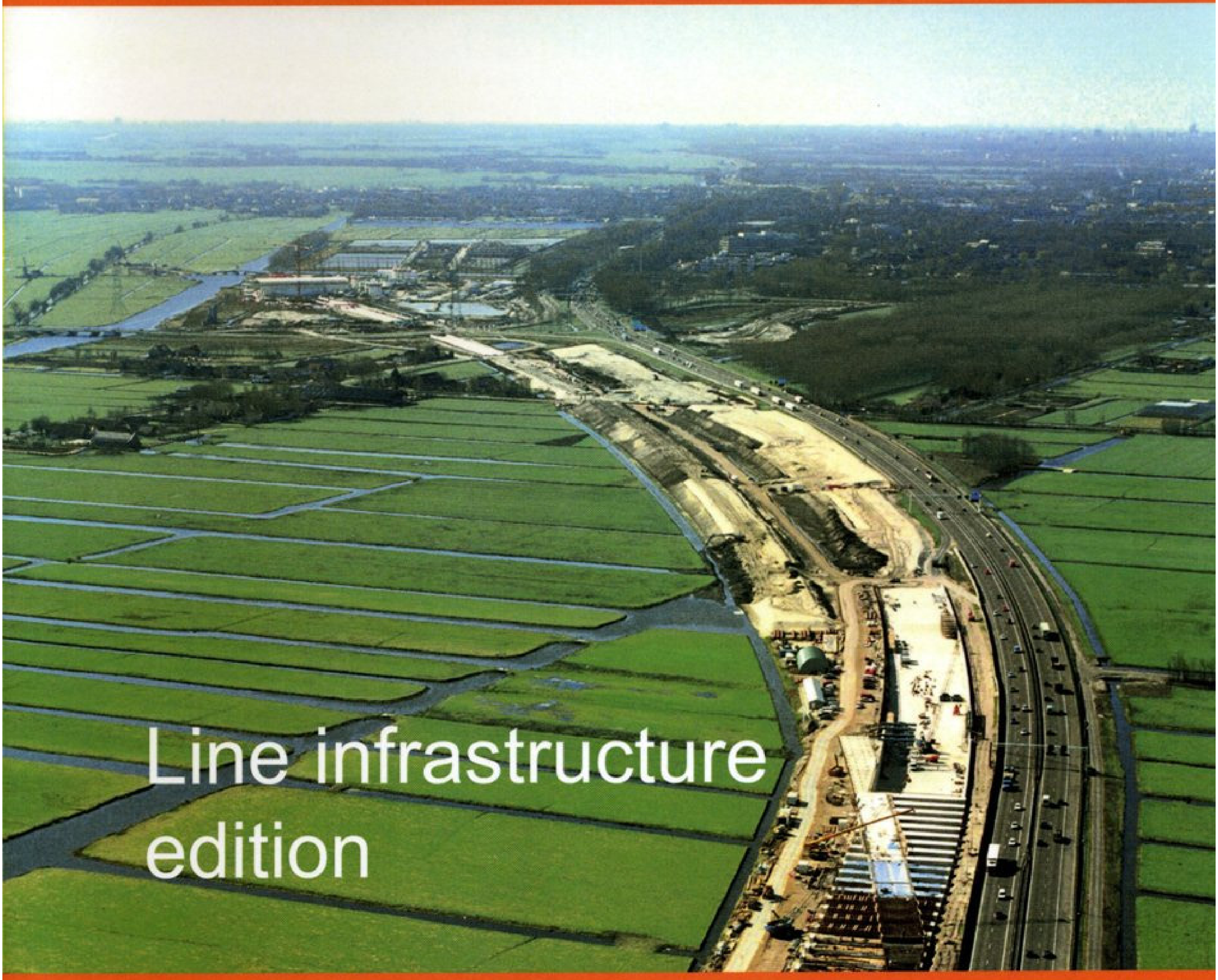
news ingeo letter

nl-iaeg ingeo kring

Vol. 10 No. 2 2003



- BeauDrain: a new system to accelerate the consolidation process
- Engineering geology in the Belgian karst belt
- The Bosporus railway tube crossing
- Geologisch model van project Spoorzone Delft

Editorial

Beste lezers,

In deze editie van de Ingeokring Newsletter staat het thema “Lijninfrastructuur” centraal. Nederland is momenteel toneel van grote (veelal lijnvormige) infrastructurele werken. De aanleg van de hogesnelheidslijn (HSL) en de Betuweroute zijn bij velen het eerste wat opkomt, er zijn echter nog talloze andere projecten. Snelwegen worden verbreed, nieuwe wegen aangelegd en tunnels worden geboord onder stedelijke gebieden. In andere delen van Europa wordt ook aan het spoor- en wegennet gewerkt. Een bijdrage uit België behandelt de aanleg van de hogesnelheidslijn door karstgebieden. Fugro heeft een artikel over de aanleg van een treintunnel onder de Bospurus geleverd. Bijdragen uit Nederland zijn BeauDrain, spoortunnel Delft en foto’s van Betuwelijn, HSL en A50. Vele ingenieursgeologen hebben in hun werkterrein te maken met lijnvormige infrastructuur, waardoor dit thema actueel blijft!

Naast de artikelen in het kader van “Lijninfrastructuur” bevat deze editie natuurlijk ook de vaste rubriek Ingeokring activiteiten. Er wordt o.a. bericht over de door Ingeokring georganiseerde lezing over de Haagse tramtunnel. De laatste ontwikkelingen op het gebied van de opleiding Ingenieursgeologie worden behandeld door Richard Rijkers en namens DIG door Jordy Mollé. De professoren column, ditmaal verzorgd door professor Kroonenberg, en Eningeering Geologists Abroad, deze keer geschreven door Marco Silvestre, worden met succes gecontinueerd!

Wij bedanken de auteurs voor hun bijdrage in deze 2e Newsletter van 2003 en wensen allen veel leesplezier!

Xander van Beusekom, Bart Fellinga, Gerben Groenewegen, Jacco Haasnoot en Robert Vuurens

From the chairman of the Ingeokring

Voor u ligt alweer de derde editie van onze vernieuwde Newsletter. Deze editie is samengesteld rond het thema ‘’Lijninfrastructuur’. Met het 1st European Regional IAEG Conference *Professional Practices and Engineering Geological Methods in European infrastructure Projects* in het vooruitzicht, vormt dit themanummer een stevige opwarmer voor deze conferentie in Luik. De aanleg van vele interessante projecten voor nieuwe wegen, spoorlijnen en tunnels vraagt de aandacht van ingenieursgeologen en toont aan dat de ondergrond aan interesse wint.

Het lijkt erop dat de ondergrond niet langer alleen vragen en problemen oplevert, maar ook oplossingen kan bieden bij technisch complexe onderwerpen. De functionaliteit en waarde van de bodem is niet alleen het (slappe) fundament, maar de ondergrond is ook een bron voor drinkwater, grondstoffen, energie en ….. ruimte. Het behoort tot de taken van een (ingenieurs-)geoloog zorg te dragen voor een duurzaam gebruik van deze bodem, immers we hebben er maar één. In 2003 kan er nog niet gesproken worden over een schaarste van de ruimte in de ondergrond, maar gelet op de bodemvervuiling van de jaren ’60-’80, de ondergrondse drukte in Japan en de recente berichtgeving over het Nederlandse kabels en leidingen netwerk in de Nederlandse kranten, moeten we wél op onze hoede zijn.
Ik zou hierbij willen stellen dat Nederland in een maatschappelijke fase is gekomen waarin de Ruimtelijke Ordening de eigenschapen van de ondergrond behoort mee te nemen bij de planvorming. Wat zijn de gevolgen en beperkingen van de aanleg van (ondergrondse) infrastructuur op de lange termijn? Wat zijn de consequenties en hoe kunnen we duurzaam omgaan met de ‘onderste laag’? Om antwoord te kunnen geven op deze vragen zal de komende jaren er intensief samengewerkt gaan worden tussen planners, geologen en civiel-technici.

We gaan roerige tijden tegemoet. De economie draait op een laag pitje en het voortbestaan van de universitaire onderwijs- en onderzoeksgroep Ingenieursgeologie TU-Delft staat nog steeds op de tocht. Dat noem ik inderdaad in één adem, want het verlies van de enige fundamentele plek waar universitair onderwijs in Ingenieursgeologie een concrete plaats had nadert. Meer maatregelen zijn aangekondigd omdat de tekortschietende derde geldstroom en de zeer lage studentenaantallen voor de TU-Delft een steeds steviger argument zijn om het onderwijs in aardwetenschappen nog verder onder druk te zetten. Echter, ik weiger te geloven dat de basis van de Nederlandse kenniseconomie draait op marktfinanciering. Continuïteit is niet gewaarborgd, en dat blijkt.

Na dankzegging aan de zeer gewaardeerde voltallige redactie van deze Newsletter, wens ik u veel leesplezier en ……….…
“beware, be prepared and be wise”.

drs Richard H.B. Rijkers
Voorzitter INGEOkring

LINE INFRASTRUCTURE

BeauDrain: A new system to accelerate the consolidation process

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This article was recently published in HANSA 140. Jahrgang - 2003 - Nr. 5

Introduction

Most of the large harbours and industrial areas in the world are located in coastal plains or deltaic areas, near rivers and other waterways. Although these locations may be favourable from an economic point of view, they are often dominated by poor subsoil conditions. Highly compressible fine grained sediments such as clays, silts and peat, generally have low shear strengths and cause settlement and stability problems when loaded by structures like roads, dikes, fills, buildings etc. Current tendencies in the construction industry to reduce the construction time and, hence, the return-on-investment period, have stimulated the development of new techniques to either accelerate the consolidation process or reduce the absolute settlements in compressible soils.
Systems to accelerate the consolidation process are all based on 2 principles: reduction of the length of the drainage path of the pore water and/or the application of a surcharge. Reduction of settlements is generally achieved by either reducing the compressibility of the cohesive strata (block stabilization) or concentrating the bearing loads onto stiff elements like piles or columns that transfer these loads to underlying, more competent strata. Some techniques combine the acceleration of the consolidation with the use of stiff elements. This paper will focus on a new system called BeauDrain that combines an innovative installation procedure with the proven technique of vacuum consolidation.
With the reduction of the consolidation period, it becomes increasingly important to monitor the development of the settlements with time and to accurately predict the final settlement in an early stage of the consolidation process as the time for corrective measures is generally limited.
The method as proposed by Asaoka [1] will be explained and used to interpret the settlement data gathered in an early stage of consolidation on a test section known as Zevenhuizen near Rotterdam, The Netherlands.

Acceleration of the consolidation process

The degree of consolidation as a function of the time depends on the stiffness, the permeability and the length of the drainage path of the dissipating pore water of the compressible strata. Although other relations exist this may be expressed as:

$$U = \frac{T_v^{\,3}}{\sqrt[6]{T_v^{\,3} + 0.5}} \qquad [1]$$

in which:

U = degree of consolidation [-]
T_v = timefactor [-]

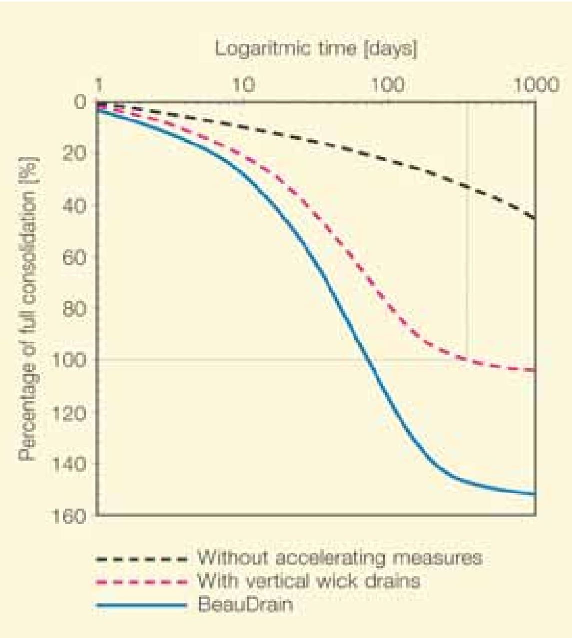

Figure 1: Theoretical comparison time settlement curves

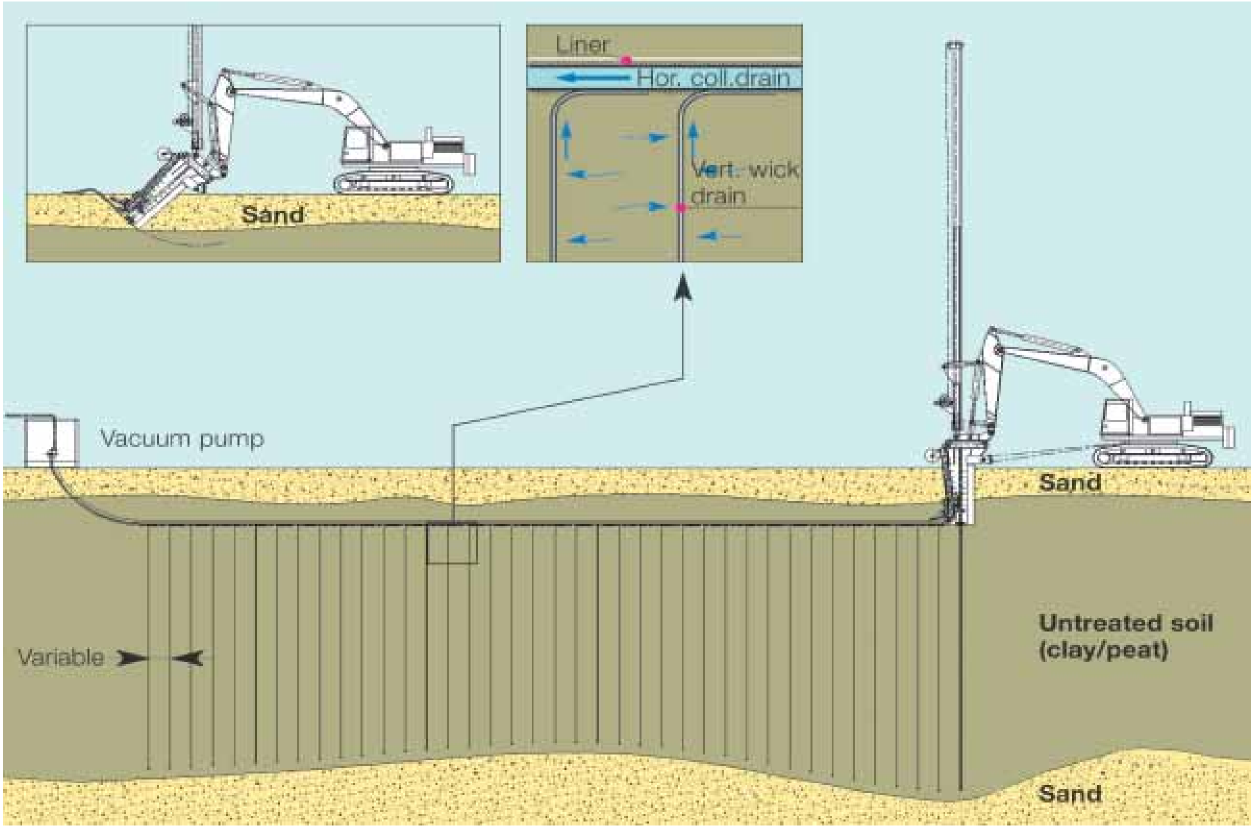

Figure 2: Installation BeauDrain system

The following formula relates the time factor T_v to the length of the drainage path of the pore water, the consolidation time and the consolidation coefficient:

$$T_v = \frac{c_v \, t}{H^2} \qquad [2]$$

in which:

c_v = consolidation coefficient [m^2/s]
H = length of drainage path of the porewater [m]
t = consolidation time [s]

Formula [2] demonstrates the influence of the length of the drainage path on the time factor and, hence, on the consolidation period. Reduction of the length of the drainage path, usually equal to the thickness (or 0.5 * thickness in case of two-sided drainage), can be achieved by installing vertical drains or other draining elements.
As follows from the formulae above, applying a surcharge does not accelerate the consolidation process, but results in a larger settlement in the same consolidation period. This implies that the required settlement can be reached in a shorter period, after which the surcharge has to be removed to avoid more settlement than strictly required. Figure 1 depicts the effects of both reducing the length of the drainage path and surcharging in combination with a reduction of the drainage length.

BeauDrain System

Most systems developed to accelerate the consolidation process use both some kind of a drainage system and a surcharge. Some methods (partially) apply the atmospheric pressure as a surcharge by reducing the pressure in the compressible strata. In its most simple form, this method of vacuum consolidation comprises a system of vertical drains and a drainage layer (usually sand) at the top, sealed from the atmosphere by a geomembrane placed at the surface. Horizontal drains installed in the drainage layer and connected to pumps in combination with the vertical drains remove the pore water from the compressible strata and reduce the atmospheric pressure in these layers.
The BeauDrain system is a recently developed technique that is based, more or less, on the same principles, but without its cumbersome intallation procedures. Through a specially designed plough that is pulled by a hydraulic crane, prefabricated vertical (wick) drains are installed and cut at predefined depths below ground level. While the plough is moving a horizontal collection drain is placed at a depth of approx. 3 m be-

Figure 3: BeauDrain installation

Figure 4: BeauDrain plough

low ground surface and is connected to the vertical drain. Before it leaves the plough, the horizontal drain is also covered by an impervious geomembrane in order to ensure a proper sealing between the horizontal drain and the atmospheric conditions. The whole system, which is usually referred to as a drainage curtain, consists of a row of vertical drains, a horizontal drain and seal. It is placed in a single pass of the plough. Figure 2 illustrates the installation of the system. After passage of the plough the compressible soil usually closes in on itself above the horizontal drain creating a natural seal additional to the geomembrane. The total system consists of a number of drainage curtains connected to vacuum pumps. The crane with plough is depicted in Figures 3 and 4.

The vacuum measured at the pumps generally varies between 80 kPa and 90 kPa (0.8 – 0.9 bar). Depending on the height difference between the pump and the horizontal drain, this usually results in a reduced pressure of approx. 50 - 60 kPa in the horizontal drain. This vacuum pressure of 50 - 60 kPa, corresponding to a surcharge equivalent of approx. 3.5 m (dry) sand, acts on the compressible strata as a load. However, unlike physical loads, it does not introduce shear stresses in the subsoil as a result of its isotropic character. It will, therefore, not cause instabilities. Moreover, it will, with ongoing consolidation, gradually increase the effective stress and, hence, increase the shear strength of the subsoil. Figure 5 clearly shows the increase of the effective stress as a result of the reduced atmospheric pressure in the soil mass. The net effect of this is an additional surcharge, which will ensure an early attainment of the required settlement, and an increased shear strength which will favour the stability (accelerated loading schemes, steeper slopes in areas with limited space).

Figure 6 shows settlement readings gathered in 2 adjacent test sections of the test site Zevenhuizen, one equipped with the conventional vertical wick drains, the other with the BeauDrain system. In both sections the initial sand surcharge equaled 1.5 m. After 100 days the thickness of this sand layer in both areas was increased to 3 m. Figure 7 gives an overview of the test area. The transition between the settled BeauDrain section (left) and the area with conventional vertical drains (right) is clearly visible. Although the type, length and spacing of the vertical drains and the thickness of the sand surcharges were identical in both sections, it is

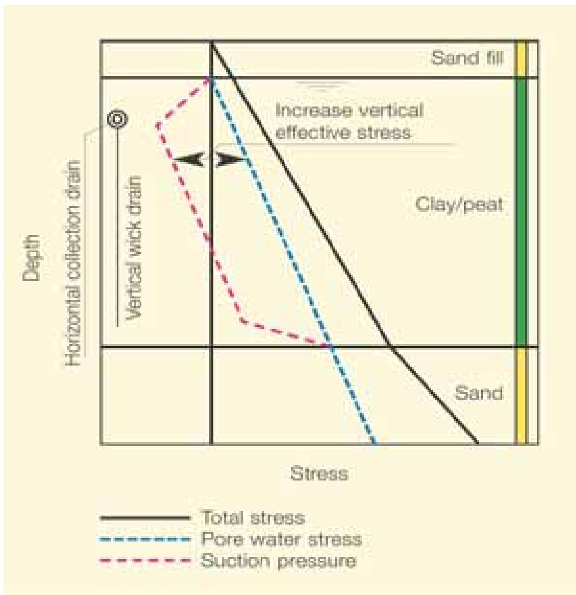

Figure 5: Increase of vertical effective stress during vacuum consolidation

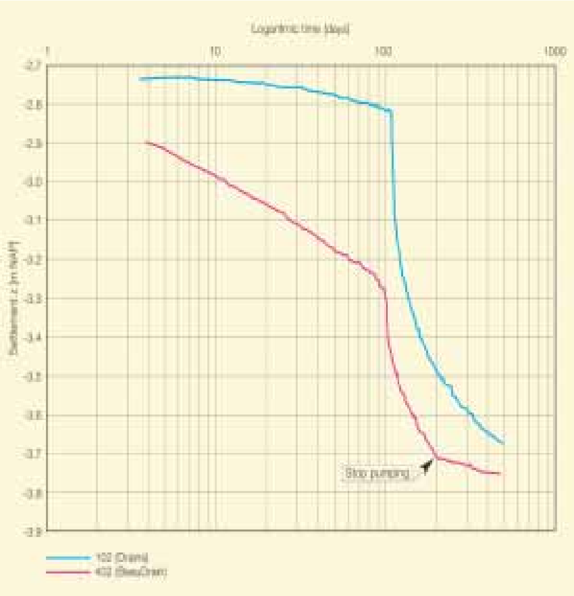

Figure 6: Test site Zevenhuizen: observed settlements of bacon 102 and 402

clear that the vacuum created by the pumping has accelerated the consolidation considerably. A back analysis suggests that approx. 50% of the total atmospheric pressure, i.e. 50 kPa had been mobilized as an additional surcharge. The pumps were stopped 206 days after the start, but because full consolidation had not yet been reached, settlement continued, albeit at a lower rate.

Asaoka Method

To avoid more settlement than strictly required, it is important to remove any surcharge in time. In case of the BeauDrain system this implies that the pumps must be stopped in time. Given the short consolidation period this means that at a relatively early stage of consolidation the final settlement should be known.
Asaoka [1] has proposed a simple method to predict the final settlement based on settlement observations at fixed time intervals. By plotting consecutive readings z(t) against z(t+1) a line will be obtained which, over a large interval, can be represented by the linear function:

$$z_{t+1} = \hat{a} * z_t + A \qquad [3]$$

in which:

â = slope of the linear section of the best fit [-]
A = intersection of the extrapolated section of the linear fit with the Y-axis

A few so-called Asaoka lines, representing various loading stages of a reclamation project, have been depicted in Figure 8.
The intersection point of the extrapolated section of this straight line and the line z(t)=z(t+1) will define the total final settlement at the moment full consolidation has been reached:

$$z_{100\%} = \frac{A}{(1-\beta)} \qquad [4]$$

The tangent of the plotted line can be related to the equivalent consolidation coefficient c_{eq} (consolidation coefficient accounting for the joint effect of the horizontal and vertical drainage of pore water) by applying the following formula:

$$c_{eq} = \frac{-5H^2 \ln\beta}{12\Delta t} \qquad [5]$$

in which:

H = length of drainage path [m]
Δt = time interval [s]

As a result of both vertical and horizontal drainage, the

Figure 7: Test site Zevenhuizen: overview

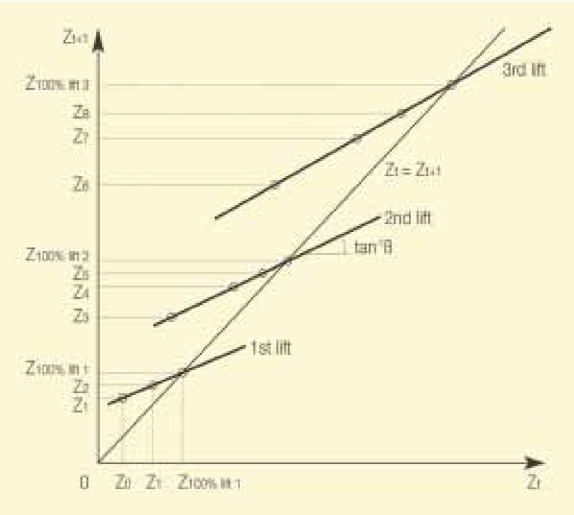

Figure 8: Asaoka lines of different loading stages

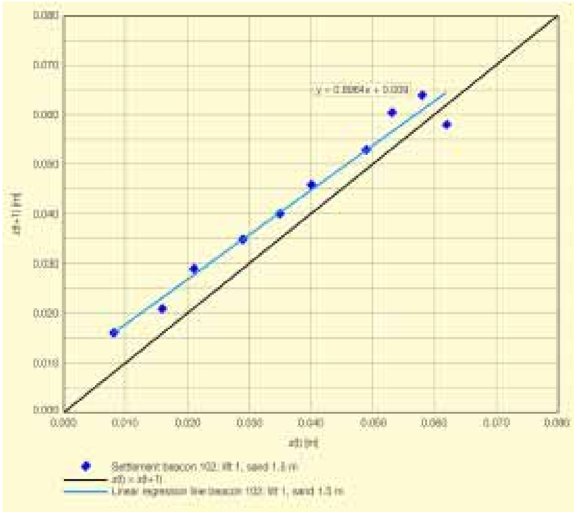

Figure 9: Test site Zevenhuizen: Asaoka line of settlement beacon 102, lift 1

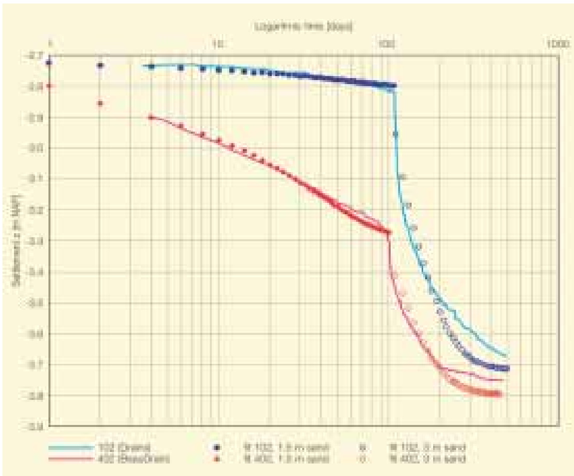

Figure 10: Test site Zevenhuizen: observed settlements and predicted settlements with Asaoka

$$U(t) = \frac{T_{eq}{}^{3}}{\sqrt[6]{T_{eq}{}^{3} + 0.5}}$$

in which

$$T_{eq} = \frac{c_{eq}\, t}{H^2}$$

In Figure 10, the settlements predicted by the Asaoke method for both lifts and both sections are compared with the actual settlement readings (settlement beacons 102 and 402 represent the conventional vertical drains and BeauDrain section respectively). It can be concluded that the calculated settlements, based on monitoring data of the initial stage of consolidation, correspond remarkably well with the field observations. The discrepancy between the rates of the predicted and observed settlements of the second lift of the section with the vertical drains (beacon 102) is attributed to a decreasing consolidation factor with an increasing effective stress.

Based on the final settlements of both the Beaudrain section and the section with conventional drains and the compressibility of the subsoil back analyses indicate that

the atmospheric pressure acting as a surcharge equals approximately 50 – 60 kPa.

Conclusions

The BeauDrain system combines the principle of the vacuum consolidation with a continuous and clean installation operation. Back calculation of various projects indicates that a surcharge of approx. 50 - 60 kPa of atmospheric pressure at the level of the horizontal drain can be mobilized by pumping. The isotropic nature of this load causes an isotropic volume reduction of the treated soil resulting in an increase of the effective stress without raising the shear stress level. This will result in a rise of the shear strength of the subsoil allowing for higher loading rates and will reduce long term settlements.

The Asaoke method proves to be a simple, powerful and reasonably accurate tool to predict the consolidation factor and the final settlement in a relatively early stage of consolidation. To improve the accuracy consolidation factors should be determined over limited intervals of the effective stress.

References

[1] Asaoka, A.: Observational procedure of settlement prediction. Soils and Foundations, (1978), Vol. 18, No.4, Japanese Society of Soil Mechanics and Foundation Engineering

LINE INFRASTRUCTURE

Line infrastructure and the role of engineering geology in the Belgian karst belt

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Introduction

Karst is in Belgium not a local feature but extends all over the country. This underground richness, or rather emptiness, causes problems when line infrastructure like railroad or highway networks are extended. Although the geology is extremely well known it still can surprise as the great construction waves of highways after the Second World War and at present times during the construction of the TGV railroads have shown. Problems related to dams in karst are especially spectacular as was discussed previously in this newsletter (Schmitz and Gyaltsen 2001). But in this contribution we will discus the problematic of line infrastructure confirm the title of this issue showing some examples of railroad, highway and tunnel construction in Belgium.

Engineering geological definition of karst

In engineering geological applications we are not primarily interested in nice caves open to mammals neither of the order of Primates nor Chiroptera. For engineering geologists the importance of karst is: any dissolution void in rock mass that represents a hydraulic or mechanical discontinuity. This is close to the definition by Richter (1989): karst involves all recent and fossil leaching and dissolution processes and the resulting surface or subsurface features (e.g. cavities or cave-ins) present in rocks prone to dissolution or leaching as well as in the overburden. Note that unlike other weathering phenomena, which show a gradual transition, karstified and sane bedrock can coexist next to each other (Höwig et al. 2003).

Where can we find karst in general

In contrast with the surface water the geological work of the subsurface water is chiefly chemical in character (Schultz & Cleaves 1955) and chemical weathering is not restricted to easily soluble rocks but attacks all rock types. The most easily weathered are limestones, of greater resistance are sandstones and shales. Igneous rocks (excluding certain volcanic rocks that weather rapidly) and quartzites are the most resistant (Blyth & de Freitas 1984) but karst landforms even develop on highly siliceous rocks (Selby 1993). Thus karst landforms developed on limestone are only the best known.

Where can we find karst in Belgium?

One method to analyse the extent of karst is to start mapping outcrops of the lithologies prone to dissolution. In Belgium the area that could be affected by karst is restricted predominantly to Wallonia. In Wallonia the karst is restricted predominantly to Palaeozoic rocks[1]. In these Palaeozoic rocks the karst is restricted to those that consist of limestone and dolomite but most karst can be found in the limestone (Ek 1996). The occurrence of the outcrops[2] of these formations is shown in figure 1. This contribution will, on the basis of the karst features in Belgium, discuss only the carbonate karst. Brussels the largest city in Belgium, Antwerp the largest in Flanders and Liege the largest in Wallonia in fact 94% of the population of Belgium is fenced in by the karst belt, shown in figure 1, starting in the west near Tournai via Mons and Charleroi to the German border, then it turns north between Liege and the Netherlands border but here in the subsurface (therefore not indicated on the map). When one wants to connect cities to Germany or France one has unavoidably to pass this border.

At which depth can we find karst?

For engineering purposes it is of course important to know at or to which depth karst still can be encountered. Therefore we need to know when the karst was weathered.

[1] Karst features outside Wallonia can be found in the Cretaceous rocks NW of Liege.

[2] The occurrence of these Palaeozoic rocks should be considered in subcrop maps up to 50m depth from the surface. These subcrop maps are at present not available. It should therefore be acknowledged that the actual extend of the karst belt is even larger. An example is given by Calembert & Monjoie (1970) discussing the problems related to the construction of the highway Liege to the Netherlands related to karst in Palaeozoic rocks north of Liege.

During engineering construction?

The solution of limestone is a relatively rapid process in a geological sense but very slow with respect to the durability of the structure. A dam site is not endangered by solution of limestone in a valley floor or flanks since concrete is generally more soluble than most limestone. Cavities filled with clayey material may let through only a little water, but its amount may increase progressively as filling is washed out from the cavities. In some cases this phenomenon has been explained erroneously as being due to rapid enlargement of caverns by the solution of limestone (Zaruba & Mencl 1976) the fact that karst cavities encountered during tunnelling are often stable and old in engineering terms can be appreciated when stalactites and stalagmites are present (John and Strappler 2003). Carbonate karst is thus not formed during construction, but it is and remains tricky to construct in or on karst. By construction the hydrogeological system can be changed but rather by changing the consistency of the plugs in karst voids than by dissolution of carbonate rocks.

The karst that is encountered must be older and commenced before engineering construction started on site. How old?

Paleokarst

As we have seen karst in Belgium is dominantly related to the karst of Palaeozoic limestone. The Palaeozoic limestone is of Carboniferous age, in Wallonia represented by e.g. the Visean, Namurian and Westphalian[1]. The contact between the Visean and the Namurian, stratigraphically a normal contact (Bouhenni 2003), corresponds on the regional scale to an old erosional surface with important karstifiacation of the limestone of the Visean (Couchard et al. 1994), the karst is filled in by Namurian sediments (Bouhenni 2003). The geotechnical properties of the infill material of the Palaeozoic karst on the contact Visean - Namurian is very heterogeneous: silt, sand, weathered shale, clays, breccia etc. (Calembert 1975). Then the Westphalian, Namurian and Visean have been extensively folded and faulted during the Variscan orogeny (Bouhenni 2003).

[1] Note that it was this Westphalian coal that outcrops in the town centre of Liege and the proximity to the Ardennes that delivered the ore, that the independency of the principality of Liege was founded in the middle ages and maintained during 800 years. Coal seams in the Namurian have never been mined because of they are too thin.

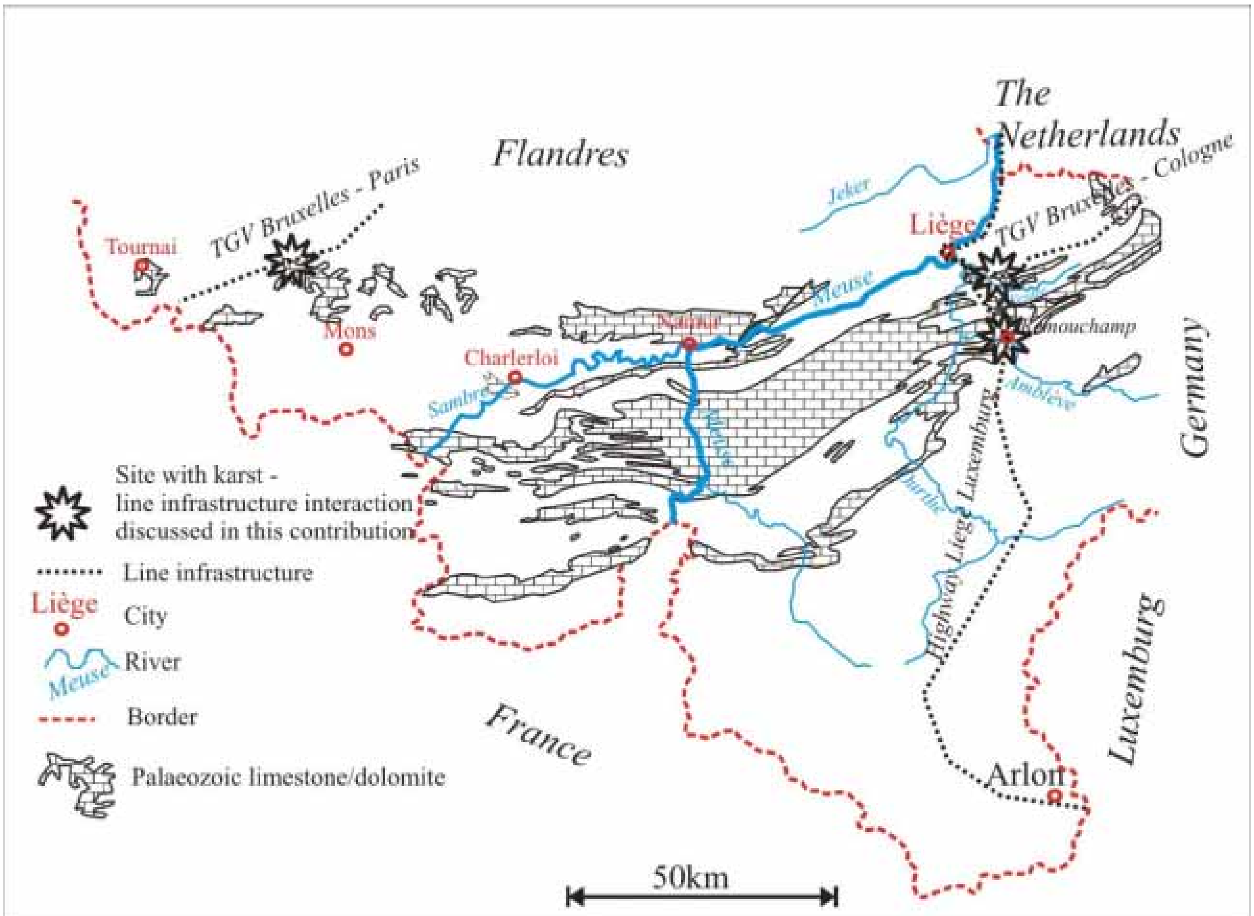

Figure 1. Palaeozoic limestone, which is located in Wallonia, contains most karst in Belgium.

The Palaeozoic limestone and dolomite have been karstified during the erosion of the Variscan mountain range (Calembert & Monjoie 1970). This paleokarst can thus be found at any level where the Visean meets the Namurian. This paleokarst was encountered during the construction of the Soumagne tunnel discussed below.

Mesozoic karst
Mesozoic karst can be found in the Palaeozoic carbonated in south western part of Belgium. These carbonates are like the carbonates described above of Carboniferous age but a little older and were deposited during the Tournaissian. Tournaissian limestones were highly karstified during the Lias[4] and are filled by Tertiary sediments (Schmitz 2004). This Mesozoic karst was encountered during the construction of the Arbre railroad viaduct, as will be highlighted later in this contribution.

Recent karst: Cainozoic
The Ardennes massif is currently uplifting. This means that rivers are incising into their beds. Thereby steep valleys are formed and the regional groundwater level decreases. Thereby karst proceeds downwards too, leaving caves higher up into the terrain dry. These are then invaded by speleologists and bats. Karst can be found to a depth of 60m below the actual riverbeds (Fetter 1994). This Cainozoic karst was encountered during the construction of the Remouchamp highway viaduct, discussed later on.

Karst and Engineering geology: site investigation
Besides boreholes with core recovery, karst morphological mapping and aerial imagery (Höwing et al. 2003) a whole suit of geophysical methods is used to reveal karst on the construction site:
refraction seismics
- lower velocity could indicate karst (Prinz 1997)
- seldom efficient to find cavities not useable in urbanised region (Monjoie 1975)
- localisation of individual voids not possible (Prinz 1997)
- good to detect filled in dolines (Bender 1984)

reflection seismics
- when diameter void is 2 to 3 times the overburden individual voids can be detected under favourable conditions (Prinz 1997)

seismic tomography
- ideal for near surface voids, if these voids are filled it is not evident to detect them (Prinz 1997)

geoelectric resistivity
- is possible to detect covered cave-ins (Prinz 1997)
- overburden must not be too large (Bender 1984)
- seldom efficient to find cavities not useable in urbanised region (Monjoie 1975)

georadar
- above groundwater table karst voids larger than 1m diameter can be discerned u to a depth of 30m (Prinz 1997) but does not work when a clay cover is present

electro-magnetic
- EM mapping is suitable to map karst discontinuities (Vogelsang 1998) (Monjoie 1975, 1979)

microgravimetry
- microgravimetry allows not only to locate empty voids but as well karst features that haven't been developed completely (Monjoie et al. 1985) if used in a dense spacing and correctly applied it is the best tool (Monjoie 1975) if not too much corrections for topography and complex geology have to be made (Waltham et al. 1986)

echo log
- can be used to determine the geometry of the karst void once a borehole has been drilled into the karst (Prinz 1997)

optical probing
- only useful above water table (Bender 1984)

On the basis of the knowledge gained during site investigation:
- the different karst features should be identifiable
- the karstified rock mass should be subdivided in zones with similar karst features
- typical scenarios for the tunnel driving through a specific karst feature should be described
- a catalogues describing suitable measures how to deal with a specific karst scenario when encountered during tunnelling should be prepared (Höwing et al. 2003)
- if the precise location or form of karst occurrences cannot be given with the help of the site investigation programme before construction then it should be insisted that the site investigation should continue during the construction phase as well.

TGV connection towards Germany: Karst in the Tunnel de Soumagne [5]

Introduction
The major project of the Belgian rail way is at the moment the construction of a new high-speed railroad from Brussels via Liege and Aachen to Cologne. The track Brussels - Liege has been inaugurated 2003 and construction is now focussed on the section Liege - Aachen. To be able to maintain a velocity of 300km/h the path of the existing rather curved railroad will not be followed. Unavoidably a tunnel needs to be constructed to pass through some hills on the way from Liege to the plateau de Herve. This tunnel is the Soumagne tunnel.

[4] *Personnel communication Prof. Thorez, Université de Liège*

[5] To follow the adavancement of the Soumagne tunnel on day to day basis: *http://www.tucrail.com/hslnet/fr/soumagne2.asp*

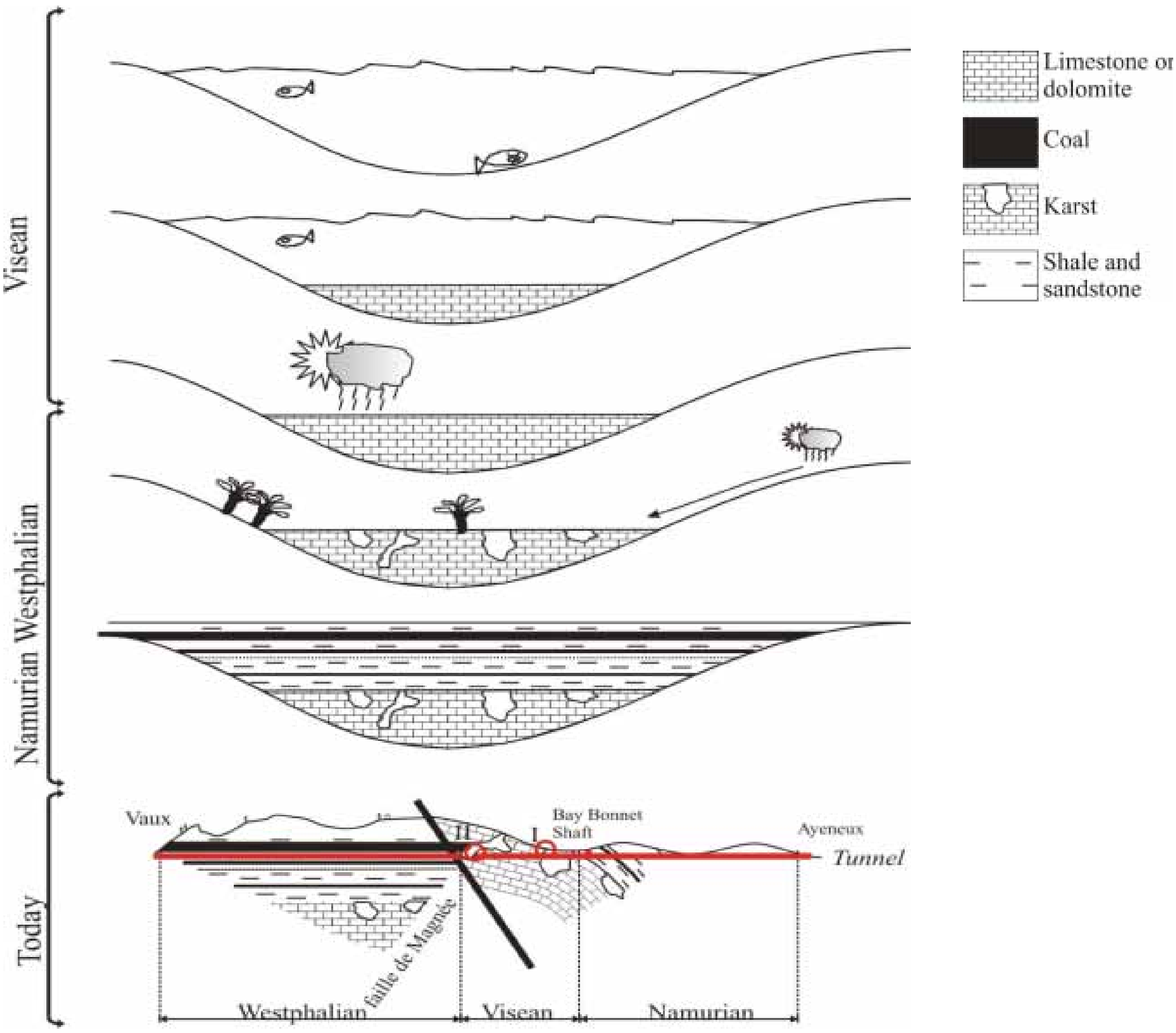

Figure 2. A simplified geological draft of the geology of the Soumagne tunnel. Due to tectonic disturbance the tunnel remains close to the Visean-Namurian interface thus within the paeokarst. Not indicated in this section is the faulting and folding of the coal and schist layers of the Westphalian.

Karst near Soumagne

The karst that is encountered near Soumagne is paleokarst. Well known and feared (Couchard et al. 1994) is the contact between the Namurian (Carboniferous; coal and shales) and the Visean (Carboniferous; limestone). This contact is feared because it contains a paleokarst on the top of the Visean. In the Soumagne tunnel this contact is crossed. As if this isn't bad due to the local tectonic setting the tunnel remains close to the former (a large part of the Namurain has been weathered away) Visean - Namurian interface. This is explained in a highly simplified way in figure 2 (a detailed geological description can be found in Monjoie et al. 1994 or Couchard et al. 1994).

During the Visean predominantly limestone was deposited. During the Namurian the environment changed and the deposition of limestone was followed by a series of shales with coal layers. During this time the limestone was highly karstified. During the Westphalian following the Namurian again coal, shale and sandstone was deposited but the coal seams were thicker and therefore exploited during the Middle Ages up to the 20th century. During the Variscan orogenis and later during the extension of the Rhine graben (Bouhenni 2003) the formations were folded and faulted.

One of the reason that the tunnel remains within the paleokarst is the Magnee fault an overthrust, which pushed the Visean over the Westphalian. In the footwall the first tens of meters shale and sandstone are deconsolidated in the hanging wall the limestone is karstified with partial clayey infill (Couchard et al. 1994). An impression of the karst cavities found at tunnel level in the Visean is given in figure 3, where karst is shown in the Visean at the surface just before this formation dips into the subsurface to meet the tunnel under construction.

Site investigation

The site investigation consisted of:

- 105 boreholes (length 8km) geophysical, geotechnical and geohydrological measurements
- pilot gallery

Microgravimetry was used (Monjoie et al. 1994) to determine the location of karst voids. Especially in the proximity of the future shaft a large anomaly indicated that karst could occur. It was verified during the tunnelling during 2002, that this anomaly was indeed related to paleokarst in the contact Visean - Namurian and in the Visean self.

Construction of the Soumagne tunnel

The tunnel (one tube, 6km long, 110m²) is constructed at 4 fronts simultaneously. One front started in the west and entered the Westphalian where it still is. Another front started in the east in the Namurian where it still is. From a near central shaft foot near the boundary between the Namurian and the Visean a tunnel front heads east to meet the eastern tunnel front in the Namurian. This breakthrough is scheduled summer 2003. The other tunnel front heads westwards to meet the western tunnel front in the Westphalian summer 2004. This latter front had to deal with the contact between the Namurian and the Visean: the zone with paleokarst until a few hundreds of meters westward the Magnee fault is passed (at the moment, mid 2003, the tunnel front has reached the end of the faille de Magnee) this discordant contact between Visean and Westphalian showed signs of paleokarst as well.

Tunnelling through karst

The difficulties of karst phenomena in tunnelling are:

- complexity of the geometrical form of the cavity created by Karst and its orientation with respect to the tunnel axis (Pöttler 2003)
- the heterogeneity of the infill of the cavity (Pöttler 2003)
- changing interactions between the karstified rock mass and the tunnel structure

Karst affect surface and subsurface structures in the same way: loss of foundation (even in tunnels). In subsurface construct however the karst can endanger the workforce not only because the lack of foundation but by cave-ins.

Methods to deal with karst

Support measures to deal with open karst cavities and karst holes with boulders in soft rock:

- spiles ahead of excavation
- sealing off cavities[6]
- backfilling cavities above support in roof with sand

These measures have to be planned schematically and to be adapted to the dimensions of the karst structures on site. An essential requirement for an effective excavation procedure is the availability of all necessary tunnelling equipment both for excavation by drill and blast and for excavation of soft rock on site (John and Strappler 2003).

In Soumagne this flexible approach is followed. Different support methods (steel arches, swellex, shotcrete) are applied depending on the geomechanical requirements. Difficult stretches are mastered using support in advance of the tunnel front like forepoling or spiling, a method commonly used in Europe to cross difficult zones like karst: construction at this moment at the Irlahüll tunnel in Germany and the Soumagne tunnel in Belgium.

Tunnel through karst: the groundwater

The management of inflow of water from the rock mass into the structure is one of the most important tasks when constructing underground. Especially dangerous is water in karst (Maidl 1997). The general difficulty of tunnelling through karst is the inflow of large quantities of ground water, which in karst areas can even for shallow tunnels approach the order of magnitude known from the great tunnels traversing the Alps (Prinz 1997). However, karstified rock masses have a low storage capacity and will deplete fast (Richter 1989). This sounds quite positive but a warning must be given: difference has to be made between karst in high and low positions. In high positions water enters the karst and leaves it to emerge at the surface again within a time delay needed to pass through the karstified massif. In the case of karst in low positions the water entering the massif is stored underground. When such a karst in low positions is pierced through during tunnelling, water is ejected with great velocity (function of the local hydrostatic head of the reservoir) into the tunnel accompanied by sand and other sediments and debris (Maidl 1997). To estimate the amount of water that might invade the tunnel is not easy. An approximate calculation is based on the delineation of the catchments area (Zaruba & Mencel 1976) but this is extremely difficult in karst areas.

In the case of the tunnel of Soumagne the major advantage is that the water table is throughout the Visean below the invert of the tunnel: near the faille de Magnee at 25m below the invert (Couchard et al. 1994) and near to the shaft foot at about the level of the invert. Therefore the contact Namurian - Visean there a large inflow of karst water hasn't to be feared (Couchard et al. 1994).

Highway connection towards Luxemburg - France

Introduction

When the highway from Liege to Luxemburg and France was constructed in the 1970th the Belgian karst belt was crossed near Remo28.

[6] For more information on injection of karst cavities: Kutzner (1991)

Figure 3. Visean limestone and karst: directly above the tunnel construction though the same formation, photo corresponds to position I in figure 2.

The name Remouchamp is synonym with karst because of the famous show caves that were discovered in 1828. To pass over the river Amblève a viaduct (940m long 80m above river) needed to be constructed.

Karst near the Remouchamp viaduct

The formations encountered were quaternary deposits, essentially deposited by the Amblève river, and Devonian deposits. The Devonian can be further subdivided in Frasnien, Givetien and Couvinien. Karst can essentially be found in the limestone of the Frasnien and the Givetian above the impermeable layers[7] that were deposited at the base of the Frasnien and in the upper Givetien (Nachtergaele et al. 1980). Like the karst which constitute the show cave of Remouchamp the karst is of Cainozoic age.

Site investigation

A microgravimetric survey was not performed because a similar survey during the site investigation of an adjacent viaduct had been found to have limited value in an area of such structural and topographic complexity (Waltham et al. 1986). Endoscopic investigation was not useful because of the limestone fines that covered the borehole wall (Nachtergaele et al. 1980). Seismic refraction was used to determine the depth to the fresh bedrock in the non karstified zone (Nachtergaele et al. 1980). Additional each pier foundation site was investigated by drilling 4 to 8 cored boreholes (Waltham 1994).

Construction

According to Murphy's law the karst had been missed but was found when foundations of the piers were excavated. Caves were found beneath 2 of the 5 footings on limestone (Waltham 1994). Solution cavities are difficult to locate during a programme of ground investigation conducted prior to construction of foundations and their presence should be suspected even if none is identified by boring (Blyth & de Freitas 1984). When the the extent of the solution cavities beneath pier 2 was recognised, much more intensive exploration was instigated at all the pier sites on limestone. The principle technique employed was probing with precise measurements of the rates of penetration. It was found to be inexpensive and very effective for identifying solution voids, which were either open or filled with young unconsolidated sediment (Waltham et al. 1986).

Methods to deal with karst

The foundation must be either be deep, if it is feasible to pierce through the karstified zone, or shallow but then the surrounding bedrock must be reinforcement (Nachtergaele et al. 1980). In the latter case the voids can either be filled which is expensive and no success can be guaranteed. Another option is to displace the pier. The washing of clay filling out of the karst cavities and their sealing with a suitable grout is very laborious and demands a long term injection programme (Zaruba & Mencl 1976). The main problem in injection is to limit the mass that should be injected so that the injectant will not escape by open fissures to any greater distance (Zaruba & Mencl 1976). The method that is used to seal the voids depends on their size:

- large caverns have to be made accessible, cleaned and sealed by concrete. If the caverns cannot be made accessible then gravel sized rocks or debris have to be injected through boreholes into the cavern followed by injection of this fill in (Kutzner 1991). Cavities are often as much as several m wide so that it is virtually hopeless to seal them with injectant since this will be washed away by flowing water. The washing of clay filling out of the karst cavities and their sealing with a suitable grout is very laborious and demands a long term grouting programme (Zaruba & Mencl 1976).
- The granular infill of small caverns can be flushed out prior to injection. Cohesive infill cannot be flushed out. Cohesive material can rest in place when it is possible to densify it prior to injection (Kutzner 1991).
- Karst channels have to hit directly by boreholes with short spacings. Depending on the discharge of water mortar with sodium silica can be used.
- Other karst discontinuities will be injected using traditional methods (Kutzner 1991).

[7] These impermeable layers are known as Macignos, a mixture of shale, calcarenite and limestone (Nachtergaele et al. 1980)

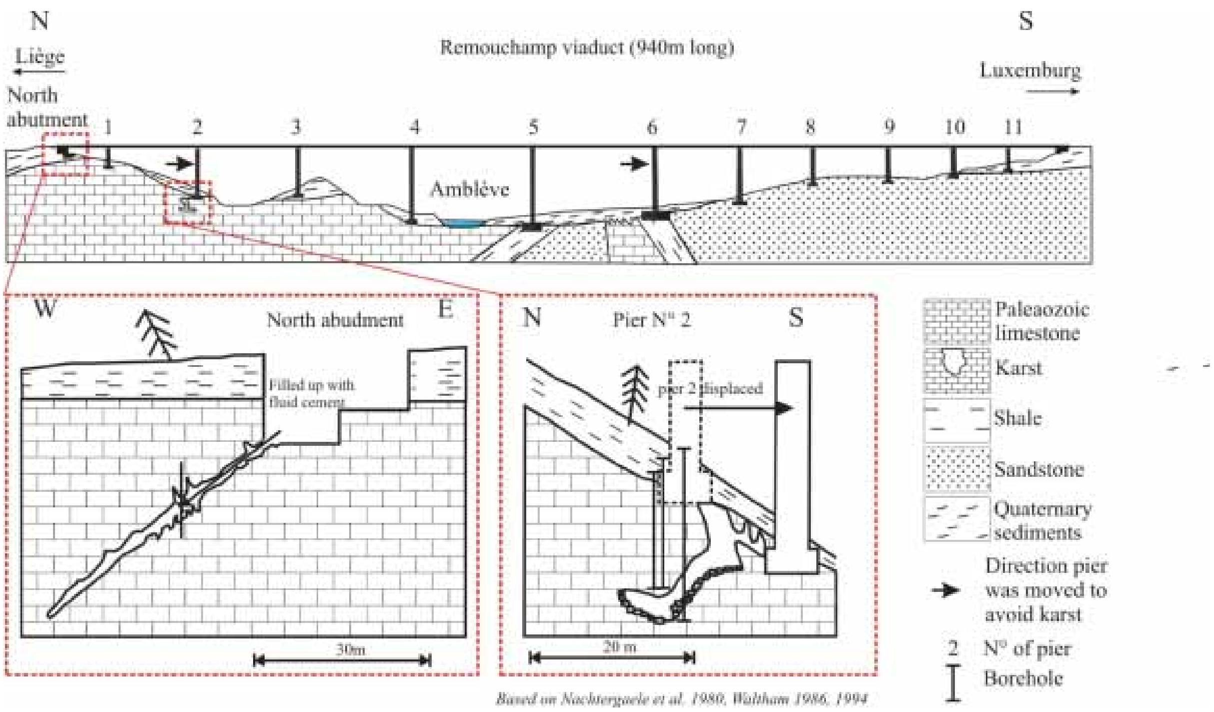

Figure 5. The viaduct over de Amblève near Remouchamp.

In Remouchamp the karst was to deep to use deep foundations. Therefore the bedrock was reinforced, karst was filled with concrete (e.g. abutment north, pier 2). Several piers were relocated (e.g. pier 2 and 6) up to 15 m to avoid the largest cave (Nachtergaele et al. 1980). Karst voids beneath the other piers were cleaned and then grouted. With the combination of these methods the viaduct was constructed and still performs well today.

TGV connection Brussels to Paris

Introduction
The TGV link between Brussels and Paris was the first TGV link to be developed in Belgium. The high speed railroad link crosses the karst belt near Arbre, where a viaduct was constructed.

Karst near Arbre
Arbre is located on the Palaeozoic rocks of the karst belt. The Paleozoic rocks were deposited during the Tournaissian. Quaternary deposits cover the Palaeozoic rocks (Couchard & Detandt 1999). The bedrock it self is affected by Mesozoic - Cainozoic karst. The karst is filled with Tertiary sediments.

Site investigation
It is known that the depth of the karst phenomena even if they are not of Palaeozoic age are independent of the current regional base level in the region which is often linked to a large river in a karst region. Cave systems can be formed above, at, or below the water table but the depth of major solution openings below base level is probably less than 60m (Fetter 1994), but can attain 100m in the Palaeozoic limestone and dolomite rocks of the Belgian karst belt (Calembert & Monjoie 1973). Cavernous zones found at these depths are well below the present water table. Warned by this an extensive site investigation was performed and it was shown that the depth of the karst extended to a remarkable 80 m below the surface. This needed to be taken into account during the planning of the viaduct.

Construction
Such deep karst voids can not be filled economically. Normal piles cannot be used in these terrains. Because compressive and tension forces (braking of train) needed to be transmitted the choice was made to stabilise the karstified bedrock with micropiles, regularly used as means of foundation of line infrastructure in karst. In Arbre 88 micropiles were installed per footing in the worst ground conditions. The micropiles were constructed to a depth of 30m (Couchard & Detandt 1999). With this method the bedrock was stabilise and the viaduct could be constructed.

Conclusion

Karst in Belgium affects predominantly Palaeozoic carbonate rocks. The rock masses showing karst near the surface or at least in the upper region where civil engineering construction takes place can be found as a half circle fencing in 94% of the population in Belgium from France and Germany. Thus at any instant when line infrastructure was constructed or is being constructed to France or Germany karst will be encountered. The

karst encountered is either the paleokarst, Mesozoic or Cainozoic karst. This paleokarst karst can be found at any depth where the Visean meets the Namurian; this contact is infamous among rock and soil mechanic engineers. Mesozoic karst is found e.g. in Tournaissian, were the karst holes are filled by Tertiary sediments up to 100m deep. Cainozoic karst, affects nearly all carbonate formations. It was developed during the ongoing uplift of region thereby allowing the rivers to erode into the riverbeds, lowering the regional water table and continue with the karst formation at a lower level: “dry” caves can be found above the present river levels e.g. Han-sur-Lesse and Remouchamp.
Three examples of line infrastructure through karst were discussed. All three constructions passed the karst belt and all three of them encountered problems related to karst. These difficulties were solved differently:
Remouchamp: the piers of the bridge was shifted when karst voids were found.
Arbre: the karst voids descended up to 80m below the present surface. The karstified rock mass was stabilised using a large number of micropiles.
Soumagne: the intensive geophysical prospection led in an early stage to the acknowledgment that a flexible construction method was needed, the karstified zone was successfully mastered.

With a mix of geophysical methods analysis of geological maps, information provided by locals and speleologists, analysis of aerial photographs one is able to indicate where there is karst but neither its seize, geometry depth or extent can be determined with an accuracy that is precise enough to satisfy engineering design. But it is important that this investigation is done as early as possible during the project. Then the construction firm knows that not a 100% detailed planning is required but flexibility.
When a bridge has to be constructed in a karstified area the position of piers might have to shift, this needs to be considered in the initial design. The permission of the controlling authority must be obtained for several ranges of pier position. When the statistical prove and permission for an adapted variant have to be obtained at a later stage e.g. during construction then valuable time is lost to get through the red tape.
When tunnelling through karst be prepared and make sure that the equipment to deal with hard rock and soft soil is on site. Select a site manager who knows that geology dictates more than in other circumstances the rate of advance. Before construction starts prepare a catalogue in which the support measures dealing with many possible karst encounters are described. Obtain the permission of the controlling officials for all these variants before the construction begins.
In all these cases there is a double task for the engineering geologist very early, less detailed, to change the psychology of the construction approach and then later on during construction on a day to day basis directly at the front or on the construction site (no fire and forget engineering geology). It was shown that when line infrastructure crosses the Belgian karst belt, that this approach, where applied, was successful to deal with the unpredictability of karst.

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LINE INFRASTRUCTURE

The Bosphorus Railway Tube Crossing

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In January 2003 Fugro conducted a soil investigation in the Bosphorus Strait in Istanbul for the proposed Railway Tube Crossing. Both a bored tunnel and an immersed tunnel construction method are considered. At the proposed location, the Bosphorus Straight is about 2km wide.

For the Bosphorus offshore soil investigation project the geotechnical/scientific dynamically positioned research vessel SRV Bavenit faced particular challenges unique due to the nature of the project. Firstly the Bosphorus Strait is a very busy shipping canal for international marine traffic. Secondly strong currents up to four knots in a the two-layer flow system – an upper level southerly current and a lower level northerly current - due to the complex water interaction between the Black Sea and the Mediterranean Sea. By using a dynamically positioned vessel instead of small jack-up platforms, it was possible to overcome these operational constraints with its speed and manoeuvrability required for working in an active shipping channel in combination with strong currents with a minimum risk for interference with the marine traffic. During the two and half week fieldwork campaign only a few hours of standby / unworkable conditions were experienced.

Working close to shore in busy traffic

For the design and installation of the Railway Tube Crossing a soil investigation was required to determine the soil conditions along the proposed Railway alignment between the Asian and European part of Istanbul. Prior to the start of the investigation Fugro reviewed existing site investigation data completed at the Bosphorus area in 1985-86 and had discussions with Alluvial Mining (AML) and geologists of Geoscience Investigation and Consulting Ltd. who have previous experience in this area. The findings indicate that there is a highly complex geological environment in the Bosphorus area and that a variety of ground conditions are expected.

For the construction of the proposed railway rock tunnel boring machines are considered for the shallow-overburden sections along both landfalls of the Bosphorus. An immersed tube solution for the centre section is proposed where sediments are up to 80 m thick.

The fieldwork scope included ten boreholes ranging from 20m depth to a maximum depth of 85m below seabed. The water depths were between 17m close to the shore and 47m in the centre part of the Bosphorus. The soil investigation techniques includes flush rotary drilling in combination with downhole geotechnical equipment such as Cone Penetrometer, a hydraulically activated push and piston samplers, and onboard geotechnical laboratory. Besides the geotechnical tests, environmental tests were carried out because of the expected contamination of the top layers. By using a seabed template fitted with pipe clamp, the drill string is immobilised when sampling and/or in-situ testing, thus minimising disturbance of the soils to be sampled or tested. For the coring work a piggy-back rotary coring system was used to advance the investigation in cemented or rock formations.

In addition special techniques were utilised, which included geophysical borehole logging and combined S-wave and P-wave seismic downhole measurements. Fugro performed the geophysical borehole logging, running gamma & neutron tool in the string to the rock head. The gamma & neutron and full-wave sonic & resistivity tools were used for the lower open-hole section in greywacke rock. A WISON CPT unit was used to advance a seismic cone penetrometer into the soil. S-waves were generated with Fugro's shearwave HUSH box placed on a seabed frame. P-wave generation was by air gun suspended from a cable in water.

The surficial geology at the proposed locations for site investigation is highly variable. The site consists of predominantly sands and silts with some clays. At most locations extensive gravel layers were encountered as well. Underlying these sand/silt/clay/gravel layers is the bedrock. The rock encountered is predominantly Sandstone and Mudstone/Shale.

LINE INFRASTRUCTURE

Foto impressie

Ronald Cozijn, Inspec BV
Bron foto's: Betuwe route, Waardse Alliantie; Project Bureau A 50; Project bureau HSL-Zuid

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Vol. 10 No. 2 2003

LINE INFRASTRUCTURE
Geologisch model van project Spoorzone Delft

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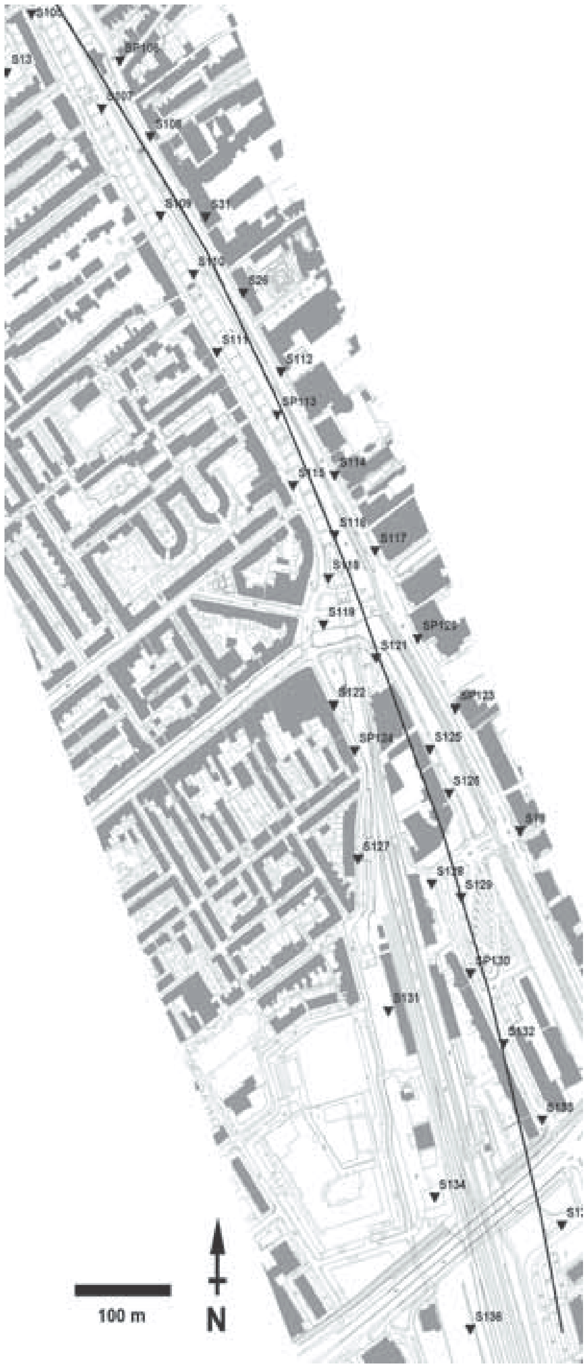
Inleiding

In 2002 ontving de combinatie DHT/TCE/Benthem Crouwel (DTB) opdracht van ProRail voor een planstudie naar 4-sporigheid Rijswijk-Delft. Uitgangspunt van de studie is een ondertunneling van het spoortraject in de spoorzone (zie figuur 1) binnen de gemeente Delft, waardoor het project bekend staat onder de noemer 'Spoorzone Delft'. Aanleiding tot het plan wordt gevonden in de hinder die het op een viaduct gelegen spoortraject veroorzaakt bij omwonenden in de vorm van vooral geluid, stof en trillingen. Daarnaast vormt het spoortracé momenteel een barrière voor het vrij verbinden van de twee stadsdelen aan weerszijden van het spoor.
Voor een dergelijk project dat in stedelijke omgeving gerealiseerd moet worden, is het zowel voor het ontwerp als voor de omgevingsbeïnvloeding van essentieel belang de geotechnische condities goed te kennen en te kunnen kwantificeren. Een betrouwbaar en degelijk driedimensionaal geotechnisch model van het projectgebied is noodzakelijk en vormt de basis voor een optimaal ontwerp en een beheersbare beïnvloeding van de omgeving.
In dit artikel wordt het driedimensionaal geotechnisch model, en de geologische interpretatie, gepresenteerd dat in de Voor Ontwerp (VO) fase werd gemaakt. Het model is gebaseerd op archiefgegevens (sonderingen met kleefmeting) en specifiek voor dit project uitgevoerd grondonderzoek bestaande uit sonderingen en pulsboringen.

Ontwerpproces

ProRail heeft de planstudie onderverdeeld in drie fasen. De adviescombinatie DTB heeft hierin een 16-tal varianten beschouwd, die variëren in tunnellengte, alignement, positie van het station, en bouwmethode. De bouwmethodes wand-en-dak, cut&cover, boortunnel, afzinken zijn onderzocht. Voor de beschrijving van de effecten op de omgeving en reizigers zijn studies uitgevoerd naar drukgolven, veiligheid, trillingen, geluid, bouwschade en –hinder. Om aan de gewenste ramingnauwkeurigheid te voldoen zijn daarnaast geotechnische onderzoeken, constructieve berekeningen, risicoanalyses en (deterministische en probabilistische) kostenanalyses



Figuur 1: Overzicht van het projectgebied. Locatie van lengtedoorsnede S-T (zie figuur 3) en de sonderingen zijn weergegeven.

Laagnr.	Geotechnische units	Geologische Formatie	Geologisch Tijdvak
1	Ophooglaag		
2 – 5	Afzettingen van Duinkerke	Westland Formatie	
4, 6, 12	Hollandveen	Westland Formatie	Holoceen
7 – 17	Afzettingen van Calais	Westland Formatie	
18	Basisveen	Westland Formatie	
19 – 22	Afzettingen behorend tot de Formatie van Kreftenheye	Fm. van Kreftenheye	
23	Afzettingen behorend tot de Formatie van Kedichem	Fm. van Kedichem	Pleistoceen

Tabel 1. De geotechnische units die in het 3D model zijn onderscheiden. 23 Hoofdlagen en 10 sublagen vormen samen de 33 geotechnische units.

uitgevoerd. Na evaluatie en toetsing aan de criteria zijn 2 varianten geselecteerd en uitgewerkt tot VO-niveau, een korte tunnel met een bovengronds station op de Prinses Irenetunnel en een lange tunnel met een ondergronds station. Beide tunnelvarianten worden gebouwd volgens de wanden-dakmethode, gebruik makend van diepwanden. De uiteindelijke keuze hangt voornamelijk af van de financierbaarheid van het project. Aan het einde van dit jaar wordt hierover meer duidelijkheid verwacht.

3DGM

Voor het maken van het geotechnisch model is een speciale software module toegepast, het 3 Dimensionaal Geotechnisch Model (3DGM). Deze module, ontwikkeld met het Geografisch Informatie Systeem ArcInfo, is door Witteveen+Bos (participant in TCE) ontwikkeld om op basis van geïnterpreteerde ééndimensionale gegevens (sonderingen, boringen) een driedimensionaal continu model te maken. In de interpretatieslag voorafgaand aan het genereren van het model is geologische kennis een vereiste. In figuur 2 wordt een voorbeeld van het driedimensionale karakter van het model weergegeven.

Geotechnisch model

Het maken van het geotechnisch model bestond uit een aantal stappen. In de eerste fase van het project werd gewerkt met archiefgegevens. Met deze gegevens werd een eenvoudig grof geotechnisch model opgezet, waaruit direct kon worden geconcludeerd dat de grondopbouw in het projectgebied zeer heterogeen van aard is. Dit rudimentaire model diende als middel om het uit te voeren grondonderzoek te plannen: speciale aandachtsgebieden konden worden geïdentificeerd waar een hogere dichtheid van sonderingen gewenst was.
Het uitgevoerde grondonderzoek bestond uit een uit 40 tal sonderingen (36 tot een diepte van - 35 m N.A.P. en 4 tot een diepte van - 45 m N.A.P.), waarvan 6 met waterspanningsmeting, en 4 pulsboringen. De boorgaten zijn ingericht als peilbuis zodat monitoring van de stijghoogte direct in het tracé van de tunnel mogelijk is.

Met deze nieuwe set gegevens werd een nieuw driedimensionaal grondmodel gemaakt. Een tweedimensionale lengtedoorsnede door dit model is weergegeven in figuur 3. In het model zijn 33 geotechnische units onderscheiden (tabel 1). Dit is een relatief groot aantal, echter door de grote variatie in bodemopbouw was dit een minimum.

In de volgende paragrafen worden de afzettingen in afnemende ouderdom besproken, met de corresponderende laagnummers, aan de hand van de doorsnede in figuur 3.

Formatie van Kedichem
De Formatie van Kedichem wordt slechts in één diepe sondering in het zuidelijk gedeelte van het projectgebied aangetroffen. Het precieze verloop van deze waterscheidende laag is onbekend, de laag komt in de andere, meer noordelijk gelegen diepe sonderingen niet voor. De aangetroffen afzettingen bestaan uit klei. De Formatie van Kedichem bestaat uit rivierafzettingen van de Rijn en Maas en is in het Vroeg Pleistoceen afgezet.

Formatie van Kreftenheye
Boven op de Formatie van Kedichem wordt een minstens 25 meter dik zandpakket aangetroffen dat een aantal kleilenzen bevat. Het betreft rivierafzettingen die bekend staan als de Formatie van Kreftenheye. De formatie is in het Laat Pleistoceen tot in het Holoceen afgezet. Dit zandpakket bestaat uit grove tot zeer grove zanden. De top van dit Pleistoceen pakket varieert van -19 m N.A.P. in het noorden tot -16 m N.A.P. in het zuiden. Op twee locaties (A en B in figuur 4) ligt de top beduidend lager: door het insnijden van geulen is de formatie hier deels geërodeerd. De top van dit pakket bevat lokaal soms een klei- of veenlaag. DSM Gist onttrekt voor haar fabrieksprocessen grondwater uit dit pakket.

Basisveen
Boven de Formatie van Kreftenheye komt over nagenoeg het gehele gebied een veenlaag voor, het Basisveen. Het Basisveen is in het vroeg Holoceen gevormd. De dikte van deze veenlaag is maximaal één meter. Slechts ter plaatse van de eerdergenoemde geulinsnijding (locatie A in figuur 4) en een andere insnijding ter hoogte van sondering S132 (locatie B in figuur 4) is het Basisveen geërodeerd. Op de bodem van de geul op locatie A is vervolgens weer een veenlaagje afgezet dat op basis van de diepte tot het Basisveen wordt gerekend.

Afzettingen van Calais en Hollandveen
De Afzettingen van Calais bestaan uit zand- en kleipakketten en dateren van 8.000 tot 3.500 jaar B.P. Alle veenlagen die boven het Basisveen voorkomen, worden gerekend tot het Hollandveen. In het profiel worden drie Hollandveen lagen onderscheiden. Deze lagen komen voor tussen de Afzettingen van Calais.

De basis van de Afzettingen van Calais wordt vaak gevormd door een kleilaag. In de zuidelijke Phoenixstraat en het stationsgebied wordt de basis gevormd door een zandlaag van 2 meter dik. Een laag bestaand uit kleiig zand is weer over het gehele gebied terug te vinden (met uitzondering van het gebied tussen de zuidelijke Phoenixstraat en de Irene tunnel). Laag 14, een zandige siltige klei komt in dit laatste gebied juist weer wel voor. De dikte varieert tussen de 1 en 5 meter. In het noorden komt deze eenheid niet voor. Alleen tussen sonderingen S107 en S116 ligt boven op deze zandige klei een dun (1 meter) kleipakket (13) met een zeer lage conusweerstand. De volgende laag in de stratigrafische opeenvolging is een veenlaag (12), Hollandveen III. Deze veenlaag heeft een variabele dikte met een maximum van één meter en komt alleen niet voor tussen sonderingen S111 en S120 (de zuidelijke helft van de Phoenixstraat).
Ten noorden van sondering S118 ligt op Hollandveen III een zandige klei (10). Dit pakket met een dikte tussen de 2 en 6 meter kan lokaal een meer zandig karakter (laag 10A, sonderingen S106 en S107) hebben. Een kleipakket (8), dat gekenmerkt wordt door een zeer lage conusweerstand, vormt de jongste geotechnische eenheid die tot de Afzettingen van Calais wordt gerekend. De laag komt over nagenoeg het hele tracé voor en is nooit dikker dan 3 meter. Het jongere Hollandveen II (6) komt over het hele tracé voor en is gemiddeld enkele decimeters dik. Deze veenlaag vormt de scheiding met de Afzettingen van Duinkerke.

Geulstructuren
Op twee locaties in het profiel worden in de Afzettingen van Calais duidelijke geulstructuren aangetroffen (figuur 3):
1. Aan het zuidelijk uiteinde van de Phoenixstraat. De geul is ingesneden tot in het Pleistocene zandpakket (Locatie A in figuur 3).Op de bodem van de geul is een veenlaag afgezet. Daarboven is een 3 meter dikke zandige klei afgezet (11). De overige geulvulling bestaat uit een ca. 11 meter dik zandpakket (9). Ter hoogte van sonderingen S125 en S126 worden de bovenste meters van de geulvulling gevormd door een siltige klei (9A), vermoedelijk afgezet in een periode van lagere activiteit, c.q. stroomsnelheid van de geul.
2. De andere geulinsnijding is gelokaliseerd ter hoogte van de Irene tunnel. De geulafzettingen bestaan uit een afwisseling van siltige en kleiige laagjes met een dikte van 10 meter. Deze geul snijdt niet in in het onderliggende Basisveen, en heeft dus alleen afzettingen van Calais geërodeerd. De geulafzettingen zijn in slechts één sondering geconstateerd. Aanvullend grondonderzoek zal nodig zijn om de afzetting beter in kaart te brengen.

Afzettingen van Duinkerke
Over het gehele gebied, met uitzondering van het uiterste zuiden, wordt de basis van de Afzettingen van Duinkerke gevormd door een zandpakket (5). De dikte van dit pakket varieert van 1 tot 5 meter. In het zuiden wordt de basis gevormd door een veenlaag (4). Deze laag, Hollandveen I, is daar en lokaal in het noorden (S102) tot 3 meter dik. De laag wordt in het overige gedeelte van het profiel ook aangetroffen, maar heeft dan een discontinu karakter en een dikte van gemiddeld 1 meter.
Laag (3), een kleipakket, komt weer wel over het hele gebied voor, zij het met wisselende dikte (0 – 3 meter). Lokaal wordt er op dit kleipakket een zandlaag (2) aangetroffen. Deze laag heeft een maximumdikte van 2 meter.

Ophooglaag
Over het hele gebied komt ten slotte een ophooglaag voor die bestaat uit zand, puin, klei en klinkers. Deze laag kan een dikte van enkele meters hebben.

Geologische interpretatie

De stratigrafische opeenvolging laat een grote variatie zien van zowel continentale als mariene afzettingen. Deze variëteit is het gevolg van een aantal zeespiegelfluctuaties die de afgelopen 10.000 jaar plaats hebben gehad.
Er is in eerste instantie een overgang waarneembaar van continentale, fluviatiele afzettingen (Formaties van Kedichem en Kreftenheye) en Basisveen naar mariene afzettingen (Afzettingen van Calais). Dit weerspiegelt een zeespiegelstijging.
In de Afzettingen van Calais en de bovenliggende Afzettingen van Duinkerke, die bestaan uit zowel mariene als continentale afzettingen, zijn enkele veenlagen gevormd (Hollandveen) tijdens regressies, perioden waarin de zeespiegel daalde. Tijdens deze

zeespiegeldalingen kwam het gebied (deels) boven de zeespiegel te liggen zodat zich vegetatie kon ontwikkelen. Deze vegetatie werd vervolgens bedekt met mariene afzettingen, waarna veenvorming (Hollandveen) plaats kon vinden.
Tijdens perioden van hogere zeespiegelniveaus bestond het afzettingsmilieu uit open zee en lagunes met geulsystemen. Afzetting van de mariene sedimenten ging gepaard met erosie en insnijden van getijdengeulen. Hierdoor komen er in het gebied lateraal grote verschillen in de lithologische opeenvolging voor zodat een differentiatie van geotechnische units niet alleen verticaal, maar ook horizontaal is gemaakt.

In het geotechnisch model worden twee geulsystemen geïdentificeerd die wijzen op activiteit tijdens het sedimenteren van de Afzettingen van Calais. Geul 1 in figuur 3 was actief tot tijdens de afzetting van laag 8 (klei). Het veen dat op de bodem van de geul wordt teruggevonden wijst erop dat de geul verschillende fasen van activiteit heeft gekend. Immers, het veen zal niet zijn afgezet op de bodem van een minimaal 14 meter diepe geul, maar in een periode waarin de geul niet actief was. Dit maakt het aannemelijk dat deze veenlaag stratigrafisch gezien tot het Hollandveen behoort. Geotechnisch gezien kan de laag echter tot de geotechnische eenheid van het Basisveen worden gerekend.
De tweede geul snijdt niet dieper in dan tot in laag 15 (figuur 3) en is dus jonger dan het Basisveen. Doordat Hollandveen II boven de geulafzetting wordt gevonden, kan geconcludeerd worden dat de geul niet meer actief was ten tijde van het afzetten van deze veenlaag.

De Afzettingen van Duinkerke zijn gevormd tussen 3500 jaar B.P. en het heden. Ze bestaan net als de Afzettingen van Calais uit zowel mariene als continentale afzettingen. De mariene afzettingen worden gevormd door geulafzettingen van het laatste actieve geulsysteem, het Gantelsysteem.

De specifieke geohydrologische situatie in Delft is bijzonder te noemen. DSM-Gist onttrekt jaarlijks ca. 13 miljoen m3 grondwater voor haar bedrijfsprocessen, waardoor de stijghoogte in het 1^e watervoerend pakket uitzonderlijk laag is (zie figuur 5). Een belangrijke uitgangspunt voor het ontwerp is of de kleilaag van Geul 1 waterdicht is en of een verschil in stijghoogte bestaat tussen de geulafzetting en het eerste watervoerend pakket. De sonderingen met waterspanningsmeting hebben hierin duidelijkheid gebracht. In figuur 6 is sondering SP120 gegeven. Deze sondering is uitgevoerd voor het pand van IHE aan de Westvest. In de zandlagen zijn dissipatietests uitgevoerd om de stijghoogte nauwkeurig te kunnen meten. Uit de SP120 blijkt dat drie verschillende waterstanden te onderscheiden zijn. Ten eerste is in de Duinkerke zandlaag het freatisch vlak goed te meten, dat op deze locatie op –0,8 m NAP bepaald is. De stijghoogte in het Pleistocene zand, dat onder directe invloed van de onttrekking van DSM-Gist staat, is op –7,2 m NAP bepaald. De stijghoogte in de geulafzetting wordt op –3,3 m NAP vastgesteld. Met een stijghoogteverschil van 4 m tussen de geulafzetting en het 1^e watervoerend pakket kan geconcludeerd worden dat de kleilaag onder de zandige geulafzetting voldoende waterafsluitend en continu is.

De boorgaten die voor het grondonderzoek zijn gemaakt zijn uitgevoerd met een filter in het 1^e watervoerend pakket. Bij deze peilbuizen zijn tevens peilbuizen geplaatst op een niveau van –12 m N.A.P., dus in de zandige geulafzetting of in de zandige Calais afzettingen . In de peilbuizen zijn divers geplaatst waarmee continu de stijghoogte gemeten wordt. Deze metingen bevestigen het beeld van sonderingen met waterspanningsmeting.

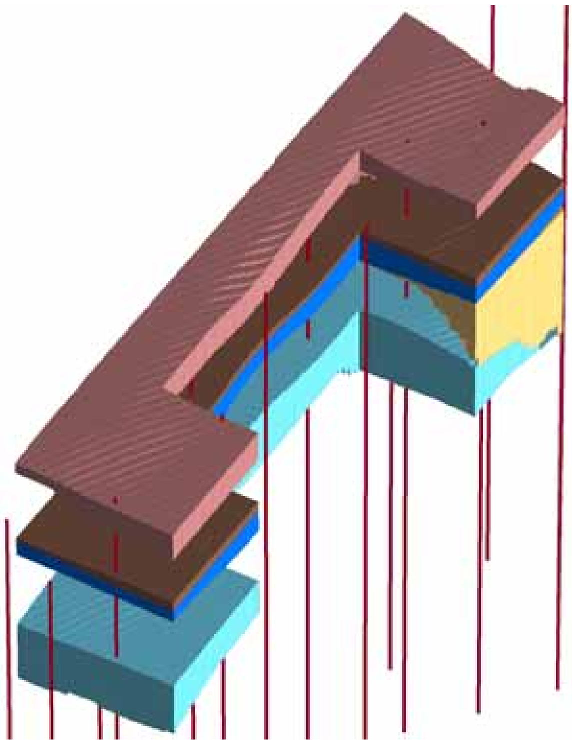
Conclusies

Bij de interpretatie van het grondonderzoek is een groot aantal lagen, geotechnische units, geïdentificeerd en is lateraal een grote variabiliteit in laagopbouw geconstateerd. Ook een complex geologisch systeem als in de spoorzone te Delft is dus vast te leggen in een driedimensionaal geotechnisch model. Voorwaarden hiervoor zijn een goede spreiding van de grondgegevens en kennis van de geologische processen.

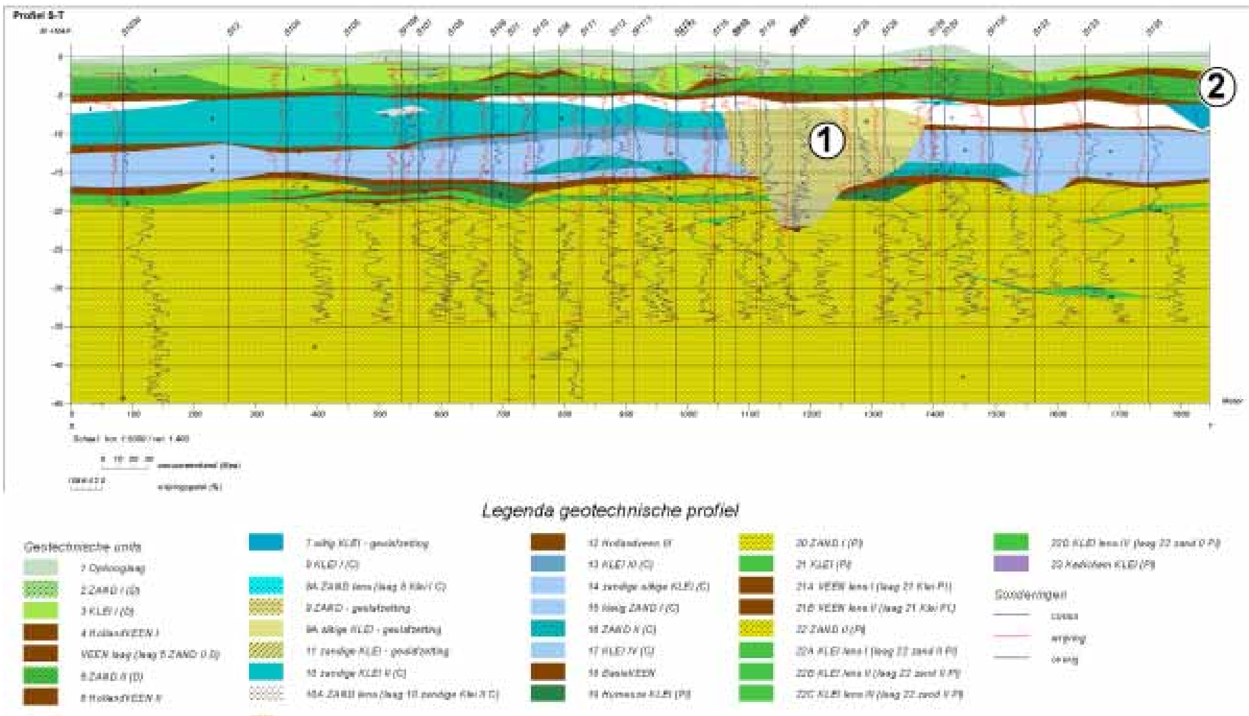
De automatische processing van het model maakt het toevoegen en up-to-date houden van het 3D model efficiënt. Grondonderzoek dat in de definitief ontwerp fase zal worden uitgevoerd kan dus eenvoudig worden toegevoegd aan het model. Met behulp van het huidige model zijn reeds locaties geïdentificeerd waar aanvullend grondonderzoek gewenst is.

Met het 3D geotechnisch model beschikt het ontwerpteam over een middel waarmee snel doorsneden gegenereerd kunnen worden in een format dat voor de geotechnische rekentools in te lezen is. Op deze wijze is een integratie van complexe geologische modellering met de geotechnische rekenprogrammatuur op een efficiënte wijze mogelijk.

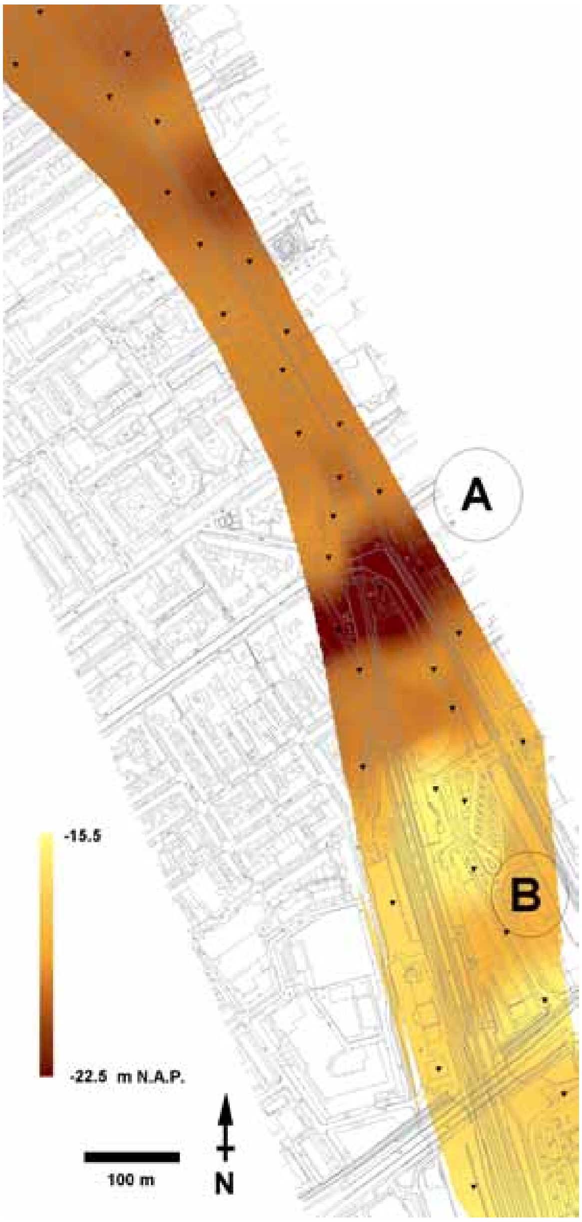
Figuren



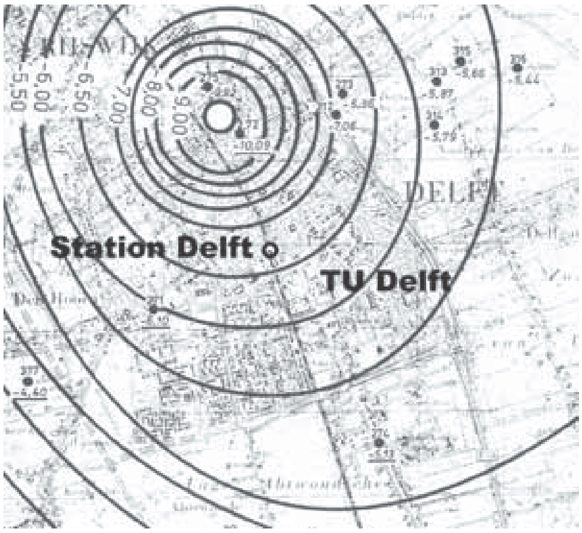
Figuur 2: Voorbeeld van een driedimensionaal beeld



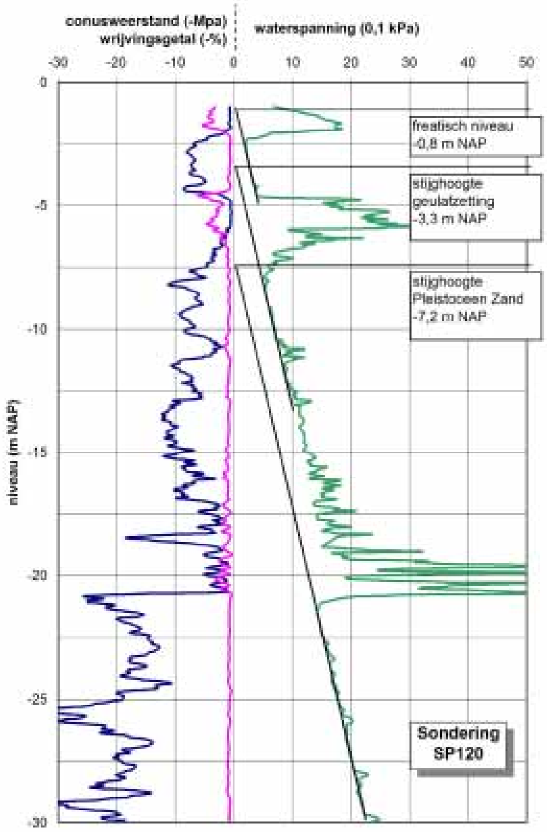
uit het model. Ook sonderingen zijn weergegeven.
Figuur 3: Lengtedoorsnede S-T. Twee geulstructuren zijn aangegeven.



Figuur 4: Diepte top Formatie van Kreftenheye. Twee geulinsnijdingen (A en B) zijn aangegeven.



Figuur 5: Verloop van de stijghoogte van het 1e watervoerend pakket (formatie van Kreftenheye). Bron: Grondwaterkaart Nederland, TNO-NITG.



Figuur 6: Sondering SP120 met waterspanningsmeting.

LINE INFRASTRUCTURE

Line infrastructure and the role of engineering geology in analysing overbreak, part I theoretical considerations

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1 Introduction

Overbreak is a considerable cost factor in underground construction. Up to date there are no efficient methods to predict the overbreak in tunnels. Therefore it is difficult for the appointed engineering geologist to find out ways to reduce it. A literature survey was started to understand overbreak and to investigate its rock mechanical background. Some of the results are presented in this contribution. The practical application of this knowledge will be discussed by S. Viroux in the following issue of the Ingeokring newsletter.

2 Engineering geological definition of overbreak

There is no general accepted definition of overbreak, for some it is the gap expressed in m, m² or m³ between the theoretical, planned excavation profile and the dimensions of the excavation that was made. Someone has to pay for this overbreak thus depending on which side you are the definitions will be slightly different. In this contribution the classification based on geomechanical considerations forwarded by Müller (1978) will be followed:

- "Genuine" overbreak

This overbreak (or underbreak) is formed by the excavation of the rock mass leaving a void, which modifies the cinematic admissibility of individual rock blocks, permitting some of them to leave the rock mass matrix. This process is "instantaneous". Note that by blasting new discontinuity sets are formed, thus new rock blocks form, which weren't present in the original massif investigated characterised (of course) by a thorough site investigation. This indicates already that the analysis of the overbreak and its reduction cannot be part of "fire and forget" engineering geology that focuses only on the site investigation phase, but requires a geomechanical counsel on a frequent basis. This "genuine" overbreak is often called "unavoidable" in the sense that it cannot be countered by intelligent support after blasting like the secondary overbreak discussed below. Therefore one likes to call this kind of overbreak "geological overbreak" in the sense that it is not the fault of the miners or the technical staff in the tunnel. This is not correct because one can take influence on this geological overbreak by selecting the wrong direction of tunnel driving with respect to the discontinuities of the massif or by poor blasting or by selecting a tunnel cross-section inappropriate for the particular rock mass. This overbreak can be found in the roof as well as in the floor of the tunnel. It includes the few cm around the blast holes shattered by the explosion. It is produced upon first blasting thus always near the tunnel front in contrast to secondary overbreak or rock burst.

- Scaling

It is assumed in this contribution that scaling after blasting does not produce any additional overbreak.

- Secondary Overbreak (instantaneous but initially cinematically not admissible)

This overbreak is often considered to be the "fault" of the miners or the technical staff in contrast to the "geological" overbreak discussed above. The "secondary" overbreak will not take place instantaneously thus there is a notion of "time". This can be:

- a real time effect related to stress redistribution in a rock mass that behaves viscously, or
- retarded with respect to the initial blasting, because the "genuine" overbreak, scaling after blasting or vibrations during the installation of initial support after the first blasting round causes additional stress or modifies the geometry, thereby allowing additional rock fall to take place. This kind of overbreak is predominantly gravity driven, thus can be found in the roof area and can be counteracted to a large extent by selecting the appropriate support and supporting technique and by applying it directly after blasting.

- Cave-in

Whereas overbreak and secondary overbreak bring about additional and substantial cost (see section 3), cave-ins are disasters that must be prevented by all means. Cave-ins are often confused with overbreak because they can (not a necessity) occur near to the tunnel front. Many engineers presume that there is a relationship between the stability of the rock mass and the overbreak, they think that less stable rock masses (or rock masses with a shorter stand-up time) produce larger overbreak than more stable rock masses (with a longer stand-up time). In fact the contrary happens:

stable rock masses consist often out of large rock blocks and have a large discontinuity spacing. Therefore, the unavoidable, primary, overbreak can be large. Contrary rock masses that are less stable have a very small discontinuity spacing, they can be excavated with less harsh methods, even without blasting, with a very small overbreak (Müller 1978). The confusion is based on the fact that less stable rock masses have a larger tendency to produce secondary overbreak and cave-ins.

- Rock burst
Rock bursts are formed by the sudden release of stored elastic energy, they are known to occur in deep tunnels and mines (Bräuner 1981). Although rock bursts are very important phenomenon having a large death-toll, they will not further be discussed in this contribution.

- Overbreak related to off set
Another form of overbreak is not related to collapse, fall or cave in of rock mass but to the restricted working space underground. The drills used to drill the contour holes have a non-zero thickness that create an off-set of 0.2 to 0.4 m as shown in figure 1.

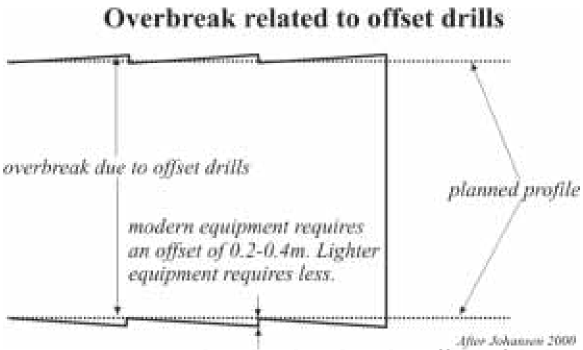

Figure 1. Overbreak related to the offset of drills

- *Overbreak by intentional over excavation in rock masses with "genuine" plasticity:*
One has to distinguish between:
- Genuine plasticity in e.g. mylonites and clays. In rock masses showing real plasticity new rock mass material will follow in the places where rock mass is extruded into the excavation, the risk of large overbreak is low, contrary the underbreak can be significant (Müller 1978).

- Pseudoplasticity: in rock masses that are stratified, foliated or fractured, pseudoplasticity is very high if the degree of separation is high or the discontinuity infill is lubricant. In rock masses showing pseudoplastic behaviour all asymmetric features given by the discontinuity matrix will become more and more asymmetric in time and thereby the overbreak increases.

As long as the rock mass plasticity provokes along the circumference a gradual and regular decrease of the excavated section, overbreak must deliberately be applied in order to obtain the minimum excavation size in due time (Müller 1978). This type of overbreak is

applied at the moment in the Sedrun stretch of the new Gotthard base tunnel (WT 2003).

3 Why should an engineering geologist care?
Because:
i) of his inquisitive nature, he is interested in all matters concerning geology and engineering.
ii) overbreak costs money.
The engineering geologist needs to convince the tunnel constructor that he is cheaper than the overbreak. Therefore he needs to know what type of overbreak most frequently occurs and how much it will cost.
If the final support is rigid then underbreak will not be accepted because a minimum excavated size is required. Because in addition underbreak is more expensive to remove than overbreak (Viroux 2003) overbreak is more frequent than underbreak. Tradition has that the client is willing to pay for a defined overbreak, considered as the "geological" overbreak. Therefore in general "gaps" are created and the major expenses of overbreak are related to the filling this gap with concrete in tunnelling and waste-rock etc. in mining. The costs depend on the size of the "gap" to be filled up. What is the size of overbreak in general?:
Overbreak in % diameter (Wahlstrom 1973, Müller 1978, Kolymbas 1998):
- 7.5 % Chippis (CH) tunnel
- 10% Simplon (CH) tunnel
- 6 - 38% for tunnels in general
- 10% average drilled and blasted tunnels
- 25% in fractured rock or if blasted incorrectly
- 22% in biotite schist and gneiss, 16% in shale, 7% in granite, 16% in closely jointed granite, 31% closely spaced horizontal joints section of the Orberts tunnel Colorado
- 5 - 10cm in compact stable rock mass with few fractures or in fractured rock with small discontinuity spacing if support is correctly selected and swiftly installed
- up to 50 cm in compact stable rock mass with large scale fractures at large discontinuity spacing
- 15 - 30 cm in fractured, unstable, rock prone to cave
- 1/6 of the diameter of excavation during blasting in rock mass with smooth planar discontinuity surfaces

The expenses include the additional concrete, the time needed to shotcrete all the voids and the time lost that could have been used to drill the following blast holes or to install appropriate support.
Müller (1973) gives some examples:
- tunnel in general: reduction of overbreak of 10 cm (average dimensions) reduces expenses equal to twice the cost of the explosives
- shaft: small overbreak, 28% excavation profile and reduction of 23% concrete could be saved

- tunnel Zillertal (AU): additional 5 m³/m tunnel (average 35 cm)
- Viroux (2003) states that the expenses to fill up overbreak equals or doubles those (including labour time etc.) of the designed shotcrete support.

It is clear that overbreak is expensive and that in the light of progress of engineering geology one should not content oneself with the large margin as they are used in general today.

4 Factors affecting overbreak

The excavation method

Regarding all excavation methods:
- drilling and blasting
- hammer and chisel
- sawing
- roadheader

excavation by drilling and blasting produces most overbreak (see figure 2).

In general explosives should be used in underground excavation as tool to dissect the rock mass and not as a "bomb" (Bouvard et al. 1988). This is reflected in appropriate ratings for the MBR blasting adjustment (Bieniawski 1984): for machine boring no correction for damage is required but for poor conventional blasting correction for severe damage must be made. What does actually happen during blasting to make this the most overbreak-prone excavation method? During excavation with blasting the vibrations cause a momentary annulation of the friction between discontinuity faces, the rock blocks loose contact and the form of the excavated section is indented. The form is given by the intersection of the planned excavation profile and the dominant discontinuities (Müller 1978).

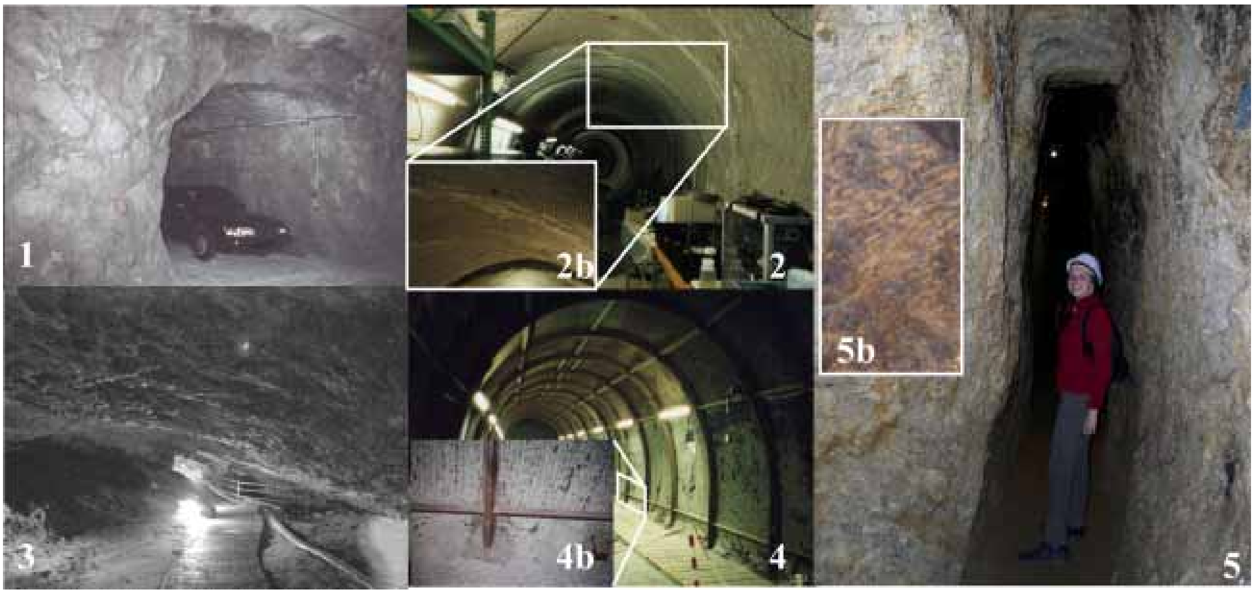

Figure 2. 1 Granite in Alps, excavation by drilling and blasting, the overbreak is clearly visible. 2) The same formation but excavated with a TBM. Only the traces of the cutter disks are visible in 2b). In 3) an example of excavation by water in a Devonian limestone show cave. The excavation follows the dip of the strata. Excavation with a roadheader in clay stone is shown in 4). Even in a close up 4b) only the traces of the cutter head can be seen but no other substantial overbreak. 5) Mine in carbonate rocks, in Alps, excavation by hammer and chisel 5b) no overbreak only traces of tools. Of all excavation techniques, drilling and blasting produces most overbreak

Does this mean that there is no overbreak with the other excavation techniques?

No, according to Müller (1978): during mechanical excavation without blasting, thus non-vibrating excavation, cavities that follow a zone of weakness in the rock or a discontinuity are only a local exception with a maximal extent in the order of decimetres. Only if very smooth planar discontinuities with an unfavourable orientation with respect to the excavation are present then parts of the rock mass can fall from the roof. But in general:
- in a bored tunnel the limit of the cross section follows the path of the tools, thereby a circular cross section is created as planned.
- in a tunnel excavated with a roadheader, pick hammer or tunnel shovel the pathways of the tools create a surface less adapted to the planned section.
- if the degree of separation is large and the rock strength high than the excavation with a roadheader can loosen rock blocks from the rock mass matrix and produce overbreak (Müller 1978).

Although blasting causes apparently the largest overbreak there is still a tremendous difference in the amount of overbreak created by poor and by good blasting.

What is the effect of poor blasting on overbreak?

The effect of poor blasting will cause higher overbreak, depending on the number of discontinuities, their strength, strength of the rock matrix etc.. Some examples are shown in figure 3.

How does one recognise poor blasting?

This is quite simple one should have a look at half-casts (see figure 4) which are present if the blasting was good. In France reduction of payment follows if a certain percentage of half-cast length cannot be found in the tunnel walls after blasting (Bouvard et al 1988).

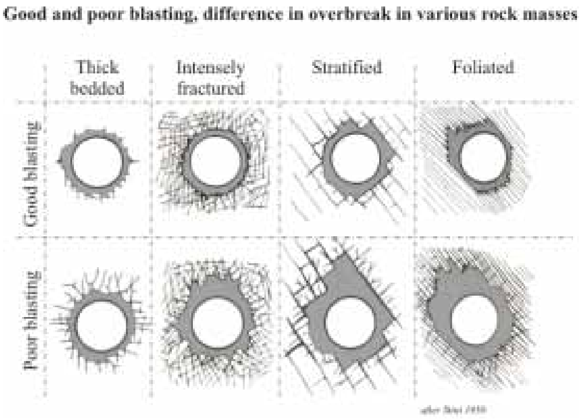

Figure 3: In this figure it is shown how poor blasting effects the amount of overbreak significantly in all type of rock masses

Figure 4: Evidence of accurate blasting: half pipes in LST. Place St Lambert, Liège.

What can be done to spare the rock mass during blasting:

First of all adaptation of the drill scheme to fit the geological conditions (Müller 1978) were made in the pre-jumbo area by experienced miners. They were specialists and took the geological conditions into account, especially those of the discontinuity matrix. Today's mechanised drilling, often done by rock-ignorant staff lost this experience and an individual discontinuity guided drilling is nearly impossible. An adaptation of the drilling scheme although theoretically possible, is not performed in praxis because neither:
- time, nor
- stimulus

is available.

The hereby caused disadvantages like:
- larger overbreak
- more cave-ins
- reduced stability
- increased water inflow caused by loosing up

are balanced by:
- faster advance => better utilisation of the available stand-up time
- more accurate in directionality => better pre-splitting without substantially increasing drill time

The disadvantages of drilling with a jumbo increase if the geology becomes more complicated, unpredictable and variable (Müller 1978).

What can be done in the post-jumbo era:

Some adaptations that can be made even today with the less flexible and adaptive drilling tools are:

A) a reduction of the depth of pull. The difference between good and poor blasting lies not in the use of high (shatter) explosives, that are (contrary to common believe) beneficial, but in a too large depth of pull (Müller 1978).

B) good blasting e.g. pre-splitting or smooth blasting

C) some simple adaptation of the drill scheme to fit the geological conditions example (see e.g. figure 5 and 6).

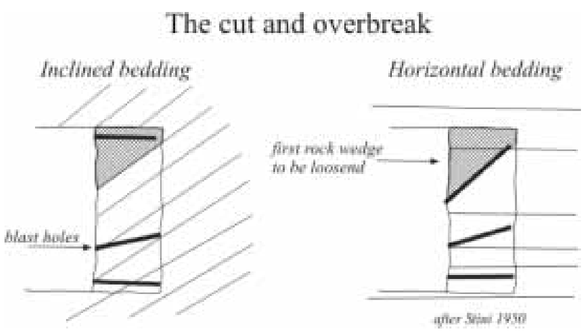

Figure 5. The inclination of the blast holes to make the cut should be inclined taking the discontinuities of the rock mass into consideration.

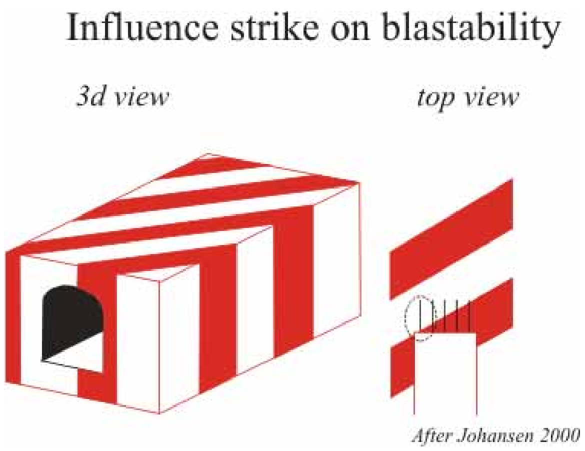

Figure 6. If the bedding planes strike at an angle between >0 <90° with respect to tunnel axis then the rock will break more easily on the left side, thus here less charge is needed.

Overbreak and primary stress

The height of the in situ stress has little influence on the overbreak but the relation between major and minor stress and the stress condition with respect to the rock strength does. If the stresses are anisotropic ($s_3/s_1>2$), a condition that prevails usually in shallow tunnels, the tendency towards:

- larger overbreak
- larger secondary overbreak
- asymmetric excavations

increases with respect to isotropic stress ($s_3/s_1<2$) often found at greater depths. In the latter case the stresses work from all sides on the tunnel and the rock blocks can form an irregular arch, leading to less overbreak (Müller 1978).

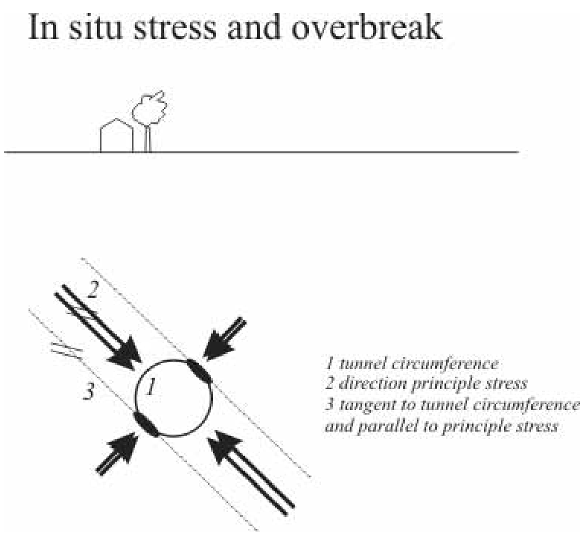

Figure 7. The direction of the principle stress has influence on the position where overbreak will form in the tunnel: the overbreak obtains a maximum value at the intersection of line 3 with the tunnel circumference

Stress/strength rock (mass) ratio

If the rock strength is high one needs large amount of explosives to excavate the rock. If a large amount of explosives are used the rock mass is loosened up considerably. If the degree of separation of the discontinuity matrix is high this leads to large overbreak values. But if the rock strength is low due to e.g.:

- weathering
- retrogressive metamorphism

it is possible to excavate close to the planned profile with very small deviations of 3cm.

Thus if the compressive strength of the rock matrix is:

< 25MPa, then the rock will break more easily along material bridges in the rock => less overbreak, especially if the degree of separation is low. However, if the rock strength is low the overbreak is low and the risk of cave-ins or secondary-overbreak rises.

> 25MPa, then a lot of explosives are needed to break the rock along material bridges but this will only increase the

degree of separation and cause more overbreak, the more if the degree of loosing up and the initial degree of separation are high (Müller 1978). These effects are shown in figure 11.

Overbreak and water

Water affects especially secondary overbreak. Water:

- reduces the strength of the infill material of the discontinuities
- generates seepage forces
- exerts hydrostatic stress

By these processes the fall of friable and small rock fragments into the excavation accelerates to such an extent that even with swift working and suitable support unavoidable overbreak is produced. In loosened up rock masses with small a discontinuity spacing, high degree of separation of the rock blocks and a lubricating discontinuity infill, the overbreak can reach a value surpassing the overbreak that would have occurred under dry conditions several times (Müller 1978).

Excavation form and overbreak

The better the form of the discontinuity matrix and the in

situ stress is followed the less overbreak will be created.

Square versus round tunnel

If for example the discontinuities are horizontal a square tunnel form can be applied with less overbreak as is shown in figure 8. In general if vertical or horizontal discontinuities are dominant rectangular profile are suggested (Müller 1976).

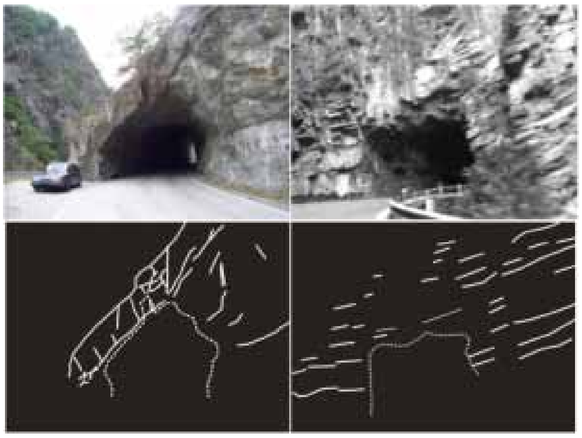

Figure 8. Schnalstal (I). If the bedding is horizontal, the shape of the tunnel can be square. Eggenschlucht (I)

Pointed arch form:
In layered (fractures, bedding planes, etc.) the shape of the excavation in the roof will be elliptical /pointed arch with the long axis perpendicular to the apparent dip of the layering (see figure 9), the stress concentration is increased in the sharp corner, the confinement of the rock blocks is increased, and thereby the stability of the underground opening is increased. A pointed arch imitates nature.

Egg-shaped and circular profiles:
Egg formed and circular excavation profiles are best suited if:
- several (more than one) discontinuity families are present, with
- inclined position to the tunnel

Profile ending in sharp corners:
Where the circumference is more sharply curved a larger arching effect occurs in the rock mass, and the rock blocks are tightly held in place, which would fall out if e.g. the roof would have been designed more flat (Müller 1976).

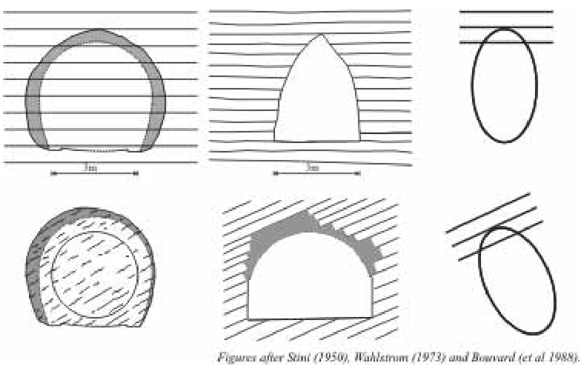

Figure 9. Shape of stable excavation with respect to the apparent dip of the layering,

Discontinuities: fractures and overbreak
Almost every stratified rock breaks readily along bedding planes. Therefore the bedding planes constitute a source of mechanical weakness. In schist cleavage planes have a similar effect. Sections of tunnels in closely jointed and faulted rocks tend to be characterised by considerable overbreak if excavated by drilling and blasting (Wahlstrom 1973). The difference in overbreak between thick and thin bedded inclined layers is shown in figure 10. According to figure 8 it is obvious that any attempt to create a circular tunnel in a rock mass with active and persistent joint sets intersecting the tunnel axis at 45° produces irregular and asymmetric overbreak.

Figure 10: Overbreak decreases with decreasing bedding thickness. Again an example demonstrating that the RMR is not directly related to the overbreak. Passeirertal (I).

Summary
A summary of the coupled parameters: rock matrix strength, discontinuity spacing, number of discontinuity families and apparent dip on the overbreak is given in figure 11.

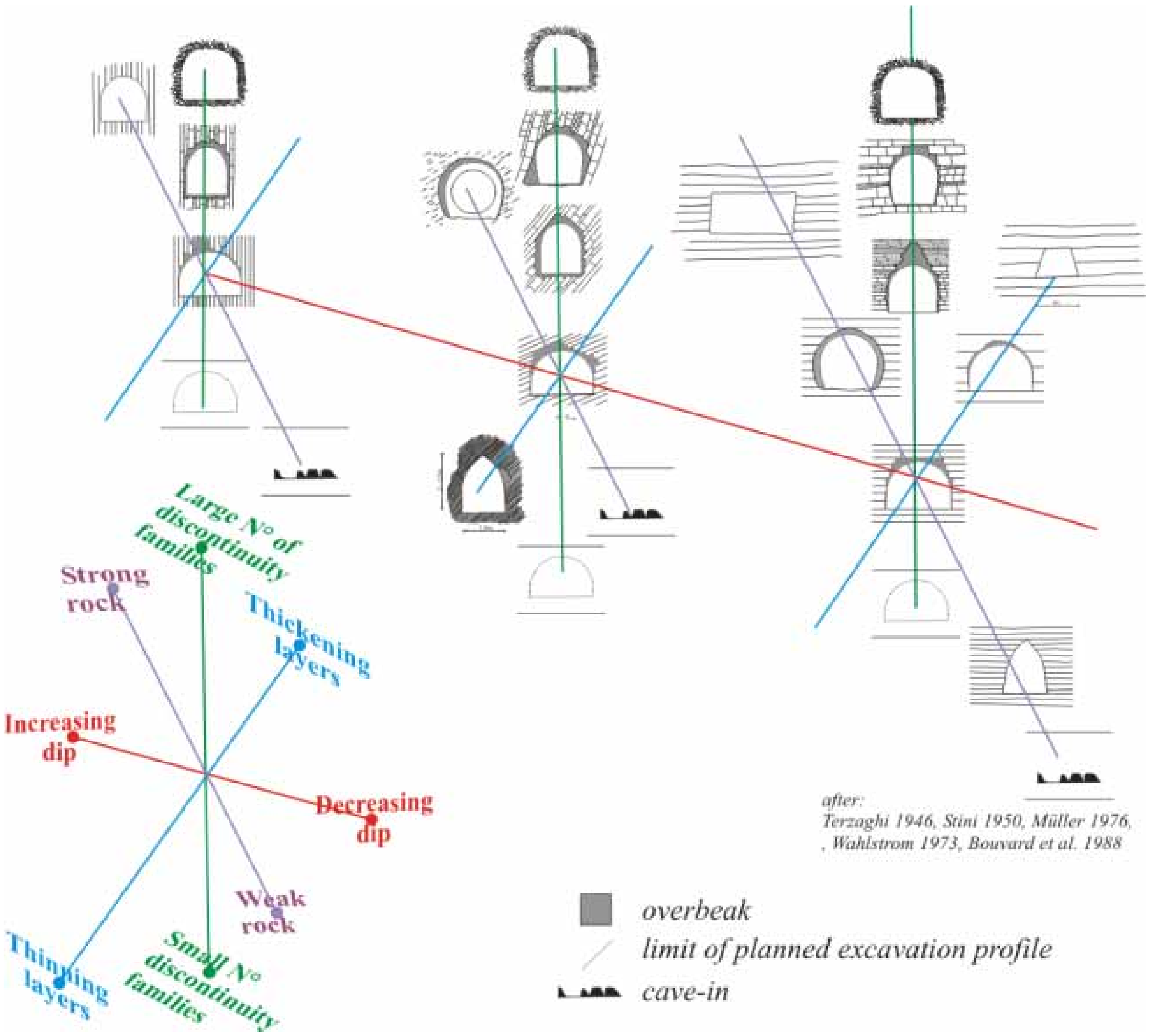

Figure 11. In this figure a summary is given of the coupling between: the number of discontinuity families, thickness of the rock blocks, apparent dip and strength of the rock matrix. With the aid of this overview different qualitative observations can be made:
- *If the apparent dip rotates from near horizontal to vertical the amount and position of the overbreak changes. The largest overbreak can be found in steeply inclined layers.*
- *If the strength of the rock mass increases and the apparent dip is near horizontal there will be no overbreak in tunnels with a rectangular form*
- *If the strength of the rock matrix decreases to zero, in all cases cave-in will result.*
- *If the number of discontinuity families increases and the strength of the rock matrix is not too weak the overbreak is generally small.*
- etc.

5 Task of the engineering geologists:

Structural analysis 3d: Degree of separation
In the analysis up to now it was assumed that the rock blocks are dissected from the rock mass matrix by a discontinuity parallel to the paper. The degree of separation in this plane is 1, i.e. there are no rock bridges left. In reality is of course not very common (see figure 12).

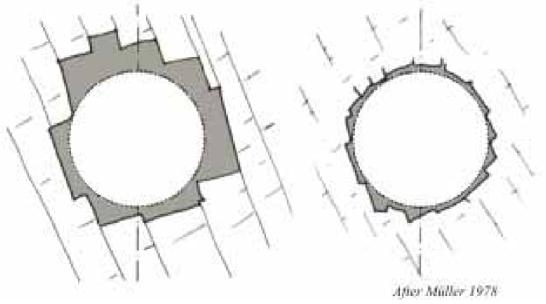

Figure 12. The only difference between the figure on the left and the one the right is the degree of separation.

To quantify this difference in overbreak shown in figure 12 one needs three parameters:

i) discontinuity frequency of a discontinuity family (figure 13).

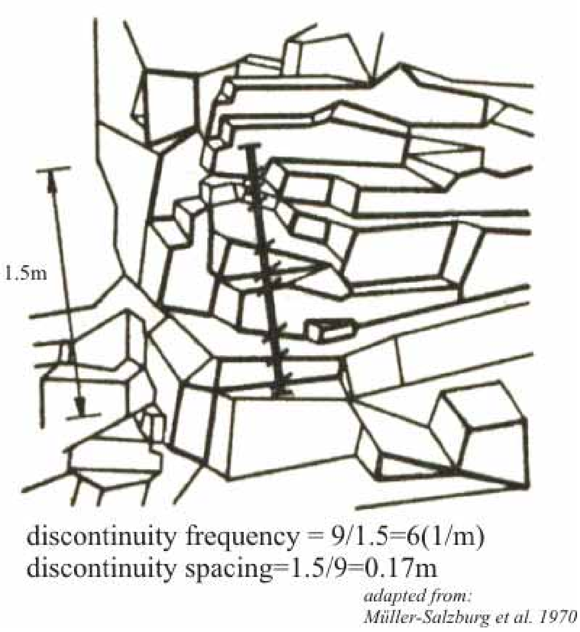

Figure 13. Only the spacing of the discontinuities does not give reliable information on the amount of overbreak that can be expected.

ii) the form of the rock blocks (figure 14)

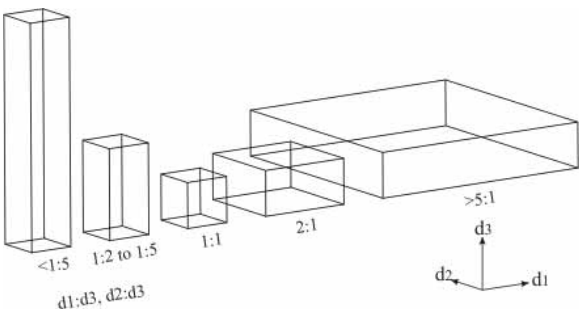

Figure 14. Next to the information about the discontinuity spacing the form of the rock blocks has to be determined.

iii) the degree of separation x (Pacher 1959, Müller 1958,1959). A summary of the theoretical background of the degree of separation can be found in Schmitz (2003).

With the three parameters mentioned above: the degree of separation, the form of the rock blocks, and the discontinuity spacing it is possible to give a quantitative prediction of the amount of overbreak like the one Müller (1978) has developed (see figure 15).

Note that the graph in figure 15 has two theoretic points:

i) at a very large spacing between two parallel discontinuities of 1 family the overbreak = 0 because the spacing is larger than the diameter of the tunnel
ii) at a very close discontinuity spacing the overbreak is near zero because now one deals more with a soil than with a rock, in which as discussed before the overbreak is negligible, the cave-in risk, however is increased.

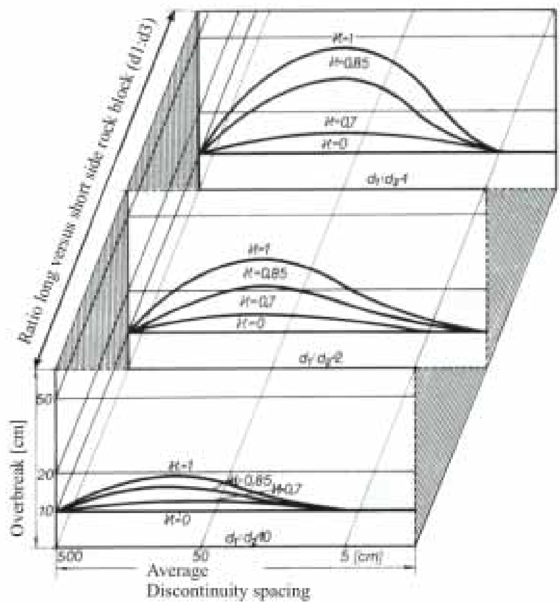

Figure 15. Prediction of the amount of overbreak on the basis of the average discontinuity spacing, the form of the rock blocks, and the degree of separation of the rock blocks from the rock mass matrix (x).

Based on figure 15 one can observe that:
- the more cubic shaped the rock blocks are
- the larger the degree of separation is,

the larger the overbreak will be. The maximum overbreak will be attained at an intermediate discontinuity spacing.

This method has recently be applied in a tunnel under construction by the university de Liège (Viroux 2003).

special case large degree of separation x = 1

In a special case the procedure described above can be simplified (especially the determination of the volumetric degree of separation is quite labour and time intensive, and time is not always available for the geologist in situ to describe the outcrop after each blast because the mucking must start as well): if
- the degree of separation is large (the rock blocks are free dissected blocks)
- the form of the rock blocks is cubic
- the strength of the rock matrix is high (i.e. rock blocks do not split due to excavation or fall)

an excavation in this rock mass will follow the rock block boundaries. It is not possible to position the charge in such a way that the rock blocks are dissected without being loosened from the rock mass matrix.

The average overbreak in this case (Müller 1978):
- is equal to the average discontinuity spacing if the fractures are closely spaced (<< diameter tunnel)
- (overbreak in the roof) can be determined by measuring the length between the outcrop of a discontinuity in one tunnel wall to the continuation of the discontinuity in the opposite wall (=discontinuity length roof) = dlr. This length should be determined taking the 3D orientation into account. If this is not possible it can be estimated using the information in 2D. The average overbreak in the roof is approximately equal to 1/4 * dlr, that in the floor equals 1/3 * (average overbreak roof). This approach is based on the assumption that the rock blocks will fall if:
- they are fully separated from the rock mass matrix, and if
- their centre of mass lies within the planned excavation profile. This simple method approached the actual overbreak in tunnels very well (Müller 1978). For more complicated rock block assemblages the overbreak can be simulated by using base friction models as is shown in fig 16.

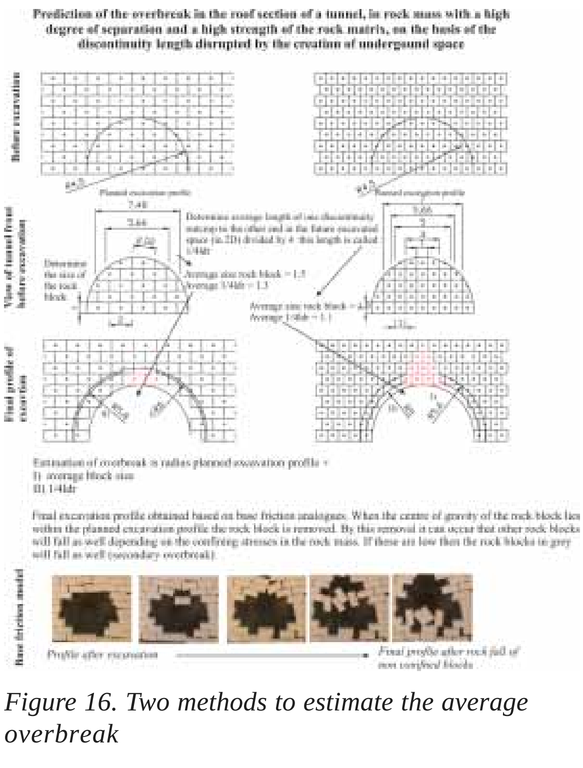

Figure 16. Two methods to estimate the average overbreak

Special case intermediate degree of separation (0.7 < x < 1)
If the degree of separation is smaller than 1 the rock blocks are not entirely separated from each other nor from the rock mass matrix i.e. rock bridges exist between the individual rock blocks. In this case the overbreak is a factor f smaller than in the case x = 1. The value of f depends on the strength of the rock bridges. The strength of the rock bridges depend on the degree of separation and on the rock strength. One has to distinguish in this case between:
α) Strong rocks. If they behave in a brittle way during a blasting round, the rock will take up the stress by continuous failure along existing path of weakness like the fractures. If these are extended to the next fracture, the rock blocks will be separated, x = 1, and the overbreak can be approximated by the method described above. If not the overbreak can be estimated by a linear interpolation of the values that would have been obtained at x = 1 and x < 0.7.
β) Weaker rocks: In weaker rocks it is not the position of the discontinuities that determine the form of the excavation but more the position of the blast holes. In this case the overbreak will attain values between half and one time the average discontinuity spacing.

Special case low degree of separation (0 < x < 0.7)
If the degree of separation x, lies between 0 and 0.7, the overbreak will be only a fraction of that if x = 1, the value will attain at maximum some cm (Müller 1978).

Structural analysis 3d: Direction of tunnelling
The discontinuities of the rock mass do not have an arbitrary orientation, but they are often organised in families. The number of families is related to the geological history. The use of families is a simplification that cannot be tolerated in some cases. This decision must be made by the engineering geologist. The stereographic projection is the most used method. After statistical treatment the orientation of the discontinuities must always be considered in relation to the direction of tunnel driving (AFTES 2003). Thus, not only the presence of discontinuity planes on which sliding is possible are important but their orientation with respect to the tunnel.
An example of the three dimensional character of overbreak is shown in figure 17. A horse shoe shaped tunnel is driven with an azimuth of 105°. The tunnel will experience some overbreak but nothing compared to the situation if it is driven at an azimuth of 15° (see figure 17).

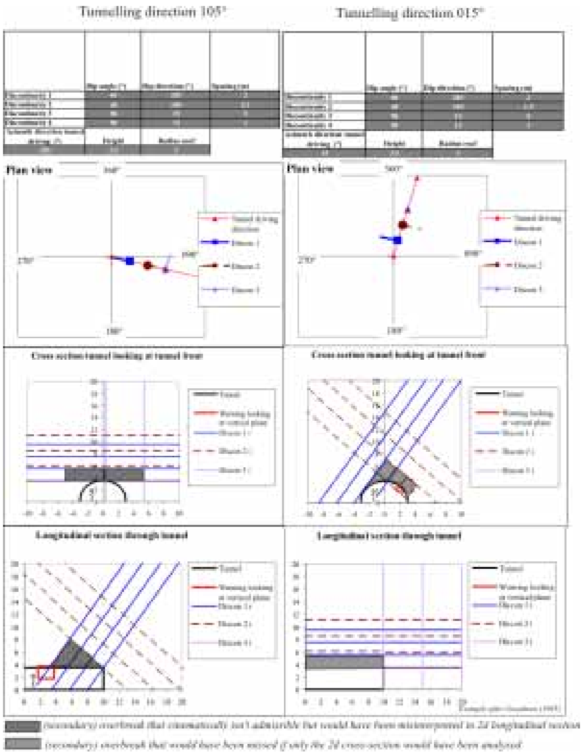

Figure 17. Same discontinuity orientation, other tunnel orientation, much more overbreak determined using a simple spreadsheet TunnelDip.

6 Conclusion

- It is a mistake to believe that if the RMR increases the overbreak decreases at all times or vice versa
- Rock masses with a low RMR are prone to cave-in, not to overbreak
- Not all the distance between the circumference of the excavation and the planned excavation profile is overbreak. It can be either an instantaneous "genuine" overbreak, secondary overbreak or cave-in or rock burst
- Of all excavation techniques blasting produces most overbreak.
- Overbreak is influenced by the presence of water, in situ stress, rock strength, layer thickness, number of discontinuity families, persistency of the discontinuities, orientation of the discontinuities with respect to the tunnel, infill etc. Most of these elements are coupled. These effects are summarised in figure 11.
- Overbreak is a 3D phenomenon and the orientations of the discontinuities should be analysed in relation to the orientation of the tunnel.

Rules of thumb:

- Average overbreak in rock masses with loose (x=1) rock blocks with cubic form, high strength rock matrix, is approximately equal to the average discontinuity spacing
- Average overbreak in rock masses with loose (x=1) rock blocks with cubic form, low strength rock matrix, is approximately equal to half the average discontinuity spacing
- Average overbreak in the roof of the tunnel in rock masses with loose (x=1), high strength rock matrix, is approximately equal to 1/4*ldr
- Average overbreak in the floor of the tunnel in rock masses with loose (x=1), high strength rock matrix, is approximately equal to 1/12*ldr
- Average overbreak in rock masses with partial fixed rock blocks (0.7<x<1), high strength rock matrix, can be obtained by a linear interpolation of the value at x=1 and x<0.7.
- Average overbreak in rock masses with fixed rock blocks (<0.7*x) with cubic form, high strength rock matrix, is only a fraction of the overbreak if x=1, often only several cm

7 Abbreviations

MBR	modified basic RMR
RMR	rock mass rating
1/4ldr	average length of one discontinuity outcrop to the other end in the excavated space (in 2D) divided by 4
x	$= x_{r\alpha}$ = Volumetric share of discontinuity planes of discontinuity family α (m^2/m^3) or short: the degree of separation of rock blocks from the rock mass matrix

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CONGRESSEN

Grouting and Ground Treatment 2003, New Orleans 10-12 februari

Jacco Haasnoot, Adviesbureau Noord/Zuidlijn (Witteveen+Bos) Entrada 231, 1096 EG Amsterdam, jacco.haasnoot@nzlijn.nl

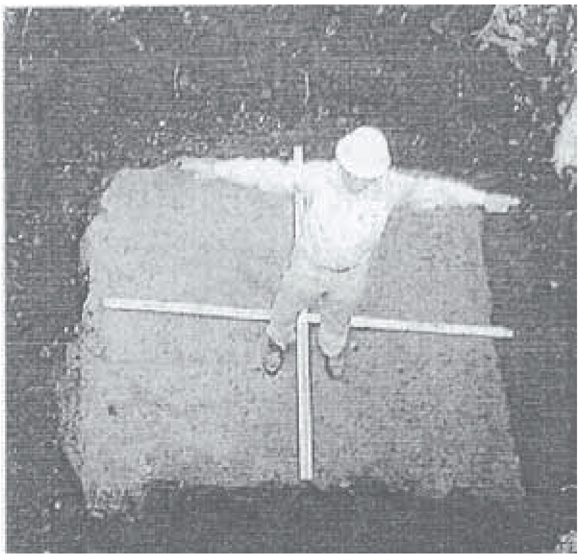
In al het congressengeweld is het Grouting and Ground Treatment congres de enige conferentie die specifieke gaat over grondverbeteringstechnieken op basis van grouts. Het congres wordt georganiseerd door het Deep Foundation Institute in samenwerking met het Geo-Institute van de American Society of Civil Engineers. Het congres wordt slechts eens in de 10 jaar georganiseerd. Het congres werd van 10 tot 12 februari voor de tweede keer in New Orleans georganiseerd.

Het congres is onderverdeeld naar de verschillende disciplines die binnen de grondverbeteringstechnieken aanwezig zijn. Met name de keynote lectures door exponenten in het vakgebied waren zeer interessant. Professor Stuart Littlejohn had drie maanden sabbatical opgenomen om uitgebreid de geschiedenis en ontwikkelingen op het gebied van permeation en compensation grouting te beschrijven. Een uitgebreid exposé over grouting in rock masses werd door Giovanni Lombardi gehouden. De Japanner Shibazaki van de Chemical Grouting Company, een pionier op het gebied van jetgrouting had een opmerkelijke ontwikkeling te melden, namelijk de vierkante jetgroutkolom (zie figuur). Volgens Japans gebruik komen de maan en de aarde er aan te pas om de totstandkoming van de vierkante jetgroutkolom te beschrijven.

Tijdens het congres werden aan vijf prominenten in de groutwereld, allen overigens afkomstig uit de USA, de Grouting Greats awards uitgereikt. Tussen de Greats ook de Engineering Geologist Ken Weaver, auteur van het boek “Dam Foundation Grouting”. Een andere opmerkelijke “Great” is James Warner, pionier op compaction grouting gebied. Warner is op jonge leeftijd, geheel volgens de Amerikaanse traditie, vanuit zijn achtertuin begonnen met het toepassen van low-mobility grouts, die vaak bestaan uit niet meer dan een grondpasta zonder toevoeging van cement. Andere opmerkelijke Warner trivia: James Warner verzamelt Wurlitzer orgels.

Op het gebied van grout materialen gaf Professor Jefferis (University of Surrey) een interessante lezing over de chemische consequenties (gevaren) aan het gebruik van grouts in de ondergrond. Bij het gebruik van cement grouts bestaat in bepaalde omstandigheden de kans op ammoniak vorming. Als bijproduct van Natrium-silicaat grouts noemt Jefferis zeepachtige materialen, waarbij hij een voorbeeld aanhaalt van een tunnelboorproject in Caïro waar het dichtblok uitgevoerd is met een chemische grout. Bij het vertrek van de slurry tunnelboormachine gaf het grout een reactie met de bentoniet suspensie waardoor de graafkamer zich geheel en al met zeepbellen vulde hetgeen de slurry niet meer verpompbaar maakte. Een anti-schuim middel heeft hierbij voor de oplossing gezorgd.

Tot slot wil ik de interessante geofysische toepassing van de uit Italië afkomstige Dr. Morelli noemen. Electro-tomography wordt gebruikt om na de injectiewerkzaamheden de omvang van het groutlichaam te bepalen. Volgens Morelli is de techniek relatief goedkoop en inzetbaar in alle typen grond. De presentatie van de resultaten, in kleurrijke 3D figuren, zijn in ieder geval indrukwekkend te noemen.



vierkante jetgroutkolom

(Re)Claiming the underground SPACE

Wij eisen de ondergrondse RUIMTE (weer) op

Richard Rijkers

Van 12-17 april 2003 vond in Amsterdam het ITA World Tunnelling Congress 2003 plaats. ITA is de afkorting van International Tunnelling Association. Deze organisatie is in Zwitserland gezeteld en promoot het aanleggen van tunnels en (voor deze gelegenheid in 2003) het ondergronds ruimtegebruik in algemene zin.

Na de formele start van de conferentie op zaterdag 12 april met een "Executive Council Meeting ("invitation only"), sloot ik me maandagmorgen in de RAI van Amsterdam aan bij het internationale congresgezelschap van meer dan 700 deelnemers uit meer dan 50 verschillende landen. De onderwerpen waren breed geprogrammeerd van *Immersed and floating tunnels* tot *TBM cutting tool performance*. Om een dergelijke conferentie enigszins nuttig te maken, beperkte ik me tot het bezoeken van de papers over tunnels in slappe grond en natuurlijk kunstmatige grondbevriezing. Dusdoende kreeg ik in krap drie dagen de status, de voortgang en hoogtepunten van tunnels te horen van alle aanstaande, lopende en afgeronde tunnelprojecten in Nederland; zoals het tunneltraject van het Pannedernsch Kanaal (Betuweroute) geboord door de opgebrachte kunstmatige dam in de voormalige zand afgraving, en de monitoringsaspecten bij 's werelds grootste TBM van de Groene Hart tunnel: "Maxima". Ik moest uiteindelijk tot één unanieme conclusie komen: alle tunnelprojecten lopen probleemloos, gesmeerd en volgens planning!

In de sessie "Research & Development" viel mij het verhaal op over "Self-compensating TBM for reducing ground settlement" (Xu, Grasso & Guglielmetti). Dit is nog steeds een onderwerp dat in Nederland steeds in de belangstelling staat vanwege de Noord/Zuidlijn onder de kwetsbare Amsterdamse grachtenpanden. Ook andere papers in deze sessie besteedde daarom aandacht aan dit onderwerp. De oplossing voor maaiveldbeheersing wordt gevonden in (1) groutdrukbeheersing, (2) het TBM-ontwerp, (3) grondcondities en/of (4) de kwaliteiten van de TBM-piloot. Na deze sessie was ik meer onthutst dan bij aanvang en moest de dag beëindigen met de gedachte dat de oplossing echt niet eenvoudig is. Uiteindelijk heb ik me gevoegd bij de borrel op de Herrenknecht stand (hier was na vijf uur koud bier beschikbaar!) om naar een meterslange maquette te gluren van het prototype voor de Noord/Zuid metrolijn. Enige discussie met ingewijden bracht mij toen wel tot nieuwe inzichten.

Zeer verhelderend vond ik de vier lezingen over de waterige gebeurtenissen in de Haagse Tramtunnel. Na jaren lezen van onbevredigende krantenartikelen hebben de aanwezige toehoorders de 'ware' toedracht tot de schade tot zich genomen: "een ongelukkige samenloop van omstandigheden, welke ook na vroegtijdig uitgevoerde risicoanalyse, niet voorzien konden worden". Dagen daarna spookte dit door mijn hoofd, maar bij mij waren blijven hangen de onverwachte hoge pH-waarde (> 12) bij de veenlagen en de gaten in de groutboogconstructie.

Het ITA 2003 congres eindigde met tien *Amsterdam Statements 2003*. Naast *There is no space like underground space* en *No guts, no underground glory* waren er ook een paar die wel hout sneden: *In future sustainability has to be a bigger issue regarding the use of Underground Space.* Na het afsluitende avondbanquet in de Beurs van Berlage was ik klaar met dit congres en verlangde ik naar mijn bed. De trein die me naar huis bracht bleef die avond bovengronds.

PROFESSOR'S COLUMN
Youth sins and the future

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It is with some hesitation that I agreed to the request to contribute to this issue of INGEO journal, as I am not an engineering geologist myself. Yet, I have to confess a few youth sins in that field, before coming to the heart of the matter.

In the seventies I studied possible quarry sites in the rainforest-clad Bakhuis Mountains of Western Suriname to extract ballast material for a railroad. The railroad was built, but not with the ballast material I indicated: the enderbitic rock I found was too good, with only 8.1% of wear in the Los Angeles abrasion test and 2-8 tons in the Brazilian splitting test, if anyone still knows what that means. Instead they used a kind of mylonite with the consistency of marshmellows. That was cheaper. But never mind, the railroad never came into operation, and the bauxite it was to bring to the Corantijn harbour is still quietly lying in the mountains. Now, thirty years later, a new mining concession for this bauxite has been given to a consortium of Suralco (Alcoa) and Billiton, the two companies that are already active in the country since decades. But the bauxite will be transported by truck, and the 400 million guilder railroad probably will rust eternally under its fresh cover of tropical rainforest. All in vain.

A second project I was involved in was tracking the trace for a new road between the towns of Tadó and Nuquí in the Chocó Department, in the westernmost part of the Colombian Andes, a virtually unexplored terrain under virgin rainforest with an annual rainfall of 10,000 mm. And it also remained unexplored by me: my colleagues and I just studied aerial photographs and radar imagery and never visited the field. Being curious geologists, we designed road cuts exactly at the sites where we wanted outcrops in the strongly folded turbidites that were known to occur there. If the road was ever built, engineers must have wondered about the strange twists the geological report recommended.

My third experience was studying the geology of an area in the eastern slopes of the Eastern Cordillera in the Colombian Andes to explore the possibilities for the construction of a hydroelectric dam in the Upper Caquetá river. But this densely forested area was so infested by guerrillas that field work again was out of the question. Nevertheless, studying the geology here was a very interesting experience. Apart from poor aerial photographs, Landsat imagery and some early reports we had the travel account of Emil Grosse, a German geologist that travelled down the Upper Caquetá on muleback during several months in 1926. His geological observations were so extensive and precise, that we could identify all geological units he described in our aerial photographs. The Colombian company Geoestudios made a new map three years ago on the basis of extensive field work and showed our map was indeed quite good. But the dam has not yet been built, as guerrilla activity has only increased since then. One of the Geoestudios geologists was kidnapped in the field and nothing has been heard of him since then. All in vain.

So far for the youth sins. If any lesson can be drawn from them, it is that engineering geologists should not be too eager to see their work turned into real infrastructure. Be prepared for disappointments and disappearances.

There is little science in all this, and that is one of the major problems that engineering geology faces in the future. Is it still an academic discipline, or rather a practical application based mainly on existing knowledge and routine procedures that preferably should be taught at polytechnical schools? I believe there are exciting new perspectives that merit a resuscitation of the subject at universities. This belief is based upon my fourth experience, which continues to the present day: working alongside engineering geologists at the Department of Applied Earth Sciences at Delft University of Technology during seven years.

Now that several underground tunnelling projects in the Netherlands have encountered unexpected difficulties, such as the stiff glauconitic Boom clays at the Westerschelde tunnel, and the high water pressure in the Holocene barrier sands under The Hague, we must realise how little we *really* know about the shallow subsurface. This is primarily due to the fact that Dutch engineers always have treated the soft-rock subsurface of their country as rippable, and therefore homogeneous. Just distinguishing between clay, sand and peat was enough. Now that new tunnelling technologies emerge that require more detailed data on the subsurface and higher demands are being put on safety and cost effectivity, this appears to be a gross simplification. We do not really know why different types of clay have different geotechnical properties. And we also cannot predict very well whether we will find clay or sand at a certain spot. Almost all our information is based on point observations: CPT's, borings, and with a bit of common-sense geology and a bit of stochastic tinkering in 3-D GIS models we think we have a reasonable control of the 3-D architecture of the subsurface.

But that is not enough for the future.

We need direct 3-D information, and just as oil companies obtain 3-D data cubes from seismics in the deep subsurface, 3-D geophysical surveys should become standard in engineering geology practice. There is a tremendous upsurge in the development of new tools in shallow geophysics, both land-based as from vessels, which will greatly improve our knowledge of the shallow subsurface. We are just in the beginning.

Furthermore, we need a much better understanding of how to translate geological and geophysical properties into geotechnical parameters. This requires much fundamental research in wave propagation through soft rock materials, clay mineralogy, compaction, overconsolidation and diagenesis. One of the most exciting developments is the direct translation of shear-wave seismic data into CPT readings. This is at the heart of engineering geology. Engineering geology is not just a bridge between geology and engineering, which you may or may not use, it is the fundamental science that studies the geotechnical behaviour of an intrinsically heterogeneous shallow subsurface. Investments in these new directions in engineering geology will not be in vain.

INGEOKRING ACTIVITIES

Duinexcursie

INGEOKRING ACTIVITIES

Lezingenavond water in de Haagse Tramtunnel

Jacco Haasnoot

Op 26 juni 2003 organiseerde de Ingeokring een lezingenavond met als titel: “Water in de Haagse tramtunnel””. Geodelft fungeerde als gastheer voor de 25 geïnteresseerden. Dirk Luger, werkzaam bij Geodelft en adviseur op het project, gaf twee presentaties. Als stand-in voor Jan Kruyt van de aannemerscombinatie Tramkom leidde Dirk Luger het project in, hetgeen gezien de complexiteit van de constructie en de ontwikkelingen rond het project geen overbodige luxe was. Vervolgens presenteerde Dirk Luger vanuit zijn expertise over de problemen met het intensieve bemalingsplan in het diepe gedeelte van de ontgraving. De derde presentatie van de avond werd verzorgd door professor Han Vrijling aangaande de probabilistische analyses die door TU Delft verricht zijn naar de jetgroutboog.

Souterrain Den Haag
Het Souterrain project is midden in Haagse binnenstad en biedt plaats aan twee ondergrondse stations voor Randstadrail en een ondergrondse parkeergarage. De tunnel wordt met behulp van de wanden-dak methode gemaakt. Voor de wanden zijn diepwanden en damwanden toegepast. Het geotechnisch profiel bestaat voornamelijk uit zand met een enkel veenlaagje. Deze geologische omstandigheden maakt het noodzakelijk een kunstmatige bodem afdichting te maken. In het gedeelte van de Grote Marktstraat is dit met behulp van de jetgroutboog uitgevoerd, ter plaatse van station Spui en de Kalvermarkt is voor een oplossing met een softgellaag gekozen.

Het project is beroemd en berucht vanwege de niet goed functionerende jetgroutboog en heeft dientengevolge een grote vertraging opgelopen. De bouw van het project is in maart 1996 begonnen, twee jaar later zijn de eerste grondvoerende wellen opgetreden. Vervolgens heeft de bouw een kleine twee jaar stilgelegen, totdat in juli 2000 de aannemerscombinatie Tramkom een nieuw contract met de gemeente Den Haag heeft gesloten. Dit contract bevat delen van het oorspronkelijk contract, zo wordt het gedeelte met de soft-gellaag volgens de besteksoplossing uitgevoerd. Voor het gedeelte met de jetgroutboog is overgegaan op een Design& Construct contract, waarbij de aannemer voor de luchtdrukoplossing heeft gekozen. Het goede nieuws is dat de tunnel nu een waterdichte bodem heeft en dat volgens de huidige planning de eerste tram eind 2004 door de tunnel zal rijden.

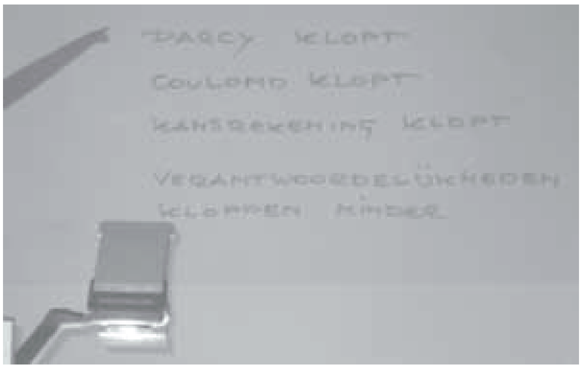
De constructie was oorspronkelijk niet gedimensioneerd op het toepassen van verhoogde luchtdruk. Bij het toepassen daarvan komen twee problemen naar voren. Ten eerste is de bouwkuip te licht en wil door de luchtdruk naar boven verplaatsen. Dit is tegengegaan door ballast in het station aan te brengen en bij de doorsnede met een damwand een nagenoeg verticaal groutanker toe te passen. Het tweede probleem is de vloer waaronder de luchtdruk aangrijpt, deze is daar niet op berekend. Waar de tussenvloer reeds gereed is, is een vakwerkconstructie gemaakt om de krachten op te nemen. Voor de doorsneden waar de tussenvloer nog niet gestort is, is overgegaan tot het toepassen van een lijnscharnier in combinatie met ballast waardoor het optredende moment in de vloer beperkt blijft. De ballastblokken schijnen afkomstig te zijn uit de Deltawerken voorraad, ook in dit civieltechnisch project blijkt eens te meer dat de Deltawerken nog steeds een grote invloed hebben op de huidige civiel-technische werken.

Verstopte filters
Na de informatieve introductie van het project vervolgde Dirk Luger de avond met de presentatie van zijn paper “Clogging of groundwater wells above a gel layer during construction of an underground station” (ITA Amsterdam 2003). Dirk gaf daarin de zoektocht weer naar de oorzaak van de verstopte filters (bemalingsputten). Uiteindelijk blijkt de combinatie van een oude uitlogende gellaag, veenlaagjes en snelle stroming funest te zijn voor de capaciteit van een bemalingsput. Binnen enkele dagen zijn de filters verstopt. Dirk Luger geeft aan het eind van zijn presentatie aan dat een ervaringsdatabase de oorzaak en dus de oplossing sneller naar voren hadden kunnen brengen. In dit kader is de paper “Long term performance of grouts and the effects of grout by-products” van Professor Jefferis op het Grouting and Ground Treatment congres van eerder dit jaar aan te bevelen (zie het congresverslag elders in deze Newsletter). In deze paper belicht professor Jefferis de chemische consequenties van het gebruiken van grouts in grond, het blijkt dat dat niet altijd zonder gevaar is!

Risico analyse groutboog
Professor Han Vrijling besprak namens Professor van Tol op authentieke wijze twee afstudeeronderzoeken die op de TU Delft zijn uitgevoerd. In het eerste onderzoek is probabilistisch bepaald hoeveel onvolkomenheden (gaten) er in de Haagse groutboog te verwachten zijn, gegeven de onnauwkeurigheden van een jetgroutboog systeem. Uit deze analyse is gebleken dat de kans op nul gaten kleiner dan 28% is en de verwachtingswaarde

4,3 gat bedraagt. Op basis van deze analyses is in 1998 dan ook de ondubbelzinnige conclusie getrokken dat “het met aan zekerheid grenzende waarschijnlijkheid te verwachten is, dat de stabiliteitsproblemen het gevolg zijn van onvolkomenheden in de putbodem”. Met putbodem wordt in dit geval de jetgroutboog bedoeld. Het tweede onderzoek heeft het zwaartepunt in het bezwijkmechanisme van een onvolkomenheid in de jetgroutboog. Het doel van dit onderzoek is om te beproeven of verdere afbouw van het station met een intensief bemalingsysteem boven de jetgroutboog mogelijk was. Met behulp van zeer illustratieve proefnemingen is het gelukt om de bezwijkmechanismen te modelleren en te kwantificeren. Op basis van dit model is geconcludeerd dat afbouw met behulp van intensieve bemaling niet tot voldoende veiligheid zou leiden. Mede op basis daarvan is voor de verhoogde luchtdruk oplossing gekozen.

Tot slot uiteraard de conclusies. Een opmerkelijke conclusie is dat een groutboog, mits goed toegepast, een uitstekend concept is. Belangrijk bij het toepassen van een constructie element met dergelijke eigenschappen is het achter de hand hebben van (nood)maatregelen in het geval dat het niet goed gaat. De enige noodmaatregelen in de Haagse situatie was het volzetten van de bouwput met water. Een mogelijkheid tot reparatie is er dan niet. De laatste conclusie van de Professor is gegeven op de bijgevoegde foto.



Laatste conclusie van Professor Vrijling

NEWS

Status herinrichting Ingenieursgeologie TU Delft

Drs Richard H.B. Rijkers, Voorzitter INGEOkring
Augustus 2003

In de eerste helft van 2003 zijn er verschillende ontwikkelingen geweest omtrent het vernieuwde professoraat Ingenieursgeologie aan de TU Delft. Via deze weg wil ik u hiervan op de hoogte brengen.

In maart dit jaar ontving het bestuur van de INGEOkring een brief van prof. Dr. Ir F. Molenkamp (CiTG-Civiele Techniek, Sectie Geotechniek) waarin een research profiel van Ingenieursgeologie (ter discussie) werd voorgesteld. Deze lijst met potentiële onderzoeksrichtingen I.G. was opgesplitst in twee delen (1) onderwerpen van nationale en internationale relevantie en (2) van hoofdzakelijk internationale relevantie. De toon in de brief was positief en het bestuur las met genoegen dat er serieuze ideeën zijn over de status en de herpositionering van Ingenieursgeologie bij de TU Delft.

Het bestuur was verheugd met de visie van Frans Molenkamp dat I.G. haar fundamentele kennis vindt in de geologische wetenschappen. De meerwaarde van I.G., zo bleek ook de visie van Molenkamp, betreft dan de erkenning en herkenning van sterkte-eigenschappen, van slappe grondsoorten en gesteenten ten behoeve van het ontwerp en de bouw van constructies op en in de bodem. De unieke positie van I.G. betreft de fundamentele kennis van de samenstellende delen van de bodem. Het was prettig om te lezen dat het unieke domein van Ingenieursgeologie de samenstellende delen van de bodem betreft (minerale korrels, poriënvolume, water en gas), alsmede de natuurlijke vormingsgeschiedenis van grond/gesteente en het geomechanisch gedrag ervan onder belasting en bij ontgraving. Prof. Molenkamp stelde dat Ingenieursgeologie veelal gaat over geo-karakterisering, veldverkenning en geometrische modellen, en dat er een zekere overlap bestaat tussen Engineering Geology, Geotechnical Engineering en GeoEnvironmental Engineering. Frans Molenkamp noemt in zijn discussiestuk dat bij buitenlandse gerenommeerde universiteiten GeoEnvironmental Engineering (~GeoMilieutechniek) ontwikkeld is, een voor de TU Delft nauwelijks bekend vakgebied. Het bestuur van de INGEOkring vraagt zich af of dit werkelijk een separaat en nieuw vakgebied is. De kennisvelden van Environmental Geology wordt reeds sedert de jaren ’60 binnen de IAEG (International Association Engineering Geology and the Environment) als een volwassen subdiscipline beschouwd. Het is wel een feit dat dit voor de TU Delft een nieuw onderwerp is.

Het bestuur van de INGEOkring is van mening dat de unieke benadering van intrinsieke en heterogene eigenschappen van de bodem, die door civiel-technici nauwelijks worden beschouwd, een aparte hoogleraarspositie Ingenieursgeologie binnen de faculteit CiTG rechtvaardigt. De INGEOkring heeft in een reactie aan de TU Delft de volgende vier hoofdlijnen van Ingenieursgeologie beschreven als maatschappelijke relevant die daarmee bijzonder geschikt zijn voor modern (toegepast) academisch onderzoek aan de TU Delft:

1. Karakterisering van heterogene eigenschappen van de bodem (incl. veldverkenning en nieuwe technieken)
- Research naar ontwikkeling van betere en goedkopere technieken van in-situ grondonderzoek: CPT, grondmonstertechnieken, monitoring, geofysica (incl. ondiepe seismiek, hoge resolutie seismiek, grondradar) en remote sensing (incl. laser, multi-spectraal analyse).

2. Ingenieursgeologische modellen:
- Research naar de ontwikkeling van (digitale) geologische, geostatistische grondmodellen, data-processing en visualisatie gereedschappen van de ondergrond ten behoeve van civiele werken.
- Research naar validatie en formalisering van grote datasets (ISO, NEN, GEF).

3 Geo-hazard, onzekerheid en risco-analyse:
- Zonering, hazard en risico-analyse (op basis van gevalideerde grondgegevens) voor het ontwerp van civiele structuren.
- Specificatie en kwantificatie van onzekerheden en betrouwbaarheid.
- (Geïnduceerde) seismische risico studies en bodemdaling voor de olie en gas exploratie/exploitatie en mijnbouw.

4. Materiaaleigenschappen en materiaal modellen:
- Sterkte-eigenschappen, kruipprocessen als functie van verwering van bodem en gesteente.
- Grondverbetering processen (injectie, bevriezing en electro-osmose).
- Geomechanische gedrag van grond en zwakke gesteente.

Het was de doelstelling van de TU Delft om met deze inventarisatie (1) het onderwijs en onderzoeksprofiel van een toekomstige hoogleraarspositie I.G. aan te scherpen, (2) bij het bedrijfsleven de maatschappelijke

relevantie van Ingenieursgeologie zichtbaar te maken en (3) daarnaast steun te vinden voor extra financiën (derde geldstroom) welke blijkbaar gewenst is. Ik heb begrepen uit de resultaten van deze inventarisatie dat er een tiental partijen gereageerd hebben op dit verzoek van Frans Molenkamp, maar dat er weinig ‘geld’ verzameld is.
Het INGEOkring bestuur gaat er vanuit dat bij het wegvallen van onderzoek ook het onderwijs in Ingenieursgeologie verslechtert. Daarnaast blijft de INGEOkring bij haar standpunt dat onderwijs a-priori niet direct afhankelijk moet zijn van de industrie. Financiële steun vanuit het bedrijfsleven is prima, maar het is daarbij verkeerd om dit vakgebied te veronachtzamen wanneer er geen gewillige donateurs zijn. Vanuit het ministerie van onderwijs worden maatregelen om het onderzoeksprogramma te versmallen aangemoedigd. Op deze wijze zou er financiële ruimte komen voor ‘excellente’ vakgroepen en universiteiten.

Echter, ……. deze inventarisatie en de herdefinitie van onderzoeksrichtingen heeft inmiddels geleid tot een onderbouwd advies aan de decaan van CiTG (prof ir. Louis de Quelerij) om een hoogleraar Ingenieursgeologie aan te stellen (voltijds). Volgens Frans Molenkamp is dit advies in serieus beraad bij de TU Delft en moet er nog aan een aantal randvoorwaarden worden voldaan voordat er een vacature wordt open gesteld. Welnu, ……. dat lijkt voor mekaar. Dit positieve nieuws wordt door de het bestuur van de INGEOkring met gepast (maar met groot) enthousiasme ontvangen. Ik gebruik de term ‘gepast’ omdat Frans Molenkamp mij vertelde dat de aanstellingseisen erg hoog zijn (op het vlak van publicaties én commerciële contacten in de industrie). Daarnaast heeft de historie bewezen dat potentiële kandidaten spaarzaam beschikbaar zijn.
Naast de herinrichting van een nieuw professoraat I.G. is ook huidige reorganisatie binnen CiTG nog een relevant punt. De (her-)positionering van I.G. binnen CiTG is de volgende kwestie die nu verduidelijkt dient te worden. Hierover hoop ik u in de volgende Newsletter meer van te berichten.

Het bestuur is erg tevreden met de huidige ontwikkelingen en hopen dat de herinrichting van I.G. op deze wijze met succes kan worden vervolgd. *Op, in* en *met* de ondergrond van Nederland zullen we nog lang leven. Ik begin zo waarlijk te geloven dat er betere tijden aanbreken voor het universitaire onderwijs en onderzoek voor Ingenieursgeologie in Nederland.

[1] een kopie van het research profiel I.G. (discussie opgesteld door Frans Molenkamp) kunt aan aanvragen bij: r.rijkers@nitg.tno.nl

COLUMN

Does a Chord Tunnel strike the right chord?

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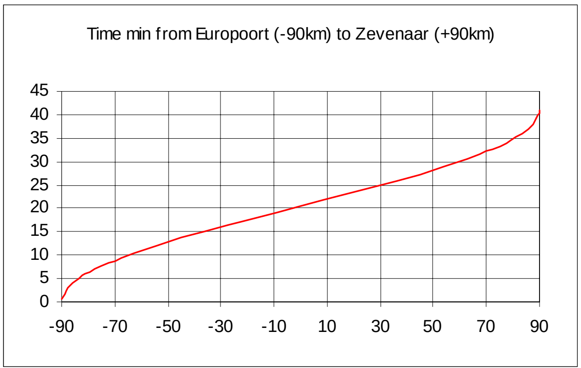
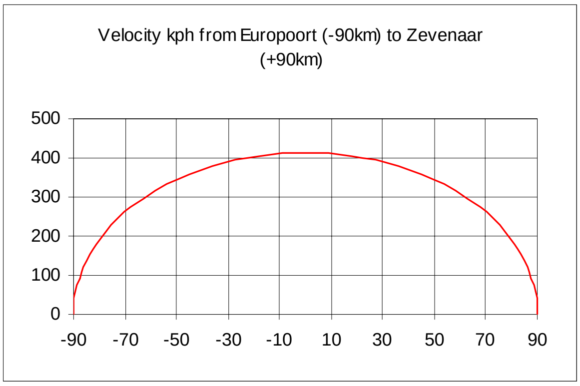
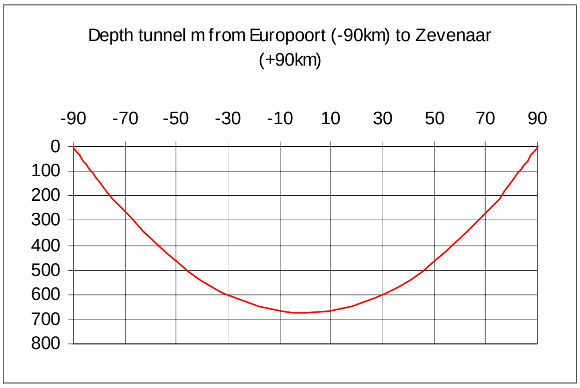
A tunnel that goes down deep into the earth but already one can see the light at the end of the tunnel from the tunnel entrance

The Betuwe Route from Europoort to Zevenaar consists solely of a goods train railway line. The line would have been an ideal candidate for a chord tunnel. What is a chord tunnel? Simply a tunnel that follows a chord through a sphere. As a first year civil engineering student in the UK a problem was set by the professor of mechanical engineering (we had a common first year with students from mechanical, electrical and chemical engineering) to analyse in a frictionless chord tunnel what the speed of the train would be at midpoint in a tunnel and the time the train needs to travel. If my present analysis serves me correctly for a 180 km Betuwe Chord tunnel the speed would reach a maximum of 414 km/hr. This would be at a depth of 675 m for a Betuwe tunnel. The time it takes for the train is 42 minutes. The 42 minutes is a constant. Hence a 100 km tunnel reaches a depth of 203 m and the train has a maximum speed of 230 km/hr in a time of 42 min These speeds would, in reality, be difficult to achieve due to frictional forces. The numbers used assume an earth radius of 6000km and acceleration due to gravity of 9.81 m/s^2 .







So why at this stage in time suggest a concept based on a chord tunnel as an alternative for the Betuwe Railway which is nearing completion? The Chord Tunnel could be regarded as a virtual project under the guise of a study more controversial projects associated with underground storage of water, oil, gas and (especially for us NIMBYs) waste (nuclear and chemical). A possible more realistic candidate for a chord tunnel is a new IJseren Lijn for the goods train line from Antwerp to the Ruhr district. This will be longer (say 200 km) so that it reaches a depth of 833 m and frictionless speed of 460 km/hr . Once the Antwerp metro is complete they can use the boring machines to make the chord tunnel. Most of the deposits in Antwerp are in the Tertiary so the machines should be suitable.

The Chord Tunnel project would, ideally, be more a project in the guise of a study to learn more about the Tertiary as, for the present, all we know is lots of speculation but little in the way of engineering properties and their distribution. Most present geotechnical parameters are from sources in the North Sea where the Tertiary nears the sea bed and often was tested for foundations of offshore structures. But what are the properties of a Tertiary clay at 675 m depth? We make assumptions about consolidation parameters taken from relatively shallow samples to calculate the settlement rates in the Netherlands and often find that correlations are difficult to make with those from level surveys. How would one take a clay sample at 675 m and retain its integrity?

Is there a history besides that first year exercise with respect to chord tunnels besides that first year engineering exercise? These tunnels, for some reason, remain part of science fiction if one visits the few websites devoted to Chord Tunnels (the best response on Google was to search for chord AND tunnel AND sphere!). Back in the 1960s when that problem was set to this then first year student TIME magazine published in their science page, soon afterwards, an article based on research work from an American professor. He had done “innovative research” by calculating what train speeds and times would be achieved along a chord tunnel between New York and Philadelphia. The idea is not so new, yet to date the first chord tunnel has not made its appearance. In mountain areas tunnels do the opposite if their entrances are at the same level to ensure seepage water drains out by gravity. In the Netherlands seepage water has to be pumped from tunnels. If 533 m or 833m is considered too deep one can always say that there are mines that go deeper. Such a tunnel would provide the novelty that one can see the exit of the tunnel from the entrance with a tunnel inclined downwards even though the gradient is just under1°.

Is it essential that we learn about Tertiary soils geotechnical behaviour at depth because of still hypothetical chord tunnel? The settlement issue has not been settled. But there are more issues:
Recently the first international Engineering Geology MSc student, Yufei Dong, received his degree at TU Delft on a study examining mechanisms of shallow induced earthquakes in the north of the Netherlands using 3D finite element modeling (DIANA). The stress and displacement fields at faults looks impressive. Yet the soil behaviour properties are assumed to be simple elastic-plastic deformation straight lines. The questions remain unanswered as to what the soil strain-stress properties are at depth. One can add more examples to the shopping list: aquifer properties as one can speculate about these especially in connection with the physio-chemical properties of soils (and rock) at these depths where temperatures and pressures rise. Could this explain that oil and gas reservoirs in carbonate rock are more susceptible to settlement than aquifers in similar rock when pore pressures are lowered: because under high pressures and temperatures there is less cementation taking place between particles?

Let us return to the Chord Tunnel. We do not only want to settle settlement issues such as differentiating between benchmarks measuring only deep settlement and those benchmarks measuring settlement of both Holocene/Quaternary and Tertiary settlement. To measure settlement in dams often horizontal tubes are installed at different levels and use is made of a level device to measure any deformation. A chord tunnel would be an ideal insitu device to measure settlement at different levels in the Tertiary! Hopefully the concept of a chord tunnel strikes the right chord to research the Tertiary geotechnical properties.

BOOK REVIEW

Siefko Slob

Grondsoorten en Delfstoffen bij Naam

Meulen, M.J., Lang, F.D. de, Maljers, D., Dubelaar, C.W. en Westerhoff, W.E. Ministerie van Verkeer en Waterstaat, Directoraat-Generaal Rijkswaterstaat. Dienst Weg-en Waterbouwkunde. Publicatiereeks grondstoffen 2002/21.

De ondiepe ondergrond in Nederland bestaat grotendeels uit zand, klei, veen, leem en grind; alleen in Zuid-Limburg en oostelijk Gelderland worden vaste gesteenten aangetroffen. “Grondsoorten en Delfstoffen bij Naam” bevat de namen en omschrijvingen van al deze grondsoorten en daarvan vervaardigde grondstoffen. Het gaat om termen zoals gebruikt door geologen, bodemkundigen, mijnbouwkundigen, civiel ingenieurs, aannemers , landbouwkundigen, producenten en leveranciers van bouwstoffen en - niet te vergeten - in de volksmond.

Voor het nabestellen van het woordenboek kunt u terecht bij TNO-NITG (030-2564850) en bij de Dienst Weg- en Waterbouwkunde (015-2518308). Contactpersonen: Michiel van der Meulen (TNO-NITG) en Hester Rijnsburger (DWW).

Handboek Grondonderzoek Grote Projecten - Geologisch onderzoek voorafgaand aan grootschalig grond- of baggerwerk, gericht op de bodemgesteldheid en eventuele opbrengsten aan zand, grind en klei.

Heijst, M.W.I.M. van, Gruijters, S.H.L.L., Gunnink, J., Kleine, M. de en Lantman, R. Ministerie van Verkeer en Waterstaat, Directoraat-Generaal Rijkswaterstaat. Dienst Weg-en Waterbouwkunde. Publicatiereeks grondstoffen 2002/21.

Dit boek biedt een overzicht van de mogelijkheden, kosten en doorlooptijd van grondonderzoek binnen een project. Daarnaast legt het de ervaring en leermomenten vast uit het grondonderzoek dat is uitgevoerd bij het Project de Maaswerken (hiertoe zijn bijdragen inclusief voorbeelden door betrokkenen aangeleverd).

Het boek is toegespitst op grondonderzoek naar de toepasbaarheid van grondstromen uit een project als bouwgrondstof zoals zan, klei, grind, etc. om na te gaan of grond binnen een project opnieuw kan worden gebruikt of kan worden verkocht. Hierbij kan bijvoorbeeld worden gedacht aan projecten als Ruimte voor de Rivier. Maar ook aan onderzoek door bevoegd gezag bij een vergunningsaanvraag voor ontgrondingen.

Voor het nabestellen van het handboek kunt u terecht bij de Dienst Weg- en Waterbouwkunde (015-2518469). Contactpersoon: Dr. M.W.I.M. van Heijst.

ENGINEERS ABROAD

'Yehaw'! Greetings from Tulsa

Ir. Marco V. Vicente Silvestre, E.I.
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Y'all Heart about Tulsa?
Tulsa once boomed and prospered from the oil industry back in the 1920's. It was the metropolitan of this industry! What is now left of this legacy are gigantic mansions built across town as a reminder. The City of Tulsa (See Figure 1) is situated in the northeast part of the State of Oklahoma. County names such as Ottawa, Cherokee, Pawnee, and Sequoyah are a constant reminder of the Native American culture on which this land was built. There are government Indian reservations, and tax free bingo halls and tobacco shops owned by various Indian Nations. Historical figures such as Geronimo, General Custer, and outlaw Jesse James once roamed the 'Heartland'.

Now, how did a Dutch engineering geologist start a career in this part of the world? Strangely as it my sound, it all began in Cameroon (West Africa). I was sent by Dowell Schlumberger to attend a two-month field engineering training in Kellyville, near the City of Tulsa. I expected dusty roads, tumble weeds, teepees, wild horses, and rowdy saloons. Not exactly what I found! Instead, I did find twisters, 3.2 percent Budweiser beer served in pitchers, skoal dipping rednecks driving in pick-up trucks, and bull riding cowboys at rodeos.

The Start of a Professional Career Path
The short-term training course turned into a five-year career. After overcoming the initial culture shock, I decided to pursue a professional career in the United States

Figure 1. Sunset in downtown Tulsa

using my engineering geological background. Little did I know, it would be a long journey to obtain a work permit and permanent residency. Besides the dealings with the governmental bureaucracy, it is also important to realize that even though the Delft University of Technology is a well recognized university in Europe; such is the opposite in Oklahoma where graduates of Oklahoma State University (OSU) and University of Oklahoma (OU) dominate the market place. For those wanting to pursue an engineering consulting career path in the United States, one must begin by having their foreign academic curriculum evaluated by the Accreditation Board for Engineering and Technology (ABET). ABET accreditation is a requirement by the Board of Registered Professional Engineers to becoming a Professional Engineer (P.E.). And if you think that you are done studying after obtaining an engineering degree, think again! First, one must pass a Fundamentals of Engineering exam after which you are recognized as an Engineering Intern (E.I.). In the State of Oklahoma, an Engineering Intern with a foreign, ABET accredited degree needs to have at least six years of experience under the supervision of a registered professional engineer in order to be eligible to take the actual P.E. exam.

A Glimpse of Engineering Geology in the 'Heartland'
The type of projects an engineering geologist gets in-

Figure 2. One of over 600 sinkholes related to subsurface mining activities.

volved in cover a wide range. Just to name a few: the construction of a new multi-story residence in Owasso for famous country singer Garth Brooks; evaluation of surface collapse potential in the abandoned underground lead-zinc Picher mining field, located in the largest Super Fund site of the United States (See Figure 2); and construction of new processing unit additions to Conoco's oil refinery in Ponca City. Unconsolidated alluvial soils and over-consolidated residual clay soils typically cover sedimentary rock formations. The types of soil range from very dense clayey chert gravel in the northeast portion of the state, to loose sandy soils along the Arkansas River, to very stiff fat clay soils present throughout the City of Tulsa. The rock formations include Pennsylvanian and Mississippian age shale, sandstone, limestone, and chert and Permian age 'red beds'.

Potential problems the local soils pose on design and construction of new structures include differential vertical movement due to consolidation of soft alluvial soils and shrink and/or swell of fat clay soils. The presence of hard bedrock, particularly limestone and chert, are important considerations for projects. Some require excavations for placement of underground utility lines, foundation construction, or site grading. Other projects require structures to be supported on deep foundations extending through soft soils and bearing into the harder bedrock formations.

See Y'all Around Town

Hopefully y'all like to come over and visit us. You will enjoy the kind hospitality of the 'Heartland', the rural touch of this city, the many restaurants, and for those of you playing golf……I do not have to remind you that the last U.S. Open was held at the Southern Hills Country Club, a few blocks from my house.

STUDENTEN

Jordy Mollé

Toekomst Ingenieursgeologie

Het zal de meeste, Nederlandse ingenieursgeologen niet ontgaan zijn dat er gedurende de afgelopen jaren een hoop onzekerheid was en is over de toekomst van Ingenieursgeologie aan de TU Delft. Mede ten gevolge van de TU brede onderzoeksportfolio van 2002 zijn er grote veranderingen opgetreden; docenten zijn vertrokken en de positie en toekomst van de sectie zijn onduidelijk. Vanwege de bezorgdheid die onder studenten over het voortbestaan van de sectie leeft, heeft de DIG een gesprek met de huidige decaan van CiTG geïnitieerd. Sabine Backx, Anneke Hommels en Gerben Groenwegen hebben de decaan op 6 mei jl. om uitleg gevraagd. Het hiernavolgende is een verkorte weergave daarvan.

Op dit moment spelen er twee ontwikkelingen rond ingenieursgeologie. Ten eerste zullen binnen CiTG de verschillende richtingen geclusterd worden. Er moeten ongeveer zeven richtingen komen, waaronder de Shallow Subsurface groep (precieze naam is nog onduidelijk). Deze groep zou zeker een onderdeel ingenieursgeologie moeten bevatten. IG zou de brug tussen diep (geologie, petroleumwinning en geofysica) en de toepassingen (civiele techniek) moeten vormen.

Ten tweede is het zo dat de mastervariant IG binnen Technische Aardwetenschappen veel gekozen wordt: ongeveer 33% van de studenten kiest IG. Mocht de grondstoffentak verhuizen naar OCP (een ander faculteit van de TUD), en IG zou verdwijnen, dan zou TA wel erg klein werden, met alle consequenties van dien. Volgens de decaan zal TA echter zeker voorlopig nog gewoon kunnen voortbestaan. De masterinstroom (ook uit het buitenland) is hierbij wel van groot belang.

Voor een doorstart zijn er echter drie voorwaarden waar aan zal moeten worden voldaan:

* Financiering
* Een duidelijke inhoudelijke focus wat betreft onderzoek
* Een goede hoogleraar

Men is bezig met de invulling van de hoogleraarpost en ook over de inhoudelijke focus wordt gediscussieerd.

Onder de studenten is de belangstelling voor gesteenten (naast slappe ondergrond) groot. Dit zal zeker binnen het curriculum blijven (dit is namelijk ook internationaal gezien van belang). Echter om hier op af te kunnen studeren is er natuurlijk wel onderzoek in deze richting nodig.

Bij het schrijven van dit stuk (inmiddels zomer) is er helaas nog niet meer duidelijkheid gekomen; noch over de toekomst, noch over de toekomstige professor.

Excursies

Naast belangenbehartiging houdt de DIG zich ook bezig met het organiseren van excursies. Daarvan zijn er de afgelopen maanden twee geweest, waarvan een in samenwerking met Ingeokring naar de mergelgrotten in Zuid-Limburg.

In april heeft de DIG zelf een excursie naar Zuid-Limburg georganiseerd. Hoewel het deelnemersaantal door het ontbreken van ITC- en TU Delft-studenten (examens, tentamens, stages en afstuderen)-beperkt was, mag er toch van een zeer geslaagde dag gesproken worden. De excursie begon met een leerzame rondleiding over een zandgroeve bij Belfeld, waarbij uitleg over de winning, sortering, toepassingen, processen, en andere gerelateerde zaken gegeven werd. In de middag is bij Rijckholt een geologische wandeling op en in de kalksteenafzettingen gemaakt onder begeleiding van een kennisrijke en vriendelijke geoloog van de plaatselijke geologische amateurvereniging. Als slot van de excursie zijn de kalksteengrotten bij Sibbe met de mountainbike bedwongen. Hierbij is het uithoudingsvermogen van de deelnemers enigszins op de proef gesteld, én is door menig hoofd de kalksteen getoucheerd. Gelukkig stond het mooie weer en het ruim opgezette programma ook het bezoek aan enkele terrasjes toe.

Veldwerk Spanje

Als onderdeel van het vierde jaar Ingenieursgeologie (en eerste jaar M.Sc.) hebben de studenten van de TU Delft, tezamen met de studenten van het ITC dit jaar veldwerk bij Cambrils (Spanje) gedaan. Net als voorgaande jaren bestond het programma uit de ingenieursgeologische kartering van een gebied, een Slope Stability analysis en enkele excursies.

In deze periode van vier weken in mei werd niet alleen de mogelijkheid geboden om fundamentele kennis te verwerven, maar werd ook ruimte gegeven voor de kennismaking met andere culturen. Dit heeft zonder twijfel de persoonlijke en vakgerichte ontwikkeling van alle studenten in positieve zin bevorderd. Kortom, uiterst leerzaam, zeer gezellig en een ‘must’ voor de jaren die hopelijk nog zullen volgen!

Allerlei

Studiereis uitgesteld

Mede door de vrij zwakke economische situatie in Nederland is het niet mogelijk gebleken om de geplande studiereis naar Canada (najaar 2003) financieel rond te krijgen. Komend collegejaar zal er opnieuw getracht worden een studiereis te organiseren, naar een iets dichterbij gelegen oord.

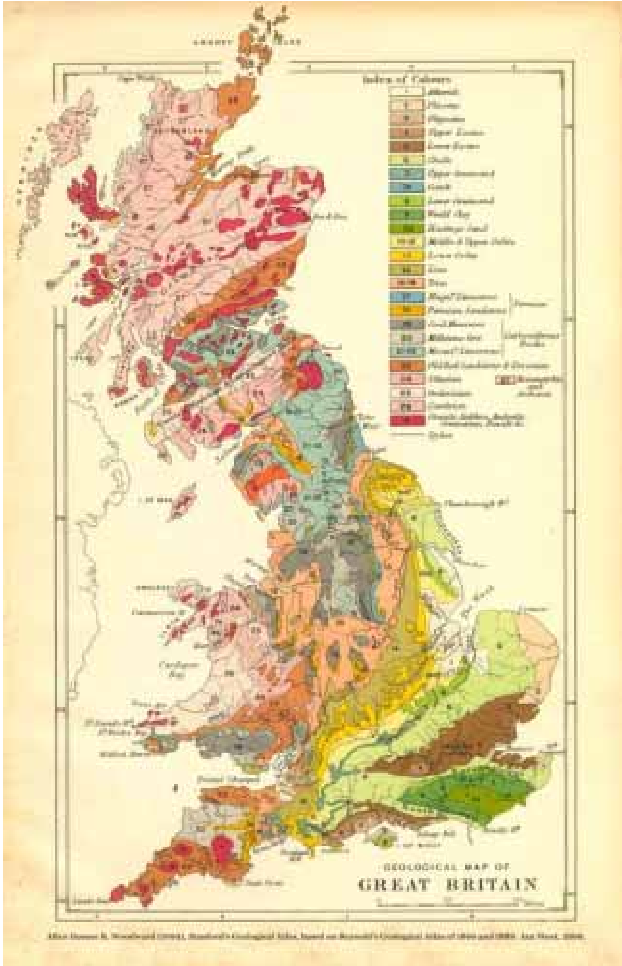
Bedrijfsbezoek

Aan het begin van het nieuwe collegejaar is het voornemen een bezoek aan Boskalis te brengen.

Uiteraard nodigen wij u ook nogmaals uit eens een bezoek aan onze website te brengen: www.dig.tudelft.nl

SURF SUGGESTIONS

Bas Vos
Robert Vuurens



http://www.soton.ac.uk/~imw/geobrit.htm

This website of the School of Ocean and Earth Sciences from Southampton University (by dr. Ian West) gives a nice introduction to the Geology of Britain, containing many maps and photographs. The structure of the website is not very clear, but if you click on some of the links, you'll find out that a large number of field guides from field work in Britain, Middel East and the Eastern Pyrenees are presented. Furthermore, high quality pictures and explanatory texts on various geological subjects make this website fun to browse!

http://www.infradocent.nl/home/default.asp

This (Dutch) website contains information on many different subjects, such as types of excavation machinery, construction of building pits, soil compaction etc. It also contains a well-organised overview of links to other informative pages.
You can even download some quizzes to widen your knowledge of 'incrowd' road builders' slang with words such as 'kielspit'.

http://bodem.pagina.nl/ en http://ondergrond.pagina.nl/

These are the (Dutch) starting pages for a wealth of different links related to (hydro)geology, geotechnics, GIS etc. You can find links to universities, societies, newsgroups, companies, museums and even the columns of Applied Earth Sciences' Geology Professor Salle Kroonenberg (for the magazine 'Intermediair') can be found here.
The page http://bodem.pagina.nl/ contains the most links by far, so it would be best to start there. The Ingeokring is also listed (under the dubious term "Bodemgemeenschappen").

http://www.sbwinfra.nl/

SBW is a (Dutch) institution that provides courses for many different subjects in civil engineering works, ranging from shovel operation to courses on RAW-contracts. It is possible for groups (i.e. company excursions) to visit the SBW information centre, visit the 'Wegenbouwmuseum' and practice shifting sand themselves by operating the shovels or bulldozers on the demonstration terrain, as some Geodelft employees have done during their latest company excursion.

THESIS ABSTRACTS

Abstract thesis Geert de Jong

Delft University of Technology,
Faculty of Applied Earth Sciences,
Section Engineering Geology

Data analysis on data of 12 years fieldwork of the formations Middle Muschelkalk (Tg21), Upper Muschelkalk (Tg23) and Keuper (Tg3) in the research area in Tarragona, Spain (Catalunya), is used to find relations between time, weathering and slope stability and the influence of slope parameters on these relations. The three formations were divided into seven engineering geological units.

A literature research appointed that topographical, geomorphologic and rock mass parameters and biological activity influence the weathering processes.

An inverse exponential relation between time and degree of weathering was derived from the interpretation of scatter plots of the degree of weathering versus exposure time; this relation was used in a Latin Hypercube simulation in Mathcad (Bootstrap percentiles). The results of the simulations for complete and lower boundary data sets can be used as indicative predictions for similar rock masses.

Besides time, the influence of the aspect, apparent dip, slope dip, bedding spacing and slope height was taken into a two variable regression analysis, in order to refine the exponential relation; the results were poor except for $Tg22_{gypsum}$, this was partly due to the large spread of the data.

An empirical relation between weathering rate, apparent dip and aspect gave better results; the weakest units (Tg22 units and $Tg3_{limestone}$) have the highest weathering rates. Slopes with small (negative) apparent dips and eastern to southern aspects tend to weather fastest.

The modelling (UDEC) and simulation (Mathcad) of a failed slope indicated the relation between the extent and intensity of weathering along discontinuities with the moment of failure and the slope stability probability that corresponds with the moment of failure.

Abstract thesis Jeroen Dijkstra

Delft University of Technology,
Faculty of Applied Earth Sciences,
Section Engineering Geology

The effect of freezing and thawing on the consolidation properties of reconstituted Eem clay.

With the construction of the North/south metro line underneath the center of Amsterdam the properties of the Eem clay have become of interest. The site investigation revealed that the Eem clay is overconsolidated to such an extent that ageing is not the sole cause for this overconsolidation. Besides the slight overconsolidation there is also a large deviation in the data set, causing ambiguous interpretations of the data cluster and of the preconsolidation pressure profiles with depth. There are several processes that could cause the overconsolidation ratios as seen in the OCR profile with depth.
Little is known about one of these processes: what is the influence of permafrost on the OCR profile with depth?

To generate more insight into the overconsolidation properties of the Eem clay a research project was proposed to investigate to what degree historic freezing and thawing affected the preconsolidation pressures of the Eem clay. To answer this question an extensive literature study on the theory of freezing and thawing and on the geology was conducted. From literature it was found that the Eem clay could have been frozen in the Weichselian and other processes like erosion had no effect on the present preconsolidation pressures. The effect of the lowering of the ground water table is not known.

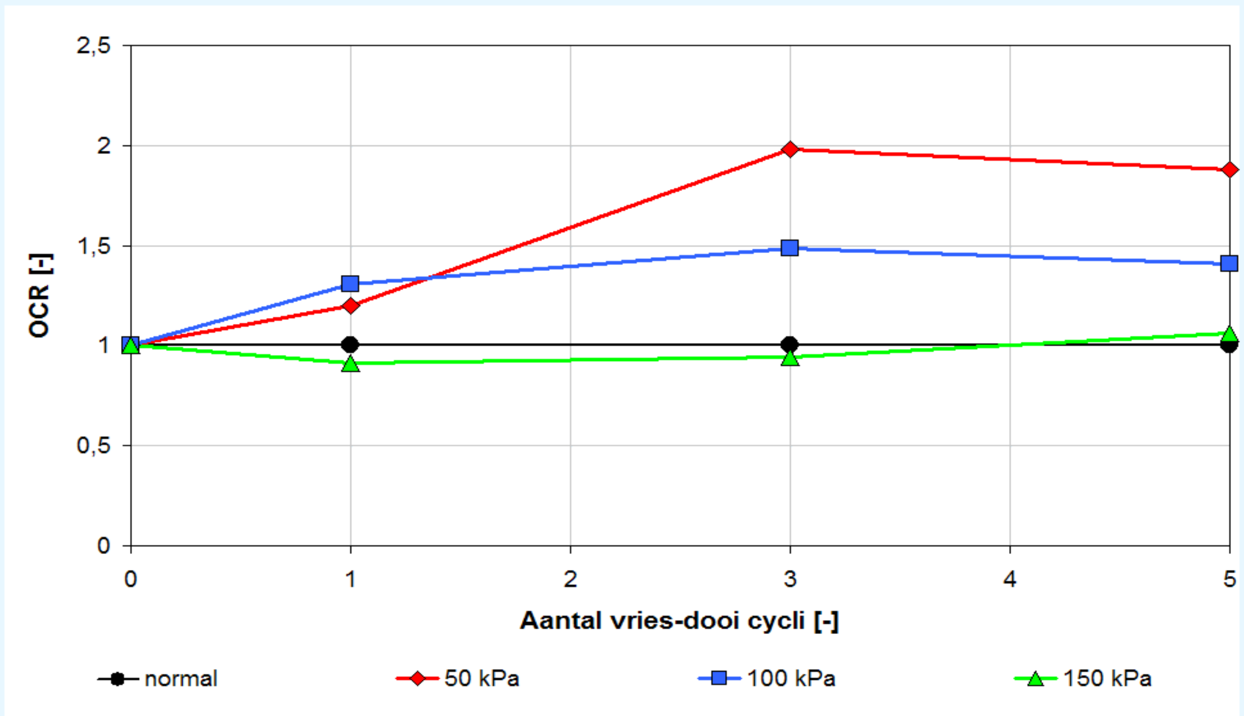
A special laboratory test program was set up to look at the effects of freezing and thawing on the measured apparent preconsolidation pressures of reconstituted samples of the Eem clay. The testing program was designed to assess freeze-thaw behavior at effective confining pressures that are similar to the in-situ stresses at the time of freezing (i.e. 50, 100 and 150 kPa or about 5, 10 and 15m below ground), under alternating cycles of freezing and thawing (i.e. 1,3 and 5 cycles).
The natural Eem clay sample was dried, crushed and reconstituted at a water content of 1.75 times the liquid limit. The reconstituted slurry was consolidated inside the Rowe cell to the desired confining pressures and samples, cut from slices of the Rowe cell clay sample,

were subjected to one-dimensional freezing inside the freezer under the same confining pressure as in the Rowe cell. Meanwhile a reference sample was aged under the same confining pressure. The freezing cell was specially developed for this research and subjected the samples to a closed-system freezing environment with limited access to water. Upon completion of the freezing and thawing program the samples were tested to determine their consolidation properties inside an oedometer.

The samples that were frozen showed different consolidation behavior from the reference samples. As expected, freezing and thawing resulted in consolidation of the sample (drop in void ratio). In addition to the drop in the void ratio, the consolidation coefficient (i.e. the slope of the compression curve in a e-10log P plot) reduced by an average of 0.075 after several freeze-thaw cycles independent of the confining pressure. The change of the Cc value is possibly due to the reorientation of the clay particles or packets after the ice has broken the bonds between the particles/packets. The change in the void ratio is explained by the expulsion of water upon thawing. In-between 20 and 25% of the moisture drains from the sample after 3 cycles of freezing and thawing.
The overconsolidation profile with depth of samples that have been frozen is similar to that of a sample that has been dried although less pronounced. It decreases with depth. The apparent preconsolidation pressure tends to increase after freezing between 0 kPa and 100 kPa, independent of the confining pressure. This is less than would be expected based on the observed change in void ratio, but the decrease in the slope of the virgin compression curve after freezing tends to reduce the apparent preconsolidation effect.

The method of determining the apparent preconsolidation pressure was investigated. Six methods were compared and it was found that the Butterfield and Becker method were more consistent and accurate than the Casagrande method. The Koppejan method tends to underestimate the preconsolidation pressures. Two methods, the Jacobsen method and the Modulus method were found to be poor indicators of the preconsolidation pressure. It is advised to encourage the use of the Butterfield method as well as the Casagrande method in practice.

The hypothesis that the permafrost from the Weichselian was of some influence to overconsolidation ratio measured is not supported by the results of this study. The research indicates that historic freezing and thawing is likely not the cause of the present pre-consolidation pressures of the Eem clay. The effect on the preconsolidation pressure is small in comparison with the present day vertical effective pressure. Probably drying in the Weichselian together with ageing are the causes of the OCR profile obtained by the site investigation.



Abstract thesis Carolina Sigarán Loría

ITC

Numerical Assessment of the Influence of Earthquakes on Irregular Morphologies – ***Analysis of Colombia, 1999 and El Salvador, 2001 Earthquakes***

A two dimensional numerical modelling is executed with the explicit FEM FLAC to assess the ground response of different geometries subjected to major earthquakes, based on data from the earthquake of Colombia in 1999 and from El Salvador in 2001. Digital elevation models are prepared to select and plot the topographical contours.

Each geometry is defined with element sizes of 4 m for the Colombian models and 2 m for El Salvador, based on an empirical relationship between wavelength and shear wave velocities of the local geotechnical units. The selection of the zone size limits the frequency content that can be introduced in the models. The signals are filtered to assure a proper wave propagation within the respective ranges (<6.5 Hz for Colombia and <6 Hz for El Salvador). The element size and number restrict the length of the cross sections according to the memory capacity assigned in the software. More elements give more accuracy, but calculations are more time consuming. The constitutive model used is the linear elastic perfectly plastic Mohr-Coulomb, that permits the development of failure. The base of the models is fixed and free-field boundaries are used on the vertical lateral boundaries. In general a damping of 5% is specified for the center frequencies.

According to the geological records, in the area of Colombia there are four main units of ashes and lapilli, residual soils, saprolite, and pyroclastic flows interstratified with lahars (from top to bottom). In Las Colinas area the main units are: pyroclastic deposits (fall and surge), brown ashes, paleosoils, and pyroclastic flows and tuffs with subordinated basaltic-andesitic lavas. The geological units are simplified and the models inserted in the FEM are a two- and one-layer model for the Colombian case, and a three layers model for El Salvador. The geological units are geotechnically comparable for both countries.

An artificial sinusoidal function with different frequencies is used as well as the frequency-independent acceleration time-history records of the earthquakes. Amplifications are higher along the top of the hills showing irregular patterns (amplification/deamplifications) within narrow ranges of amplification factors. Along the slopes, the amplifications generally decrease towards the base with a close-to exponential shape, with rates of change apparently higher than the patterns observed in depth for the steeper geometries (>50 degrees). In depth, the reduction on the amplifications present a linear trend in almost all the cases. The change of frequency affects the magnitude of amplification. Stiffer geological units cause a slight decrease on the response, but the trends have the same patterns. Higher amplification does not cause instability in a slope, the instability is more function of the geometry.

Abstract thesis Dionísio Pedro de Amurane

ITC

Decision support system to assess settlement in a peaty Holocene deltaic environment

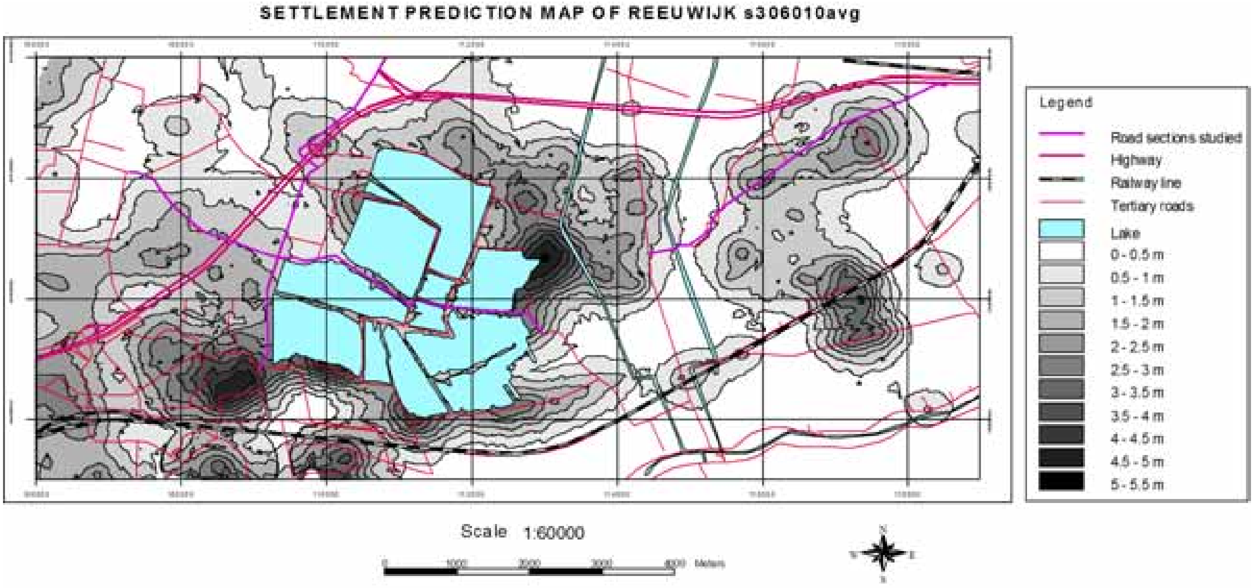
The implications of settlement problems are several and they may involve many variables, which make difficult its assessment.
The present study addresses the problem through the perspective of management of tertiary roads. The approach used attempts to provide the support that a decision maker may need when dealing with high costs of road maintenance triggered by a systematic deterioration process. Through the method developed in this study, support to the decision makers is provided after processing of borehole data in a number of steps incorporated in a Decision Support System framework. The framework consists of four main steps:

· Problem definition
· Evaluation criteria
· Alternative generation
· Decision matrix

The study was carried out in Reeuwijk, The Netherlands, where the settlement problems along tertiary roads are believed to be related to groundwater withdrawal. Therefore, the fluctuation of groundwater level is used as a parameter to carry out a sensitivity analysis while calculating settlement from the borehole database, in the step of alternative generation. The other parameters are applied load and coefficient of compressibility. The alternatives are settlement prediction maps created by interpolation of settlement point data. The alternatives are then ranked and combined with criteria attribute maps (elaborated in the Evaluation criteria step) that contain the attributes related to specific objectives of the decision makers.

Most of the steps of the framework require the use of

GIS to allow interpolation settlement point data, overlay analysis and presentation of the results in maps and graphs.



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