Review of the papers and general comments

Examen des rapports et commentaires généraux

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ABSTRACT

This report attempts to review the contributions to discussion session 2.2 and give some general comments concerning the problems of groundwaterlowering. Finally a number of conclusions are drawn.

RÉSUMÉ

Dans ce rapport il est essayé de donner un aperçu des contributions à la session de discussion 2.2 et quelques commentaires généraux sur les problèmes de rabattement de la nappe. A la fin un quelques conclusions sont données.

1 REVIEW OF PAPERS

The papers presented for discussion session 2.2 are not all dealing with groundwater in urban areas. They can be grouped as follows:

Subject	Number of papers
Case histories	4
Settlements due to groundwater lowering	2
Hydraulic barriers:	
- ground freezing;	2
- sheet piles;	1
- injections;	1
- cement bentonite walls;	1
- compacted clay liner (CCL)	1
Heave and swell pressure	1
Water extraction by trees	1
Slope stability	3
Swelling soils	1

1.1 Case histories

Bock & Markussen describes the realisation of a hughe groundwater lowering for a 17 m deep excavation at the location of the new Copenhagen Opera. The paper describes the:

- difficult geological conditions;
- the former experience from the groundwater lowering for the reparation of a nearby dry dock;
- the preliminary investigations with long term and short term pumping tests in different layers;
- the stabilisation of the excavation slopes with sheet piles till 10 à 13 m depth;
- the design of the groundwater lowering;
- the infiltration to protect the foundations of historic cranes and storehouses;
- the monitoring program;
- the infiltration with sea water (250 m^3/h in 32 wells).

During spring and summer a fast grouth of organisms has been observed in the re-infiltration wells. So cleaning of the wells by acid-treatment was necessary and filters with meshes of 0.4 mm have been installed on the pump intakes.

The paper clearly demonstrates that groundwater lowering for open pit excavations is still possible in very difficult conditions when good preliminary investigations and detailed monitoring programs are provided.

Hulla et al describes two very interesting excavations in the city of Bratishava. Both excavations have been realised within a diaphragm wall enclosure.

At the Opera Garage several problems occurred:

- during the construction of the diaphragm wall settlements of up to 25 mm occurred. After

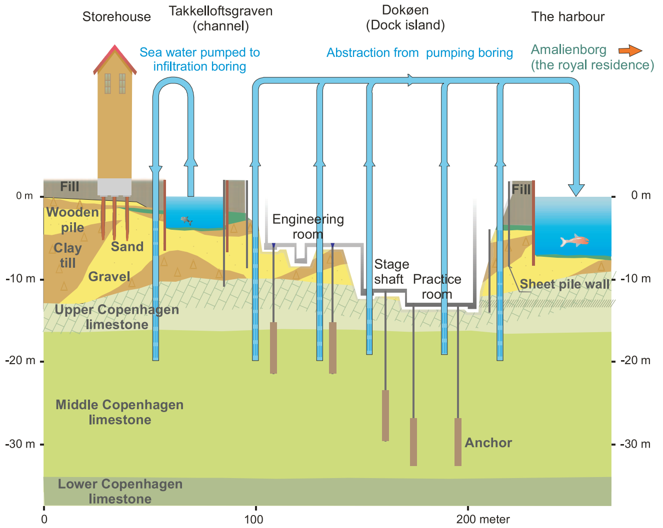

Figure 1. Principal vertical section of the excavation and the groundwater lowering system.

changing the execution technology much lower settlements occurred;

– after the realisation of the diaphragm wall, designed for 3 underground levels a 4th underground level had to be installed. During the realisation of the 4th underground level the horizontal displacements of the diaphragm remained very small (max. 3.5 mm).

At the Carlton Garage also several problems occurred:

– the realisation of a second row of ground anchors situated ca 3 m underneath the groundwater level was not possible as great quantities of sand flowed into the construction pit after the realisation of the boreholes. So braces between the diaphragm wall and the already realised central part of the foundation slab had to be installed;
– the diaphragm wall consists of prefabricated elements with reinforced self hardening suspension between them. Serious leakages occurred during excavations. Injections with polyurethane resins have been performed and a drainage system has been installed on the foundation slab;
– to ensure the uplift stability of the garage a pumping system is installed, which allows to lower the water pressures in the soil beneath the foundation slab in periods of flooding in the Danube river, cfr. Figure 2.

Paci describes the problems that occurred with the execution of a 7.5 m deep excavation in Tirana. The retaining wall consisted of tangent piles ∅ 0.70 m with a length of 12 meters. A reinforced concrete beam of 70 cm × 70 cm has been installed on top of the piles. The subsoil consists of fill, clay-sand, sand and clay-stone.

Calculations have been performed with the Plaxis software. For the clay-sand and upper clay-stone layer very high values of the cohesion and E modulus have

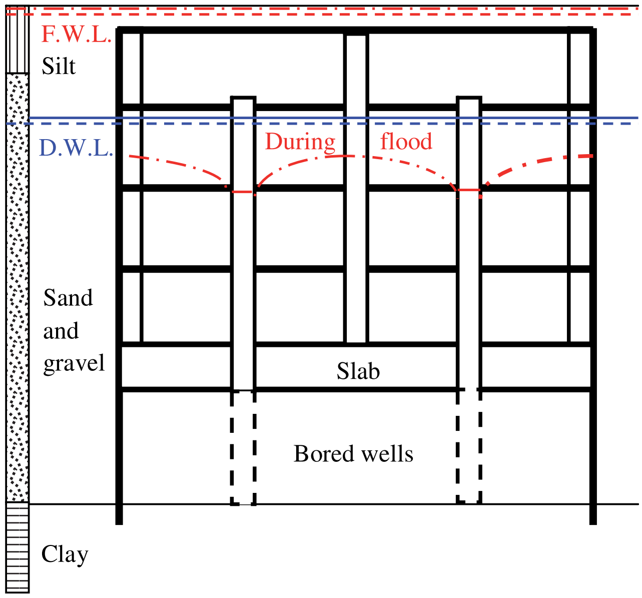

Figure 2. Stability assurance of the Carlton garage for uplift during a flood in the Danube (D.W.L. – design water level, F.W.L. – flood water level).

been introduced. The calculated displacements were rather small.

After a rainy storm the excavation was filled with water from a drainage pipe diameter 1.5 m situated behind the wall, and it took 1 week to pump the water out of the excavation.

After this accident a second calculation was performed with modified characteristics of the sand layer and the upper part of the clay-stone and a much larger horizontal displacement of the top of the wall was obtained (44.5 mm instead of 26.5 m). According to these results it is quite normal that cracks occurred in the masonry building situated just behind the wall.

This example clearly illustrates that it is always risky to install cantilever walls along existing buildings with direct foundations. Even when former experience is available it remains very difficult to predict the displacements to be expected. Cantilever walls are also more sensible for damages when unforeseen events occur.

Justa Camara describes the measures that have been taken to have access to the cutter head of a shield tunnelling machine.

The first case history concerns a project where a tunnel diameter 12.06 m was bored under more than 10 m water head (= above the upper level of the shield) and through sandy gravels with boulders. As the boulders were much larger than expected, the cutter head had to be repaired during the tunnelling operation. Diaphragm walls have been installed in combination with jet grout columns in order to create an access to the cutter head. To control and reduce the hydrostatic pressure over the access shaft two discharge wells have been installed.

The second case history concerns Metro line 9 in Barcelona, where an improved ground block has been realised by means of jet-grouting. In order to limit the disturbance on the street level the jet-grouting has been executed from a small working room.

1.2 Settlements due to groundwater lowering

Ivanov & Mihova describes the different approaches used to predict the settlements due to the groundwater lowering necessary for the construction of two ventilation shafts of the Sofia Metropolitan.

- unique deformation modulus: max. predicted settlement 31.3 cm;
- deformation modulus increasing with depth: max. predicted settlement = 12.11 cm;
- consolidation taken into account max. predicted settlement = 5.5 cm after 1 year.

They conclude that the settlement predictions are only tentative ones, due to the lack of sufficiently precise data.

Mecsi describes a model to calculate the soil deformation due to a lowering or rising of the groundwater table. The model has only a demonstrative character as stated by the author.

1.3 Hydraulic barriers

1.3.1 Ground freezing

Thumann & Hass describes the construction of a cut of wall underneath an existing metro tunnel by means of ground freezing, cfr. Figure 3.

As the freeze pipes had to be installed in very difficult conditions till a depth of 40 meters, a detailed monitoring of the location of the pipes was necessary and additional pipes had to be installed. Based on the real position of the pipes thermal calculations have been performed. So it was found that warming pipes have to be installed to avoid that the foundation piles of the new station (= Tube× piles) have to be drilled through frozen soils.

Frost heave calculations have been performed to estimate the potential impact on the existing tunnel due to freezing operation. Laboratory tests have been performed to deduce the frost heave parameters.

Finally the monitoring program is described.

Sres et al describes a 3D model elaborated within the framework of a research program into artificial freezing for tunnelling. The so called TH model allows to simulate combined thermal and hydraulic process.

Large scale tests ($1.2 \times 1.3 \times 1.0\,m^3$) have been performed with a wide range of seepage flow conditions to verify the model, cfr. Figure 4.

After the verification of the TH-model a parametric study was started to illustrate the effect of seepage flow on the closure time necessary for the creation of a frozen wall.

1.3.2 Grouted sheet piles

Lehtonen & Sintonen describes the execution of a field test where continuous grout has been continuously injected during the installation of sheet piles in order to decrease the permeability, cfr. Figure 5.

The obtained permeability was less than 9×10^{-11} m/sec. As the field test was performed in

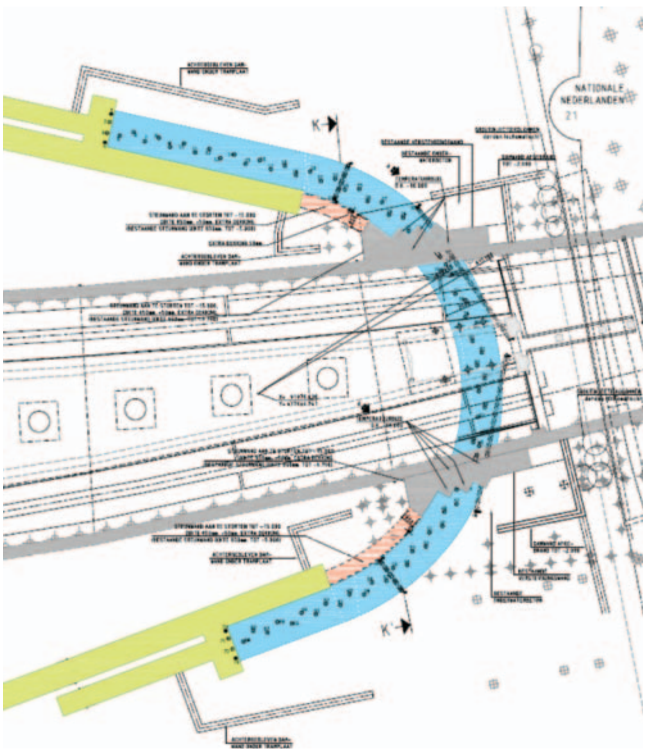

Figure 3. Plan of designed collar construction around the tunnel at station platform elevation, showing frozen soil area (blue) and supporting diaphragm walls (green). Distance between vertical freeze pipes is ca. 0.9 m.

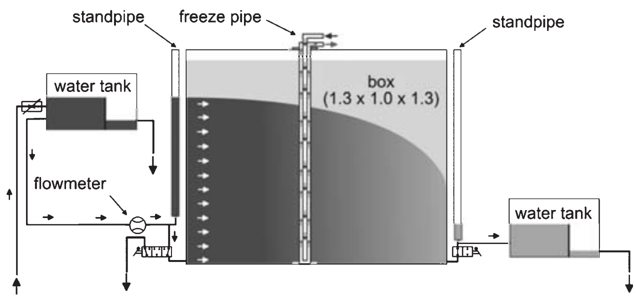

Figure 4. Plan of the large-scale laboratory model.

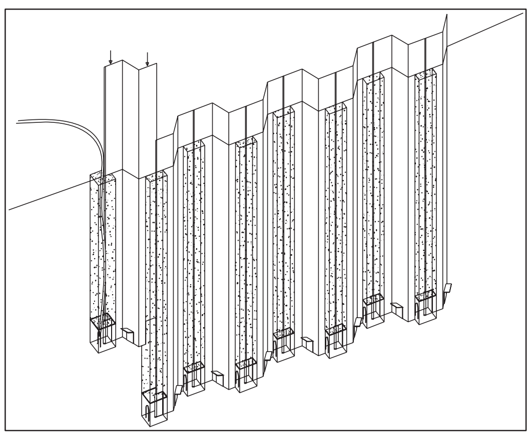

Figure 5. Sheet piling and continuous cement grouting.

very soft clay the additional driving resistance due to the presence of an collar above the pile tip was limited.

1.3.3 Injections

Lambert et al describes the risks related to water retaining walls.

When water retaining walls are constructed to limit the groundwater lowering outside a building pit it is important that:

- possible preventive measures are already considered during the design stage;
- a detailed quality control and monitoring is performed during construction of the wall.

When excessive leakage takes place leak detection methods can be used to find the exact location of the leak(s).

Full scale tests performed in the Netherlands clearly demonstrates that Biosealing is a new technique which can even solve leakage problems that could not be solved until recent times. This because Biosealing does nog require to be able to reach and even not to know the exact location of the leak. The injected nutrition searches for and clogs in the leak. Actually further research is going on.

Lipinsky et al describes the test performed to determine the characteristics of cement-bentonite SOLIDUR 2745 used for the realisation of a vertical cut-off wall around sanitary landfills in Poland.

The permeability tests have been performed in triaxial cells after isotropic consolidation under an effective stress of 150 kPa and according to the flow pump technique (= steady flow is imposed and pore pressure (head) at the bottom of the specimen is measured). Tests have been performed on reconstituted samples and undisturbed samples taken form a 3 years old wall.

Based on the results of the performed tests formulas have been set up to calculate the coefficient of permeability of self hardening SOLIDUR 2745 at any stage of the hardening process and for any material density.

1.3.4 Compacted clay liner (CCL)

Heerten discusses the capping of non-hazardous landfills and more particularly the material that should be used for the realisation of an impermeable mineral layer.

Although the discussion is very interesting it falls completely beyond the scope of this discussion session.

1.4 Heave and swell pressure

Oung et al describe the back calculation of the heave measured during the construction of a 20 m deep excavation in Rotterdam.

The 80 m×25 m building pit was surrounded by diaphragm walls, with a tickness of 1.50 m and installed into an impermeable layer at 40 m depth. Two extensometers have been installed in the centreline of the building pit to measure the vertical soil displacement during excavation. The vertical soil displacement at different levels is given in Fig. 6.

Back calculations have been performed with the Plaxis hardening soil model. A good agreement between the measured and calculated heave was obtained when the normally applied coefficient of consolidation is multiplied by 4.2, cfr. Fig. 7.

The paper contains also a very interesting discussion on the determination of the swell pressure.

The conclusion is that Plaxis provides smaller values of swell pressure when a fictive floor with zero deformation is introduced than when a counter load with zero vertical soil deformation is introduced. In the case study, the swell pressure calculated with a counter load, agrees with the practice.

1.5 Slope stability

Görög & Török describe the study of the stability of a 60 m high slope of an ancient clay quarry. The cross-section is given in Fig. 8.

The characteristics of the different soil layers have been obtained from laboratory tests. For the landfill undisturbed samples could not be taken, so conservative characteristics had to be introduced.

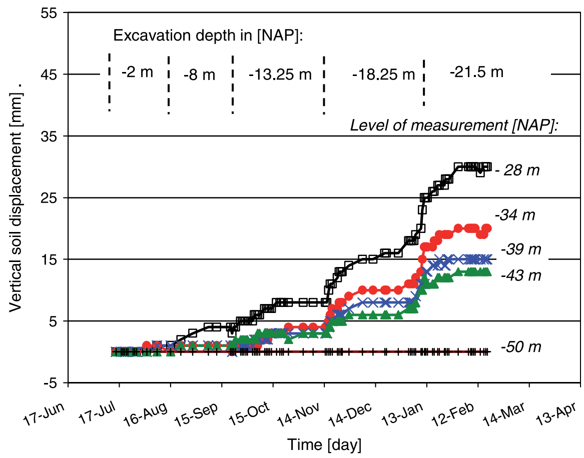

Figure 6. In-situ measurements of extensometers.

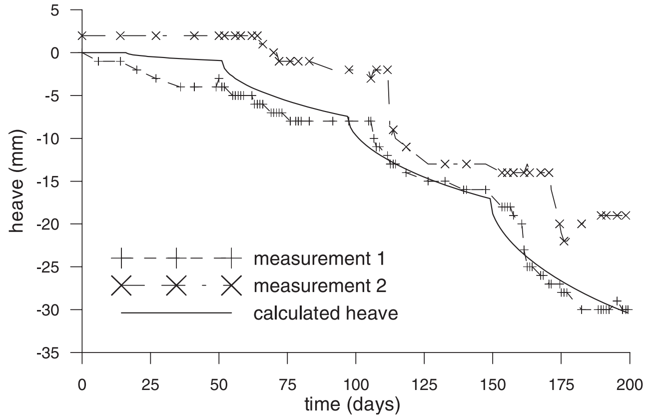

Figure 7. Calculated vs. measured heave (analytical method).

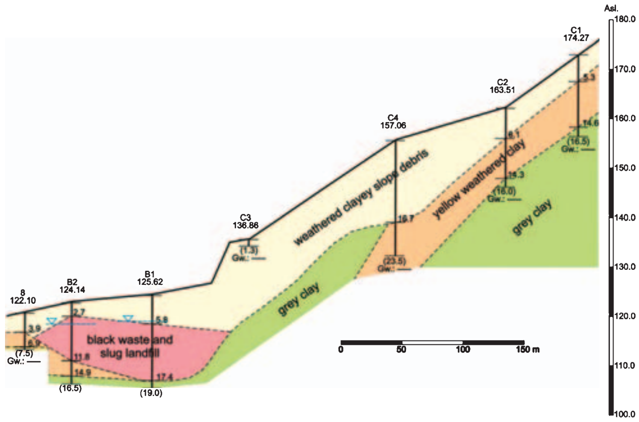

Figure 8. The section of the analysed slope.

Stability calculations have been performed by means of the analytical program Ge04 and the finite element program Plaxis. Comparable values of the safety factor have been obtained.

Although the stability calculations indicated that there is no considerable decrease of the safety factor when buildings are installed on the slope, the conclusion is that the risk is to high. This conclusion is based on the experience that the fiction angle of the soil layers can decrease radically when the water content of the clay increases due to water infiltration.

Boldini et al describes the study of a large unstable slope (length 500–600 m, height $\sim$100 m and inclination 11–12°). The subsoil consists of clayey material (slide debris) on softened and intact clay. Over a long period pore water pressures and displacements have been measured.

The obtained results clearly indicates that there is more than one slip surface and the inclinometer measurements shows a cyclic displacement. The irreversible component is due to the very slow movement of the upper layers. The cyclic component is related to the pore water pressures and is ascribed to the swelling and shrinking of the upper layers.

Calculations with the Finite Element Code Seep/W were carried out to understand the influence of infiltrating rainfall on the pore water pressure variation. It is remarked that the influence of rainfall is overestimated and that the discrepancies between measured and calculated pressures can probably be limited by introducing the run off component and the water retention in the loam top-soil layer.

Navarro et al describes the studies performed concerning the damages of buildings along the Rondilla Street in Alcazar de San Juan which occurred after the realisation of a utility trench.

This study comprised:

– a detailed geotechnical investigation with the boreholes, SPT-tests, DPSH-tests and investigation pits;
– observation of crack widths.

As the subsoil underneath the foundations consists of lean clay and loam, also the variation of the moisture content with depth was measured.

The influence of the trees has been studied by means of a finite difference program, modelling the water flow. The results indicated that the trees have hardly any effect on the metric suction near the building façades. So the drainage caused by the trees has been associated with a loss of restraint produced by the lateral shrinkage.

The conclusion is that the damage was caused by the shrinkage – lateral movements and a loss of restraint due to the excavation of the services trench.

Finally the trees have been removed and remedial works have been carried out.

Tomboy et al describes the full scale tests performed at Limelette where the upper layers consists quaternary loam and the groundwater level is very deep. A 3 m deep and 20 m long excavation has been realised and the negative porewater pressures and displacements have been monitored over a long period. A first collapse occurred after an important rainfall and was followed by other collapses. The measurements clearly indicates that at that time the negative pore water pressures were almost zero near to the surface and reduced considerably at larger depths.

The results of the full scale test will be used to elaborate a guidance for contractors who want to realise temporary excavations with very steep slopes in unsaturated soils.

1.6 Swelling soils

Ejjaaouani and Shakhirev discuss the very difficult problem of the heave of foundations due tot the wetting of swelling soils. The considered problem can be compared to the calculation of the settlements of foundations due to the lowering of the groundwater table. However in the last case the calculations are normally performed for the Greenfield conditions and it is assumed that no damage will occur when the settlements for Greenfield conditions are smaller than certain values, f.i. 20 mm.

When the settlement or heave of a foundation has to be estimated, the stress distribution underneath the foundation has to be introduced and the obtained results are influence considerably by the method used for the calculation of the stress distribution underneath the foundation.

In the contribution of Ejjaaouarri and Shakhirev a simplified method is proposed to calculate the stress distribution underneath the foundation and the heave of the foundation. The heave values calculated according to the proposed method have been compared with the experimental data obtained on a job site at Ouarzarate.

Venda et al describes the influence of the initial stiffness of the soil foundation on the behaviour of an embankment.

Although the subject is very interesting it is completely beyond the scope of the discussion session.

2 COMMENTS

2.1 General comments

The review of the contributions to the discussion session 2.2. "Dealing with groundwater in urban areas" clearly illustrates that the subject is very complex.

When looking to the number of references given at the end of the different papers one can see that there is a very large scatter and that for problems as slope stability, unsaturated soils, waste disposal there is a lot of information available, cfr. Table 2.

From the number of references of the case histories one may deduce that there is only little information available on the real subject of groundwater in urban areas. This is certainly true.

Possible explanations for that are that:

– modelling of the groundwater lowering for excavations or infrastructure works is mostly done by hydro geologists or geologist and not by geotechnicians;
– there are almost no standards and reference works on the subject (= artificial groundwater lowering, calculations of settlements due to groundwater lowering, installation of hydraulic barriers, control of hydraulic barriers ...);
– lowering of the groundwater table is more and more forbidden due to the risk that existing contaminations are displaced by the groundwater seepage.

The problem is clearly illustrated by the fact that TC288 has never accepted tot elaborate a Euronorm on groundwater lowering although it has been proposed several times.

When looking to the past any one can see that the subject has been loosing a lot of his interest during the last decennia. In 1987 the European Conference of ISSMGE was completely devoted to this subject as the theme of the conference was : groundwater effects in Geotechnical Engineering.

Following sub themes have been discussed:

1) Field and laboratory testing;
2) Groundwater control;
3) Environmental problems and seepage;
4) Groundwater problems in embankments, dams and natural slopes;
5) Special problems soils;
6) Dynamic effects;
7) Groundwater in foundations and excavations;
8) Groundwater modelling;
9) Filters.

Table 2.

Subject	Number of references
Case histories	0,4(0),3(0),2(0)
Settlements due to groundwater lowering	3(1),9(1)
Hydraulic barriers:	
- ground freezing;	1(0),8(1)
- sheet piles;	3(0)
- injections;	4(1)
- cement bentonite walls;	2(2)
- compacted clay liner (CCL)	15(0)
Heave and swell pressure	8(0),9
Water extraction by trees	13
Slope stability	9,9,12
Swelling soils	18

2.2 More specific comments

The subject of groundwater in urban areas is indeed so complex that on has to split it up in more specific subjects, f.i.:

– artificial groundwater lowering:
  – execution
  – influence
  – modelling
– settlements due to groundwater lowering;
– environmental impact of groundwater seepages;
– hydraulic barriers:
  – execution;
  – control;
– recharge;
– slope stability.

2.2.1 Artificial groundwater lowering

The knowledge on the execution of an artificial groundwater lowering is almost completely in the hands of the specialised contractors. The interest from the geotechnical world on that subject is rather small. However, the execution of the discharge wells and the evacuation system may have a major influence on the capacity of the wells and the obtained groundwater lowering.

A very complicated item is the prediction of the radius of influence of a groundwater lowering. For groundwater lowering in phreatic water many geo technicians are still using the formula of Sichardt

$$R = 3000 \cdot \Delta \cdot \sqrt{k}.$$

The knowledge of the radius of influence is very important to determined the area of which piëzometers have to be installed and settlements have to be measured to control the influence of the groundwater lowering.

At the moment powerful programs are available to model groundwater seepages. In most cases the necessary information on the permeability and storage capacity of the different layers is not large enough to obtain very detailed information. In most cases calculations performed for the non steady state provides a false impression precision.

2.2.2 Settlements due to groundwater lowering

Generally estimation of the settlements due to groundwater lowering is done for the Greenfield condition. When calculating the settlements due to a groundwater lowering one has to be take into account that:

– the stress increase due to the groundwater lowering is rather small and that therefore rather high stiffness values have to be considered. A practical solution that often is applied is to consider the normal stiffness values and to divide the obtained settlements with a factor 2;
– when the groundwater level has already been lowered before, much smaller settlements have to be expected. In that case the unloading reloading stiffness can be introduced in the calculations.

When the settlements of foundations are calculated one has to consider that the stress distribution underneath the foundation, used for the calculation of the initial stress may have a large influence on the calculated settlements. This is certainly the case when the compressible layers are at a certain depth underneath the foundation level.

2.2.3 Environmental impact of groundwater seepages

In most cases hydraulic barriers are foreseen when a large environmental impact is expected. So the available information on the environmental impact of groundwater lowering is rather limited.

2.2.4 Hydraulic barriers

Due to the environmental problems created by groundwater lowering underground works are realised more and more within the protection of hydraulic barriers.

When possible vertical hydraulic barriers are realised till a soil layer with a low permeability. Although vertical hydraulic barriers are realised quite often, there are not so much publications on the quality control of these barriers during and after construction.

When a closed box is created the efficiency of the hydraulic barrier can be checked by means of pumping tests. However the interpretation of the results of such pumping tests is not easy. The obtained permeability of the barrier is strongly influenced by the variation of the storage within the dewatered layers.

Actually new methods are available to check the homogeneity of hydraulic barriers. Although these methods are used quite regularly there is not so much information on results obtained on real projects.

Vertical hydraulic barriers are often used in combination with a permanent drainage system underneath the foundation slab. When a permanent drainage system is used imperfections of the hydraulic barrier have to be treated. When the location and type of the imperfections are known it is generally quite easy to treat them. When the location and type of the imperfections is not known generally long discussions arise between the contractor and the owner on the cost of the treatment. In such cases it is very important to have a good quality control during the execution, f.i to be sure that the barrier is anchored over his full length in the less permeable layer.

When ground layers with a small permeability are only encountered at very large depths, horizontal hydraulic barriers are more and more realised, f.i. by means of je-grouting or injections. During the last decade a lot of research has been performed on the design and the control of such horizontal hydraulic barriers.

2.2.5 Groundwater recharge

Some decades ago recharging of the groundwater was quite often used to limit the environmental impact of a groundwater lowering. During the last decades this method is loosing his interest and more and more hydraulic barriers are installed.

2.2.6 Slope stability

It is well known that the groundwater level and/or the piëzometric head have a great influence on the stability of slopes. Although there is a lot of information available it is not always possible to explain the observed deformations. One of the reasons for that is that in many cases the deformations or instabilities are created by high water pressures that occur in very thin permeable layers, situated between less permeable layers, f.i. small sand layers within loam or clay layers.

High water pressures may occur after periods of heavy rainfall. With the actually available equipment it is very difficult to measure the water pressures within such very thin layers.

2.3 Eurocode 7

In EN 1997-1:2004 par 5.4 is dealing with groundwater lowering. This paragraph has a length of 1 page and the given information is very general. Paragraph 3.3.9.1 is dealing with permeability and consolidation of soils and rocks. Also this paragraph is very general.

In the Designers' Guide to EN 1997-1 by Frank et al. (2004), only halve a page is dealing with dewatering. Here more specific information is given such as:

– it should be checked that the settlements induced by the groundwater lowering do not lead to settlements of neighbouring structures, which may cause damage or impair their serviceability;
– the design of dewatering schemes may very often benefit from the use of the observational method.

3 CONCLUDING REMARKS

When looking to the publications on dewatering of the last decades one has to conclude that the interest for the subject is decreasing. This is also clearly illustrated by the fact that EN 1997-1:2004 contains only some very general informations on dewatering.

The contributions to the discussion session 2.2 confirm that:

- dewatering is still an interesting technique for the realisation of deep excavations, cfr. example of the Opera house in Copenhagen;
- more and more attention is given to the realisation of hydraulic barriers, this in order to limit the environmental impact of groundwater lowering;
- the prediction of settlements due to groundwater lowering remains very difficult;
- pore water pressures have a large influence on the stability of slopes.

The contributions to the discussion session 2.2 also clearly illustrate that there is no or only a very small progress compared to the ECSMFE of 1987.

It is the authors opinion that a real progress can only be realised when:

- the observational method is applied more systematically for the design and execution of groundwater lowering projects;
- the available information from groundwater lowering projects (= permeability, radius of influence, settlements, . . .) is brought together in data bases;
- a risk assessment is performed systematically, based on the available soil information and information from preliminary groundwater lowering projects.

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