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Application of bursting pressure and conventional drained behavioural theory to the Brumadinho failure

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The Brumadinho tailings dam failure in 2019 has had a profound impact on the way tailings engineers analyse and interpret stability, with the focus of assessments moving away from conventional drained behaviour to consideration of undrained and post-liquefaction resistance. One of the contributing reasons is that the Expert Panel investigation of the failure concluded that, despite the drained factor of safety being adequate, the failure took place in an undrained manner. This paper examines the Brumadinho failure from a conventional drained perspective using two-dimensional limit equilibrium and finite difference modelling. Collectively, these analyses show that the failure can also be explained by a drained triggering mechanism and therefore suggest that several additional lessons may be learnt from the failure.

Keywords: tailings, upstreaming, bursting pressure, limit equilibrium, Brumadinho

INTRODUCTION

The Brumadinho tailings dam failure in 2019 has had a profound impact on the way tailings engineers analyse and interpret stability, with the focus of assessments moving away from conventional drained behaviour to consideration of undrained and post-liquefaction resistance. One of the contributing reasons is that the Expert Panel investigation of the failure (Robertson *et al* 2019) concluded that, despite the drained factor of safety being adequate, the failure took place in an undrained manner. Specifically, the mode of failure was attributed to a combination of ongoing internal strains owing to creep, and strength reduction owing to loss of suction in the unsaturated zone. This finding has led to the view that tailings facilities characterised by low factors of safety for undrained behaviour can fail suddenly and unpredictably, and that the hitherto widely used observational approach is not to be trusted to anticipate and forestall failure. This view is not supported by experience in Southern Africa, where some tailings facilities with low undrained factors of safety (a good proportion with factors of safety of less than one) have existed in a stable state for many decades (Wates 2023). Therefore, for the finding of the Expert Panel to be accepted as such a defining case, it is important to be certain that the Brumadinho failure cannot be explained by a conventional drained mechanism.

This paper examines the Brumadinho failure from a conventional drained perspective using two-dimensional limit equilibrium and finite difference modelling. Collectively, these analyses show that the failure can also be explained by a drained triggering mechanism, and therefore suggest that several additional lessons may be learnt from the failure.

GEOMETRY OF THE BRUMADINHO TAILINGS FACILITY

An aerial view of the Brumadinho facility before it failed is shown in Image 1. The facility was approximately 86 m high when deposition ceased in 2016 and contained approximately 12 million m^3 of tailings. The face of the Brumadinho tailings facility, as it began to fail on 25 January 2019, is shown in Image 2 (a video of the failure may be viewed at **https://g1.globo.com/mg/minas-gerais/noticia/2019/02/01/video-mostra-o-momento-exato-em-que-barragem-da-vale-rompe-em-brumadinho.ghtml**). The slumping of most of

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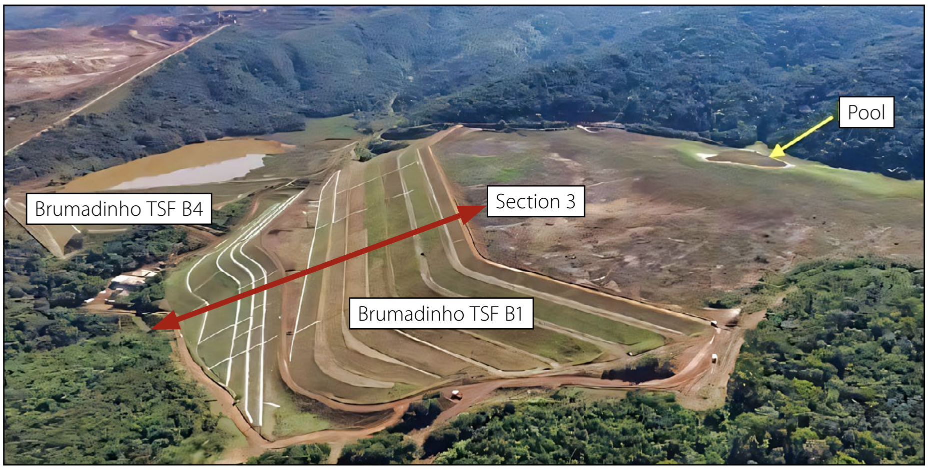

Image 1 Aerial view of the Brumadinho tailings storage facility (TSF) in 2018 before failure

Image 2 Image of Brumadinho at onset of failure initiation (25 January 2019)

the crest and bulging of the face above the deepest point in the valley can be seen in the image. The section that best represents the weakest point in the embankment is Section 3 from the Expert Panel Report, as shown superimposed on Image 1. The cross-section at Section 3, on first examination, would not be considered to represent a risk since the overall drained factor of safety for a slope of 1v:3h, combined with a friction angle typically expected for iron ore tailings (34 degrees), would be expected to be much greater than 1.5, even with an elevated phreatic surface.

The Expert Panel Report established that the elevation of the phreatic surface at the time of failure was approximately coincidental with the bench at elevation 900 m amsl (shown in Figure 1 from Robertson *et al* 2019). The predicted phreatic surface was found, by the Panel, to be realistic because of relatively poor foundation drainage and possibly poor performance of the drains that had been installed behind the starter embankment and subsequent raises. Actual piezometric levels and pressures derived from cone penetration test work (CPTu) are also shown in Figure 3 (see page 31) and illustrate a lower phreatic surface when compared to that predicted by the Expert Panel. Specifically, the piezometric and CPTu dissipation data give a relative consistent water elevation of about 878 amsl upstream of the starter wall.

KEY OBSERVATIONS

An important observation in the period leading up to the failure was the emergence of drilling fluid some distance away from the drill rig during installation of horizontal drainage hole DHP-15 (located about 200 m towards the right abutment from Section 3, and around elevation 880 about 20 m below the bench at level 900). The Prosecutor's investigations (Arroyo & Gens 2021) confirmed this observation by stating that "*... a serious incident occurred during perforation of DHP-15 on June 11, 2018. Installation of DHP-15 was suddenly halted when a localized flow of water and mud (sic tailings) was noted close to a drainage channel at a point 15 m to the side and 7 m above the drain perforation point. The drain was sealed, and flow contained with sandbags. That flow lasted until at least June 14 (three days later), when the outlets of two older drains were unblocked, releasing significant flows of water with suspended solids. These older drains were*

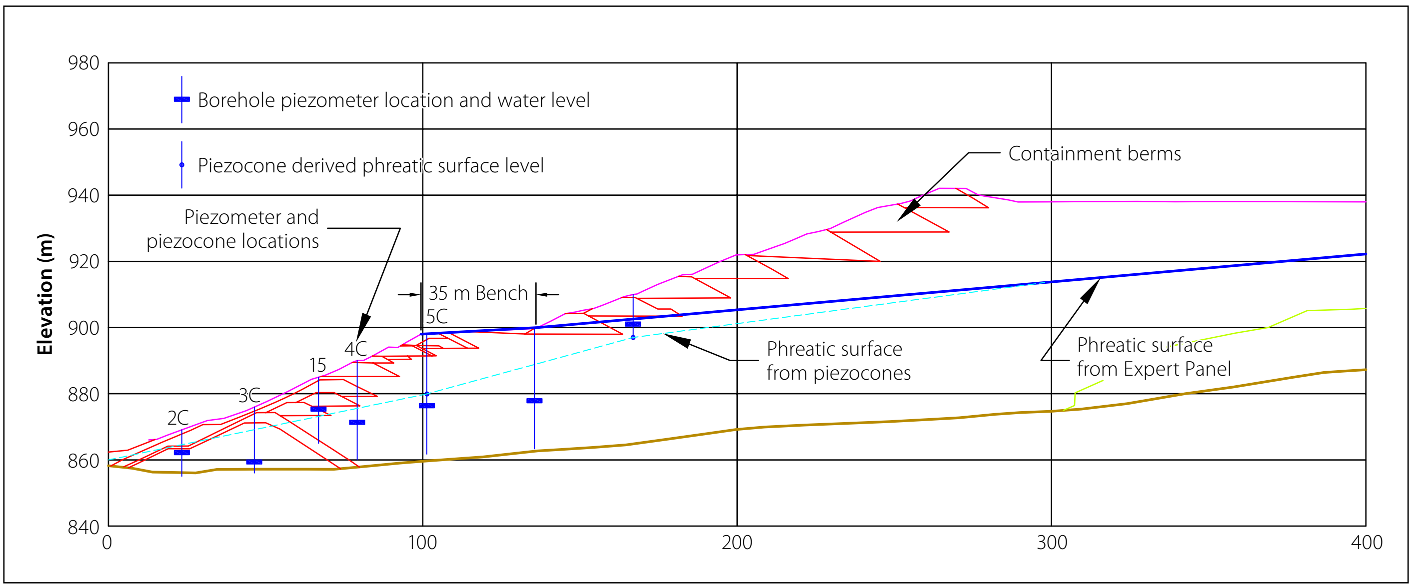

Figure 1 Section 3 illustrating location of phreatic surface from the Expert Panel compared to measured piezometer and piezocone dissipation levels (figure modified from Robertson *et al* 2019)

reported to have been located more than 150 m towards the left abutment of the injection point. During the same inspection visit, emerging flows were noted from older drains located close to and at the same elevation as DHP-15, and from a drain 250 m towards the left abutment. Once the older drains were unblocked, the flows gradually reduced at all points. The incident raised, for a few days, the water pressure by 0.6 and 3.5 m in piezometers PZ-07 and PZ-09, respectively, which were in the vicinity of DHP-15." The piezometers did not reflect the actual increase in water level, which was at least 7 m above the drilling point.

The observation implies that either the drilling fluid entered the drains directly or that hydraulic fracturing connected the drill hole to the drains while the drilling was taking place. The great distance (approximately 250 m) over which the interaction between the flows from DHP-15 and flows elsewhere was observed is remarkable, and implies that very significant pore pressure changes, over a wide front, could have been created by the drilling, and that the drains were indeed capable of transmitting water rapidly over a large distance. Overall, the relationship between the drilling activity and the observed responses strongly suggests that the observed emergence of water and mud (tailings) was associated with a connection between the drains, and that the alleged poor performance of the drains was probably more likely associated with blockage of the outlets than fouling of the drains. The loss of tailings from the interior of the facility would also have led to stress relaxation and possibly to the development of a cavity with potentially significant loss of support for the tailings in the interior of the facility.

DRAINED ANALYSIS FOR BRUMADINHO

The Expert Panel performed limit equilibrium analyses for drained conditions and concluded that the overall factor of safety was high enough to preclude drained failure, even if the phreatic surface was coincident with the bench elevation. This conclusion was independently checked by the author using the same parameters adopted by the Expert Panel and the factor of safety was found to be greater than 1.6 for failure of the full slope (see Figure 4 on page 32). Overall failure of the facility would therefore not have been expected to be initiated by a conventional drained mechanism.

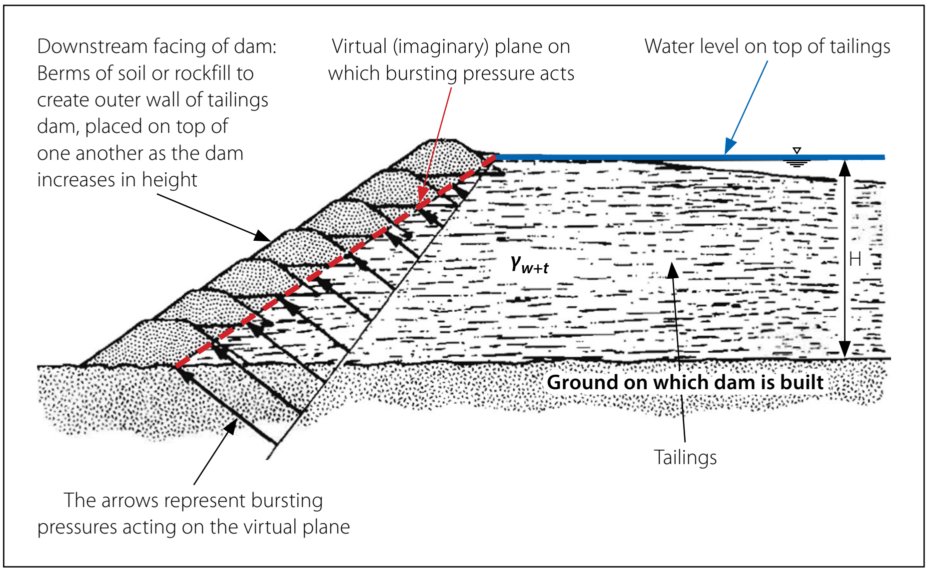

Figure 2 **Illustration of bursting pressures by Casagrande and McIvor (1970) as adapted by the author in Wates (2023)**

FAILURE MECHANISMS EVALUATED BY INVESTIGATORS

The Expert Panel and the Prosecutor's investigators do not agree on the failure trigger, although both concluded that an undrained failure had been triggered. The respective findings were as follows:

- Expert Panel: "*The sudden strength loss and resulting failure of the marginally stable dam was due to a critical combination of ongoing internal strains owing to creep, and a strength reduction owing to loss of suction in the unsaturated zone caused by the cumulative rainfall since the end of tailings deposition*".
- Prosecutors Experts: "*The undrained failure was triggered by over pore pressure from the drilling fluid at the drill hole being installed at the time of the failure*". (This was a vertical hole being drilled just below the crest.)

While the two reports agree that the tailings liquefied, they do not agree on the trigger mechanism. Neither the Expert Panel nor the Prosecutor's team considered that the drain hole DHP-15 could have triggered the failure, probably owing to it having been drilled about seven months prior to the failure.

The author is of the view that neither of the proposed triggering events is persuasive, primarily owing to the absence of a precedent. Alternative mechanisms therefore warrant further consideration for the following additional reasons:

- Failures (other than those induced by seismicity) can usually be explained by drained initiating mechanisms.
- Failures usually start at the toe of an embankment (usually preceded by cracks at the crest).
- The factor of safety of a facility is expected to increase and the probability of failure to decrease after decommissioning as the phreatic surface drops away.

The possible alternative initiating mechanisms which have been selected to better satisfy the three experiential rules described above are examined below.

Bursting failure mechanism

Casagrande and McIvor (1970) stated that the shell that provides resistance to failure of a tailings facility must be of sufficient width to retain "bursting pressures". Casagrande and McIvor's definition of a bursting mechanism is illustrated in Figure 2 (as adapted by Wates 2023).

Limit equilibrium analysis using the method of slices, does not adequately model the "bursting" pressures to which Casagrande and McIvor (1970) were referring, since pore pressure is assumed to dissipate down the face of an embankment or at the interface with a drain that intersects the phreatic surface. Limit equilibrium models assume that the pore pressure at the base of each slice is equal

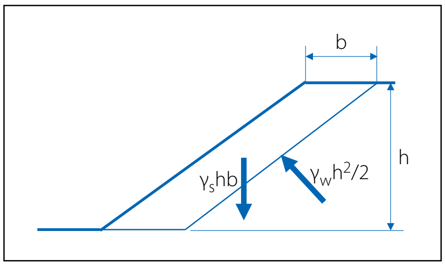

Figure 3 **Representation of thrust that develops on face of relatively impervious embankments**

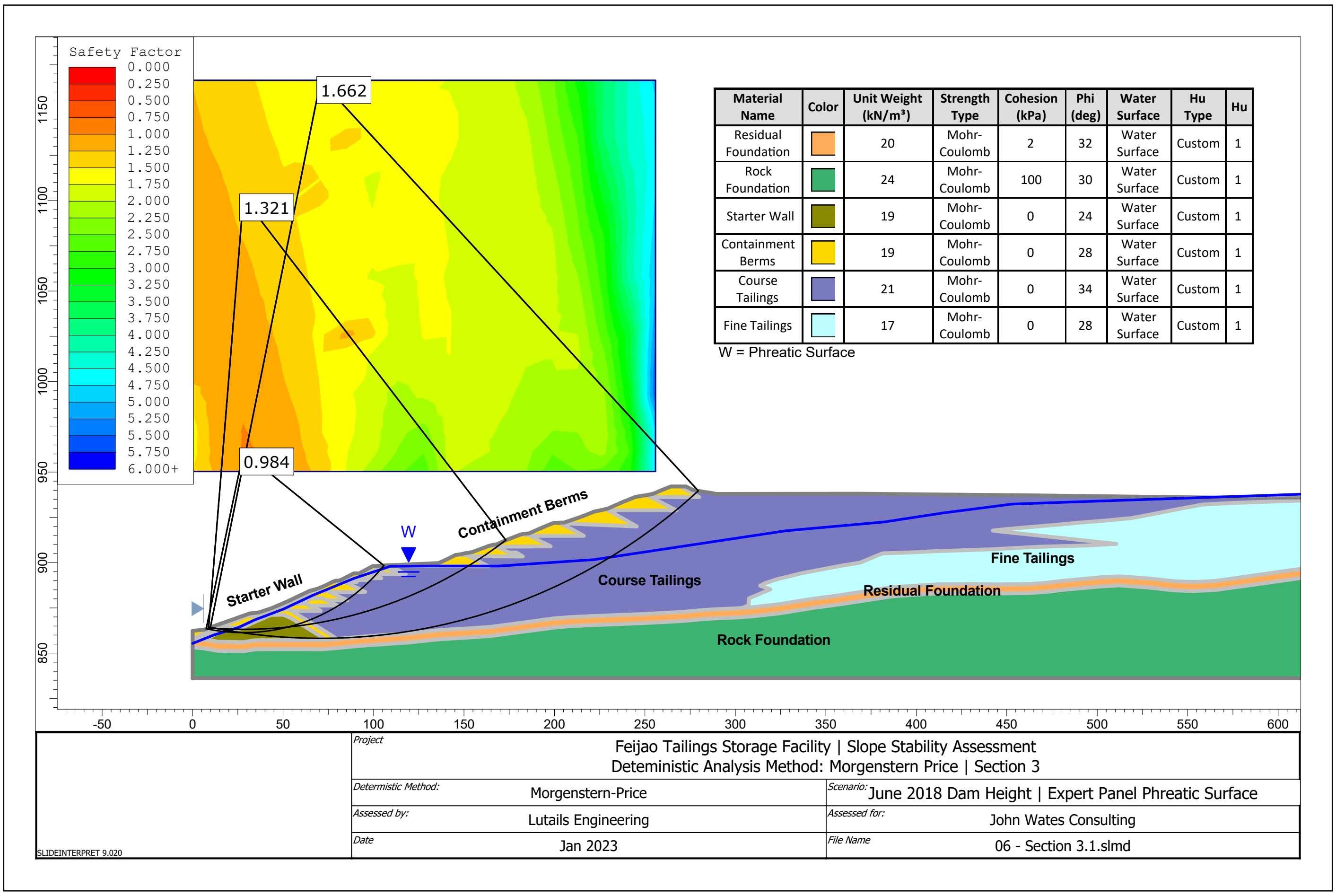

Figure 4 Shallow limit equilibrium result for bench and for full slope

to the product of the elevation difference between the free surface and the slice base multiplied by the unit weight of water. For the bursting mechanism the pressure is equal to the elevation difference between the point of exit of the phreatic surface and the base of the slice multiplied by the unit weight of water. Limit equilibrium analysis, using the method of slices, might therefore not correctly represent the Brumadinho case, since low permeability embankments may have allowed full hydrostatic pressure to build up behind the embankments (referred to as containment berms in the Brumadinho sections) if the drains were not fully functioning and adequately sized, or if drill holes were to introduce water into the drains at a rate faster than the water could be discharged. The equilibrium between the thrust (bursting pressure) arising from the hydrostatic pressure that exists behind relatively impervious embankments and the resisting mass is illustrated in Figure 3 on page 31.

In the configuration illustrated by Figure 3, the factor of safety against bursting failure (if the soil has no tensile strength) would be given by Equation 1.

$$\text{Factor of safety} = 2\,\gamma_s\, b\, h\, \text{Cos}\alpha/\gamma_w h^2 \qquad (1)$$

Where:

γ_s = bulk density of soil
b = width of resisting wedge
h = slope height
α = slope angle
γ_w = density of water

For the Brumadinho section, for a factor of safety of one, using Equation 1, the width of the resisting wedge calculated for hydrostatic pressure behind the full slope up to the bench would be between 7 m and 10 m depending on the bulk density of the embankments. Locally, especially where the containment berms are narrowest, factors of safety could thus have been lower than one. The calculation therefore suggests that the stability of the bench could have been compromised by the increase in water pressure behind the containment berms that was brought about by the installation of DHP-15. The increase in pressure could thus have caused deformations (which were not visible on the surface at the time), and which in turn could have altered the stress regime within the tailings below the bench. The presence of excess water pressure is corroborated by the observation that water emerged about 7 m above the position of DHP-15 while it was being drilled, suggesting that the internal pressure had been sufficient to cause local hydraulic fracturing.

The "bursting pressure" calculation has been compared with the conventional drained limit equilibrium analysis illustrated in Figure 4, using the Morgenstern Price model (Morgenstern 1965), which also shows that the factor of safety of the outer face of the slope below the bench would have been close to one.

Wedge failure mechanism

Equations to analyse the stability of a two-wedge translational failure mechanism proposed by Xuede *et al* (2003) were adapted to include a destabilising hydrostatic pressure acting along the failure scarp of the active wedge. This was done by assuming the hydrostatic pressure resulted in a horizontal and vertical force which could be incorporated into the force equilibrium equations used to obtain the factor of safety. The cross-section and two wedges assumed for the analysis are shown in Figure 5. This analysis also gives a drained factor of safety of about one for the Brumadinho slope geometry, assuming the water level is at the elevation of the first bench. This correlates well with the

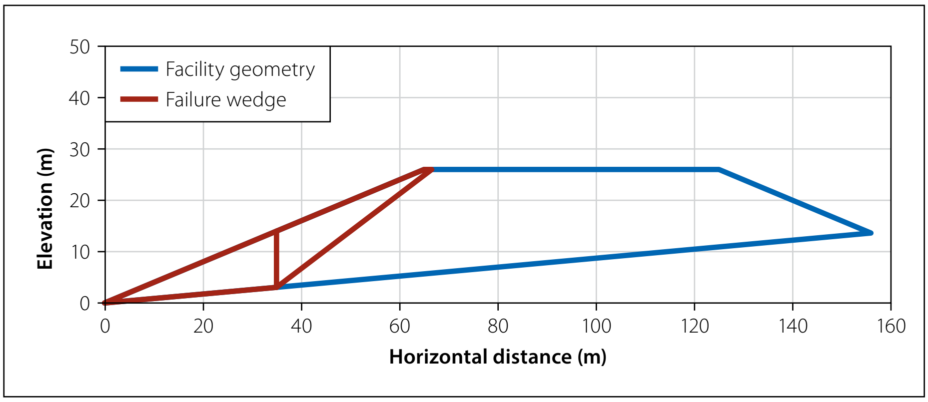

Figure 5 Cross-section illustrating two wedge analyses using the Xuede *et al* (2003) limit equilibrium model

Table 1 Comparison of factors of safety for method of slices, "bursting pressure" and two wedge limit equilibrium models

Case	Description	Factor of Safety
1	Full slope drained with Expert Panel piezometric level (method of slices Morgenstern & Price 1965)	1.6
2	Full slope drained with actual piezometer level (method of slices)	2.4
3	Bench only drained with phreatic surface at bench level (method of slices)	0.98
4	Bench only drained bursting pressure (7 m wide prism Casagrande & McIver 1970)	1.00
5	Bench only drained two wedge analysis (Xuede *et al* 2003)	1.00

"bursting pressure" and limit equilibrium calculations.

Table 1 compares the results of the calculations for the method of slices (Morgenstern & Price 1965) with the "Bursting Pressure" factor of safety (Casagrande & McIver 1970) and the adapted Xuede *et al* (2003) model.

Since the video of the failure clearly shows that the full slope failed, the drained analysis of the bench (level 900 amsl) only suggests that the slope below the bench may have existed in a meta-stable state up to the time of the failure of the full slope. The impact that deformation of the slope below the bench could have had on overall instability was tested with a limit equilibrium model that assumed that the support from the slope up to level 900 amsl had been removed by assuming that this portion of the slope first failed (deformed) having a factor of safety of one or less. The results of this computation are shown in Figure 6.

The analysis suggests that the removal of support for the slope above the bench up to full height could have initiated a conventional drained failure at any time after the support had been removed. The removal of support could well also have led to the creep to which the Expert Panel attributed the failure.

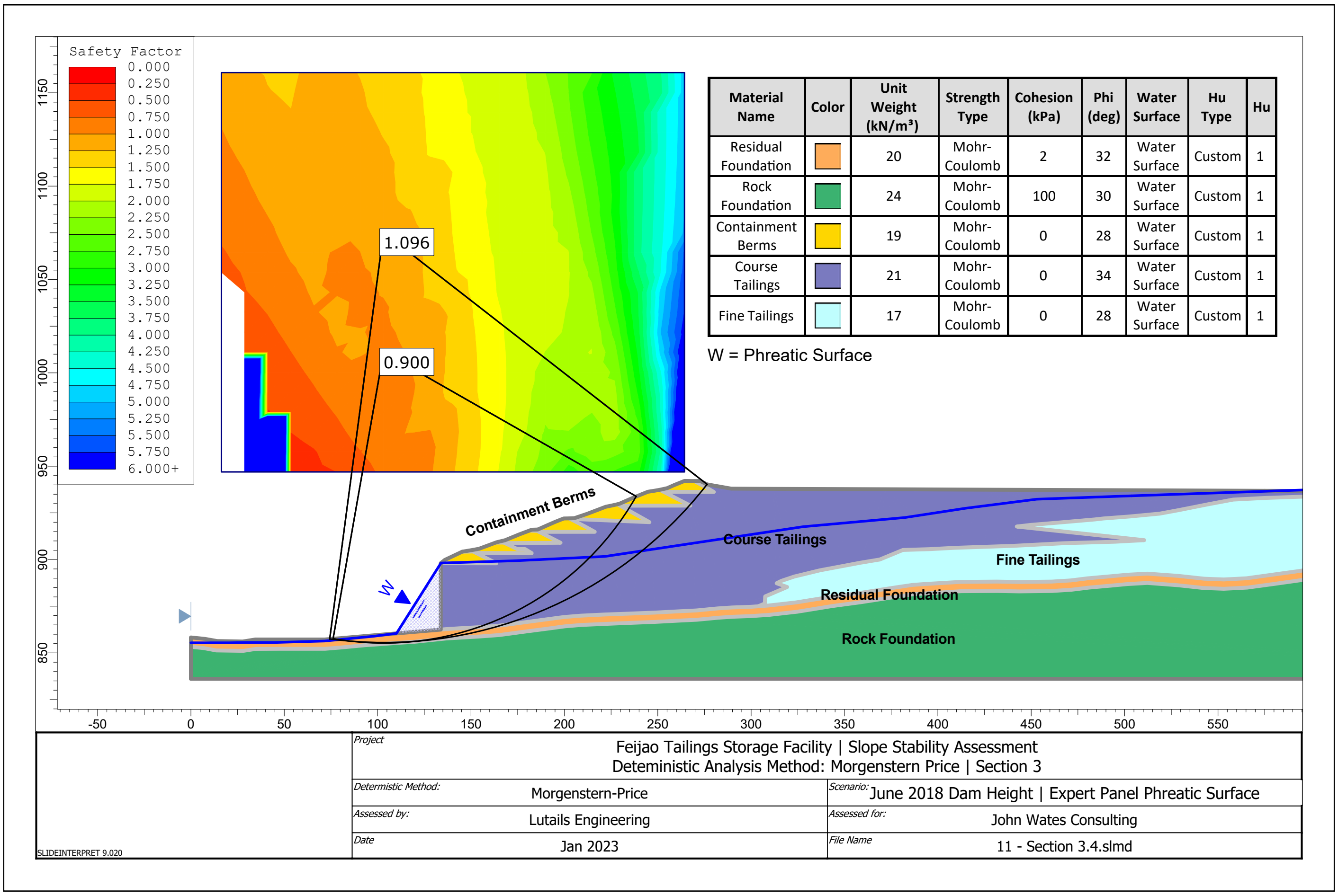

Figure 6 Limit equilibrium result assuming removal of support of the slope below level 900

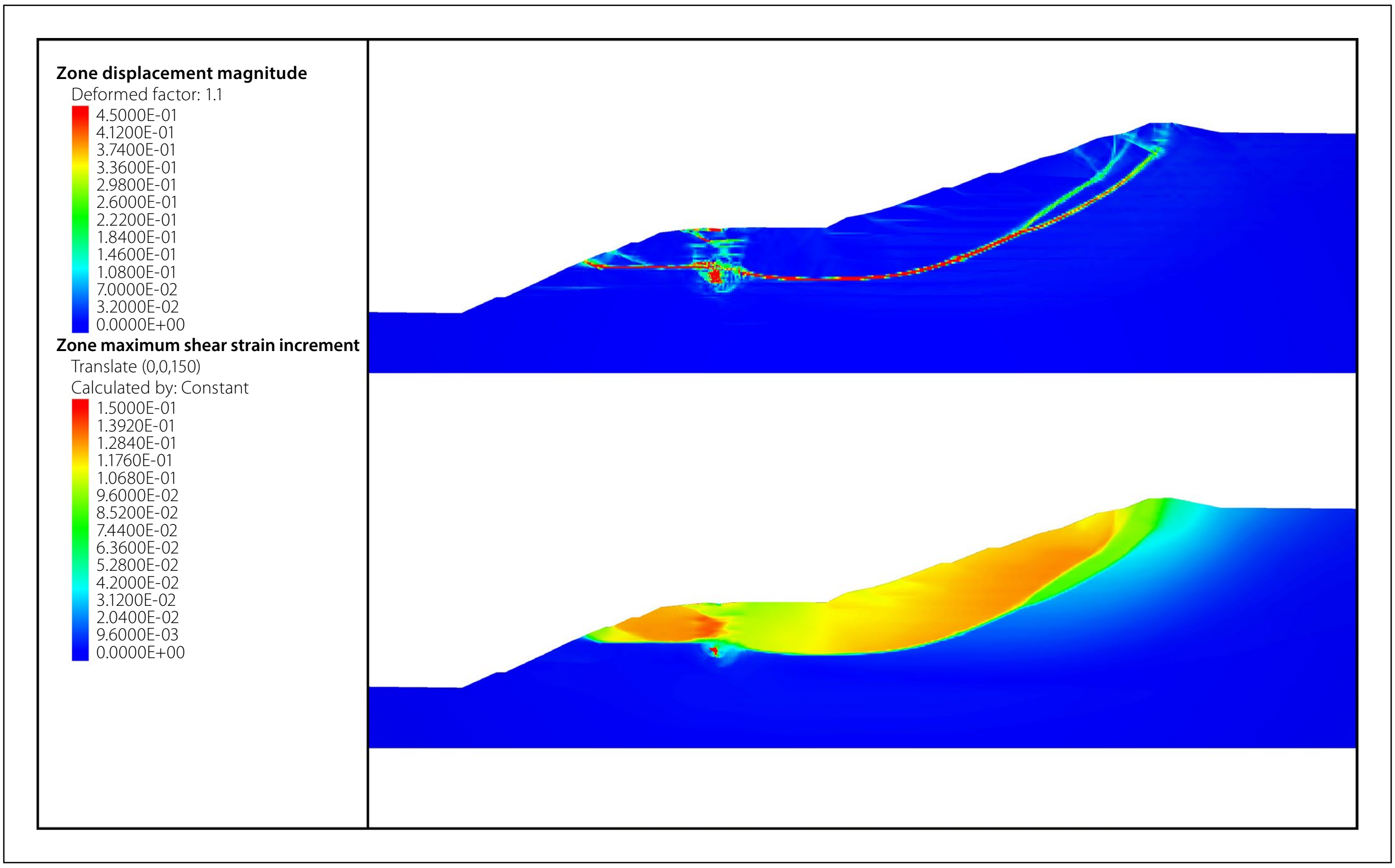

Figure 7 **Displacement and shear strain results (video of progression of failure may be viewed on YouTube https://youtu.be/-Wj23wiNBxs or the ARC website https://arq.co.za/?p=2656)**

Internal deformation mechanism

The video of the failure appears to show the dam crest settling a fraction of a second before the toe begins to bulge (Robertson *et al* 2019). This is an unusual way for a slope to fail, since failure normally initiates at the toe of a slope. Since the tailings is relatively incompressible, it is not likely that the slope can collapse internally in such a manner as to allow the crest to move downward without the toe moving outward, unless a void is created into which the tailings can move.

It is of interest to note that significant loss of solids was reported to have occurred during and after DHP-15 was partially installed. This loss of solids would have been certain to have led to stress relaxation and may have left a cavity fully or partially filled with water and/or low-density tailings in the interior of the dam. In turn, the collapse of tailings into the cavities may have provided a gap into which tailings from above could have collapsed before the toe moved outward. The above considerations, however, raise the question as to why the facility had not failed when DHP-15 was drilled some seven months before the failure occurred. The only realistic explanation would be that relaxation was delayed and developed gradually after installation of the DHP drains with related release of solids.

Given the uncertainty it is useful to consider the sequence of events that would most probably have followed installation of DHP-15. Evidently, the pore pressures behind the starter wall and raises had increased over a large area at the time of installation of DHP-15. Thereafter, it is reasonable to assume that excess pressures would have dissipated over time. This would have returned the slope to the same state of equilibrium as had existed before drilling of DHP-15. The worst-case factor of safety for overall stability would exist when the water filling the cavity (formed by loss of solids) had been fully dissipated. This condition could thus have developed between the time of drilling of DHP-15 and when the failure occurred.

To investigate the hypothesis that failure would take time to develop, two cases and two scenarios were tested in a finite difference analysis. The first case involved the assumption of 1 m and 2 m diameter cavities that remained filled with water. In both instances the state of stress was found to alter but not sufficiently to cause propagation of a failure. In the second case the finite element analysis was run for the 1 m and 2 m diameter cavities assuming that the water had drained away. The stress release for the 1 m diameter cavity increased as the water drained away but did not lead to ultimate failure, while the stress release and deformations for the 2 m cavity led to failure.

Staged construction in 15 steps was incorporated into the simulation. The states of stress within the model were "tracked" at selected points. This allowed for development of the stress paths for each of the selected points during the staged construction, as well as after the void had been created. This enabled an assessment of whether the stress paths were realistic.

At the end of the staged construction, the void was introduced with a water pressure applied to the void surface. The position of the void is shown in Figure 7 where the stress concentration is initiated (the red zone). The model converged here, indicating that it was stable under these conditions. The water pressure applied to the void surface was then removed and zones with a high strain rate were switched to an undrained state. This resulted in the global failure for the 2 m diameter cavity shown in Figure 7.

Figure 8 provides the stress paths for development of the facility for selected points with the dots indicating the stress state when the void is introduced with water pressure still applied, and the colour

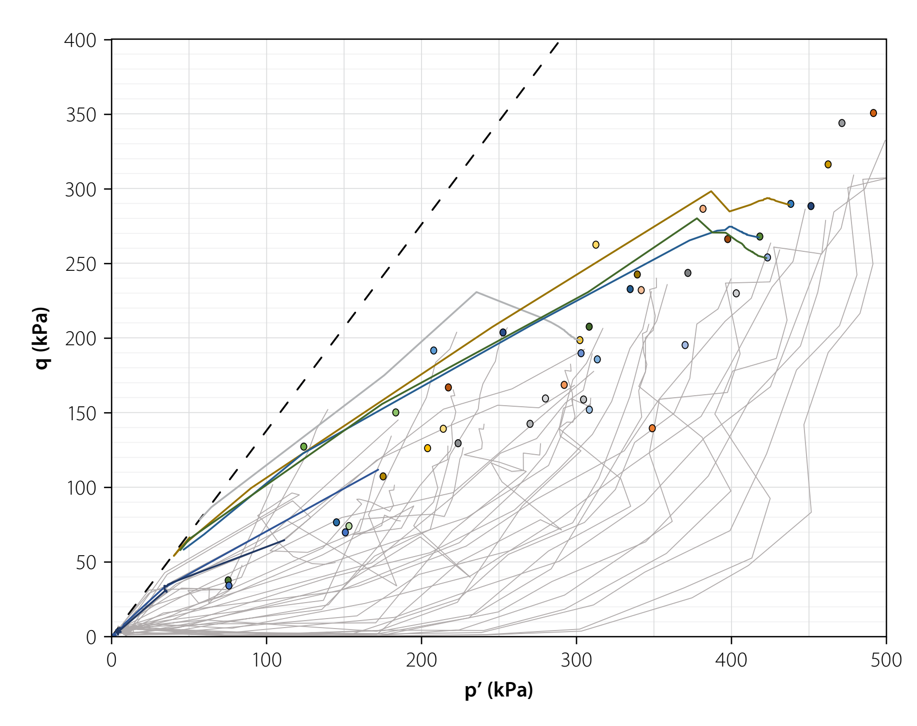

Figure 8 Stress paths in p':q projection

paths show the stress development of a few of the queried points which fall on the shear zone of the failure once the water pressure is removed. All experience a decrease in mean effective stress and loss of strength, and the paths all move towards the CSL (black dashed line). The plots in the e:p' projection confirm undrained behaviour in the zones as there is no change in void ratio and confirm that realistic results have been obtained from the analysis.

While it is acknowledged that the two-dimensional finite difference analysis does not accurately represent the actual three-dimensional condition, it does demonstrate the consequence of stress relaxation in the zone below the toe of the upper slope.

DISCUSSION

One may ask why this is an important illustration of how the failure may have been triggered. The author's view is that since the Brumadinho failure occurred, tailings engineers have discarded much of the experience of many years and have assumed that undrained failures can occur unexpectedly and without warning. This conclusion, if drawn from the Brumadinho experience alone, is not justified and can lead to engineers being distracted from defining more complete credible modes of failure. The situation is also being exacerbated by the implication that trigger mechanisms must not be considered when evaluating undrained failures. On the contrary, the paper illustrates how important it is to identify, understand and mitigate the potential triggers that might present the greatest risks, to focus monitoring on those mechanisms that may trigger a failure.

Conventional wisdom would suggest that the phreatic surface in a tailings facility would drop away after the pond is removed at decommissioning even in wet environments. This would suggest that the facility should become safer with time when viewed from both a drained and undrained perspective. The phreatic surface level measured in the piezometers and the dissipation tests from the piezocones prior to the Brumadinho failure was relatively low and suggested that the drains combined with natural drainage were able to draw down the phreatic surface over time after decommissioning. The facility should therefore not have failed.

The emergence of drilling fluid a substantial distance from, and well above the DHP-15 drill hole, and well above the phreatic surface level, demonstrated that the introduction of drilling fluid could change pore water pressures substantially and rapidly. However, the observation that some drains needed to be unblocked to drop water pressure meant that the drain outlets did not have the capacity to evacuate much more water than was being produced by the natural recharge to the drains. It can therefore be postulated that drilling of the near horizontal drainage holes and DHP-15 (in particular) raised the phreatic surface and probably caused one or more cavities to form when tailings was released. Over time the excess pressure dissipated, thus reducing support for the slope above which began to deform under drained conditions until failure occurred. It is not known how much time would have been required for the excess water to drain away and hence it may be argued that the model presented in this paper does not model the failure accurately. It should, however, be noted that the formation of cavities and the eventual development of collapse in tailings facilities have been shown by experience with other incidents to take time, much like the timing of collapse of sinkholes in karstic terrain takes time and cannot be predicted precisely.

CONCLUSION

The Expert Panel concluded that Brumadinho failed by liquefaction, and this was confirmed by the Prosecutor's team. The assertion that the eventual result was liquefaction is not disputed. This paper, however, shows that the trigger could have been a drained mechanism that removed support for the tailings, and that this led to undrained deformation and ultimately to liquefaction of the tailings. This distinction is of great importance since it calls into question whether static liquefaction can occur without a clear and unambiguous triggering event.

This conclusion further supports the view that the call to eliminate or ban upstreaming in regions with low seismicity, based on the assumption that the Brumadinho failure proves that undrained failures can be triggered for unpredictable reasons, is unjustified. Provided that the tried and tested rules for upstreaming as described by McRoberts *et al* (2017) are applied, there is no reason why future upstream raised facilities cannot be designed and constructed to be as safe as centreline and downstream facilities. The continued use of existing upstream raised facilities can thus be motivated, provided there is strong engineering backing and comprehensive assessment of the probability of the development of trigger mechanisms that could induce undrained behaviour, and if monitoring is put in place to provide early warning of the development of the credible triggers.

The Brumadinho case, however, clearly and powerfully illustrates the importance of ensuring that designs are robust and resilient. The Brumadinho section had a low factor of safety for undrained behaviour, and as such was vulnerable to relatively minor changes in state of stress. The design was therefore not robust. This is the key learning that should be appreciated by practitioners where undrained factors of safety are below the accepted norms.

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