Cite this article
Shuttle D, Marinelli F, Brasile S, Jefferies M and Ong YH (2022)
Discussion: Validation of computational liquefaction for tailings: Tar Island slump.
Geotechnical Research **9(1)**: 56–60,
https://doi.org/10.1680/jgere.22.00003

Discussion Contribution
Paper 2200003
Received 07/01/2022; Accepted 11/01/2022
Published online 08/02/2022

Geotechnical Research

ice Publishing

Discussion: Validation of computational liquefaction for tailings: Tar Island slump

Dawn Shuttle PhD, MICE, CEng, PEng
Consultant, BGC Engineering, Vancouver, BC, Canada (corresponding author: dawn_shuttle@hotmail.com)

Ferdinando Marinelli PhD
Assistant Professor, Department of Civil, Architectural and Environmental Engineering, University of Naples Federico II, Naples, Italy

Sandro Brasile PhD
Senior Manager, Geotechnical Research, Plaxis BV, Delft, the Netherlands

Michael Jefferies PEng
Consultant, BGC Engineering, Vancouver, BC, Canada

Yin Hoe Ong BEng, MEng, PhD, PrEng
Director, Dr Toh Associates SDN BHD, Kuala Lumpur, Malaysia;
Geotechnical Consultant, Kuala Lumpur, Malaysia

Contribution by Yin Hoe Ong

Shuttle *et al.* (2021a) presented an interesting computational study of static liquefaction, but their results raise several questions where further clarification would be useful. The issues relate to both how liquefaction may be modelled/analysed and to aspects of soil behaviour revealed.

In terms of the case-history itself, was there direct evidence that the beach below water (BBW) (or beach above water (BAW)) switched from drained to undrained response? The presented simulations applied an undrained condition, something that would usually be associated with rapid loading; whereas in this case-history the loading rate appears sensibly constant. More generally, looking to other situations, could the authors elaborate as to how the undrained condition develops in sand tailings? Can rising of the water table in the dam (such as the case the authors mentioned of Aberfan in Wales) cause static liquefaction? Is static liquefaction a failure mode that only happens during the construction or filling of a dam? Most importantly, if a dam has been stable for months and years, can static liquefaction still happen?

Looking to the details of the modelling, what was the basis for the geostatic stress conditions of $K_0 \sim 0.6$ for the tailings? And could the authors elaborate on their elastic model, which they suggest was a key to accurately simulating how the slump developed? Is it a standard procedure to model static liquefaction by switching drainage condition from drained to undrained to simulate static liquefaction? Or was a fully coupled stress-seepage analysis used to simulate the perturbation, and which indicated the BBW was in undrained condition due to the rate of loading or blockage? The contributor is not able to link the process in the soil to 'switching from drained to undrained' because rate of loading at Step 5b did not seem to be exceptionally high compared to other lifts in Figure 7 and it seems that the authors wanted the dam to fail at Step 5b rather than simulating the actual process in the soils.

Finally, considering application of the methodology used in the paper to other dams: can static liquefaction analysis be carried out without CPT tests, perhaps using the SPT instead? Are the CPT calibrations in Figures 21 and 22 site specific or universal? What is the minimum type and number of tests to be carried to enable one to analyse static liquefaction using Plaxis?

Authors' reply

Introduction

Dr Ong requests further details about the authors' idealisation of the Tar Island slump (Shuttle *et al*., 2021a) as well as about the Plaxis model of the slump. Dr Ong then asks about how the authors' results might be carried forward to other dams and has questions about the physics of liquefaction. These questions about 'physics' bear on the modelling details, so the authors address these physics issues before discussing how these aspects were captured in the Plaxis model. Some of the points raised by Dr Ong were covered in a webinar on the Tar Island slump (based on the paper), and a video of that webinar was uploaded to YouTube by the Calgary Geotechnical Society (CyGS, 2021). This webinar may be of interest as a companion to this reply.

Characterisation of the slump

There were no piezometers installed in the BBW or BAW; thus, there were no direct measurements showing drained and/or undrained behaviour. The slump analysed in the paper was one of four such events during the period 1972–1974 as the construction method was developed (Plewes *et al*., 1989), with engineers of the day considering each event to be liquefaction. Their reasoning for this view appears to be the rapid and large settlements in each instance, a situation that cannot develop with drained conditions even for loose sand. Further, the remarkable extent of the movements upstream cannot be reconciled with drained behaviour. The authors concur with these views from the era, but the authors' analysis could be viewed as a hypothesis that is then found to match reality in quite remarkable detail – and, most importantly, using data at 'face value' with no contrived 'operating strengths'. Going further, there is considerable similarity between the video of computed liquefaction at Tar Island (in the online supplementary material accompanying the paper under discussion) with the 'security cam' record at Brumadinho, and Brumadinho did have piezometers that showed drained conditions until liquefaction was triggered (Robertson *et al*., 2019). Thus, whether building on the judgement of the engineers of the day or treating the authors' work as a hypothesis, the conclusion is the same: liquefaction from a prior drained state is consistent with all that is known at Tar Island.

Of course stress analysis starts from a prior stress state. Although the evolution of stress can be modelled throughout the construction

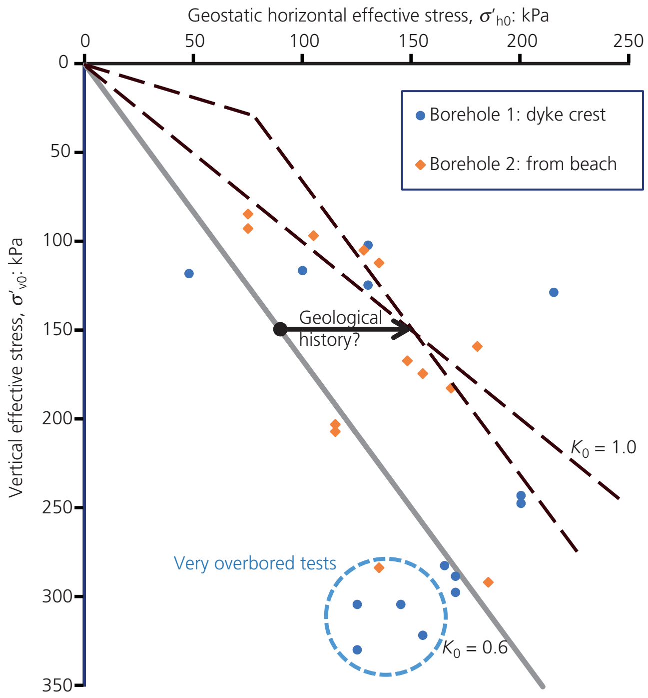

Figure 35. Geostatic stress in oil sand tailings at Muskeg River mine (Shuttle *et al.*, 2021b)

history, the present ability to do so misses details such as the effect of machinery movement on the fill, freeze–thaw cycles, drying and ageing – in essence, the true geologic history of the tailings and dyke fill, hence the preference for using at least some in situ stress measurements to condition the stress state immediately prior to the raise construction. There were no such in situ stress measurements at Tar Island, but a similar, adjacent tailings facility (Muskeg River mine) did include considerable self-bored pressuremeter (SBP) testing. These SBP tests have been analysed using 'iterative forward modelling' to develop the in situ geostatic conditions, the results of which are shown in Figure 35. The authors' assignment of K_0 for the Tar Island analysis was largely based on this SBP analysis supplemented by other experiences on geostatic stress in hydraulic fills (see Jefferies and Been, 2006).

Turning the question of the in situ state parameter, Figures 21 and 22 are calibrated to the properties of the Alberta tar sands. The methodology is universal, but differing M, λ and so on produce different calibrations (see Ghafghazi and Shuttle, 2008). Originally, the methodology relied on parametric simulations, but current computing power makes calculating soil-specific cone penetration test (CPT) inversion coefficients a near-trivial task (see the paper by Shuttle and Jefferies (2016); the software is public domain). More generally, this methodology is conditioned on CPT calibration chamber data – and those data are nearly entirely on dilating soils ($\psi < 0$), leading to possible uncertainty when applied to loose soils. However, the methodology was validated for loose tailings of state comparable with Tar Island at Cadia (see figure E4-5 in the report by Morgenstern *et al.* (2019)), and the methodology appears robust.

The elastic shear modulus was determined using the 'seismic' CPT with inbuilt geophones, which arguably are the industry standard, leading to Figure 14, where the data are grouped by depositional situation. Inspection of this figure shows the densest zone (compacted dyke fill) had about double the G_{max} of the loosest zone (the BBW). This effect of density on soil stiffness is widely known, and Figure 14 is uncontroversial. Where there is far less acceptance is how such dependence on the void ratio (or density) should be represented. Pestana and Whittle (1995) considered various elastic idealisations for soil, identifying those that were acceptable from those that were physically incorrect. The authors' preference is an idealisation that is consistent with their critique but goes one step further by requiring that the soil must not compress elastically to a void ratio less than its minimum value. This additional physical constraint is achieved by Equation 4, where $G_{max} \Rightarrow \infty$ as $e \Rightarrow e_{min}$.

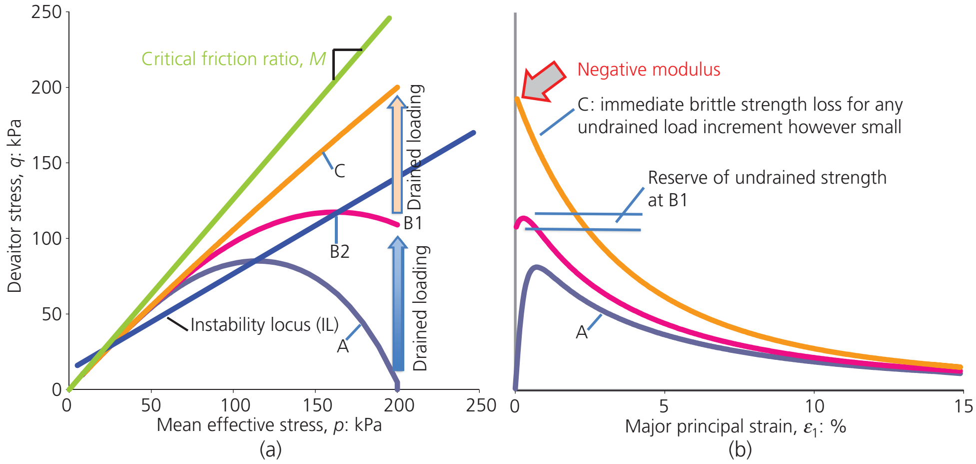

Figure 36. NorSand simulations illustrating the development of drained instability

As compressibility is the inverse of modulus, this can equally be viewed as compressibility tending to zero as the soil void ratio tends to its minimum. Strictly, the elastic modulus should tend to that of sandstone or siltstone (as appropriate) rather than becoming infinite, and e_{min} may also depend on the stress level.

Liquefaction from drained conditions

The drained-to-undrained transition is perhaps best understood by considering the response of a single element of the soil. The starting point is that 'drained' does not correspond to 'no excess pore pressure'. Rather, any loading must be broken into an instantaneous undrained step, repeated at periodic intervals, followed by time for dissipation. If the dissipation time is less than the rate at which the undrained steps occur, then true drained conditions exist. If not, then the behaviour mode switches to undrained. The kernel issue in this behaviour-mode change is the rate at which excess pore pressure develops.

Figure 36 shows three stress–strain curves for the same soil, computed using the NorSand (NS) model, for a simplified loading path approximating the path in the BBW below the toe of the raise at Tar Island. The behaviour annotated as 'A' is a standard undrained test of loose soil, with the familiar associated static liquefaction once the undrained strength is reached. In 'B', soil with the same ψ is first loaded drained to B1 before then further loading undrained, but this change to undrained conditions is not dramatic, as the stress at the change to undrained loading is less that its undrained strength (B2). The soil can accept the excess pore pressure and allow drainage to occur. The further step is shown as 'C', with drained loading at the same ψ continuing beyond the undrained strength at B2 before switching to undrained conditions. As can be seen in the figure, instant brittle strength loss develops despite there being zero excess pore pressure at that instant: the soil has a negative modulus to a rapid, small perturbation. This perturbation could be a quickly placed berm raise, operation of machinery (e.g. a bulldozer) or an earthquake. What happens next depends on the ability of the adjacent soil to accept load from the part experiencing negative modulus, as that overload must be accepted to allow time for dissipation. This redistribution of load will propagate as a plastic wave if the adjacent soil then also becomes overloaded in turn. This propagation of brittle strength loss at Tar Island can be seen in the video provided as online supplementary material. In particular, notice in that video how liquefaction develops in a small zone with the switch to an overall undrained slump being the consequence of rapidly propagating strength loss. The 'undrained' situation is not something imposed by large-scale rapid loading but instead develops as a consequence of brittle strength loss that propagates at about one quarter the elastic shear wave velocity.

In Plaxis this last berm raising was simulated in a so-called stage construction style, with the additional weight progressively applied through a control parameter going from 0 to 1, allowing, albeit in a quasi-static fashion, gradual plasticity propagation and stress redistribution. As undrained softening was experienced, loads were initially redistributed to adjacent elements, but when

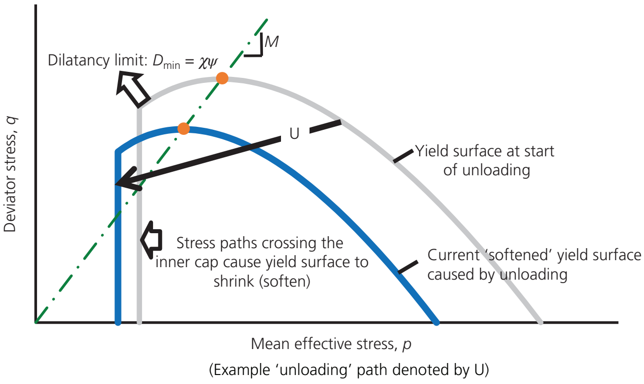

Figure 37. Softening of NS yield surface during unloading

the current load level became unsustainable (and further evolution of deformations could not be linked to an equilibrated stress state), the Plaxis 'arc-length control' mode was adopted, allowing the analysis to 'follow' the propagation of liquefaction through adaptation of the load to a sustainable level. This mode automatically reduces (or increases) the step size to account for severe (or mild) non-linearities associated with non-linear behaviour and ensures that equilibrium is satisfied globally while incipient/localised plasticity is captured locally, thereby enabling the numerical model to follow the Tar Island liquefaction slide.

Dr Ong raises two further and related points regarding the drained–undrained transition. In the case of the question as to whether liquefaction can 'suddenly' happen if the dam has been stable for months and years, the mathematics suggests not. Referring back to Figure 36, there must be a 'rapid perturbation'; absent such perturbation, the dam will remain stable. Of course, engineering is more than mathematics, and the real question is, what sort of perturbations can cause liquefaction? Earthquakes and similar vibrations, as well as rapidly applied additional load or excavation at the toe, are well-known examples. But, what else? This leads to Dr Ong's point regarding Aberfan.

NS uses a limiting dilatancy that is a linear function of ψ. This limiting dilatancy (or, equivalently, strength through stress dilatancy) is widely found in data, but it is more than a useful correlation, as it follows from theory (Jefferies, 2021). A corollary is that the yield surface must have an internal cap for self-consistency with the limiting dilatancy (Jefferies, 1997) (see Figure 37). Stress paths crossing this internal cap, which specifically include those associated with increasing seepage pressures, cause the yield surface to shrink. While the stress path lies on the internal cap, plastic strains are small. However, once the yield surface has shrunk to the extent that the imposed path now lies at the limiting dilatancy, large plastic strains occur. If $\psi > 0$, these plastic strains will cause positive excess pore pressures and large distortional strains – static liquefaction, again from drained conditions. Thus, Aberfan. This description is based on NS, but any constitutive model anchored to the state parameter will show comparable response, thus the realistic simulation of Brumadinho presented by Arroyo and Gens (2021).

Plaxis model

The authors did not carry out a coupled seepage–stress analysis. Rather, the authors' preference is to mimic Figure 36 and apply an undrained load. If that load does not cause failure, then the authors step back and reapply that same load but now drained.

An alternative procedure would be to apply all load steps drained and to test the ratio η/η_{IL} (where η_{IL} is stress ratio at which a negative modulus arises, commonly called the instability locus, which varies with ψ) throughout the domain of interest. However, from an analytical viewpoint, this is an ill-conditioned criterion. Referring to Figure 32, the stress path of point B approaches its instability locus obliquely – minor changes in properties can cause much larger changes in when triggering is predicted, hence the preference for a direct numerical test.

In the case of Tar Island, all but the last lift had been successfully constructed and this was consistent with the stress path lying below the instability locus at point B. There was also a recorded increase in the average construction rate for the final lift (see Figure 7) to provide a 'trigger', although the bulldozer operating on the fill might also have been the rapid perturbation. Thus, triggering was not investigated further.

As to whether either of these two modelling procedures, or indeed fully coupled analyses, is standard, the current answer must be this: there are no standard procedures as yet. The prior work in connection with Fundão (Morgenstern *et al*., 2016), Cadia (Morgenstern *et al*., 2019) and Brumadinho (Robertson *et al*., 2019) is consistent with the η/η_{IL} approach. Equally, the alternative view of Brumadinho (Arroyo and Gens, 2021) is consistent with the authors' analysis of Tar Island. In part, which method is used depends somewhat on the software: tracking load redistribution within the finite-element domain as load continues to be shed as the failure develops is a severe test of numerical methods. A 'standard method' will emerge, but the authors guess that the profession is at least 5 years away from that. Also, the profession needs to see more case histories assessed in the manner presented.

Application to other situations

The origin of critical state theory was work by the US Army Corp of Engineers (USACE), 1935–1940, to avoid static liquefaction of dams (sand flow failure as it was then called). However, the USACE was unable to measure the void ratio in situ with any reliability, and this prevented practical application of the ideas. The 'game changer' was the Canadian oil industry, which was constructing hydraulic fill islands in the Arctic in 1980–1990 and used the ideas of the USACE but then added in careful calibration of the CPT to ψ. It had to be the CPT, as the methodology was based on cavity expansion theory, which is the familiar analogue for quasi-static penetration; see Shuttle and Jefferies (2016). No such analogue exists for the standard penetration test (SPT), nor does the SPT have the body of calibration data that exists for the CPT.

Going further, a central issue with Tar Island was assessing the characteristic state. The CPT readily provides the needed basis to do this, shown in Figure 23. An equivalent of Figure 23 cannot be developed from the SPT because the SPT neither provides the continuous record needed nor has sufficient repeatability to distinguish testing error from true soil variability. Indeed, this can be seen with Brumadinho, where there were many instances of matching SPT boreholes to CPT soundings, yet that SPT data contributed nothing to the understanding of the failure.

However, it is not just a question of CPT against SPT; there are also issues with what type of CPT. When processing CPT data, 'soil behaviour type' is used to assess how the soil properties may vary within the domain of interest, and that requires accurate measurement of sleeve friction. Thus, CPT equipment for liquefaction assessment must have independent friction measurement (i.e. do not use 'subtraction' transducers) with thermal drift compensation and downhole analogue-to-digital conversion; equipment of this nature is widely available. Public domain software (open-source 'freeware') is available for CPT data processing, as well as some inexpensive commercial programs. The authors' preference is for open-source software, as that is easily adapted to the needs of engineering practice.

Lastly, there is the question of supporting laboratory tests. The first step is to examine the CPT data to identify representative strata. In the case of tailings, a single gradation may be sufficient to be representative. Equally, there may be segregation during deposition leading to two to three possibly characteristic gradations. Each identified gradation requires testing. In principle, just three triaxial tests are sufficient for each gradation. However, not all tests will be perfect and require repeating, while it is helpful to have a range of void ratios. Thus, real test programmes tend to be five to eight triaxial tests rather than the conceptual minimum of just three. The Tar Island case history was typical in this regard with four soil gradations tested and with each using eight or fewer triaxial tests. Jefferies and Been (2016) provide detailed guidance on test programmes and procedures.

REFERENCES

Arroyo M and Gens A (2021) *Computational Analyses of Dam I Failure at the Corrego de Feijao Mine in Brumadinho*. Vale S.A. and International Centre for Numerical Methods in Engineering, Barcelona, Spain. See http://www.cimne.com (accessed 26/01/2022).

CyGS (Calgary Geotechnical Society) (2021) Mike Jefferies – analysis of static liquefaction with Plaxis: 1974 Tar Island slump. *YouTube,* 1 December. See https://www.youtube.com/watch?v=xl8RGFYQoZU (accessed 26/01/2022).

Ghafghazi M and Shuttle D (2008) Interpretation of sand state from cone penetration resistance. *Géotechnique* **58(8)**: 623–634, https://doi.org/10.1680/geot.2008.58.8.623.

Jefferies MG (1997) Plastic work and isotropic softening in unloading. *Géotechnique* **47(5)**: 1037–1042, https://doi.org/10.1680/geot.1997.47.5.1037.

Jefferies M (2021) On the fundamental nature of the state parameter. *Géotechnique*, https://doi.org/10.1680/jgeot.20.P.228.

Jefferies M and Been K (2006) *Soil Liquefaction: A Critical State Approach*. Taylor & Francis, London, UK.

Jefferies M and Been K (2016) *Soil Liquefaction – A Critical State Approach*, 2nd edn. CRC Press, Boca Raton, FL, USA.

Morgenstern NR, Vick SG, Viotti CB and Watts BD (2016) *Fundão Tailings Dam Review Panel: Report on the Immediate Causes of the Failure of Fundão Dam.* Cleary Gottlieb Steen & Hamilton LLP, New York, NY, USA. See http://fundaoinvestigation.com/ (accessed 30/08/2021).

Morgenstern NR, Jefferies M, Van Zyl DV and Wates J (2019) *Report on NTSF Embankment Failure, Cadia Valley Operations*. Newcrest, Melbourne, Australia. See http://www.newcrest.com/sustainability/dams-and-tailings-management (accessed 26/01/2022).

Pestana JM and Whittle AJ (1995) Compression model for cohesionless soils. *Géotechnique* **45(4)**: 611–631, https://doi.org/10.1680/geot.1995.45.4.611.

Plewes HD, O'Neill HD, McRoberts EC and Chan WK (1989) Liquefaction considerations for Suncor tailings ponds. In *Proceedings of the Dam Safety Seminar in Edmonton, Alberta*. BiTech Publishing, Vancouver, Canada, vol. 1, pp. 61–89.

Robertson PK, de Melo L, Williams DJ and Wilson GW (2019) *Report of the Expert Panel on the Technical Causes of the Failure of Feijão Dam I.* Skadden, Arps, Slate, Meagher & Flom LLP, New York, NY, USA. See http://www.b1technicalinvestigation.com/ (accessed 26/01/2022).

Shuttle DA and Jefferies MG (2016) Determining silt state from CPTu. *Geotechnical Research* **3(3)**: 90–118, https://doi.org/10.1680/jgere.16.00008.

Shuttle D, Marinelli F, Brasile S and Jefferies M (2021a) Validation of computational liquefaction for tailings: Tar Island slump. *Geotechnical Research*, https://doi.org/10.1680/jgere.21.00007.

Shuttle D, Martens S and Jefferies M (2021b) Geostatic stress in oil-sand tailings. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering*, https://doi.org/10.1680/jgeen.21.00114.

How can you contribute?

To discuss this paper, please submit up to 500 words to the editor at journals@ice.org.uk. Your contribution will be forwarded to the author(s) for a reply and, if considered appropriate by the editorial board, it will be published as a discussion in a future issue of the journal.