Some Experience in Measuring Pore Water Suction in Dublin Glacial Till

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Several recent articles and discussions in GIN, by Penman (2002), Thomann et al. (2003), Ridley (2003) and Sellers et al. (2003) highlight the importance of pore water pressure and suctions and in particular the difficulties associated with measuring suction. This article outlines some recent experience in measuring suctions, which were developed in cut slopes in a very stiff glacial till during the Dublin Port Tunnel (DPT) project in Ireland. A particular feature of the project was the execution of a fully instrumented 12 m deep trial excavation (Menkiti et al., 2004).

Initially some background to the project and to the problem will be given. Then some details of the specification, installation technique and some examples of the measurements obtained will be outlined. Finally some conclusions will be made on the lessons learned and recommendations will be made for future similar applications.

Dublin Port Tunnel – Northern Cut and Cover Section

Further details of this project can be found in the papers by Long et al. (2003) and Menkiti et al. (2004). This article focuses on the works within the 800 m long northern cut and cover section, where the following design constraints apply (Figure 1):

- Very tight limitations on land take exist.
- This section of tunnel runs along the footprint of the existing M1 Motorway, now diverted to run 2m from the slope crest. The diverted motorway (and adjacent properties and homes) must be protected throughout construction work.
- Tight costs and programme constraints also apply.

Tender stage site investigations suggested the general presence of competent Dublin Boulder Clay (DBC), which is a very stiff to hard, dark grey, slightly sandy clay, with some gravel and cobbles. Local experience (Long et al., 2003) confirmed that steep excavations, up to 8 m or so, could stand unsupported for periods of three to four months. Since temporary support is only required for a period of three to six months and the excavation depth is 12m or less, it was decided right from tender stage to construct the works within steep cuts supported by soil nails where appropriate. The basic design solution required nails installed over the full slope height. However an observational approach was developed whereby rows of nails are omitted unless required by adverse geology or unsatisfactory monitored performance. By partially utilizing the excavation induced soil suctions, the partly nailed slopes were specifically designed to have a limited “stand up” time of about 12 months in order match construction requirements plus an adequate margin of safety.
The adopted solution is an application of the observational method originally codified by Ralph Peck in his Rankine lecture (Peck, 1969). He stated, “Inherent in the use of instrumentation for construction purposes is the absolute necessity for deciding, in advance, a positive means for solving any problems that may be disclosed by the results of the observations. If the observations should demonstrate that remedial action is needed, that action must be based on appropriate, previously anticipated plans”. In our case history, as a key aspect of the design submission, a series of appropriate contingency measures were developed for each stage of the works, and time interval for their implemented on site justified. Further details of the design and construction of this case history can be obtained from (Milligan et al., 2004)

This observational approach relied on thorough logging of the excavated materials as well as a daily review of readings from instruments, which included inclinometers, piezometers with a suction measuring capability, precise leveling and conventional geodetic surveying.

Pore Water Pressure in Cut Slopes

The pore water pressure conditions in the slope before and after excavation are shown diagrammatically on Figure 2. Before excavation the pore water pressure (u) is simply given by the hydrostatic head below the groundwater table, which is at about 2 m depth, i.e.

\[ u = \gamma_w z \]
The change in $u$ caused by the excavation can be determined (approximately at least) from Skempton’s (1954) classical equation for pore water pressure change:

$$\Delta u = B[\Delta \sigma_3 + A(\Delta \sigma_1 - \Delta \sigma_3)]$$

In this case both the all round pressure ($\Delta \sigma_3$) and the deviator stress ($\Delta \sigma_1 - \Delta \sigma_3$) reduce due to the excavation-induced stress relief. This means that $u$ reduces and, depending on its initial value, could become negative. If $u$ reduces then the effective stress increases, thus improving, in the short term, the stability of the slope. The length of time over which this reduced pore water pressure can be sustained is a complex issue and depends on the soil type, its fabric, permeability, the sequence of construction, slope protection, weather, etc.

**Figure 2. Pore water pressure reduction in cut slopes**

**Calculation of Reduced Pore Water Pressures / Suctions**

As can be seen from Skempton’s formula above it is necessary to determine $\Delta \sigma_3$ and $\Delta \sigma_1 - \Delta \sigma_3$ in order to calculate the pore water pressure after excavation. This is not a trivial matter. Some elastic based solutions exist. However in this project use was made of the finite element approach. Further details of these analyses can be found in Menkiti et al. (2004). This work was carried out by the Geotechnical Consulting Group (GCG), who made use of the Imperial College, London, geotechnical finite element code called ICFEP.

Initially, a full-scale monitored trial excavation was carried out to accurately calibrate the numerical model, and to develop procedures for safely implementing the observational method in the main excavation. No soil nails were used in the trial excavation. Then, various analyses were carried out with the calibrated model to investigate the influence of the excavation slope, slope protection measures, granular lenses in the till (which encourage recharge and suction dissipation), the presence of soil nails etc. (Menkiti et al, 2004). Some typical output for a homogenous clay till, without any permeable lenses or horizons, and with no soil nails (i.e. similar to the trial excavation) is shown on Figure 3.
A maximum horizontal movement of 15mm is predicted near the slope crest due to essentially undrained excavation. The induced stress changes depress the pore water pressures as discussed earlier to give high suctions of various intensities, up to -75kPa (-10.9 psi) at the slope toe. In the analyses, it is these suctions that maintain the stability of the slope.

**Specification for Piezometers**

In view of the above it was clear that the pore water pressure sensors to be used in the project had to have a suction measuring capability. The following parameters were included in the specification, which was sent to the tendering instrumentation contractors both for the trial trench and for the main works. Tendering was for a “supply and installation of instrumentation” to be carried out by a subcontractor to the general contractor. Separate tenders / contracts were used for the trial trench (which was part of the design development) and later for the main works.

![Figure 3. Finite element output for trial trench (assumes homogenous clay slope)](image)

- Direct measuring piezometers are required. Indirect measuring systems, such as thermal conductivity and electrical resistivity sensors, are not acceptable.
- The piezometer shall be calibrated and be able to operate in the range -85kPa to +250kPa (-12.3psi to +36.3psi).
- They shall have an output resolution of 0.025% full scale (FS) and an accuracy of 0.1% FS. Resolution, meaning the smallest division on the readout scale, was specified to give the Designer’s confidence that the accuracy can be achieved. If a resolution of 25% of the required accuracy cannot be achieved, then the system must very closely examined.
- Each piezometer shall be installed in a separate borehole and the measuring unit inserted, into a cleaned smaller diameter hole at the base of the main borehole, so as to fit as tightly as possible against the natural ground. (However, as a result of budgetary constraints during tender negotiations for the main works, this technical requirement was relaxed so that up to three piezometers could be installed in a single borehole. The relaxation was coupled with stringent requirements to prevent vertical hydraulic connectivity between instruments in the same borehole.)
• It is then sealed into the hole using plaster of Paris before the borehole is grouted up using a bentonite / cement mix.
• The piezometer should have the facility to be de-aired, prior to reading, e.g. by flushing with de-aired water.
• Output from the piezometers shall be logged at 15-minute intervals. Sufficient data logging capability should be provided to permit downloading weekly.

Why Use a Plaster of Paris Seal?

Installing piezometers in boreholes and completely surrounding them with grout (of permeability higher than that of the ground) is not a new idea. This technique was used for the instrumentation of embankment dams in the 1960’s. Vaughan (1969) describes some experience at Balderhead Dam in the UK. He developed an approximate relationship between the error in the piezometer reading and the relative permeabilities of the ground and the grout. Vaughan also showed that the grout must have permeability considerably higher than the ground for the piezometer reading to have a significant error.

Subsequently experience in Europe had shown that plaster of Paris is a rigid but permeable material that can provide a good snug seal around the piezometer, without trapping air voids. Depending on the adopted mix, the permeability of the set plaster of Paris is similar to that of a fine sand / silt (i.e. of the order of $10^{-7}$ to $10^{-6}$ m/s). This is many orders of magnitude higher than the permeability of the cohesive glacial till (which has a permeability of the order of $10^{-9}$ to $10^{-10}$ m/s). In essence, the plaster of Paris performs the same function as filter sand commonly used in the response zones of standpipe piezometers, except that the plaster of Paris arrangement gives a system that is capable of rapidly transmitting the local ground water pressures to the instrument for measurement. Furthermore, the whole piezometer-plaster of Paris tip arrangement is easy to keep saturated.

The plaster of Paris is only used over a short length of borehole around the piezometer (typically 0.5m long). Above this, the borehole is filled with low-permeability cement-bentonite grout to avoid hydraulic connectivity vertically along the borehole.

Details of Piezometers Actually Used

The piezometers used in the works were supplied by ITM Ltd. and comprised the Soil Instruments, de-airable vibrating wire, 3 bar (43.5 psi), piezometer, which was fitted with flushing lines. A section through and a photograph of the instrument are shown on Figures 4 and 5 respectively.

Some special and particular features of the piezometer are as follows:

1. Unit is approximately 50mm diameter, 300mm long and with a 50mm long filter
2. The filter unit is constructed with 1 bar high air entry (i.e. high resistance to air entry) value porcelain, supplied by Fairey Industrial Ceramics Ltd. It has maximum pore size of 1 \( \mu m \) and an apparent porosity of 40% to 45%.

3. The volume of water in the chamber between the filter unit and the pressure measuring diaphragm is approximately 12.5 cm\(^3\).

4. The de-airing flushing lines consist of two 3.7mm internal diameter tubes (one marked in and one marked out).

**Figure 4.** Section through de-airable, flushable, vibrating wire piezometer

**Figure 5.** Photograph of piezometer
Alternative Proposal

As the works are approximately 800 m long with monitoring being required at about 20 m intervals on both sides of the excavation, and with up to three pore water pressure instruments at each location, economic factors significantly influenced the choice of instrument used.

An alternative, but significantly more expensive, instrument was also suggested by one of the tendering contractors. This comprised low air entry filters with solenoid or hydraulic valves at the tip to be installed in a fully-grouted borehole. The chief advantage of this system is in the ease of installation, i.e. only one medium is needed for the installation. However there were concerns about some aspects of the system, which was still under development at the time. For example there was a much higher risk of vertical connectivity between the three instruments if all were installed in a single hole, particularly in the case of a perched water table within the surface made ground. These concerns, together with cost considerations, led to the selection of the chosen solution.

Installation of Instruments

Prior to installation, several small scale tests were conducted under laboratory conditions, in which trials were made of the powder / water ratios required for the grout and the plaster of Paris, setting times required and optimum sequence of installation. It was also found that, due to the low permeability of the till, in general the boreholes were dry. This allowed all three instruments to be installed in a single borehole in most cases (see piezometer specification section above). For these conditions, the liquid plaster of Paris mix could be tremied straight to the bottom of the dry hole. However if the hole is flooded, the liquid plaster of Paris tends to dissociate in the borehole water into a thin colloidal mix that coats the side of the borehole over the wetted perimeter. This tends to generate unacceptable hydraulic connectivity over the flooded section of borehole. The following procedure was therefore adopted, as shown on Figure 6.

1. Assemble piezometer and pre-saturated ceramic filter tip. Cut cables and tubes to required length.
2. Encase piezometer tip in plaster of Paris using a mould.
3. Once set, saturate the piezometer tip and keep under water until installation.
4. Drill a 146mm borehole using Geobore ‘S’ tools to a point 300 mm deeper than the installation depth of lowest instrument. Dip the hole to check that it is dry.
5. Tremie a pre-measured volume of thoroughly mixed plaster of Paris into the borehole.
6. Insert the piezometer into the plaster of Paris and leave for at least 10 minutes to set.
7. Carefully tremie well mixed cement / bentonite grout into the borehole to within 1 m of next installation tip and allow 10 minutes to settle down.
8. Thereafter, add the required amount to bentonite pellets to bring the level up to 0.3m below next installation. Bentonite pellets were used in combination with cement / bentonite grout for the following reasons: (i) it was found from lab trials that this combination gave the best seal around multiple cables and tubes; (ii) bentonite pellets
also provided a stable platform for installation of the next piezometer, and (iii) the bentonite pellets prevent intermixing plaster of Paris with the cement / bentonite grout.

9. Tremie water onto the bentonite pellets until water level is level with top of pellets (using a dip meter). Allow 20 minutes for pellets to go off by expanding and absorbing all the free water.

10. Repeat steps 5 to 9 with remaining instruments.

11. Once last instrument is in place, and the plaster has set, bring the remaining level up to 0.4m below ground level with cement / bentonite grout.

12. Concrete headworks (plastic covers with caps) into the ground. Insulate de-airing tubes against frost.

13. Wire up to datalogger

14. After 24 hours, de-air instruments.

**Commissioning Procedure**

The following procedure was adopted to de-air and commission the instruments and to check whether there was any connectivity between each separate installation.

1. Connect the de-airing equipment to the piezometer “in” valve. Keep the piezometer “out” valve closed. The de-airing equipment consists of a system of foot pump, pressure gauge and collection chambers. The system is used for driving de-aired water through the piezometer installation while measuring the volume of water circulated through the piezometer. A system by ITM was used for supplying de-aired water. This was produced by boiling water in a reinforced chamber with a one way outlet valve. Boiling was maintained for a minimum of 10 minutes, preferably much longer, to de-air the water and drive out any air in the chamber. Then, the valve was closed and the system allowed cool.

2. Set the data logger to log at the highest frequency.

3. Using the foot pump and pressure gauge pressurize the lowest piezometer first, taking care not to exceed the pressure limits for the instrument.

4. Once at a reasonable pressure (150 - 200 kPa) (22 - 29 psi) is attained, lock off and maintain that pressure for 30 minutes, periodically introducing more pressurised water as the pressure dissipates.

5. Gently release the pressure applied to the piezometer to complete the connectivity test. Then, de-air the instrument by circulating de-aired water through the piezometer to complete the commissioning process. The volume of de-aired water circulated through the piezometer should be several times the volume of the piezometer chamber and connecting tubes.

6. Repeat steps 1 to 5 for the other piezometers in the borehole, working upwards.

7. Download data logger and analyse results, pressure peaks should only occur at one instrument at a time.
Figure 6. Cross section through piezometer installation

Typical Measured Data – Main Works

A cross section at a typical location (Chainage 1320W) on the main works is shown on Figure 7. Piezometers were installed in relatively homogenous low permeability clayey till at 4 m, 8 m and 12 m and the ground water table was at about 2 m. Taking the 8 m piezometer, for example, initially the pore water pressure should respond to the ground water table 6 m above it (i.e. 58.9 kPa or 8.5 psi). Subsequently it was expected, from the finite element analyses, that the excavation induced stress relief would reduce the pore water pressure to a value in the range −25 kPa to +25 kPa (- 3.6 psi to + 3.6 psi), with a “best guess” being about −10 kPa (−1.5 psi).

Recorded data for the three piezometers at Ch. 1320W is shown on Figure 8. As was expected the 8 m piezometer initially recorded a pressure of about +58 kPa (+8.4 psi). The effect of the four stages of the excavation can be clearly seen with the final stage having the most pronounced influence. The lowest value recorded in this piezometer was about +5 kPa (+0.7 psi), a value consistent with the finite element prediction.
12 m excavation in 4 lifts

Before excavation
\[ u = 6 \times 9.81 = 59 \text{ kPa (8.6 psi)} \]

After excavation
stress relief reduces \( u \)
FEM predictions
- 25 kPa to + 25 kPa
(-3.6 psi to + 3.6 psi)

4 m
8 m
12 m

Figure 7. Typical cross section – Main works (Chainage 1320 W)

Figure 8. Pore water pressure data – Main works Ch. 1320W
(Note y-axis range of –20 kPa to +100 kPa = -2.9 psi to +14.5 psi)

The lowest values of pore water pressure were recorded by the 12 m piezometer, with a minimum value of about –8 kPa (-1.2 psi). This instrument was de-aired a few occasions and the effect of the de-airing process can be easily seen in the data. It can be seen that the readings recorded by the piezometer recovered rapidly following de-airing and thus the length of time during which the piezometer was out of commission was very small.
Typical Measured Data – Trial Excavation

Output for all of the instruments, located within 0.5 m of the slope crest, in Zone 2 of the trial excavation is shown on Figure 9. At this location, the slope was protected by a geotextile. A local failure occurred about 35 days after excavation. The failure was attributed to persistent high rainfall which occurred for several days preceding the slip and the presence of a high permeability lens at about 5 m depth (Menkiti et al., 2004).

The finite element analyses predicted that suctions of about –20 kPa or –2.9 psi (2.5 m) and –10 kPa or –1.5 psi (5.5 m) would be induced by the excavation in the upper two piezometers and that the 7.5 m piezometer should indicate pore water pressures of about zero. In actual practice it can be seen that all the recorded values were more positive than the predictions. Only the 2.5 m deep instrument recorded negative values of about –10 kPa (-1.5 psi).

Subsequently it can be seen that this piezometer (P2A at 2.5m depth) responded to the persistent rainfall which eliminated all of the suction. As it was located on the failure surface, negative pore water pressures were induced due to dilation on shearing until cavitation occurred at about -15 kPa (-2.2 psi).

![Figure 9. Pore water pressure data – trial excavation Zone 2](image)
(Note y-axis range of –20 kPa to +100 kPa = -2.9 psi to +14.5 psi)
Conclusions and Discussion on Measured Values

Some conclusions on the measured piezometer data are as follows:

- The excavation-induced suctions were modest and the instruments generally successfully recorded these.
- These suction values were lower (i.e. closer to zero) than predicted by finite element analyses, probably due to local higher permeability zones in the microfabric, not accounted for in the analyses.
- The installation procedure adopted, including the use of plaster of Paris to encase the piezometer tips, was successful. However a high level of engineering supervision is essential.
- The piezometers responded very well and rapidly to routine de-airing.
- Some instances of cavitation were noted at suctions in the range –20 kPa to –30 kPa (–2.9 psi to –4.4 psi).
- Site experience suggests that although de-saturation of the filter tip and the relatively large body of water within the piezometer may have contributed to the instances of cavitation, these were not the major factor.
- It is likely that the most significant contributing factor to the instances of cavitation was the length of the de-airing lines. Occasionally when the valves at the top of the lines were opened for de-airing air bubbles emerged from the water body.

Lessons Learned / Proposal for Future Works

- Very significant cost and time savings were enjoyed in the case history described by applying the observational method with controlled use of induced soil suctions. This illustrates the potential benefits that could be accrued, for temporary works in particular, by taking induced soil suction into account in a quantitative manner. However, the design and implementation of these procedures requires careful thought.
- For this reason the trend is increasingly for the contribution of soil suction to be taken into account in design work. In parallel, it is likely that there will be increased demand for suction measurement instruments. Consequently more work is needed to develop reliable and cheaper instruments for suction measurement in the ground.
- The piezometers in the case study described were found to be suitable and cost effective for suctions up to about –20kPa (-2.9 psi).
- In cases where it is essential to measure higher suctions, other piezometers that contain a smaller volume of water would be more appropriate. However, these are generally more expensive.
- For similar work in Dublin Boulder Clay, sufficient experience has now been gained to warrant substantial reduction in instrumentation quantities to say half of those used for the initial application of the novel design. For application to different ground conditions instrument numbers should be selected based on careful consideration of: (i) sensitivity of the design to measured suctions; (ii) consequences of collapse; (iii) variability of the ground conditions, etc.
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