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The use of vibrating wire piezometers to measure matrix suction in dams

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SYNOPSIS A knowledge of pore water pressures in embankment dams and in mining dams is essential to monitor performance. In many instances, this knowledge forms part of a critical risk control to prevent a high consequence event, such as global instability and release of containment. Yet the field measurement of pore water pressures can be difficult. This is particularly the case when unsaturated conditions prevail for long periods. Vibrating wire piezometers are used in many instances to monitor negative pore water pressures in dams, both in the foundation and in the fill; yet these instruments, which can measure small sub-atmospheric pressures, have not been designed to operate in an environment of sub-atmospheric pressures indefinitely. This paper touches on two topics that are of interest to the dam engineer: (i) the effect of degree of saturation and matrix suction on liquefaction potential, and (ii) the measurement of matrix suction in the field using vibrating wire piezometers.

INTRODUCTION

A knowledge of the amount of pore fluid (soil-water wetness) or the pressure within the pore fluid (soil-water potential) in a soil-fluid system is needed to predict the performance of a geotechnical structure. This is particularly the case for mining dams located in seismic areas, where a sufficiently low volume of pore fluid, or a sufficiently low pressure within the pore fluid, is used to manage the risk of liquefaction during an earthquake. Although this statement also applies to embankment dams founded on potentially liquefiable deposits, it is most relevant in the case of mining dams that often rely on the strength of the deposited tailings for stability. In many instances, the tailings, which consist of fine sand and silt grains derived from the grinding of ore, will have been deposited as a slurry. Densities may be low, and both the soil-water wetness and the soil-water potential within the tailing mass will govern their potential for liquefaction.

FIELD MEASUREMENT OF SOIL-WATER WETNESS AND SOIL-WATER POTENTIAL

The amount of pore fluid in a soil can be expressed as a gravimetric water content (ratio of water mass to dry soil mass), volumetric water content (ratio of water volume to total soil volume), or degree of saturation (percentage of void space that contains water). Gravimetric water content can be measured from soil samples by drying the material. This, together with a knowledge of bulk density and specific gravity of the soil grains, allows determination of the in situ volumetric water content and degree of saturation.

An estimate of volumetric water content can be obtained the field by measuring the electromagnetic properties of the soil-fluid system, using techniques such as time domain reflectometry (TDR) or nuclear magnetic resonance (NMR). However, conversion of volumetric water content to degree of saturation requires a knowledge of gravimetric water content and specific gravity of the soil grains, which can only be obtained from soil samples. This means that although it is possible to determine the degree of saturation of material in a dam or foundation at a particular instance, it is not possible to monitor its variation with time. In addition, the electromagnetic properties of the soil-fluid system are influenced by factors other than water content, which adds to the difficulty in interpreting the data.

The soil-water potential of soil in the field can be measured with instruments that make direct contact with the pore fluid. Positive pore fluid pressures (pressures above atmospheric pressure) can be measured directly with one of several types of piezometers available, including hydraulic piezometers, vibrating wire piezometers, and electrical resistance piezometers. Negative pore fluid pressures (pressures below atmospheric pressure) can be measured directly with a tensiometer. It is also possible to measure negative pressures indirectly using, for example, electrical conductor sensors or thermal conductor sensors. Indirect methods measure a property related to the negative fluid pressure and require calibration of the sensor.

A NOTE ON NEGATIVE SOIL-WATER POTENTIAL

Soil-water potential, or the potential energy per unit mass in the soil, includes several components, of which gravitational potential and pressure potential are the most relevant for engineering practice. Potential can be expressed in three equivalent ways: energy per unit mass, energy per unit weight (hydraulic head), and energy per unit volume. It is customary to report pressure potential in terms of hydraulic head (units of length) or energy per unit volume (units of pressure).

Gravitational potential is given by the elevation of a point relative to an arbitrary reference level. Pressure potential is measured in relation to atmospheric pressure. Soil-water at a hydrostatic pressure greater than atmospheric pressure is defined as having a positive pressure potential; when the soil-water pressure is below atmospheric pressure, the pressure potential is taken as negative and is referred to as matrix suction (reported as a positive quantity).

Matrix suction results from both capillary and adsorptive forces between the soil water and the soil matrix. This quantity captures the total effect resulting from the affinity of water to the matrix of the soil, including its pores and particle surfaces, which bind water in the soil and lower its potential energy below that of bulk water. Formally, matrix suction is defined as the negative gauge pressure, relative to the external gas pressure on soil water, to which a solution identical in composition with the soil solution must be subjected to be in equilibrium through a porous membrane wall with the water in the soil. In practice, matrix suction (s) is calculated by taking the difference between the pore-air pressure (u_a) and the pore-water pressure (u_w); i.e. $s = u_a - u_w$.

EFFECT OF DEGREE OF SATURATION AND MATRIX SUCTION ON LIQUEFACTION OF SOILS

There is now an extensive body of literature that considers the liquefaction resistance of unsaturated coarse-grained soils, such as sands and silty sands. This work, conducted in the laboratory, has focused primarily on assessing the effect of changes in degree of saturation on resistance to cyclic stress-induced liquefaction. In addition to degree of saturation, some recent studies have also reported matrix suction prior to and during a cyclic test.

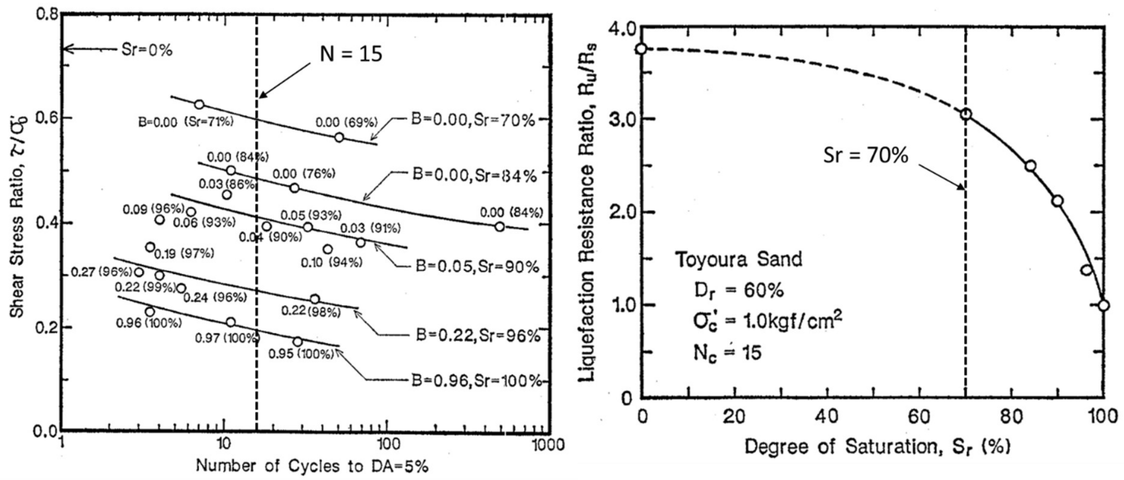
Cyclic-induced liquefaction, which can result from ground shaking during an earthquake, is caused by the densification of loose material during cyclic stress changes and principal stress rotation, which can result in an increase in pore water pressures. For dense soils, cyclic stress changes will result in a reduction in stiffness and potentially in deformations during loading (cyclic mobility). For loose soils, cyclic loading can result in an undrained strength reduction and brittle failure (liquefaction). Cyclic-induced liquefaction is one of two types of liquefaction phenomena, the other being static liquefaction. The latter results from a large undrained strength reduction due to an increase in pore water pressure during monotonic stress change (loading or unloading). Static liquefaction is associated with brittle failure.

Cyclic liquefaction in the laboratory is normally determined by measuring the number of uniform cycles required to reach a particular failure criterion, such as (i) 5% double amplitude (DA) strain, or (ii) excess pore water pressure equalizing the initial effective confining stress. During a test, different levels of uniform cyclic stress are applied to the sample, and the data is presented in the form of cyclic stress ratio (CSR) against number of cycles to reach failure (N). The CSR is the cyclic shear stress normalized by the initial normal stress.

Figure 1(a) shows a plot of CSR (labelled *Shear Stress Ratio* τ/σ_o') against N (labelled *Number of Cycles to DA = 5%*) obtained by testing Toyoura sand in a hollow cylindrical torsional shear (Yoshimi et al 1989). Toyoura sand is a research material widely used in Japan with the following characteristics: d_{50} = 0.175 mm, d_{10} = 0.129 mm, coefficient of uniformity (C_u) = 1.52, and fines content (FC) = 0%. Liquefaction during a cyclic test was defined as the number of cycles required to yield a double amplitude shear strain of 5%. The figure includes B-values[1] measured during initial consolidation together with degree of saturation prior to the application of the cyclic load (labeled as *B* and S_r, respectively, in the figure). As the initial degree of saturation decreases from 100% to 70%, the cyclic resistance of Toyoura sand increases markedly.

Figure 1(b) shows the variation in the shear stress ratio required to cause a double amplitude shear strain of 5% after 15 uniform cycles (corresponding to an earthquake of magnitude 7.5). The figure also shows the static shear strength of dry sand, defined as the shear stress at a shear strain of 2.5 %. The greatest increase in cyclic resistance in Toyoura sand takes place as the degree of saturation reduces from 100% to 70%. The ordinate in Figure 1(b), labelled *Liquefaction Resistance Ratio,* R_u/R_s, corresponds to the cyclic resistance at a particular degree of saturation (R_u) normalized by the cyclic resistance of saturated material (R_s).

[1] Ratio of the increase in pore water pressure to the increase in cell pressure.

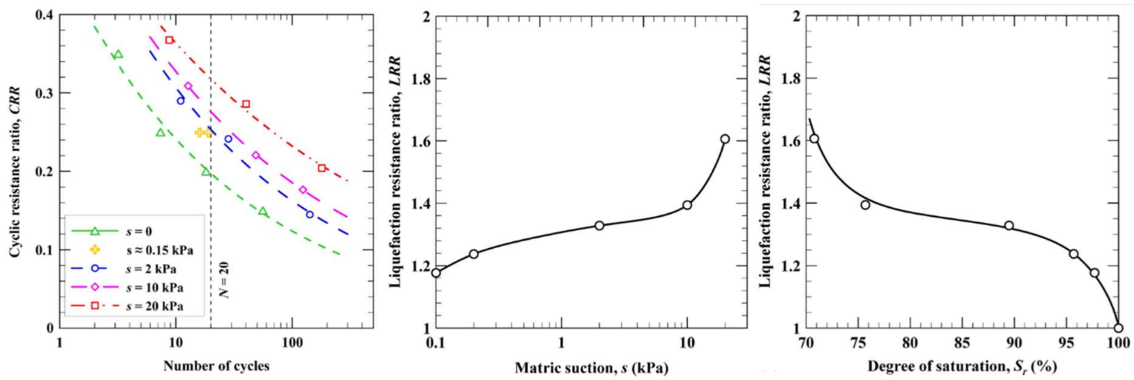


(a) (b)

Figure 1. Results from hollow cylindrical torsional shear tests on Toyoura sand (modified from Yoshimi et al 1989)

Figure 2(a) shows the results from cyclic loading tests carried out on a silty sand (50% FC) using a triaxial cell capable of monitoring matrix suction during a test (Banerjee et al 2022), with suction being controlled and measured using the axis translation technique. The target relative density of the soil at the start of the test was 50%. Samples were first saturated and thereafter dried to the desired initial matrix suction, which ranged from 0 kPa to 30 kPa (corresponding to degrees of saturation of between 100% to 70%). During the undrained tests, pore-air pressures and pore-water pressures were measured independently to record changes in matrix suction. Tests were stopped after the double amplitude axial strain had reached 5%, or after the number of cycles had exceeded 300. The figure plots the variation in CSR (labelled *Cyclic Resistance Ratio, CRR*) with N (labelled *Number of cycles*) for different initial values of suction. An increase in matrix suction results in an enhanced cyclic resistance of the silty sand.

Figure 2(b) and Figure 2(c) show the variation in cyclic resistance at 20 uniform cycles with changes in matrix suction and degree of saturation, respectively. The ordinate in both figures is given in terms of a liquefaction resistance ratio (LRR), corresponding to the cyclic resistance of the unsaturated material normalized by the cyclic resistance of the saturated material. For the silty sand tested, a large increase in cyclic resistance occurs as degrees of saturation reduce from 100% to 90%, corresponds to an increase in matrix suction from 0 kPa to around 2 kPa. The data indicates that a reduction in degree of saturation below 75%, associated with an increase in matrix suction above 10%, is accompanied by a marked increase in cyclic resistance.



(a) (b) (c)

Figure 2. Results from triaxial tests on silty sand (modified from Banerjee et al 2022)

AN EXAMPLE OF THE NEED TO MONITOR MATRIX SUCTION IN THE FIELD

Both Figure 1 and Figure 2 presented in the previous section show that cyclic resistance to liquefaction is very sensitive to changes in degree or saturation and matrix suction. A reduction in the initial degree of saturation in Toyoura sand from 100% to 90% translated to a doubling of the cyclic resistance when N was 15 (Figure 1). A similar reduction in degree of saturation in the silty sand tested with the suction controlled triaxial cell, associated with an increase in matrix suction from 0 kPa to 2 kPa, was accompanied by an approximately 40% increase in cyclic resistance when N was 20 (Figure 2). Although a reduction in degree of saturation (and an increase in suction) results in an enhanced response of a soil during cyclic loading, the test results presented in Figure 1 and Figure 2 also show that unsaturated material with a high degree of saturation can experience cyclic mobility, and possibly cyclic liquefaction if the initial state is loose enough; i.e., it cannot be assumed that if the degree of saturation falls below 100% the risks of cyclic mobility and liquefaction disappear. This presents a challenge to the engineer, as explained below.

Figure 3 shows a confining dam part of a tailings facility now under active care. There has been no deposition of tailings in the facility for the past 40 years and work is progressing towards final closure. The dam was constructed in the upstream direction using the coarse fraction of the tailings to create an outer shell and then tailings were deposited in the impoundment hydraulically as a slurry.

Interpretation of cone penetration test (CPT) soundings indicates that most of the tailings in the dam are dry or have low degrees of saturation: dynamic and equilibrium pore pressures are negligible. This material is labelled as 'Dry tailings' in Figure 3. The CPT data also indicates the presence of layers of fine tailings near the base of the dam where dynamic pore pressures are high. The material in these layers is interpreted to have a high degree of saturation and the layers are labelled as 'Wet tailings' in Figure 3. In addition to being wet, the normalised tip resistance corrected to an equivalent clean sand value ($Q_{tn.cs}$), proposed by Robertson and Wride (1998), obtained in this wet material is below 70. $Q_{tn.cs} \leq 70$ is the criterion given by Robertson (2010, 2016, 2022) to determine, at a screening level, if a soil is susceptible to undrained brittle response and to liquefy. Therefore, the layers of wet tailings depicted in Figure 3 are assumed to have the potential to liquefy during an earthquake.

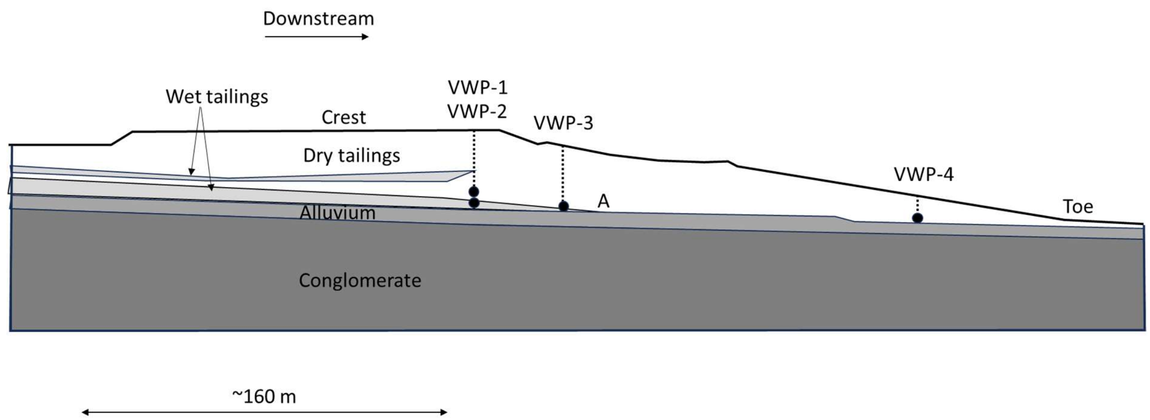


Figure 3. Example of a mining dam with layers of potentially liquefiable tailings, labelled as "wet tailings".

The basal layer of wet tailings with the potential to liquefy has been interpreted to extend from the impoundment to Point A (Figure 3). Beyond Point A, only dry tailings are thought to be present in the dam. Although the dry tailings beyond Point A are in a loose state ($Q_{tn.cs}$ value are still below 70), given the low degrees of saturation, the material is thought not to be susceptible to experience liquefaction during an earthquake.

A two-dimensional limit equilibrium stability analysis assuming liquefaction of the basal layer of wet tailings that extends to point A gives a high factor of safety (FoS) for global instability of around 2.6. This corresponds to a post-earthquake loading condition and indicates that the dam would be stable during an earthquake even if the wet material underwent an undrained brittle response, with the strength of the tailings in the basal layer reducing to the undrained residual strength. The high FoS is due to the stabilizing effect of the unsaturated material near the toe of the dam, beyond Point A, which is assumed to retain its strength during an earthquake. The material near the toe of the dam, however, is in a loose state. This means that an increase in degree of saturation at the base of the dam beyond Point A could potentially result in material in this zone becoming susceptible to an undrained brittle response and to liquefy during an earthquake. If the basal layer of wet tailings is extended from Point A to the toe of the dam, the post-earthquake FoS reduces to 0.9. An increase in saturation could occur, for example, due to a rising water table or from the prolonged storage of water in the impoundment.

Currently, conditions at the base of the dam, within the tailings mass, are monitored with four non-vented, non-flushable vibrating wire piezometers (VWP). The locations of piezometers are shown on Figure 3. Instruments have been labelled as VWP-1, VWP-2, VWP-3, and VWP-4 in the figure. VWP-1 and VWP-2 were installed within the same borehole at different depths in July 2017; whereas VWP-3 and VWP-4 were installed in separate locations in August 2019. The piezometers are fitted with low air-entry (LAE) filters and were installed by the fully grouted method.

The response of the four piezometers since installation until November 2021 is shown in Figure 4. The plots indicate the location of the sensor and the total head recorded over time. VWP-1, VWP-2 and VWP-3 have been reporting negative pore water pressures since installation, with maximum suctions of 10 kPa (VWP-1 and VWP-3) and 50 kPa (VWP-2) measured up until November 2021. Readings in VWP-4 have fluctuated within the range of ±2 kPa during the reporting period.

A fully saturated, non-vented VWP that is making direct contact with the pore fluid in the surrounding soil will record barometric pressure fluctuations. This appears to have been the case in the four piezometers after installation, where the initial response shows fluctuations in piezometric readings. The period during which fluctuations are observed, labelled as 'Fluctuation' in Figure 4, ranged from 5 to 14 months, with the longest period corresponding to piezometer VWP-4 (where readings fluctuated between positive and negative values). After this initial period, the variation in piezometric readings with time in all four plots traces a smooth curve, suggesting that the piezometers may have desaturated.

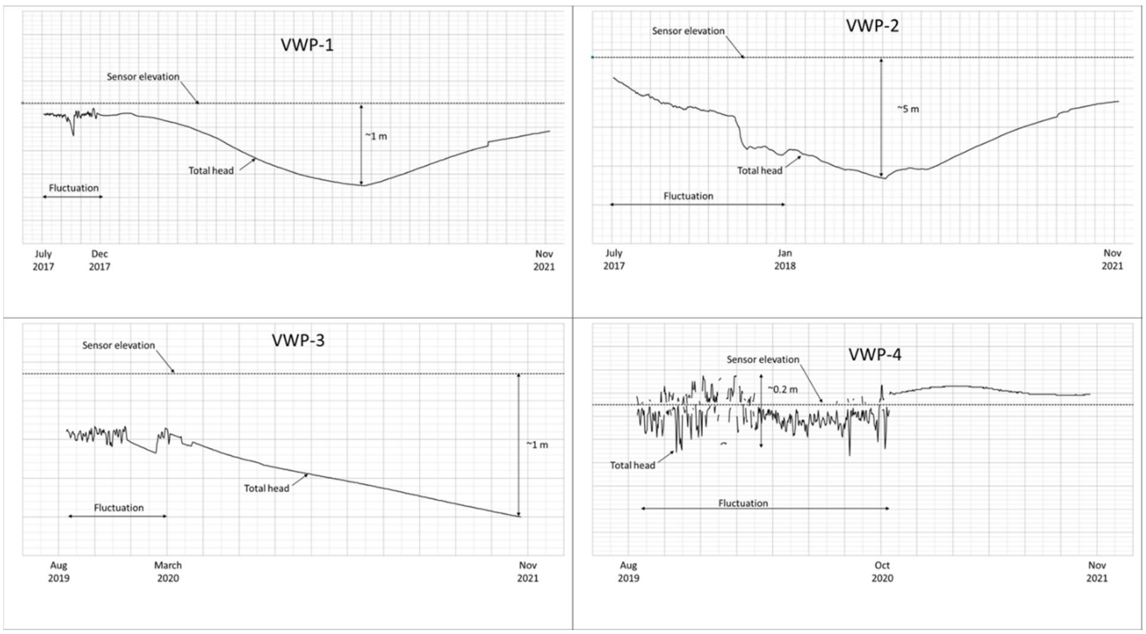


Figure 4. Readings recorded in VWP installed by the fully grouted method

Piezometers VWP-1 to VWP-4 are currently used to monitor conditions at the base of the dam shown in Figure 3. The expectation is that the instruments will respond to changes in pore water pressure, and hence alert of an increase in saturation and an associated risk of liquefaction during an earthquake. These piezometers are part of a critical control for the dam.

Considering the possibility that piezometers VWP-1 to VWP-4 may have desaturated, two questions arise:

- How reliable are long-term measurements of matrix suction derived from non-flushable VWPs installed with the fully grouted method?
- Should non-flushable VWPs installed with the fully grouted method be used in situations where matrix suctions in the surrounding soil can prevail for long periods?

THE USE OF VIBRATING WIRE PIEZOMETERS INSTALLED WITH THE FULLY GROUTED METHOD TO MEASURE MATRIX SUCTION

The generic term for an instrument that measures matrix suction directly is a tensiometer. This consists of a porous filter and a means of measuring stress, which are separated by fluid retained in a reservoir. Tensiometers work in a similar manner to piezometer: they allow water to flow (in the case of a tensiometer, out of the device) until the internal energy of the water filling the tensiometer's reservoir reaches a state of equilibrium with the internal energy of the soil-water. This, however, does not mean that the tensile stresses in the tensiometer and in the soil-water are similar, since both capillary and adsorbed components of potential are present in the latter case.

Any piezometer fitted with a diaphragm, such as a VWP, has the potential to measure matrix suction (i.e. it can be used as a tensiometer); however, the successful measurement of matrix suction requires (i) that the water in the piezometer reservoir is in contact with the water in the soil, and (ii) that the piezometer remains saturated (Ridley 2015). If the first condition is not met, the water in the piezometer reservoir will not be able to reach equilibrium with the soil-water; if the second condition is not met, the accuracy of any suction measurement will be uncertain.

There are four factors that restrict the measuring range of a tensiometer, including VWPs (Ridley 2015): (i) the procedure used to remove air from the tensiometer, (ii) the volume of water in the tensiometer reservoir, (iii) the material used to manufacture the body of the tensiometer, and (iv) the pore size of the porous filter (given by the air-entry value). The first three factors are associated with the formation of vapour cavities as the water in the tensiometer reservoir is subjected to a hydraulic tension. Cavitation (the formation of vapour cavities) can occur within the liquid or at the boundary between the liquid and the tensiometer reservoir wall. The fourth factor has to do with the ingress of air into the tensiometer reservoir when the difference between the tensile stress in the water within the tensiometer reservoir and the atmospheric air pressure outside the tensiometer reservoir reaches the air-entry value of the porous filter. This causes air to be drawn through the filter under the influence of the difference in pressure.

Over the past three decades there has been an increase in the use of VWPs installed by the fully grouted method in geotechnical projects. Contreras et al (2008) discuss the subject and build on work originally carried out by Vaughan (1969). The authors present results from finite element analyses that indicate how errors in the measurement of pore water pressure are only significant when the permeability of the cement-bentonite grout is three orders of magnitude greater than the permeability of the surrounding soil. If the permeability of the grout is lower than the permeability of the surrounding soil, measurement errors will be minimal. The authors also present several examples of the successful use of the fully grouted method for piezometer installation in geotechnical practice. Additional examples are given in Dunnicliff (2008).

Besides simplifying the installation method, the use of cement-bentonite grout as backfill for piezometer installation offers the additional advantage of remaining saturated when in contact with soils that have high matrix suction (something unlikely to happen when a sand pack is used as backfill around a VWP). The use of grout is, therefore, preferable when soil suctions are likely to be encountered in the field. Given that the pore size of the porous filter in a VWP will restrict the measuring range of the instrument, it would seem appropriate to use a VWP fitted with a high air-entry porous filter, together with a fully grouted installation, when soil suctions need to be measured in the field.

Simone and Sorensen (2018) carried out a study that looked at the performance of VWPs fitted with both high air-entry (HAE) and low air-entry (LAE) filters placed in fully grouted boreholes. Seventeen non-flushable VWPs were installed in very low permeability, stiff, overconsolidated clay. VWPs from two different manufacturers were used and the HAE filters were saturated using five different methods. In addition, three different cement-bentonite mixes were employed. An additional test was carried out in the laboratory by sealing a VWP with a HAE filter in a block of cement-bentonite grout and placing the instrument in a 3m high pipe filled with water.

The authors report that within weeks of installation the piezometers with HAE filters started to give erroneous readings. After eight months, only one of the nine piezometers fitted with a HAE filter gave credible readings. The main reason for the poor performance was attributed to unsatisfactory filter saturation, with the cement-bentonite grout being the main problem. This conclusion was based on the observation that similar VWPs with HAE filters were able to measure successfully positive pore water pressures when placed in direct contact with the clay.

Simone and Sorensen (2018) concluded that non-flushable VWPs fitted with HAE filters have a high risk of malfunctioning when placed in fully grouted boreholes. When employing this method of installation, they recommended the use of LAE filters.

The above recommendation is captured in the current ISO standard on measurement of pore water pressures using piezometers (ISO 2020). Annex E, which is normative, considers the installation of piezometers with the fully grouted method. It states that "*high air entry porous filters shall not be used with the fully grouted method unless there is a means of removing air from the piezometer*". Furthermore, Annex F, which is also normative, includes the following two statements:

- *"To successfully measure soil suctions all parts of the piezometer system (e.g. the backfill material, the porous filter and the fluid reservoir) shall remain saturated at all times and the water in the piezometer shall be in continuous contact with the water in the soil at all times.*
- *If air forms inside the piezometer it shall be removed and saturation of the device shall be restored. NOTE: Air can be removed by flushing water into a flushable piezometer or by removing the piezometer and resaturating it."*

The above implies that that non-flushable VWPs installed by the fully grouted method, even if fitted with HAE filters, should not be used to measure matrix suction in the field for long periods, given that (i) there is uncertainty in the performance of a HAE filter embedded in cement-bentonite grout, (ii) saturation of the piezometer system cannot be ensured, and (iii) it is not possible to resaturate the instrument once in place. The measurement of suctions in the field requires the use of a piezometer that can be retrieved and resaturated if needed, or the use of a flushable piezometer. An example of the successful use of a flushable piezometer to measure suctions is given, for example, in Ridley et al (2003).f

SUMMARY

This paper has briefly touched on a couple of topics that are of interest to the mining dams engineer and, to a lesser extent, to the embankment dams engineer. The first subject has to do with the effect of degree of saturation and matrix suction on the potential for a soil to liquefy during an earthquake. Lower degrees of saturation and higher suctions translate into enhanced resistance during cyclic loading; however, experimental data suggests that unsaturated materials still have the potential to experience cyclic mobility. The second topic has to do with the measurement of matrix suctions in the field. An example is given of a situation where this forms part of a critical control for a dam. Measurements are currently done with non-flushable VWPs installed by the fully grouted method. Although this method of installation offers advantages, non-flushable VWPs installed by the fully grouted method, even if fitted with HAE filters, appear not to be suitable for the task of measuring matrix suctions. The measurement of suctions in the field requires the use of a piezometer that can be retrieved and resaturated if needed, or the use of flushable piezometers.

REFERENCES

Banerjee A, Puppala A J and Hoyos L R (2022). Liquefaction assessment in unsaturated soils. *Journal of Geotechnical and Geoenvironmental Engineering,* **148(9)**.

Contreras I A, Grosser A T and Ver Strate R H (2008). The use of the fully-grouted method for piezometer installation. *Geotechnical News,* **26(2)** 30-37.

Dunnicliff J (2008). Geotechnical instrumentation news - discussion of "the use of the fully-grouted method for piezometer installation" by Contreras et al (2008*). Geotechnical News*, **26(2)** 38-40.

ISO (2020). *Geotechnical investigation and testing — Geotechnical monitoring by field instrumentation — Part 4: Measurement of pore water pressure: piezometers. Standard No. ISO 18674-4*. International Standards Organisation (ISO), Geneva, Switzerland.

Ridley A M (2015). Soil suction—what it is and how to successfully measure it. In *FMGM 2015: Proceedings of the Ninth Symposium on Field Measurements in Geomechanics*. Australian Centre for Geomechanics, Crawley, Western Australia. 27-46.

Ridley A, Brady K C and Vaughan P R (2003*). Field measurement of pore water pressures. TRL Report 555.* Quality Services, Civil Engineering Division, Highways Agency, Crowthorne, UK

Robertson P K (2010). Evaluation of Flow Liquefaction and Liquefied strength Using the Cone Penetration Test. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, **136(6)** 842-853.

Robertson P K (2016). Cone penetration tests (CPT)-based soil behavour type (SBT) classification system – an update. *Canadian Geotechnical Journal*, **53(12)** 1910-1927.

Robertson P K (2022). Evaluation of flow liquefaction and liquefied strength using the cone penetration test: an update. *Canadian Geotechnical Journal,* **59(4)** 620-624.

Robertson P K and Wride C E (1998). Evaluating cyclic liquefaction potential using the CPT. *Canadian Geotechnical Journal*, **35(3)** 442–459.

Simonsen T R and Sørensen K K (2018). Performance of vibrating wire piezometers in very low permeable clay. In *Proceedings of the 10th International Symposium on Field Measurements in Geomechanics* (FMGM2018), 1-20.

Vaughan P R (1969). A note on sealing piezometers in boreholes. *Géotechnique,* **19(3)**, 405-413.

Yoshimi Y, Tanaka K and Tokimatsu K (1989). Liquefaction resistance of a partially saturated sand. *Soils and foundations*, **29(3)**, 157-162.