INTERNATIONAL SOCIETY FOR
SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING

SIMSG ISSMGE

This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

https://www.issmge.org/publications/online-library

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Panel contribution: Comments on use of performance monitoring for underground works

Contribution à la discussion: Commentaires sur l'utilisation de l'auscultation dans les travaux souterrains

K. Karlsrud – *Norwegian Geotechnical Institute, Norway*

ABSTRACT

This contribution discuses the importance of properly defining the scope of a monitoring program and how necessary it is that this is well understood by all parties involved. It also raises questions about our ability to foresee an impeding failure based on monitoring results. The points made are illustrated by the collapse of a tunnel in Norway.

RÉSUMÉ

Cette contribution discute l'importance de définir de façon adéquate le but et l'étendue d'un programme d'auscultation et la nécessité pour que le programme et son envergure soient bien assimilés par chacun des acteurs dans le projet. La présentation se penche aussi sur notre habilité à prévoir une rupture imminente sur la base des résultats d'auscultation. Les aspects soulevés sont illustrés avec l'effondrement d'un tunnel en Norvège.

INTRODUCTION

Performance monitoring serves an important role in many construction projects and is often a part of a "design-as-you-go" or "observational" approach. It is, however, important to properly define the scope of the monitoring program and the role of the different parties involved (owner, consultant and contractor) in dealing with the results of the monitoring during construction.

KEY TO THE SUCCESS OF A MONITORING PROGRAM

Many performance monitoring programs do not live up to the expectations of the intended users of the results, or to those that finally pay the bill, because of lack of a clearly defined purpose or scope of work. The scope or purpose may be broadly defined by the following 7 levels, partly adopted from Peck (1969) and Karlsrud (1986):

Level A: Verification of the design basis and soundness of the design. This may be warranted when there are significant uncertainties about the interpretation of ground conditions, external loading conditions or design methods adopted.

Level B: Detection of unacceptable performance that may lead to failure or have other serious consequences, in due time for remedial measures to be undertaken.

Level C: To help optimize the design during construction or in other words, deliberate use of "design-as-you go" or "observational" method.

Level D: Documentation of possible influence on surrounding structures. Today this has become a must in almost every construction project to avoid unnecessary claims and litigations from third parties.

Level E: Documentation of the quality of construction. In this respect it is my experience that when a contractor knows that his works will be closely followed by performance monitoring, it will improve the quality of his works.

Level F: Provide data to help improve design procedures for future works of a similar nature.

Level G: To enhance our general understanding of ground response and basic mechanisms. This is in other words more general and long-term R&D related.

In most cases, the scope will be a combination of two or more of the levels quoted above. In any case, it is also vital to the success of a performance monitoring program that it's scope and purpose is well understood by all parties involved, including the client, the designer and the contractor. This may be an obvious point, but it an aspect which should be given more attention. In Norway as well as internationally, there seems to be a tendency for lesser if any, involvement by the consultant or the designer after the contract documents are completed. Furthermore, the client and the contractors seem far more concerned-, and hazel about,- financial and contractual aspects, than they are concerned about the quality and safety of construction.

To be successful, a performance monitoring program at Levels A, B and C, which is strongly directed towards the safety of the works, also requires that the following conditions are fulfilled:

1. The designer has a priori made a realistic prediction of the expected performance
2. Similarly, that the designer has considered consequences of less favourable ground conditions on predicted performance, and carefully, and with an open mind, studied what factors that are really important to the safety of the works. This should include ground parameters as well as well as unexpected defects that could arise during construction.
3. The designer, or someone else hired by the client who has been made thoroughly familiar with all relevant aspects of the design and performance monitoring program, is given continuos access to the data collected, inspects the site on a regular basis, and has the authority to act if unacceptable performance is detected.
4. Possible actions to prevent failure or critical situations have been carefully thought of in advance.
5. If one deliberately work with a large risk of failure, one should also have the equipment for undertaking preventive measures ready at hand. The same applies to use of the "design-as-you-go" or "observational" methods.

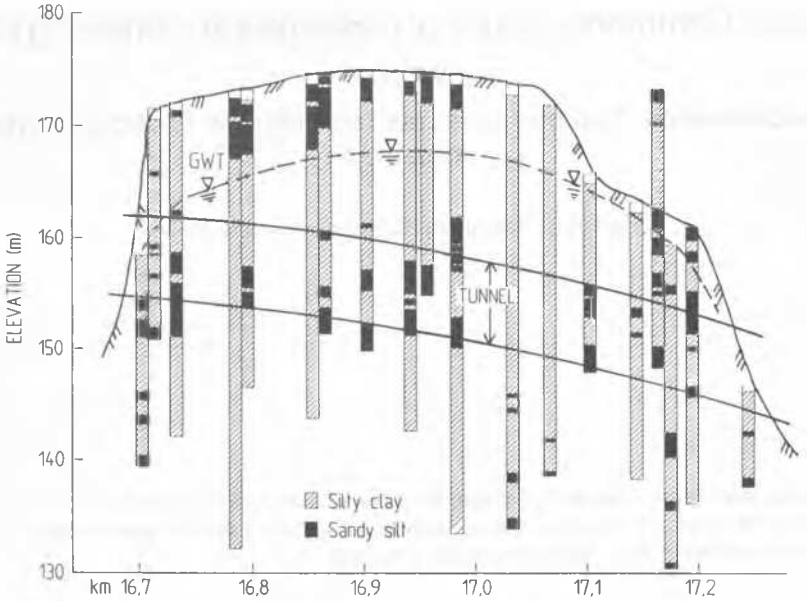

Figure 1 Illustration of ground conditions along the Eidsvoll tunnel

To what extent we actually can foresee a catastrophic failure based on monitoring data is a question that we as a profession should pay more attention to. We may tend to be too optimistic. It is surprising how many failures have occurred very abruptly and with essentially no warning in terms of for instance magnitudes or rates of displacements. On the contrary, there have been many failures where measured displacements prior to failure were quite stable and judged to be at an acceptable level. Reference is for instance made to the numerous failures which have been summarized by HSE (1996) in relation to use of the New Austrian Tunnelling Method (NATM).

One may therefore, question whether we fully appreciate the "brittleness" of the overall system, and that rapid "brittle" failures may occur, not only as a result of the brittleness or strain softening nature of soils or rock masses involved, but also due to "brittleness" of the support system and the possibility of sudden changes in ground conditions or loading conditions.

THE EIDSVOLL TUNNEL EXPERIENCE

A year ago there was a dramatic collapse in a soft ground tunnel in Norway, which will be used to illustrate some of the points made above.

The Eidsvoll tunnel as it is called, is a 440 m long single track rail tunnel (Fig. 1). The depth from the ground surface to the tunnel crown is 13 to 17 m, apart from near the entrance and exit points. The ground conditions at the site are illustrated in Fig. 1. Briefly described it consists of a medium stiff marine clay deposit with meter- thick inclusions of non-cohesive fine sandy silt. These silt inclusions showed a very erratic distribution both horizontally and vertically, but made on average up about 25 % of the total soil volume along the tunnel. The ground water table was 4 to 7 m below the surface, but there was an underpressure compared to a hydrostatic distribution with depth, which corresponded to about 3 to 5 m of head water at the tunnel level. The marine clay has an undrained shear strength which increase from typically 50-60 kPa at the crown to 80-90 kPa at the invert of the tunnel.

NGI was engaged by the client to undertake the feasibility study for the tunnel. As part of that study, it was considered a wide range of tunnelling concepts, ranging from use of various forms of tunnelling shields, concepts based on establishing a

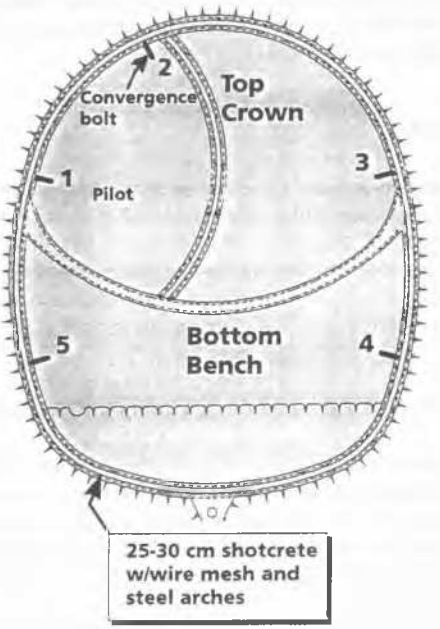

Figure 2 Tunnel cross section with construction sequence

support ring by micro-tunnels, ground freezing or jet- grouting, as well as the so-called NATM concept. NGI ended up recommending use of an open gate shield, but several of the other concepts could also be technically and economically feasible. One concept that was specifically turned down, however, and regarded as not feasible under the prevailing ground conditions, was use of the NATM concept. Without going into details, this conclusion regarding the feasibility of NATM was not so much based on rational analyses, but more on "engineering judgement". It was felt that the combination of relatively soft clay and waterbearing silt was not a good case for a traditional NATM approach, and face stability as well as overall stability problems were foreseen.

Following this feasibility study, the client decided to let the contract out on a turn-key basis, but the contractor's had to specify their choice of tunnelling method. That was also the end of NGI's engagement for the client. The lowest bid came from a

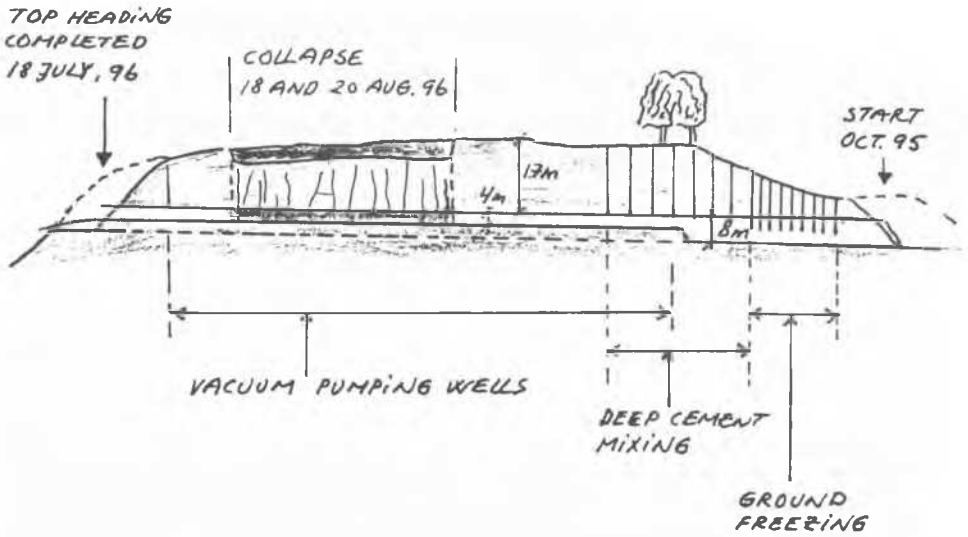

Figure 3 Ground improvement and collapse along the Eidsvoll tunnel

group which based itself on use of the NATM concept, which was accepted by the client.

The chosen concept implied dividing the tunnel excavation into a top and bottom heading, but the top heading was also divided in two (Fig. 2). The tunnel was to be temporary supported by a 30 cm thick mesh-reinforced sprayed concrete lining with steel arches. Excavation and temporary lining was carried out in sequences of 80 cm at a time.

Very shortly after they started driving the pilot and top heading, collapses occurred in the unsupported crown area by intrusion of water-bearing silt. The collapses eventually extended right to the ground surface. Various stabilizing schemes were tried and implemented during the subsequent driving of the top heading (Fig. 3). First ground freezing an umbrella from the surface, then stabilization by the deep cement mixing method, also from ground surface, and finally by installing vacuum pumping wells every 8 m along each side of the tunnel. These wells were put successively in operation over a distance 15 m ahead of the tunnel face and 10 m behind, and turned out to be quite successful in relieving the pore pressure in the silt layers and preventing collapse before the temporary lining was in place.

The real disaster occurred, however, about one month after the top heading was completed along the entire tunnel length. First, on 18 August, 1996, the top heading collapsed over a length of 80 m, then two days another 50 m collapsed (Fig. 3).

The collapses seemed to occur without any warning from the performance monitoring system. This system consisted primarily of convergence measurements in the roof and walls of the temporary lining (Fig. 2), which have been reported to show that displacements had stabilized completely one to two weeks after the tunnel had advanced beyond a given point (Fig. 3). There were also other measurements taken, like surface settlement and pore pressures, but the extent and results have not been reported so far.

The client, contractor and insurance companies have of course initiated some studies as to the causes of the collapse. No official explanations have been given so far, but the contractors consultant reported at an open meeting that they believed that the failure primarily was a result of the invert of the tunnel being flatter than intended, and the curvature in the transition between invert and walls to sharp. This conclusion was based on extensive parametric finite element analyses carried out after the collapse and which revealed that the bending stresses in the mid span of the invert and the transition to the walls were very sensitive to the exact geometric shape in those areas. The observations after the failure that the top part of the tunnel was more or less intact, tends to support the conclusion that it was the bottom invert that actually failed first.

Without speculating further about the possible causes of the collapse, this case emphasizes the following aspects, also seen in light of the earlier comments on use of performance monitoring:

- The potential for a collapse of the form that actually occurred was apparently not envisaged, as evidenced by lack of measurements of displacements and/or other measurements in the top heading invert area. In other words, the design team had not found in advance that this could be a critical failure mechanism.
- If the design was very sensitive to the exact shape of the invert, that aspect should have been given special attention during design, and not least, during construction.
- The collapse seems to have been of the very fast and "brittle" kind, which in general should require relatively large safety margins, especially on critical elements, like in this case-, the temporary lining. As a side remark, the temporary lining needed to stand up for many months, and thus may be considered permanent in relation to safety aspects.

When or if all data becomes publically available, there may also be quite a few other lessons to learn from this collapse. It was indeed good fortune that the collapse occurred on a Sunday, with nobody in the tunnel.

REFERENCES

HSE 1996. Safety of New Austrian Tunnelling Method (NATM) tunnels. Published by *Health and Safety Executive, U.K.*

Karlsrud, K. 1986. Performance monitoring of deep supported excavations in soft clay. *Proc. 4th Int. Geot. Seminar on Field Instrumentation and In Situ Measurements*. Publ. by Nanyang Technological Institute, Singapore.

Peck, R.B. 1969. Advantages and limitations of the observational method applied in soil mechanics. *9th Rankine Lecture, Geotechnique, 19, 2, pp. 171-187.*

Peck, R.B. 1980. Where has all the judgement gone? *The 5th Laurits Bjerrum Memorial Lecture*. Printed by Norwegian Geotechnical Institute, Oslo, Norway 1980.