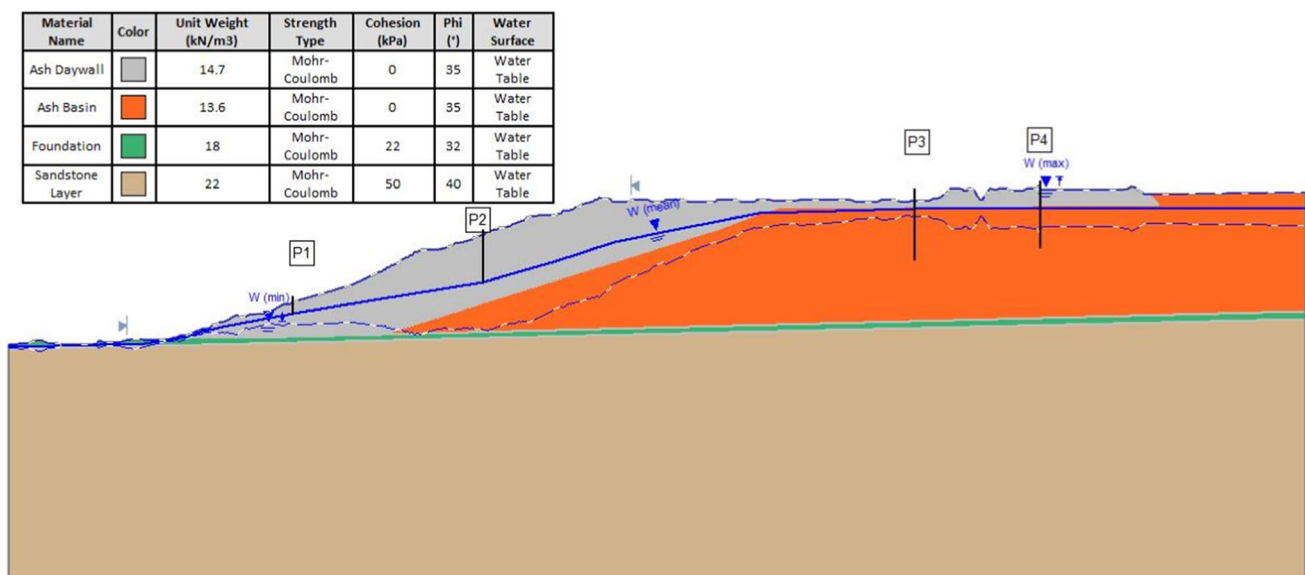

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface
Ash Daywall		14.7	Mohr-Coulomb	0	35	Water Table
Ash Basin		13.6	Mohr-Coulomb	0	35	Water Table
Foundation		18	Mohr-Coulomb	22	32	Water Table
Sandstone Layer		22	Mohr-Coulomb	50	40	Water Table

Fig. 1 Model geometry, material regions and phreatic lines

strength ratios, overconsolidation ratio, state parameters etc.) that are essential for slope stability analyses. Due to the cost related to CPTu, alternative solutions such as open-end standpipe piezometers and vibrating wire piezometers are more commonly used in fly ash dam facilities in South Africa.

3 Methodology

3.1 Seepage Analysis

A seepage analysis was performed to calculate the seepage-induced pore-water pressures within the ash wall and ash basin. In this study, a 2D Slide2 [11] was used to estimate pore water pressures through finite element groundwater seepage analysis. Slide2 is slope stability software package that applies limit equilibrium method, with a built-in finite element groundwater seepage capability for both steady state and transient flow conditions. In this type of analysis, the same model can be utilised for the slope stability problem and the groundwater seepage analysis. The boundaries of the problem, once defined, are used for both the groundwater analysis and the slope stability analysis. The approximate number of three-noded triangle mesh elements in the model is 1500. The number of elements was determined by checking how the mesh size influences the accuracy of the results. The mesh sizes were gradually reduced to 1500 until no changes in the results were observed. The analysis was performed in a steady-state condition. The upstream side to the right hand was assigned a total head of elevation $+88$ m. The downstream side to the left was not assigned a total head. Upstream was assigned as not known boundary conditions. The bottom boundary is set to no flow condition. The dry beach length was assigned 100 m.

The value of hydraulic conductivity adopted for the seepage models for the ash wall was $k = 5 \times 10^{-4\ 5}$ cm/s and for the ash basin $k = 3 \times 10^{-4\ 5}$ cm/s, for the foundation clay layer $k = 4 \times 10^{-5\ 6}$ cm/s. Quantities carried out by Roussev [12] and by Fourie and Strayton [13], at Lethabo ash dump in South Africa, suggested a value of $k = 4 \times 10^{-5}$ cm/s. Data trends from published reports a mean a range of values of measured hydraulic conductivity which span three orders of magnitude, from 4×10^{-4} to 4×10^{-7} cm/s, with a mean value of 3.8×10^{-5} cm/s. Based on the above, the value of hydraulic conductivity adopted for the seepage models for the ash wall was $k = 5 \times 10^{-5}$ cm/s and for the ash basin $k = 3 \times 10^{-5}$ cm/s, for the foundation clay layer $k = 4 \times 10^{-6}$ cm/s. The analysis was conducted considering vertical permeability (k_v) to be smaller relative to the horizontal permeability (k_h). For the ash daywall, and the ash basin, $k_h/k_v = 10$, while for the sandstone layer, and foundation, $k_h/k_v = 1$, was considered. Bachus et al. [2] also suggested that segregation and layering are pervasive in water-deposited ash based on laboratory testing and X-ray images of recovered specimens.

3.2 Seismic Activity

Seismic activity has a major impact on slope stability and thus, with or without liquefaction seismicity, must be considered as per the Global Industry standards on tailing management (GISTM, 2020) [14]. In the current example, the pseudo-static deterministic approach was used. The seismic load for a pseudo-static analysis is usually represented as an equivalent horizontal load. The simplification of the pseudo-static principle was used considering the horizontal component of an earthquake vibration, which will have the most destabilising effect. In this study, a seismic acceleration of $a = 0.120$ g was assumed. This assumption was made based on information obtained from the seismic hazard map of South Africa, which is used to map the expected peak ground acceleration (PGA). The PGA is specifically mapped with a 10% probability of being exceeded within a 50-year period. The chosen seismic acceleration, as per the hazard map, aligns with a return period of 475 years, SANS 10160 [15]. This is specific to the region of Mpumalanga where most of the coal-fired power plants are located. To balance for the probable remoulding of materials during shaking, the peak undrained shear strength was subjectively reduced by 20% (Makdisi & Seed [16]). An assumption is made that the fly ash tailings were adequately drained and consolidated and therefore not expected to liquefy with a PGA of 0.12 g.

3.3 Analysis Methodology

In this study, the analysis was repeated for different scenarios (i.e. static, seismic loading conditions, a variable phreatic line and seepage analysis) and the calculated results are summarised in Table 2. The pore pressure was initially established based on limited readings from standpipe piezometers W(mean). The water table location W(mean) represents the measured phreatic surface levels from standpipe piezometers.

Table 2 Summary of different scenarios considered for slope stability analysis

2D/3D	Scenario	Pore-water pressure Standpipe piezometers
CASE A	Static	W(mean)
CASE B	Seismic	W(mean)

An array of 4 piezometers were installed in 4 boreholes (Fig. 1) in this example, for the purpose of monitoring pore-water pressures. The piezometers are in the ash daywall, two at the toe, one in the middle of the slope and one at the slope crest. The pore-water pressures' measurements were used to determine the initial seepage boundary conditions and to adjust the seepage parameters to realise the best agreement between the measured and calculated pressure heads. The hypothetical cross-section relates to a case study where only 4 boreholes were installed. The authors argue that the number of readings and piezometers are not sufficient; therefore, a FS calculation is proposed considering porewater pressure as were calculated from seepage analysis. A sensitivity analysis was performed to estimate FS value for the range of assumed maximum and minimum water table level. Sensitivity analysis was performed in order to check the FS value for the tested scenarios for the range of maximum W(max) and minimum W(min) water levels. Seepage-induced pore water pressures were used in the subsequent slope stability analysis.

The analysis was conducted using limit equilibrium method (method of slices for 2D and method of columns for 3D, adopting Morgenstern-Price [17]).

In theory, a slope is stable when FS > 1. For design purposes, due to uncertainties related to input parameters, a FS higher than 1.3 for short-term stability and FS higher than 1.5 for long-term stability is necessary (Szymański M.B. [18], Hawley and Cunning, [10]).

A probabilistic slope stability analysis was performed with Slide2 based on the slip surface which was located by the regular (deterministic) slope stability analysis. The safety factor was re-calculated for $N = 1000$ times for the overall slope, using numerous randomly generated input variables for each analysis. For the probabilistic analysis, model input parameters, friction angle and unit weight were introduced as random variables. A normal distribution was adopted for all three variables; standard deviation and min and max values were completed for each variable, to define the statistical distribution of each random variable. It is expected that for the requirements of a normal distribution, 99.7% of all samples to fall within three standard deviations of the mean value. Thus, a relative minimum and relative maximum value of three times of the standard deviation was followed, to ensure that a complete (non-truncated) normal distribution is defined.

3.4 3D Limit Equilibrium Analysis

Three-dimensional (3D) limit equilibrium analysis has recently become more popular, as 3D limit equilibrium software is widely commercially available. While 2D procedures have found to be a reliable analysis method, there are circumstances where 3D analysis is required to define the slide surface and slope geometry more precisely. Early studies by Cavounidis [19] and Hungr et al. [20] suggested that factors of safety calculated using 3D analysis of an ellipsoidal slip surface are larger than those estimated using 2D analysis.

In the current study, the 3D slope was produced by importing the 2D Slide2 cross-section into 3D Slide3 [21], and extruding the model to a length of 618 m to realise a wall of a ring-dyke impoundment. After the extrusion of the 2D slope in Slide3, a focus section was added to include the entire slope. The focus section was analysed using a grid search and the factor of safety was determined. The grid search method is the search method for locating the global minimum safety factor for non-circular slip surfaces in 2D, generating spherical and ellipsoid slip surfaces in 3D. In 3D slope stability analysis, the factor of safety is computed for all slip surfaces, and the minimum safety factor is the critical slip surface determined. The slope stability analysis was performed for both spherical and ellipsoid slip surfaces.

4 Results

The limit equilibrium analysis results for static conditions are illustrated in Fig. 2. The seepage analysis results are illustrated in Fig. 3, simulating the pore pressure regime through seepage analysis. Figure 4 shows the results of the 3D slope stability analysis. The results for all the scenarios are summarised in Table 3 for 2D analysis and Table 4 for 3D analysis.

In 2D analysis, the estimated safety factors were 1.54 for static conditions, with water table values based on reading from standpipe piezometers, and 1.04 pore water pressures resulting from seepage analysis. The safety factors resulting for seismic conditions were estimated 1.11 and 0.86 respectively. The outcome of the probabilistic analysis suggested a probability of failure for static conditions with water table level values based on piezometer reading (PF) of 8.2% (8.2 out of 1000 slopes failed), and for seismic conditions 28.52% (28.52 out of 1000 slopes). Upper and lower bound envelopes (Hawley and Cunning, [10]) were used, as presented in Fig. 5.

In 3D analysis, the calculated safety factors (1.63) were similar for the ellipsoid slip surfaces, as illustrated in Fig. 4. The slip surface corresponding to the minimum safety factor is the one with aspect ratio of the ellipsoid closer to 1. In this case, the end effects diminish, and the factor of safety is expected to approach the 2D safety factor which is 1.55. A section was subsequently created from the 3D model and a 2D analysis was performed. The resultant factor of safety of 1.58 was computed, and the probability of failure was calculated as 0.1% for static conditions and 37% for seismic scenario.
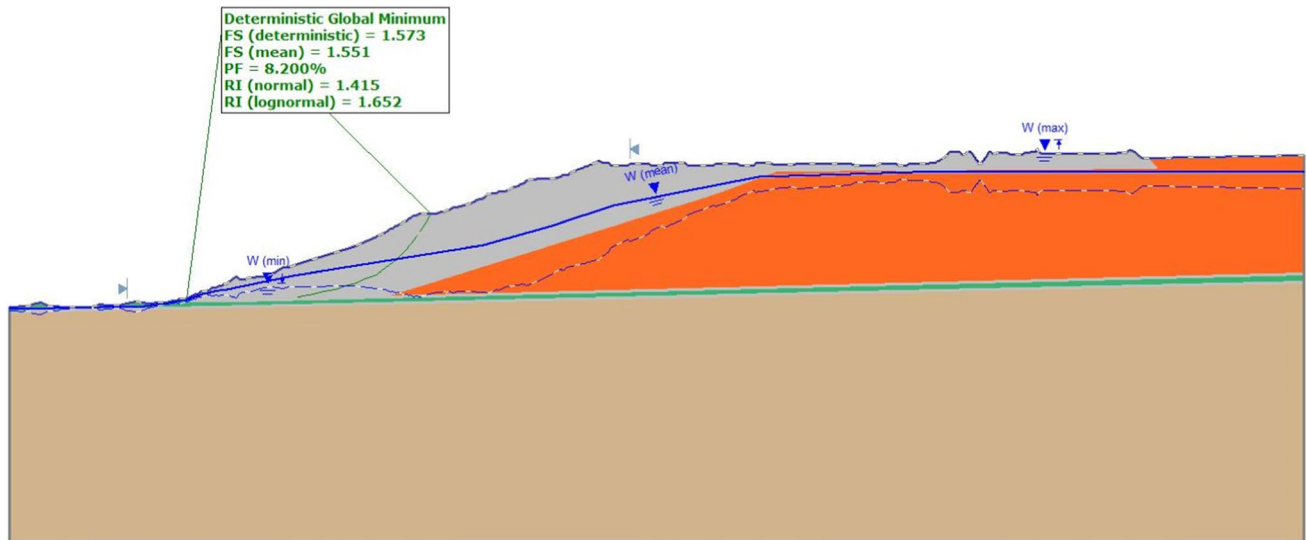
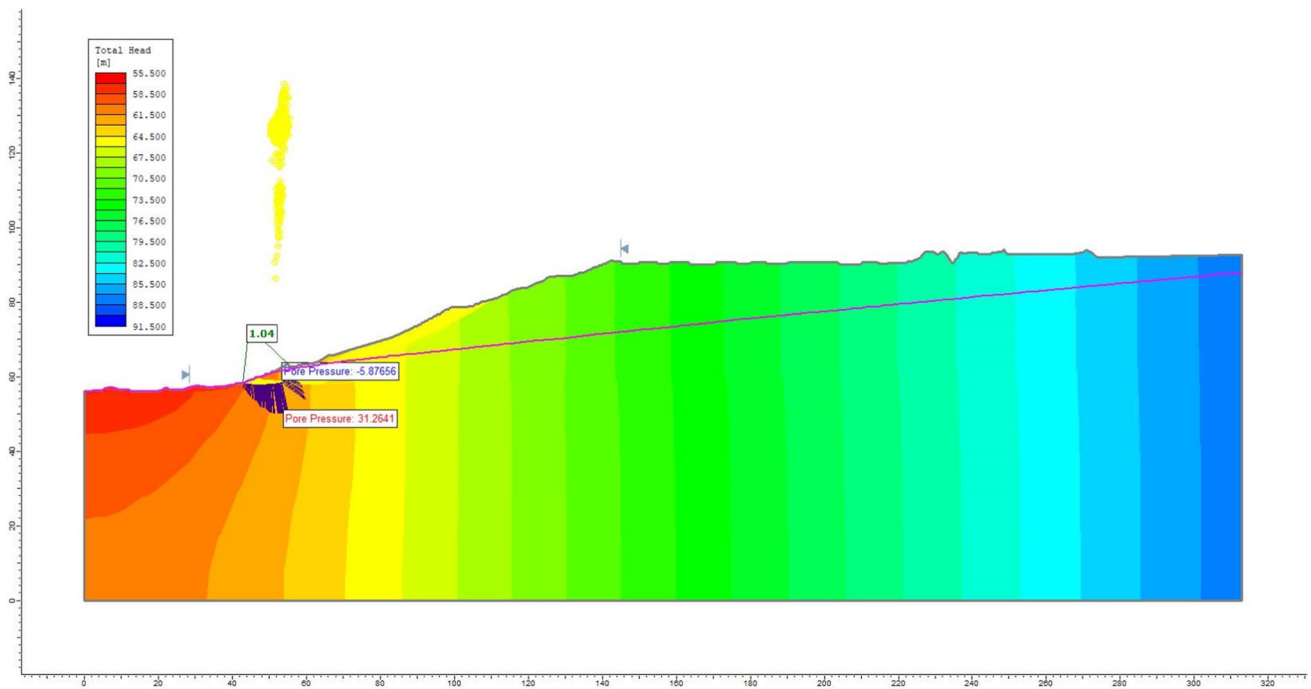**Fig. 2** Results of 2D slope stability analysis, based on *W*(mean) phreatic line

Fig. 3 Results of 2D slope stability analysis and seepage analysis

The safety factor for static conditions considering pore pressures based on the phreatic line (resulting from stand-pipe piezometers) is computed to be 1.55 in 2D analysis, and 1.63 in 3D analysis, mobilising a relatively shallow failure surface. The failure mode suggests that the shearing occurs so slowly that no excess pore pressures would be generated. The safety factor when the phreatic line from seepage analysis was considered is calculated to be 1.04.

The computed safety factors for all the studied cases are listed in Table 3 for 2D analysis. The 3D analysis results are summarised in Table 4.

4.1 Qualitative Risk Assessment

The suggested stability acceptance criteria for the design of waste rock dumps and stockpiles are summarised in Table 5
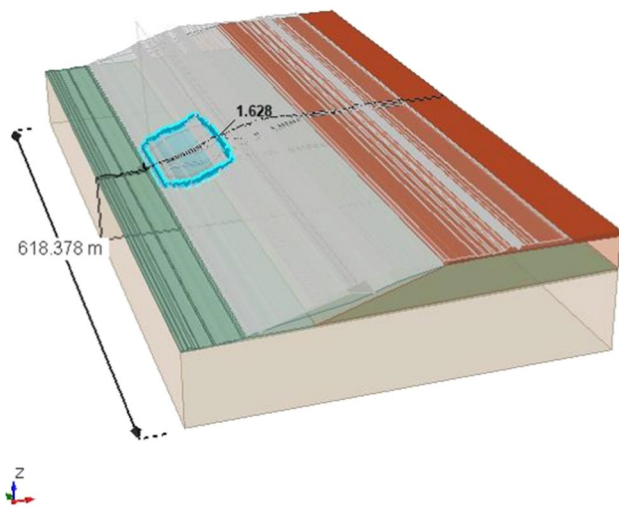
Fig. 4 Results of 3D slope stability model, based on piezometer readings *W*(mean) phreatic line

Table 3 Summary of 2D slope stability analysis

Case	Scenario	Safety factor	
		Pore-water pressure measurement (standpipe piezometers)	Seepage analysis
A	Static *W*(mean)	1.54	1.04
B	Seismic *W*(mean)	1.11	0.74

Table 4 Summary of 3D slope stability analysis

3D analysis	Scenario	Safety factor
		Pore-water pressure measurement (standpipe piezometers)
A	Static *W*(mean)	1.63
B	Seismic *W*(mean)	1.11
C	Section created from the 3D model (static)	1.61

(Hawley and Cunning, [10]). The criteria were followed, as they presented a range of FS values for each category as well as a range of probabilities of failure. These criteria follow the approach adopted to define stability acceptance criteria in Guidelines for Open Pit Slope Design (Read and Stacey 2009). The table uses a similar approach but substitutes 'Consequence' for 'Scale'. It introduces some additional complexities to consider 'Confidence' (or reliability) in the key input parameters and analytical technique used. The 'Consequence' involves both the impact of potential instability, which is often related to the scale or size and mechanism of instability and the structure's service life. The inclusion of service life under 'Consequence' is intended to retain the sensitivity of the acceptance criteria to the design basis for the structure (i.e. short-term (construction/operations) versus long-term (closure)).

The suggested acceptance criteria are also illustrated graphically on the following charts, presented in Fig. 6, after Mark Hawley and Cunning [10]. The charts represent the suggested acceptance criteria using a matrix approach through a qualitative risk assessment. According to this concept, an indirect index of risk is proposed based on the consequences and confidence. For example, a low consequence rating combined with a high confidence rating leads to a relatively lower overall risk and results in a lower required FS. A simplified approach is adopted in these diagrams, and the PF is related to both FS and consequence.

The qualitative risk assessment method, using the matrix approach for the conducted analysis, suggests that for static conditions, considering confidence (reliability) and potential impact (consequence), the results are acceptable for MODERATE confidence − MODERATE consequence conditions, and the PF is ≤ 8.2%. The results of the seismic analysis suggest HIGH confidence − LOW consequence with a PF of 28.52% resulting from the 2D analysis, and 37%, resulting from the 3D analysis.

5 Conclusions

In this paper, slope stability analysis of a fly ash pond facility was performed to assess the stability when little information is available to estimate the material characteristics and pore water pressures in the facility. The scope of the study is to demonstrate the sensitivity of the safety factor value, and the implication of adopting representative material properties, and water table levels, even in cases where limited data are available, and assumptions need to be made to assess the long-term stability of a fly ah dam facility and yield reliable results. To run the analysis, informed assumptions were made mainly based on the literature, and were used to calibrate a numerical model and perform seepage analysis. This analysis also aimed to compare critical failure surfaces and resulting safety factors under static and seismic loading conditions in 2D and 3D modelling. The 3D slope stability analysis was considered to investigate the effect of lateral constraint and compare the resulting safety factors and associated critical surfaces in the 2D model.

The results suggest that it is essential to make the informed assumptions regarding the failure mechanism, the material properties and the pore pressure distribution within a fly ash pond.
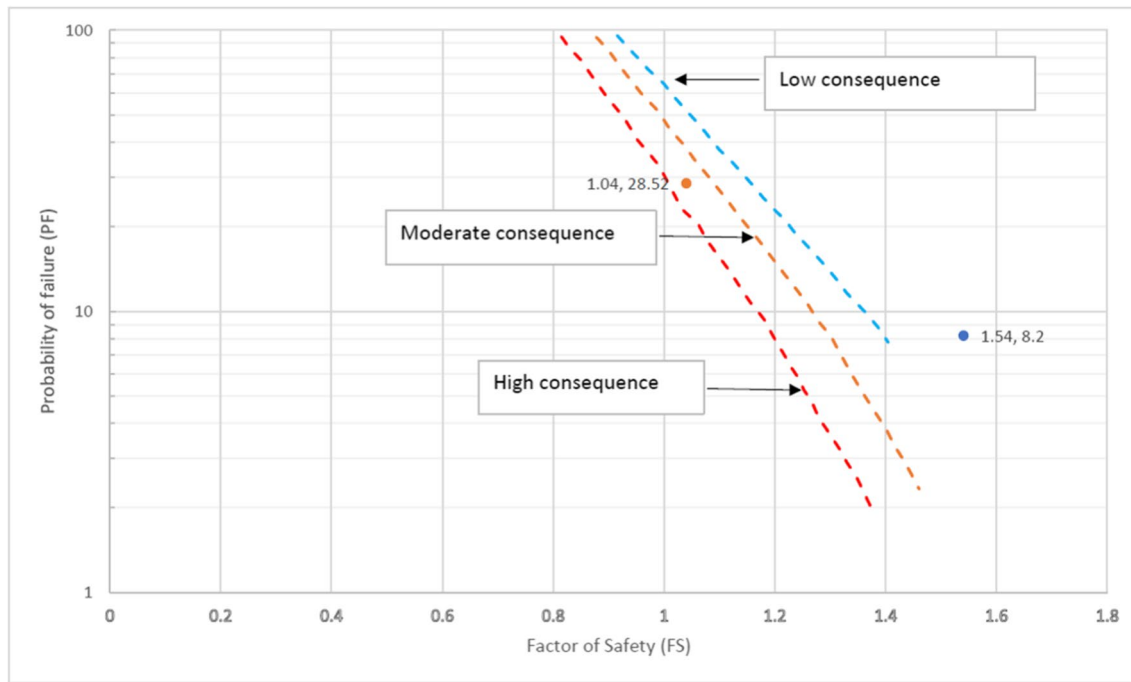
Fig. 5 Factor of safety versus probability of failure, (reconstructed from Hawley and Cunning, [10])

Table 5 Suggested stability acceptance criteria for stockpiles and waste rock dumps adopted from Hawley and Cunning [10]

Consequence	Confidence	Static analysis		Seismic analysis	
		Min. FS	Max. PF	Minimum FS	Max. allowable strain*
Low	Low	1.3–1.4	10–15%	1.05–1.1	≤1
	Moderate	1.2–1.3	15–25%	1.0–1.05	≤1.5%
	High	1.1–1.2	25–40%	1.0	≤2%
Moderate	Low	1.4–1.5	2.5–5%	1.1–1.15	≤0.75%
	Moderate	1.3–1.4	5–10%	1.05–1.1	≤1%
	High	1.2–1.3	10–15%	1.0–1.05	≤1.5%
High	Low	≥1.5	≤1	1.15	≤0.5%
	Medium	1.4–1.5	1–2.5%	1.1–1.15	≤0.75%
	High	1.3–1.4	2.5–5%	1.05–1.1	≤1%

*For the purposes of these acceptance criteria, displacement or deformation estimates derived using these methods can be converted to an 'equivalent' strain by dividing the estimated cumulative displacement or deformation by the length of the critical failure path and expressing the result as a percentage. If the length of the critical failure path is unknown or not easily obtainable, the overall slope length or height of the embankment can be used as a conservative estimate for the length of the critical failure path. If numerical techniques are used, then convergence of the numerical model is necessary before the cumulative displacement can be calculated (Hawley and Cunning, [10])

When data are available, further research is necessary to study the effect of the dry beach length or the upstream slope on the calculated safety factor values. Based on numerical simulations of seepage and stability analysis of tailing dams, Zhang et al. [22] suggest that the longer the dry beach, the higher the safety factor of the dam.

Another important item to highlight is the limitation of standpipe piezometers. In studies, by Van der Berg [23] and Geldenhus et al. [24], the limitations of standpipe piezometer readings to determine the FS of tailing storage facilities were presented. They stressed the importance of
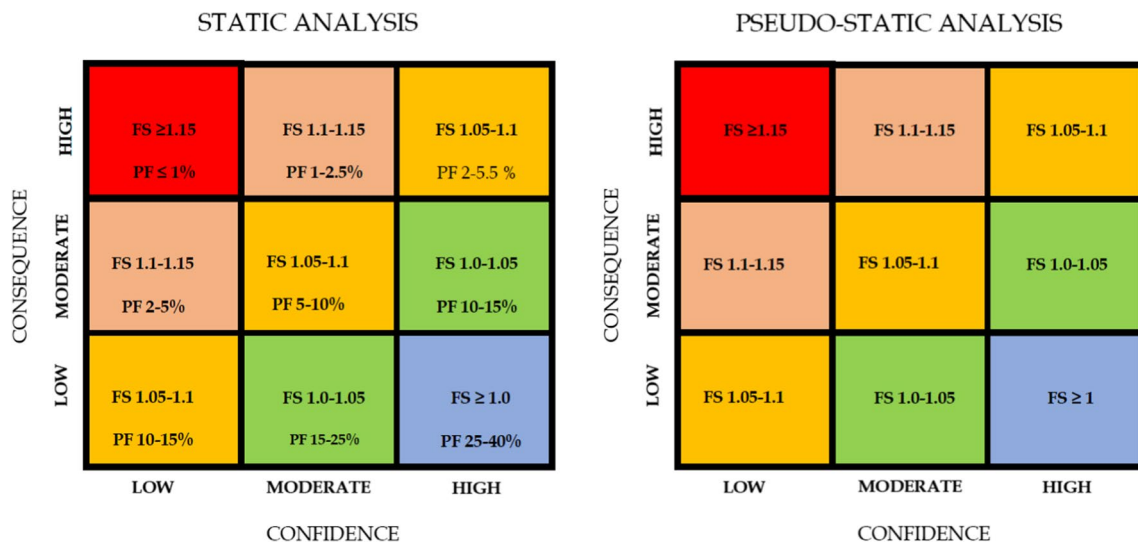
STATIC ANALYSIS

PSEUDO-STATIC ANALYSIS

Fig. 6 Acceptance criteria charts for static and seismic conditions (reconstructed from Hawley and Cunning, [10])

considering the tip level of the piezometers when inferring the phreatic surface, or, even better, the use of CPTu data.

In the current study, the results stemming from the 3D analysis appear to yield higher safety factors that meet the acceptance criteria for static conditions and seismic conditions. When calculated using 3D limit equilibrium methods, the FSs are 5% higher than in 2D analysis. The case study utilised in this paper showed consistency of failure surfaces in 2D and 3D analysis and safety factors in 2D and 3D. An interim conclusion that can be drawn from the current study is that different methods of analysis and 3D effects could be misleading, and it is always recommended to test alternate solution to a particular problem. It could be concluded that 3D geometry can generally increase the calculated safety factor. Therefore, engineering judgement and critical approach are usually expected in this type of analysis.

Declarations

Competing interests The authors declare no competing interests.

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