Special edition
30 years of Newsletters

Colophon

Ingeokring, founded in 1974, is the Dutch association of engineering geologists. It is the largest section of KNGMG (Royal Geological and Mining Society of the Netherlands). Ingeokring also forms the Netherlands National Group of the International Association for Engineering Geology and the environment (IAEG).

With over 200 members working in different organizations, ranging from universities and research institutes to contractors, from consultancy firms to various governmental organizations, Ingeokring plays a vital role in the communication between engineering geologists in the Netherlands.

The objective of the Newsletter is to inform the members of the Ingeokring, and other interested parties, on topics related to engineering geology, varying from detailed articles, book reviews and student affairs to announcements of the Ingeokring and current developments in the field of engineering geology. The Newsletter wants to make engineering geology better known by improving the understanding of the different aspects of engineering geology.

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Subscription to the Newsletter

Each member of the Ingeokring receives at least once a year a new edition of the Newsletter. Membership fee for the Ingeokring is € 18,-, student membership fee is € 9,-.

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Foreword

Dr. Peter N.W. Verhoef, former Ingeokring Nieuwsbrief editor (1983-1993)

With great pleasure I introduce to you this colourfully covered Special Edition celebrating 30 years of Newsletters. A good idea from the editorial team of the Newsletter. Michiel Maurenbrecher, probably the only person who still possesses all editions of the Ingeokring Nieuwsbrief/Newsletter, immediately started a new project by scanning pages of the old issues. Michiel, Gerhard Wibbens, Jordy Mollé and Erik Schoute went through all the Newsletters and made the selection presented here. The nice cover designed by Erik brings back memories, as I have been the editor of most of the red covered Nieuwsbrief editions.

Jan Nieuwenhuis, the first chairman of the Ingeokring, has laid out the aims of the Nieuwsbrief in his foreword to the first issue in 1977. When I read it I thought that the function of the present Newsletter still answers to the ideas put forward by Jan. I am not surprised to learn that the birth of the Newsletter was contrived by three gentlemen including David Price. Professor Price was appointed "Lector" for Engineering Geology at Delft in 1975. He spent quite some time in the first years to prepare a document that contained a master plan to introduce the Engineering Geology profession in The Netherlands. This included an active professional body, which would need a newsletter. David was also the first "redacteur". When I came to Delft in 1982 as the first scientific collaborator of David, he immediately appointed me editor of the Nieuwsbrief. We were always keen to seduce people to write for the Newsletter, but this turned out to be a difficult task. Writing papers is hard work for most people and scientists tend to prefer to send their production to scientific journals. On the other hand it is very nice to explain to your friends the details of an interesting case or research study, without the burden of the constraints put to scientific papers (such as originality, scientific soundness and scrutiny, enough data), which make such papers often boring to read. In fact another type of paper made its way to the Newsletter. No doubt the tone was set by Michiel Maurenbrecher who is without doubt the most regular contributor to the Newsletter. He likes to enliven his papers with personal opinions and observations, as we can read again in his paper re-published in this Newsletter. He has also written congress reviews and interesting excursion reports. Many people have taken his contributions as an example to write their own contribution I think. So many papers, interviews, congress and excursion reports which have been published in our Newsletters over the years are relaxed and pleasant to read. The idea of Niek Rengers in 1994 to give the Newsletter a good "make-over" was certainly successful.

Today we are pleasantly surprised when we receive a copy of the Ingeokring Newsletter on our doormat. The surprise is for most of us probably twofold: first because we had nearly forgotten that there is a Newsletter (since the target of a publication twice a year is often not met) and secondly that it is thick, looks very nice and contains a lot of papers. For me, I am happy when I receive my copy. Having been a lecturer at Delft for more than 20 years, I know many of the writers and I am eager to know what everybody is doing nowadays and of course I am interested to see what we Engineering Geologists are working on. I hope you enjoy this special edition with a selection of papers that illustrate how useful Engineering Geology is for our country and industry, I think you will notice with me that the subjects that are dealt with are still of relevance today. I end by encouraging you to contribute (again) to our Newsletter by sending material to our enthusiastic Newsletter editors.

First issue: May 1977

NIEUWSBRIEF

van de Ingeokring

Ingenieurs- Geologische Kring
Netherlands Section of Engineering Geology
Secretaris: Ir J.R.Willet, Postbus 264, Arnhem (The Netherlands)
Postgiro 3342108 t.n.v. Penningmeester Ingeokring, Assen

Inhoud Nieuwsbrief no. 1

Inleiding

Voor u ligt de eerste, enigszins haastigvervaardigde, Nieuwsbrief van onze Ingenieurs Geologische Kring.

Toen een half jaar geleden Hageman, Price en ondergetekende bijeenkwamen om over een mededelingsblaadje te praten, zweefde ons een concept-inhoud voor ogen die, zij het nog in onvolkomen vorm, ongeveer overeenkomt met de inhoud van deze Nieuwsbrief.

Het leek ons niet zinvol het blad als convocatieblad te gebruiken, omdat daartoe de verschijningsfrequentie, twee hooguit drie maal per jaar, te gering zou zijn. Het leek ons evenmin gewenst het blad een wetenschappelijk karakter te geven door het opnemen van uitgebreide boekbesprekingen of literatuuroverzichten.

First issue: May 1977

Wat zou er wel in moeten? In de eerste plaats informatie voor de leden over nationale en internationale conferenties op het gebied van de ingenieurs geologie. Verder, informatie over activiteiten van de Nederlandse universiteiten en hogescholen op ingenieurs-geologisch gebied (cursussen, lezingen van buitenlandse bezoekers, e.d.). Voorts voor alle leden belangwekkende inlichtingen overhet werk van de IAEG (International Association for Engineering Geology). Omdat een aantal van onze leden persoonlijk lid van de IAEG is en derhalve op de hoogte is van IAEG-activiteiten, werd vooral gedacht aan het werk van IAEG-werkgroepen en de Nederlandse bijdragen hieraan. Een uitgebreide rubriek "personalia" leek ons uiterst zinvol, omdat de Kring zich beweegt op een vakgebied, waarvan de beoefenaars elkaar niet of nauwelijks kennen.

Tenslotte zouden korte persoonlijke bijdragen van de leden welkom zijn, waarin iets wordt verteld over toepassingen van de ingenieurs geologie in de praktijk. Het staat de auteurs vrij de grenzen van het vak in deze bijdragen naar eigen voorkeur te definiëren.

De weerslag van de wensen en gedachten van de initiatiefnemers vindt u in deze Nieuwsbrief terug. Ik zei het u reeds, nog in onvolkomen vorm. De rubriek "Universiteiten en Hogescholen" is onvolledig. Wij dachten ook aan afstudeerders en promoties. De rubriek "personalia" is beperkt tot nieuwe leden, bedankjes en adreswijzigingen.
Wij hadden verder gedacht aan overplaatsingen, functiewijzigingen, onderscheidingen of andere voor onze leden belangwekkende persoonlijke gegevens. Het verstrekken van deze gegevens is uiteraard een privézaak waarin de een terughoudender is dan de ander, maar ik zou u toch willen opwekken niet al te terughoudend te zijn. Onze Kring stelt zich immers onder meer ten doel de beoefenaars van de ingenieurs geologie te "verenigen". En bij "verenigen" behoort mijns inziens kennis van elkaars rol als vakbroeders.

De rubriek toepassingen van de "ingenieurs geologie", zo u wilt case studies, kan evenals de voorgaande rubriek, slechts bestaan bij de gratie van uw medewerking. Het is niet onze bedoeling tientallen gevallen per nummer te publiceren, maar 2 à 3 per nummer, dat wil zeggen 5 à 10 per jaar, moet met de medewerking van 100 leden toch mogelijk zijn.

Deze eerste Nieuwsbrief is gedeeltelijk in het Nederlands en gedeeltelijk in het Engels geschreven. De heer Price, die de Engelse stukjes heeft geschreven, heeft nog onvoldoende vertrouwen in zijn beheersing van het Nederlands en de overige auteurs achten een Nederlandse vertaling niet noodzakelijk. Mocht u dit mengelmoes van Nederlands en Engels ergerlijk vinden, laat u ons dat dan weten. De volgende Nieuwsbrief kan aan de wensen van de meerderheid van de leden worden aangepast.

De Nieuwsbrief heeft één redacteur, de heer D.G. Price Msc., lector in de ingenieurs geologie aan de T.H. Delft. Zijn adres is: Technische Hogeschool Delft, Afdeling der Mijnbouwkunde, Mijnbouwstraat 20, Delft 8.

De heer Price verzorgt de rubrieken "Komende conferenties", "Literatuuroverzicht", "Universiteiten en Hogescholen" en "Toepassingen van de ingenieurs geologie". Mocht u mededelingen voor een van deze rubrieken hebben, of mocht u nieuwe rubrieken willen toevoegen, stuur dan uw tekst of suggesties (in Nederlands of Engels) naar bovenstaand adres.

De contactpersoon voor de rubriek "Personalia" is ir. J.R. Willet, secretaris van de Kring. Zijn adres is opgenomen in de rubriek "Personalia" van dit nummer van de Nieuwsbrief.

De contactpersoon voor de rubriek "IAEG" is ing. H, Wiegers, penningmeester van de Kring, Heemskerkstraat 36, Assen.

Mocht u het allemaal te ingewikkeld vinden dan staat het u vrij al uw mededelingen aan ir. Willet te sturen.

Met de overige initiatiefnemers hoop ik dat u het verschijnen van een Nieuwsbrief een goede zaak voor de Kring vindt.

J.D. Nieuwenhuis, voorzitter

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I.A.E.G. International Symposium on Landslides and other Mass Movements, Prague, 15th - 16th September 1977

The Symposium was generally a great success in terms of organisation and the general quality of the papers presented. The greater number of these came from Czechoslovakia and the U.S.S.R. Field trip excursions were well organised and illuminating. However, it would be hard to point out any significant new idea or process presented in any of the papers submitted.

For the writer the high point of the Symposium was the visit to the laboratory of the Mining Institute in Prague where the work on subsidence, mine- and tunnel stability with the aid of models was most impressive.

The I.A.E.G. Council met before the Symposium and meetings of the Commissions on Slope Movements, Engineering Geological Mapping and Site Investigations also took place outside of Symposium sessions.

The Commission on Site Investigations, of which David Price is Chairman and Niek Rengers is Secretary, held its first meeting. In this, and the Council meeting, the general purpose of the Commission (outlined in the last Nieuwsbrief) was confirmed. The contents of the proposed report were agreed and will now be amplified by the Chairman and Secretary and distributed for comment to Commission Members.
Those interested in obtaining copies of this document should contact Niek Rengers in I.T.C.

CASE STORY

New commercial Harbour of Bandar Abbas (Iran)

door

ir. J. Kruizinga en J. Thomas M.Sc. D.I.C. F.G.S., ingenieurs bij het Laboratorium voor Grondmechanica te Delft

1. Inleiding

Nedeco, het Nederlandse ingenieursbureau voor werken in het buitenland, heeft kort geleden van de "Port and Shipping Organisation" (P.S.O.) van Iran opdracht gekregen voor "designreview" en "supervision" met betrekking tot de bouw van een nieuwe handelshaven bij Bandar Abbas in het zuiden van de Perzische Golf.

Het ontwerp en de uitvoering van het project is in handen van een grote Italiaanse bouwcombinatie.
Voor Nedeco is het ingenieurs- en architectenbureau van Hasselt en de Koning te Nijmegen in grote mate bij het project ingeschakeld.

Het Laboratorium voor Grondmechanica te Delft is op verzoek van Nedeco nauw bij de grondmechanische aspecten van de diverse onderdelen van het project betrokken.

2. Situering van het project

De nieuwe handelshaven wordt bij Bustaneh ca. 20 km westelijk van de bestaande haven van Bandar Abbas gebouwd (fig. 1).
De haventerreinen en -bassins zijn gedeeltelijk op het vaste land en gedeeltelijk op een thans bij eb droogvallend gebied geprojecteerd (gemiddeld tijverschil ca. 2,5 m).

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De bassins worden tegen de sterke stroming en golfaanval beschermd door in zee uitgebouwde havendammen. Een schets van de layout is gegeven in fig. 2.

3. Enige kenmerken van het gebied

3.1 Tektonische aardbevingen

De haven van Bandar Abbas bevindt zich in een actief seismisch gebied, zoals door een recente, ernstige aardbeving is aangetoond (6.8 op de schaal van Richter). In dit gebied grenzen drie grote aardschollen aan elkaar:

- de Iraanse en de Arabische schol die tegen elkaar drukken, als gevolg van de noord-noordoost beweging van de laatste
- de Indische Oceaan schol die onder de Iraanse schol schuift; op deze laatste is de stad Bandar Abbas gelegen.

3.2 Locale geologie

Het betrokken gebied bestaat uit een kustvlakte van bescheiden omvang, die omgeven is door een achterland van paleozoïsche en tertiaire origine. Het wordt verder gekenmerkt door een anticline waarvan de as min of meer evenwijdig aan de kust loopt. De kustvlakte zelf bestaat uit jonge marine sedimenten waarin voornamelijk siltige zanden en kleien (met een hoog zoutgehalte en mineralen als dolomiet en calciet) voorkomen.

De kenmerkende siltige zand- en kleilagen duiden naar alle waarschijnlijkheid op afzetting tijdens het pleistocene en post-pleistocene tijdvak (12.000 - 6.000 jaar jeleden). Hierin deden zich zekere schommelingen in klimaat voor, resulterend in veranderingen in de omvang van de polaire ijskappen, waardoor het zeeniveau variëerde.
In het gehele gebied wordt een tamelijk samendrukbaar kleimineraal, attapulgiet, aangetroffen dat door enige deskundigen in verband wordt gebracht met het bestaan van een veel natter klimaat tijdens het post-pleistoceen.

De reeds eerder genoemde zeeniveaufluctuaties en verdamping kunnen het zoutgehalte plaatselijk beïnvloed hebben.

4. Terrein- en laboratoriumonderzoek

Een uitgebreid terrein- en laboratoriumonderzoek heeft in de loop der tijd plaatsgevonden (door verscheidene bureaus).
Grote aantallen boringen, standard penetration tests (S.P.T.'s), sonderingen zijn uitgevoerd. Vele geroerde en ongeroerde monsters zijn gestoken voor nader onderzoek in het laboratorium, zoals door middel van:
samendrukkingsproeven, triaxiaalproeven, vrije prismaproeven, Atterbergse grenzen, korrelverdelingen, etc.
In het kort kan het grondprofiel ruwweg tot een drielagensysteem (de lagen variëren in niveau, dikte, samenstelling en vastheid) geschematiseerd worden:

- laag I : een siltige topzandlaag, die zeewaarts gaande verdwijnt
- laag II : een kleiïge siltlaag, die onderscheiden kan worden in een bovendeel IIa en eronder een wat vastere laag IIb
- laag III : een harde diepe zandlaag met harde tussenkleilagen

Elke laag kan gekarakteriseerd worden door een aantal relevante grondparameters (b.v. γ, c, c_u, ϕ, etc.).

5. Onderdelen met belangrijke grondmechanische aspecten

5.1 Havendammen

Het constructieontwerp van de dammen bestaat uit "quarry run" afgedekt via een filterlaag door een "armour rock" laag.

Uit stabiliteitsoverwegingen (vooral tijdens aardbevingen) wordt eerst een cunet

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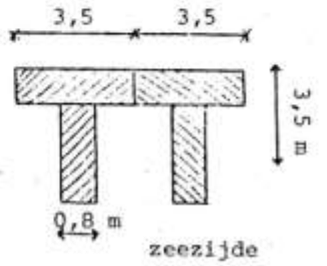
in de klei- en silthoudende lagen gebaggerd en gevuld met een zand-grind mengsel, alvorens de dam wordt opgetrokken.

De schuifweerstand van de overgebleven cohesieve laag onder het cunet is bepalend voor het ontwerp en dient dan ook aan een bepaald criterium te voldoen. Door het uitvoeren van zeer gevoelige conusweerstandmetingen kan in het werk bepaald worden tot welke diepte gebaggerd moet worden om aan het schuifweerstand-criterium te voldoen.

5.2 Kademuren

De begrenzing tussen de havenbassins en de vaste wal wordt voor het grootste deel gevormd door een kademuur die als een diepwand wordt uitgevoerd. De hieronder geschetste T-vormige diepwand secties van de kademuur worden achterwaarts verankerd aan een doorgaande ankerwand (ook als diepgaand uitgevoerd).



De grootste totale hoogte van de hoofdwand bedraagt ca. 33 m met een maximum kerende hoogte van ca. 20 m. Deze laatste komt tot stand door na fabricage van de wand en het aanspannen van de ankers de grond voor de diepwanden weg te baggeren. Bij de sterkte- en stijfheidsbeoordeling van het ontwerp speelt de bepaling van de in te voeren relevante grondparameters een hoofdrol.

5.3 Terreinophogingen

Het haventerrein wordt op ca. 3 m boven het gemiddelde zeeniveau gebracht, hetgeen vlak achter de meest zeewaarts gelegen kademuren een ophoging van max. 5 m impliceert.
Aan de kwaliteit, samenstelling, vastheid van het ophoogmateriaal zijn bepaalde eisen gesteld. Tijdens de uitvoering wordt door middel van zekere grondmechanische controlemetingen nagegaan of aan de gestelde eisen wordt voldaan.

5.4 Loodsen

Op de opgehoogde terreinen achter de kademuren zullen loodsen worden gebouwd die op staal gefundeerd zijn gedacht.
Resultaten van zettings- en consolidatietijd-berekeningen zijn hier onontbeerlijk bij het ontwerp en de bepaling van tijdstip van uitvoering van de loodsen.

6. Bandar Abbasproject in enkele cijfers

- de oppervlakte van het nieuwe havencomplex zal ongeveer evenveel ruimte in beslag nemen als de stad Delft
- de aanneemsom bedraagt ruim ƒ 2.000.000.000,-
- voor supervisie is ca. ƒ 60.000.000,- gereserveerd
- 5,5 km havendam wordt aangelegd
- ca. 300.000 m^3 "armour rock" wordt verwerkt
- ca. 5 km kademuur zal als diepwand worden uitgevoerd
- 9 miljoen m^3 droog grondverzet
- 40 miljoen m^3 baggerwerk

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FORTHCOMING CONFERENCES

Date	Title and Place	Contact
4-8 September 1978	3rd International Congress of the Association of Engineering Geology, Madrid	Dr. A. Garcia Yagüe, Secr., 3rd Int. Congress of the I.A.E.G., Servicio Geológico de Obras Publicas, Avenida de Portugal 81, Madrif 11, Spain
9-12 October 1978	Foundation Aspects of Coastal Structures, International Symposium on Soil Mechanics Research and Foundation Design for the Oosterschelde Storm Surge Barrier , Delft	Foundation Aspects of Coastal Structures, c/o Delft Soil Mechanics Laboratory, P.O. Box 69, Delft

PERSONALIA

David Price is travelling to Cebu in the Philippines on January 6th in connection with the NUFFIC project for co-operation between the TH-Delft and the University of San Carlos, Cebu City on the subject of water resources. Upon his return he has been invited to lecture at the Asian Institute of Technology and to the Australian Geomechanics Society and should be in Delft on February 18th.

A new organisation has been formed, E.A.R.S. B.V., who are engineering consultants in the field of environmental analysis and remote sensing. Those interested in hearing more of their activities should contact Ir. Ko Bijleveld at Kanaalweg 1, Delft (phone 015-562500)

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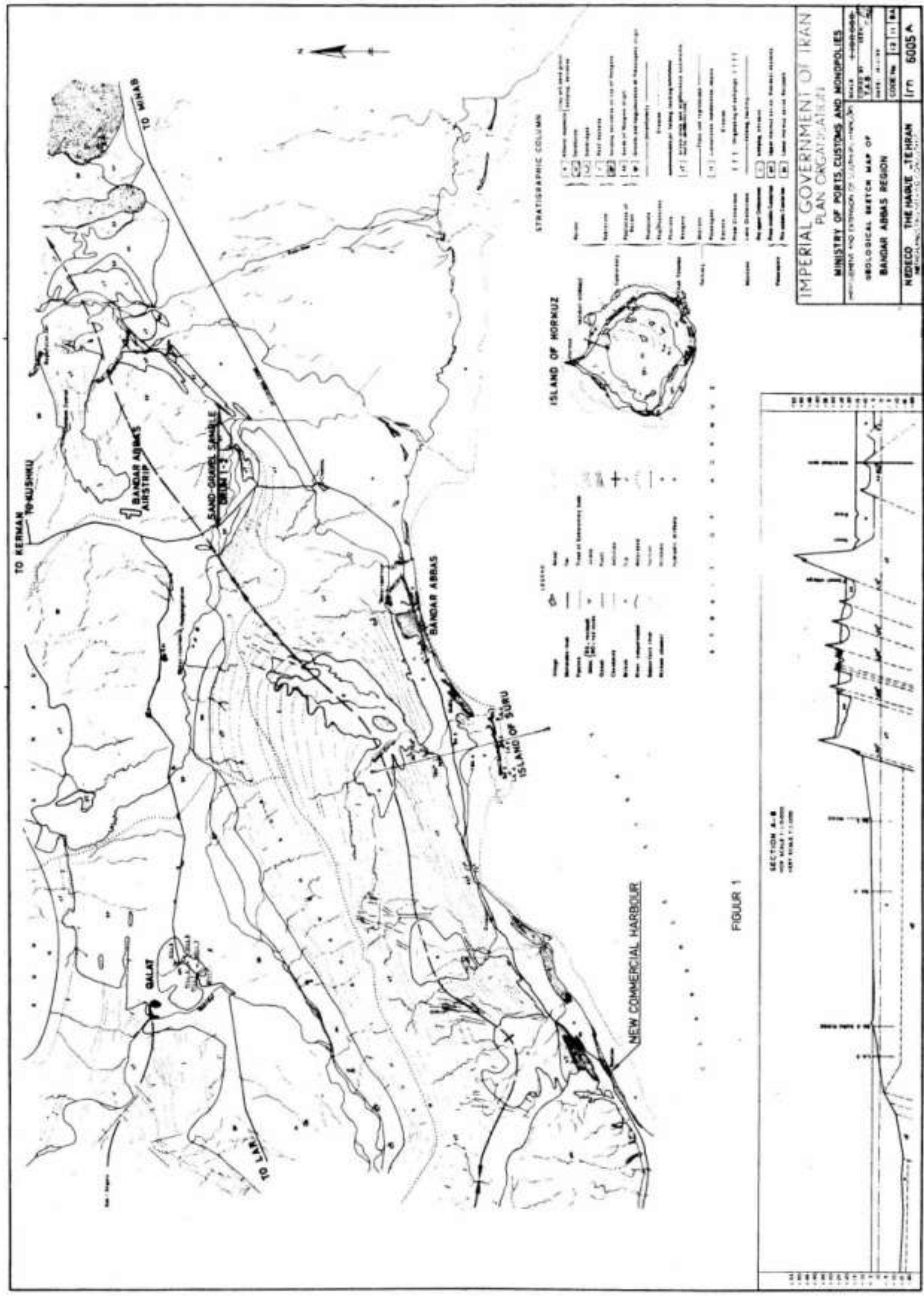
TO MINAB
TO KERMAN
TO KUSHKU
BANDAR ABBAS AIRSTRIP
BANDAR ABBAS
ISLAND OF HORMUZ
ISLAND OF SURU
NEW COMMERCIAL HARBOUR
TO LAR
QALAT
STRATIGRAPHIC COLUMN
LEGEND
SECTION A-B
FIGUUR 1
IMPERIAL GOVERNMENT OF IRAN
PLAN ORGANISATION
MINISTRY OF PORTS, CUSTOMS AND MONOPOLIES
GEOLOGICAL SKETCH MAP OF
BANDAR ABBAS REGION
NEDECO THE HAGUE TEHRAN
6005 A

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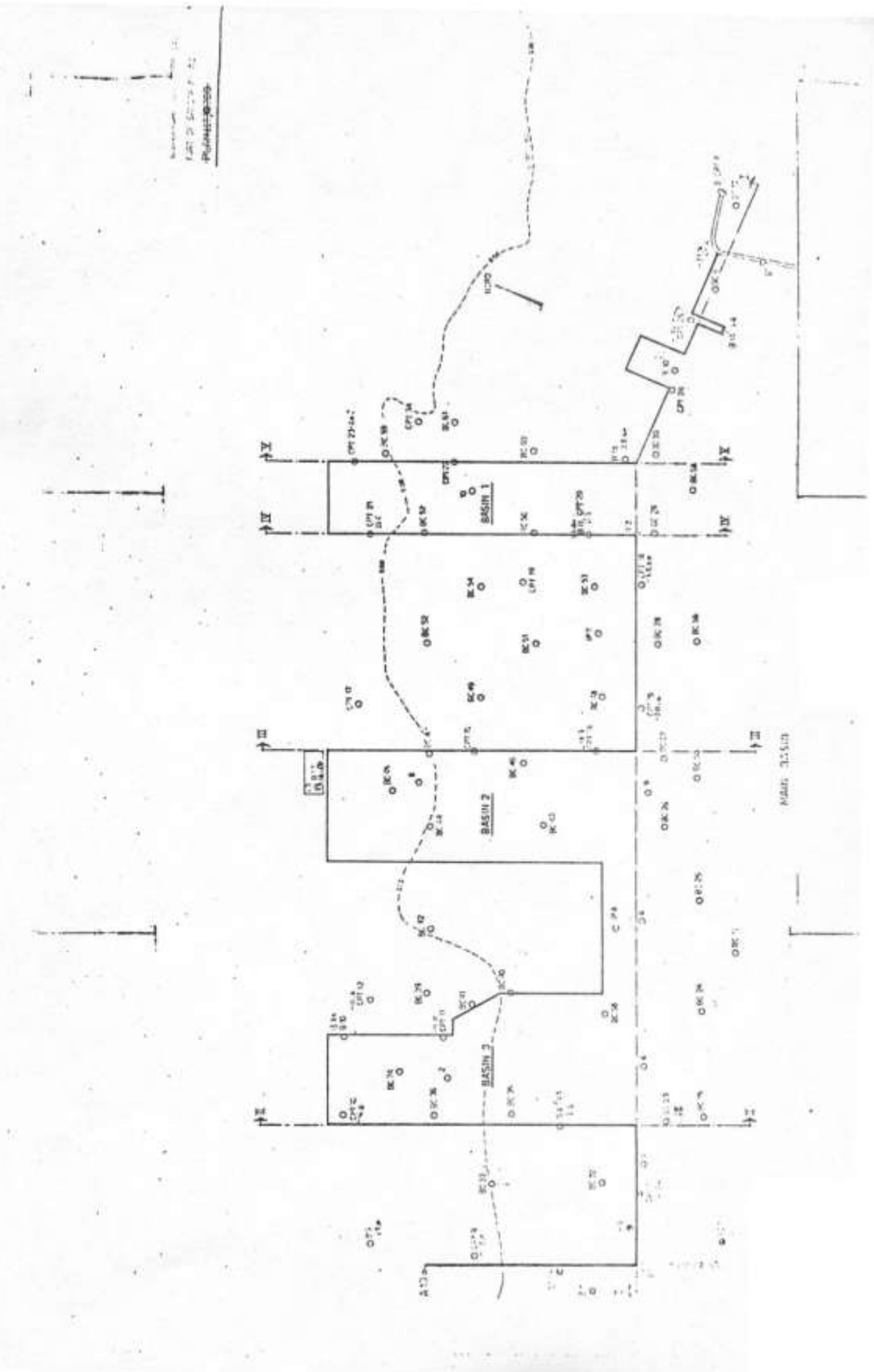

Figuur 2.

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NIEUW ONDERZOEK NAAR DE GEOTECHNISCHE EIGENSCHAPPEN VAN GECEMENTEERDE ZANDEN.

Inleiding

Bij de Sektie Ingenieursgeologie van de Afdeling der Mijnbouwkunde van de Technische Hogeschool Delft is onlangs een onderzoek naar de geotechnische eigenschappen van gecementeerde zanden van start gegaan. Het feit dat los zand zich kan verkitten, zodat uiteindelijk een harde zandsteen ontstaat, is niet onbekend. De processen die hiervoor verantwoordelijk zijn, zijn al sinds jaar en dag bestudeerd door geologen (met name sedimentologen), maar deze kennis is nog maar weinig toegepast bij geotechnische problemen. Het onderzoek van de Sektie probeert alle kennis die voor geotechnische projekten van belang kan zijn te verzamelen. De medewerking van bedrijven die op dit gebied ervaringen hebben opgedaan, is daarbij van essentieel belang.

Wat is cementatie?

Het belangrijkste proces waardoor een zand in een zandsteen verandert, staat bij geologen bekend onder de naam "cementatie". De mineralen die de verbinding tussen de individuele korrels tot stand brengen, zijn meestal opgelost in het grondwater aanwezig. Als om de een of andere reden een overkoncentratie ontstaat, dan zullen de mineralen uit het grondwater neerslaan en aldus de verkitting tot stand brengen. De mineralen kristalliseren eerst uit op de punten waar de korrels elkaar raken en later worden ook de overgebleven poriën opgevuld. De mineralen die zo een zand kunnen verharden, zijn calcium carbonaat (calciet, aragoniet), ijzeroxiden, silikaat, zout en nog enkele minder vaak voorkomende mineralen.

Bij cementatie speelt het klimaat een belangrijke rol. Het neerslaan van de opgeloste mineralen wordt onder andere bevorderd door hoge temperaturen en een sterke verdamping. Dit is dan ook de reden van het veelvuldig voorkomen van recente zandstenen in het Midden Oosten en rond de Middellandse Zee. In de landen met een gematigd klimaat vindt ook cementatie plaats, maar dan op dieptes van 100 m of meer.

Een vooral in het Midden-Oosten bekend verschijnsel is de vorming van sabkhas (figuur 1) en caliches. In beide gevallen veroorzaakt sterke verdamping een capillaire werking in de bodem, zodat relatief koel

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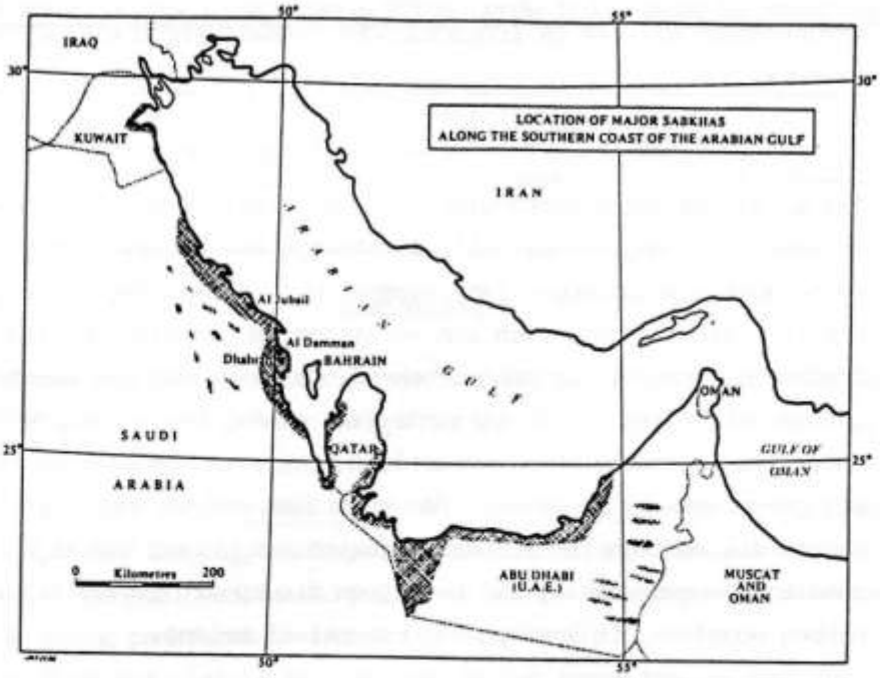
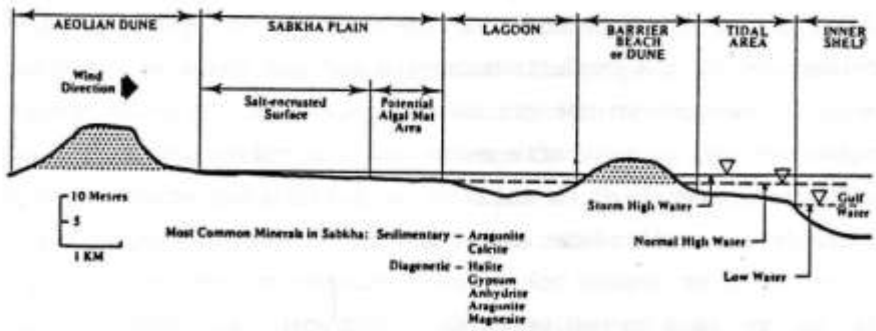


Map showing location of major Sabkhas along the southern coast of the Gulf.



Generalized cross section across a typical coastal Sabkha with typical surface features.

Figuur 1: Sabkhas aan de zuidelijke kust van de Perzische Golf (Akili & Torrance)

grondwater als het ware omhoog gezogen wordt. Als voldoende water verdampt is, zal aan of dicht onder de oppervlakte kalk neerslaan (caliche). Bevat het grondwater weinig kalk, dan zullen na verdere verdamping ook de goed oplosbare zouten neerslaan. Deze zouten kunnen aan de kust ook neerslaan uit verdampend zeewater en vormen dan meestal een harde korst die op de oudere afzettingen ligt (sabkha). De bovengenoemde gevallen zijn slechts twee voorbeelden van de vele milieus waarin cementatie kan optreden.

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De sterkte van gecementeerd zand

Daar het vrijwel onmogelijk is om van losse en zwak gecementeerde zanden ongeroerde monsters te nemen, zal het onderzoek van de Sektie zich koncentreren op in-situ proeven zoals de Standard Penetration Test (SPT), sonderingen, geophysische methoden etc. De overgang van los zand naar harde zandsteen is geleidelijk en in de praktijk kan men met elke overgangsvorm te maken hebben. Het is zelfs niet ongebruikelijk dat men op een site een afwisseling van losse en gecementeerde zanden tegenkomt. Dit heterogene voorkomen maakt dat sommige onderzoekingsmethoden beter geschikt zijn dan andere. Bovendien blijkt dat de resultaten van standaardproeven die ontworpen zijn voor losse zanden niet zonder meer bruikbaar zijn in gecementeerde zanden. Zo is de uit de SPT gevonden waarde (de 'N-value') niet langer geschikt om de relatieve dichtheid van een zand uit te rekenen. De sterkte van de cementatie-bindingen speelt namelijk een overheersende rol.

De uit sonderingen verkregen conusweerstanden zijn nutteloos als de cementatie verloren gaat ten gevolge van te grote deformaties die door belasting in de grond op kunnen treden. Tot nu toe is vastgesteld dat cementatie een bepaalde cohesie aan zanden geeft. De hoek van inwendige wrijving (ø) van een gecementeerd zand is nagenoeg even groot als die van een ongecementeerd zand. Als door deformatie (of door oplossing) de verbindingen tussen de korrels verbroken worden, dan neemt de cohesie af tot bijna nul en wordt de sterkte van het zand geheel bepaald door de inwendige wrijving. In figuur 2 zijn enkele resultaten uitgezet van een uitvoerig grondonderzoek dat door FUGRO B.V. in Saoedi Arabië is verricht. Dit onderzoek is gedaan ten behoeve van het ontwerpen van machine-funderingen. De seismische snelheden zijn bepaald met behulp van de 'crosshole' methode. Deze geophysische techniek is door FUGRO al veelvuldig met succes toegepast. Met behulp van deze resultaten kan op een betrouwbare manier het dynamische gedrag van de ondergrond voorspeld worden.

Baggeren van gecementeerd zand

Ook bij baggerwerkzaamheden moet men bedacht zijn op het feit dat zanden gecementeerd kunnen zijn. Weliswaar moet ons onderzoek op dit terrein nog van start gaan, maar 'common sense' doet inzien dat:
a.: De mate waarin een zand verhard is, bepalend is voor de benodigde

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energie en de optimale methode om het zand te baggeren.

b.: Cementatie de zogenoemde 'stand-up time' verlengt; dit is de tijd waarover een steile helling (tussen een reeds gebaggerd gebied en een nog te baggeren zone) overeind blijft staan.

Samenwerking met het bedrijfsleven

De Sektie is van plan om laboratorium-onderzoek te gaan doen aan kunstmatig gecementeerde zanden. Het overgrote deel van de gegevens zal echter uit praktijkervaringen moeten komen. De samenwerking met FUGRO B.V. is al van start gegaan. Wij hopen dat ons onderzoek naar het baggeren van gecementeerde zanden binnenkort kan beginnen. Al degenen die in het onderzoek geïnteresserd zijn; die suggesties willen doen en die van de vorderingen van het onderzoek op de hoogte gesteld willen worden, worden vriendelijk verzocht om kontakt op te nemen met ondergetekende.

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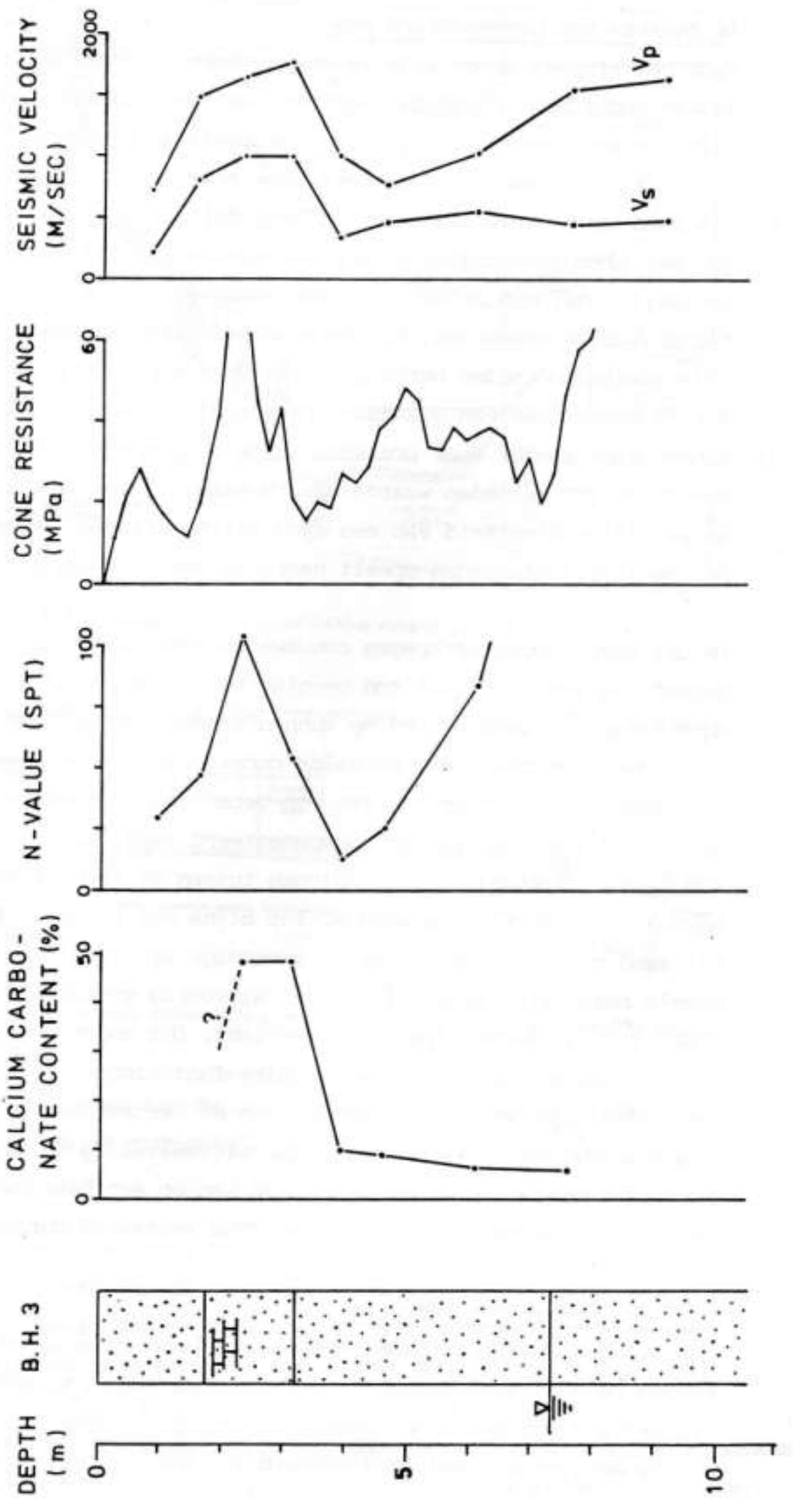

Figuur 2 : Resultaten van een grondonderzoek in Saoedi Arabië. Op 2 à 3 m diepte is een gecementeerde zandlaag aangetroffen.

December 1988

ENGINEERING GEOLOGY IN FINLAND: ROCK ENGINEERING

Engineering geology covers a broad field of activities concerning the interaction between the earth and the construction. The discussion in this note is restricted to a general view about the activities in rock engineering in Finland.

The bedrock of Finland belongs geologically to the Fennoscandian shield (the Precambrian era) and is approximately 1000 to 3000 M.Y. old. After orogeny in the Precambrian era, already in early Cambrian, Finland was eroded to a peneplain. No deposits are present from the period between Precambrian and Quarternary. The bedrock in Finland is composed of igneous rock as granite, peridotite and gabbro, and of metamorphosed igneous and sedimentary rock as quarzite, gneiss, migmatite and schist. The bedrock is covered by a relative thin layer (10 - 30 m) of Quarternary deposits, e.g. sand, gravel, till, which are the result of erosion and deposition during and after the ice ages in the Pleistocene. The peat bogs are formed during the Holocene.

Due to the fact that the total soil thickness all over Finland is thin and because the topography of the bedrock is rather irregular, nearly all the surface constructions are faced with rock engineering. Civil engineering constructions worth to be mentioned are build during the last 20 to 30 years, this in contradiction with the mining industry which started about 300 years ago. Caused by economical growth and the technical development, both in civil engineering as in mining engineering, the amount and the size of the rock excavations at the surface and sub-surface are increased enormously since 1950. The mining industry concerns the production of:

- metal ores as copper, nickel, zinc, vanadium and cobalt ore;
- non-metal ores as limestone and soapstone;
- building material as crushed rock used as aggregate and granite blocks for ornaments.

The ores and building material are mined in open pits, quarries and underground mines e.g. sublevel stopes. In the field of civil engineering, the excavations are made for various use. Surface constructions are e.g. road and railway cuts, foundations, open pit for waste dump or waste water settlement ponds. Some examples are: cut and fill construction for a smooth alignment of the motorway to the east and to the west from Helsinki. With the pre-splitting technique the rock faces are smoothened and stabilized. In the sub-surface, excavations are made for the purpose of transport, storage etc.. Tunnels are build for the transport of water for consumption or cooling and for the sewerage, caverns are build for the storage of oil and bulk goods. Other constructions are: sand silos in rock, civil defense shelters, parking- and sport facilities [3]. These civil rock structures are concentrated in the southern part of Finland (figure 1), especially in and around the towns of Helsinki, Turku and Porvoo. Space in this area is sparce, and because

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the rock is of a good quality in general, it is of economical benifit to build underground space instead of surface constructions.

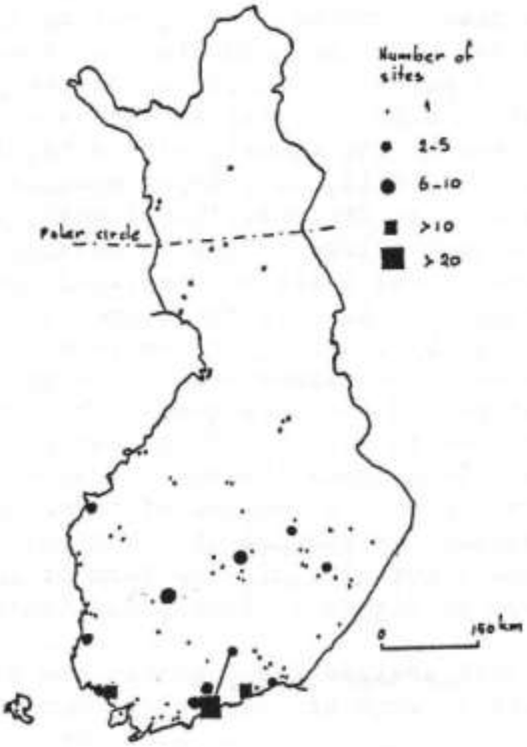

Figure 1. Amount of rockstructures in Finland build in the period from 1960-1986. (including open excavation > 50 000 m3)

The following table [2] gives an impression of the amount of underground volume, approximately 4 million m3 nowadays in the town Helsinki.

TUNNELS	LENGTH	VOLUME	CAVERNS	VOLUME
watertransport	36.8 km	407 000 m3	oil storage	955 000 m3
sewerage	42 km	418 000 m3	water storage	55 000 m3
railway	1 km	50 000 m3	sand silos in rock	50 000 m3
metro	8 km	500 000 m3	shelters	600 000 m3
multipurpose service	6.7 km	110 000 m3	other	105 000 m3
other	8.8 km	148 000 m3		

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The rock in Finland is, comparing with other countries, strong and requires drilling and blasting technique for both surface and sub-surface excavations. This type of excavation technique is expensive, but the rock strength is of such a value that in proper design no expensive lining has to be installed. In some places rockbolting, grouting and shotcreting is needed to create the required stability. In Finland, the total excavated volume for constructions is 3 - 5 million m3 per year. In the mining industry, another 8 million m3 per year is excavated. The raw water transportation tunnel, the Pijnne tunnel, with a total length of 120 km and a cross-sectional area of 15.5 m2, is an example of the largest rock engineering projects in Finland. This tunnel, the longest of the world, transports water from the lake Pijnne to Helsinki and the surrounding municipalities and it was build between 1973 and 1982. Important factors in the design were to keep the tunnel as short as possible and to minimize leakage by avoiding broken rock zones. Another type of constructions of enormous dimensions are the large rock caverns which are needed for oil storage and for bulk goods. The national oil company of Finland, Neste OY, has in Porvoo (50 km east of Helsinki) a total of 4.8 million m3 underground space. Several large caverns are build since the last 20 years. In 1982 a complex of three caverns with a total volume of 800 000 m3 became operational [1]. Not only did the volume increase during the years but also did the form of the cross-sectional area, as can be seen in figure 2. Economical factors play a role in choosing the form.

New developments in rock engineering concerns the storage of municipal and industrial waste in rock or disposal of nuclear waste of low and medium level [4].

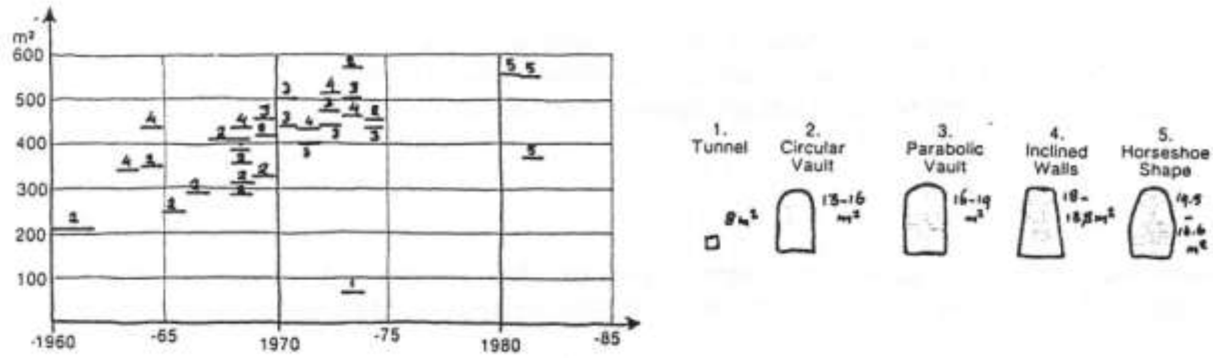

Figure 2 Development of cross-section area of unlined oil storage caverns in Finland between 1960 and 1983.

Together with increasing investments in rock constructions, activities in research and innovation of equipment and methods are raised to afford a better understanding of the behaviour of the rock, safety and a higher productivity. Two research institutes are: the Helsinki

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University of Technology and the Technical Research Centre of Finland (Valtion Teknillinen Tutkimuskeskus), both settled in Espoo. Another institute worthwhile to be mentioned is the Geotechnical Department of the City of Helsinki. Besides these institutes, consulting, construction and mining companies are active in developing new methods, instruments and machinery for site investigation and excavation techniques. The expertise in rock engineering has been applied in Finland as well as throughout the world. In Finland, several associations are representing the companies and institutes e.g.: Finnminers which is active in the field of mining engineering, SKOL, an association of consulting firms, and the Finnish Tunneling Association. The latter association has recently published two books about rock engineering. Both books are recommended for those who like to read more about rock constructions applied in Finland. Both books are listed in the references.

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Summary of results of doctoral-thesis on

GEOPHYSICAL DETECTION OF SOLUTION PHENOMENA

by Drs I.K.Deibel, August '88
Section of Engineering Geology
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INTRODUCTION

A major part of the province of South-Limburg, the Netherlands is underlain by limestones of Upper Cretaceous age.

Where the limestone is situated beneath a thin cover of Tertiary sediments or superficial deposits, the surface of the limestone is often highly irregular. Localized solution weathering of the limestone surface can produce vertical cylindrical pipes or funnel shaped sink-holes. Usually the pipes and sink-holes are filled with gravel, sand or clay derived from the overlying deposits, but the fill may be loose and contain weakly bridged cavities.

For these reasons, pipes and sink-holes are potentially a hazard to surface structures and where suspected they should be detected and fully investigated by appropriate site investigation techniques.

SUBSIDENCE MECHANISM

Solution pipe genesis can result in two common forms of metastable structures created in the ground.

* If the cover deposit is a low cohesive permeable granular deposit, solution subsidence can sometimes produce loose zone ground conditions. Consider a sand and gravel layer undergoing solution subsidence to produce loose zone ground conditions; the loss of basal support to the layer, below a circular area, caused by solution subsidence, encourage a local downward movement of the sand grains and gravel clasts. Assuming the layer was compact to start with, the localized solution subsidence effectively cause a portion of the layer to take up a larger volume of space in the ground than it occupied previously. This volume expansion results in a disturbance of the original

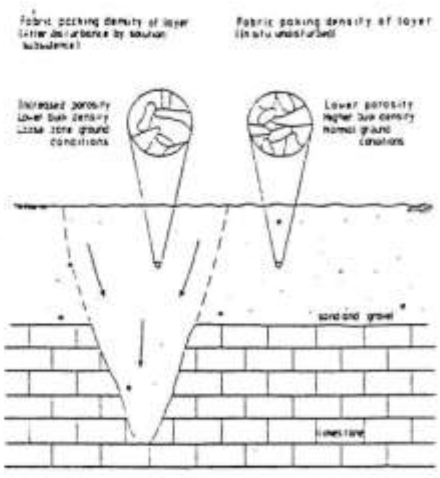

Figure 1 Formation of loose zone ground conditions by solution subsidence.

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compactness of the undisturbed in situ fabric. The volume increase of the layer, occurs by an increase of void space i.e. porosity and hence a lowering of the bulk density. The extent to which the deposit fabric can expand to accommodate the downward movement depends on angularity and grading of the deposit; figure 1.

If the downward movement is more than can be accommodated by expansion of the layer, the overlying layers will be effected too.

* If a more cohesive or resistant layer is present in the superficial sequence then instead of loose zone ground conditions continuing to migrate up through the sequence, the movements halt at the base of the resistant layer. As the solution pipe continues to deepen, the deposits below the resistant layer continue to undergo gravitational settlement creating a metastable cavity roofed over by the resistant layer. Cavities produced have curved walls with arched roofs and their volumetric size tend to be a function of the overburden pressure (confining stress), their size often decreasing with increasing overburden pressure; figure 2.

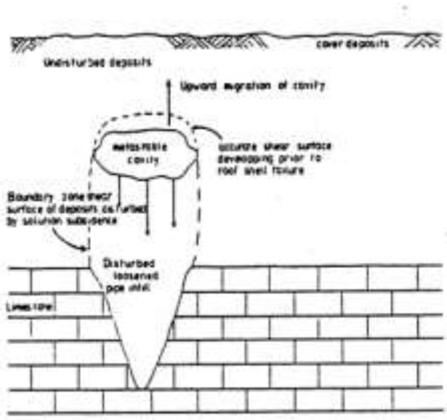

Figure 2 Formation of metastable cavity by solution subsidence

The problem of solution pipes is aggravated by their localized character and the frequent absence of surface evidence.

solution subsidence can take place slowly (low cohesive overburden) or suddenly (forming of cavities), either way it can be disastrous for surface structures and where suspected they should be detected and fully investigated by appropriate site investigation techniques.

The investigation techniques can be divided into three categories namely,
I : remote sensing
II : direct methods
III: geophysical methods

A geophysical investigation was carried out on a test site in South-Limburg, the Netherlands.

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GEOPHYSICAL DETECTION OF SOLUTION PIPES

Introduction

The success of geophysical methods for locating filled solution pipes is controlled by four main factors: penetration (how deep can you measure), resolution (what is the anomaly), signal-to-noise ratio (can you distinguish the anomaly from the noise) and contrast in physical properties (this determines among other things the signal-to-noise ratio and the resolution).

The following methods are used on a test site in South-Limburg:

- magnetic methods (the OMNI 4 protonmagnetometer)
- electromagnetic methods (EM 31 and EM 34)

Location and geology of the site

A test site with a known solution pipe was chosen. This was done to see if a cavity with subsided overburden would also give an anomaly in the measurements. Figure 3 shows the location of the site, it is situated in the south-east of Limburg in the wooden areas (Vijlenerbossen) near Vaals.

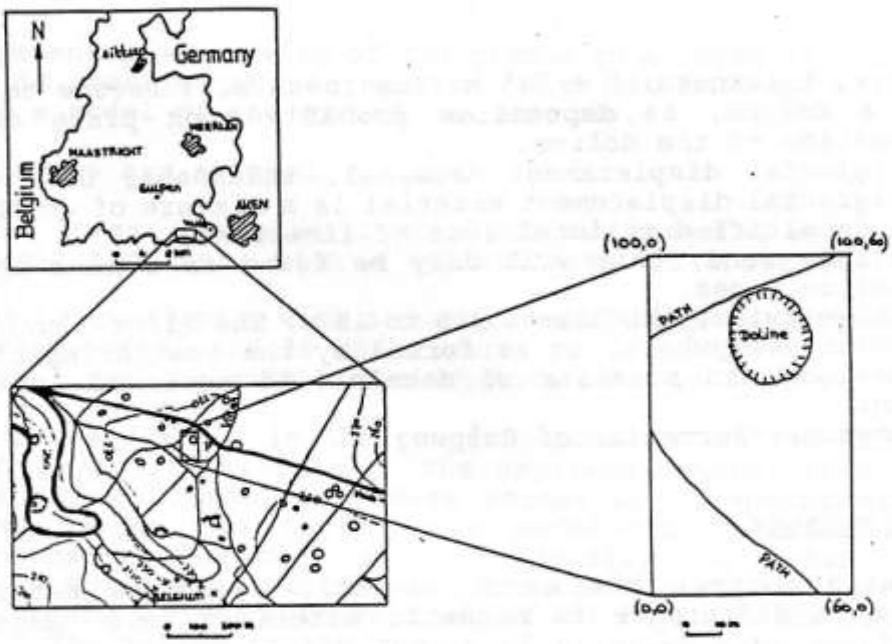

Figure 3 Location of the test site

A possible profile of the investigated area is given in figure 4.

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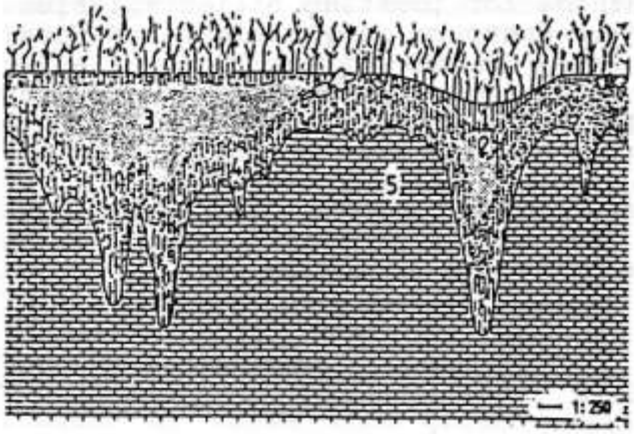

Figure 4 Possible profile of the investigated area

1 = Loess, thickness: 0 - 2.5 m. The loess will become thicker in a doline, as deposition probably took place during formation of the doline.
2 = Periglacial displacement material, thickness: 0 - 3.5 m Periglacial displacement material is a mixture of weathered and decalcified residual loam of limestone.
3 = Tertiary sand, this will only be found as a lens in the solution pipes.
4 = Flint-eluvium, thickness: 3.5 to 15 m. The flint-eluvium is present everywhere, it is formed by the weathering of the limestone and consists of decalcified residual loam and flint.
5 = Limestone (Formation of Gulpen).

Magnetic methods

In magnetic methods the earth magnetic field is measured, indicating a difference in magnetic materials (e.g. magnetic minerals).
The measurements were taken on N-S lines with a station spacing of 2 meters and a line spacing of 5 meters.

However, the differences in the measured values were too small (at the most 20 nT) to indicate anomalies suggesting a possible solution pipe.
It is not sure whether this is only noise (resulting from concentrations of magnetic minerals which obscure the target anomaly) or an indication of possible structures in the ground.

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Electromagnetic methods

In electromagnetic methods a magnetic field is induced by electrical currents. The magnetic field is proportional to the terrain conductivity. The conductivity of the ground can be an indication of soil properties such as water content, porosity and mineralogy.

The EM 34 consists of separate portable transmitter and receiver coils. These are connected by a flexible cable.

On the test site measurements were taken with vertical (exploration depth = 7.5 m) as well as with horizontal coils (exploration depth = 15 m) and with an intercoil spacing of 10 meters. The lines along which the measurements were taken are oriented N-S with a station spacing of 5 meters and a line separation of 5 meters.

The apparent conductivity of the ground to a depth of 7.5 and 15 meters is measured at the surface. With the help of a computer program a model of the ground can be made.

Three layers are assumed:

1) The loess, periglacial displacement material and tertiary sands together, with a conductivity of about 0.6 to 0.8 mS/m.
2) The flint-eluvium with a conductivity which lies between the values of 22 and 26 mS/m, and
3) The limestone with a conductivity of 5 to 6 mS/m.

These values are put in the computer program and the thickness of each layer is determined. The apparent conductivity at the surface is calculated for these values and compared with the measured values. In this way a model can be made of the underground of the test site. Especially the depth to the limestone surface is of interest for the detection of solution pipes. Figure 5 gives a contour map of the depth to the limestone surface. The doline is the result of solution subsidence, thus is situated above a solution pipe and borings carried out by the RGD (Dutch Geological Survey) revealed a thickening of the loess layer in the surrounding of the other possible solution pipe in the figure.

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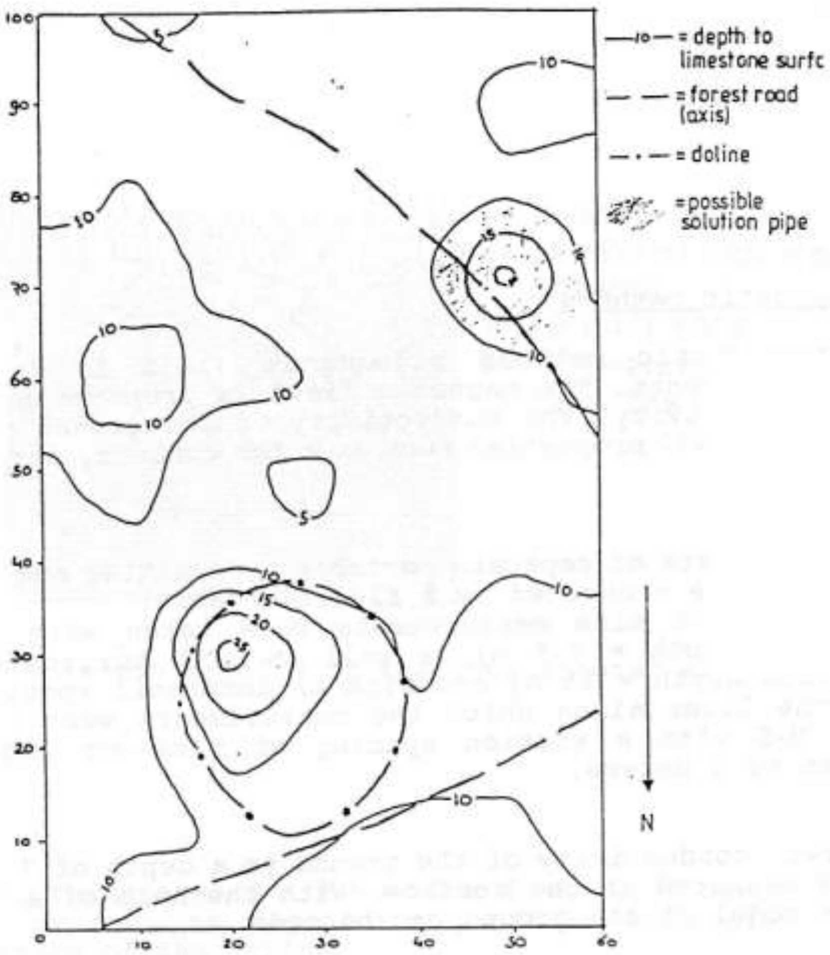

Figure 5 Contourmap of the depth to the limestone

The EM 31 has an exploration depth of only 6 meters and will work in case of near surface phenomena, but is not suitable for solution pipes located deeper than 6 meters. It seems that the lower density and thus the higher porosity (i.e. higher water content) above a solution pipe is not sufficient enough to indicate the solution pipe.

CONCLUSIONS

Solution phenomena can seriously damage structures. It is therefore wise to detect them before subsidence takes place.

Geophysical methods which offer fast continuous profiling (e.g. magnetic- and electromagnetic methods, ground radar) have a distinct advantage over methods where one has to set up equipment for each reading separately (e.g. gravity, resistivity, seismic). For the two methods tried, only the EM 34 seems to be successful. However, in order to do a reasonable reliable prediction more measurements with different coil-separations (different exploration depths) has to be taken, also borings to verify the possible location of a solution pipe should not be forgotten.

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LINEAR CUTTING TESTS IN ARTIFICIAL SAND-CLAY MIXTURES TO STUDY THE INFLUENCE OF BRITTLENESS ON ABRASIVE WEAR OF CUTTING TOOLS.

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Abstract.

Linear cutting tests on an artificial kaolinite clay were performed to distinquish different failure modes during the drying process. The influence of brittleness on abrasive wear of cutting tools was studied by drying an artificial sand-clay mixture.
Three different failure patterns were examined: a ductile mode, at which the chip sticks to the cutting blade, a transitional mode and a brittle mode, at which chips were broken off.
Brittleness is expected to influence the wear mechanism during cutting: the more brittle the rock or soil, the more three-body abrasion will occur. In ductile material two-body abrasion is expected to be of main importance.

1 Introduction.

Abrasive wear of rock-cutter picks is one of the main problems occurring at many rock-cutting dredging projects. To develop reliable site-investigation techniques which can predict the severity of abrasive wear, one has to get a clear insight of all parameters influencing the abrasive wear process.
One aspect which is expected to influence this is the brittleness of rock [1]. According to Robberts [2], rock can fail in a brittle and a ductile manner during cutting. In purely brittle material, a rock chip would break off after penetration by a wedge-shaped bit and the bit would again penetrate the fresh rock. In a ductile rock the contact between the rock and wedge is continuous. From this point of view the abrasive wear is expected to be larger in rock which fail in a ductile manner [1].

In a study of the influence of failure mechanism on abrasive wear of cutting tools, the main problem is to obtain a test specimen which can deform in both brittle and ductile failure mode. Furthermore, to compare the abrasive wear in both modes, the other properties of the specimen which influence the wear process have to be constant. This pilot study describes how a ductile and a brittle failure mode were obtained in different clay specimens.
For the experiments a French shaper, trademark Guillemin-Sergor et Pegard (GSP), was used.

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2 Linear cutting tests with a drying kaolinite clay.

2.1 The test procedure.

The clay used for the testing, was an artificial kaolinite clay (table 1). The mixture has been chosen for its excellent drying properties. No shrinkage cracks appeared during the drying process.

Table 1 Properties of the kaolinite clay.

Property		Value	
Atterberg Limits:			
Liquid Limit (LL) (cone penetrometer)	37.0%		
Plastic Limit (PL)		16.5%	
Plasticity index (PI)		20.5%	
Moisture Content		22.1%	
Shear Strength (lab. vane test)			
peak			27.2 kPa
residual		13.7 kPa	

This kaolinite clay is an artificial mixture of 60% clay, mainly kaolinite, 30% quartz powder, and 10% calcite.

To prepare the samples for the cutting tests, specimens measuring 21 cm in length, 12 cm in width and 7 cm in thickness were cut from the clay blocks.
In order to observe the failure modes during the linear cutting, 7 samples with different moisture contents were tested. These different moisture contents were developed in the clay sample by allowing them to air-dry. The average moisture contents at which the specimens were tested were: 22.1% ; 20.2% ; 18.6% ; 16.7% ; 13.7% ; 11.6% and 7.7%.
Within each specimen, two types of cutting were performed:

Type A: The cutting was performed at the center of the sample. The effects of the sides as well as the proceeding of the chip could be studied.

Type B: the cutting was performed at the side of the specimen. The cutting pattern at the cutting edge could be studied.

The following mechanical test parameters have been used:

- cutting angle: 30°
- cutting velocity: 6 m/min
- cutting depth: 5 mm
- stroke : 150 mm

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The cutting blade was of a simple chisel shape, 49 mm wide, 150 mm in length, a surface roughness of 0.2 μm and made of steel 37.
The test arrangement is shown in figure 1.

2.1 Discussion and results.

From the linear cutting tests, it was found that during the drying of the clay specimens three types of failure behaviour can be distinguished. Samples with a moisture content greater than 17% failed in a ductile manner. Samples with a moisture content lower than 16% showed a brittle failure mode and between these failure modes there exists a transition zone.

To characterise the different failure modes four parameters have been studied: the behaviour of the soil chips, the sides of the cut, the cutting surface and the cutting pattern in front of the cutting edge.

The ductile mode (fig. 2):

This failure mode has the same characteristics as the "flow" type of failure described by two Japanese investigators Hatamura and Chijiiwa [3]: a zone appears where shear deformation occurs continuously when the cutting blade proceeds. The soil chip sticks to the cutting blade along its whole length. In the direction of the cutting edge the thickness of the soil chip increases slightly. The cutting sides are straight and the cutting surface is smooth.

The transitional mode (fig. 3):

As the cutting blade proceeds, a chip develops on the cutting blade face as is the case during the ductile mode. However, instead of sticking to the blade, the chip curves away from it. The cutting side pattern and the cutting surface shows the same characteristics as developed during the ductile mode, but the soil chip retains the same thickness within its whole length.

The brittle mode (fig. 4):

As the cutting blade proceeds, chips are broken off in front of the cutting edge. A slightly curved failure line appears as is shown in figure 4. A "break out" effect at both sides of the tool was observed. A small contact area between the chip and the blade exists, which decreases with decreasing cutting depth.

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More detailed investigation of the wear processes within the ductile mode is necessary to obtain a measurable quantity of wear. Since it is known that cohesive soils containing quartz can be the source of excessive wear, this is certainly an important topic for further investigation.

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Note from the editors:

This article has been modified from its original version. The last 2 sections of paragraph 2.1 on page 28 were not printed correctly in the 1990 issue of the Newsletter due to a typographical error. The correct text has been extracted from Maarten Reinking's original thesis.

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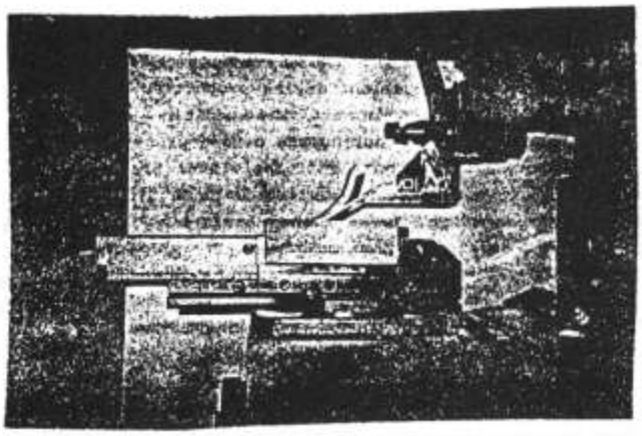

Figure 1. The test arrangement with the test specimen clamped lengthwise in a bench vice.

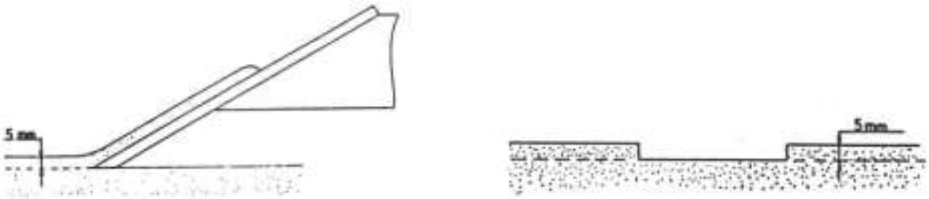

Figure 2. The ductile type of failure.

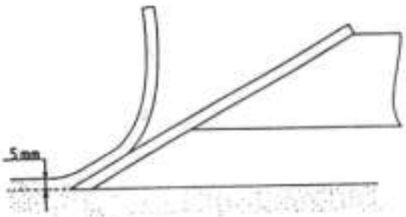

Figure 3. The transitional type of failure.

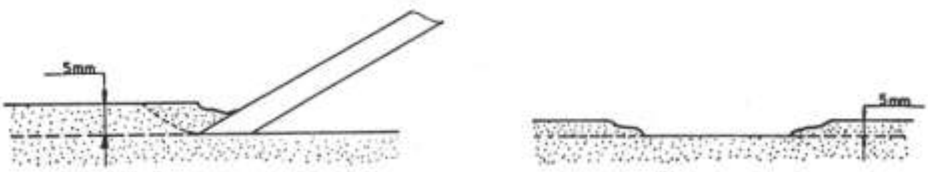

Figure 4. The brittle type of failure.

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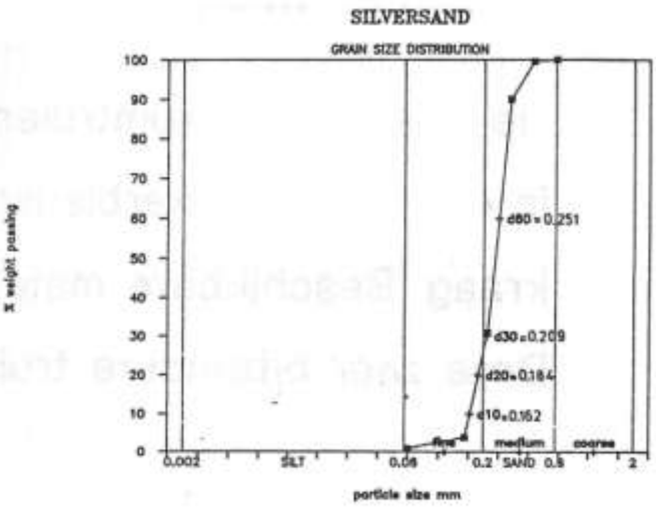
Table 2. Properties of the sand-clay mixture.

Sand-clay mixture characteristics		
Property	Test	Value
Atterberg Limits:		
Liquid Limit (LL)	cone penetrometer	24.6%
Plastic Limit (PL)		12.75%
Plasticity Index (PI)		11.85%
Linear Shrinkage (LS)		5.29%
Moisture content (Mc)		14.5%
Shear strength:		
Peak	lab. vane test	31.5 kPa
Residual	lab. vane test	11.1 kPa
Brittleness index		1.84
Sensitivity		2.84
Cohesion	undrained triaxial test	30.0 kPa
angle of internal friction	undrained triaxial test	4.5°
Unconfined Compressive Strength (UCS):		
- dry (110 °C)	UCS test	2700 kPa
- wet (Mc = 14.5)	UCS test	37.3 kPa
Bulk density		2.17 g/cm³
Dry density		1.89 g/cm³



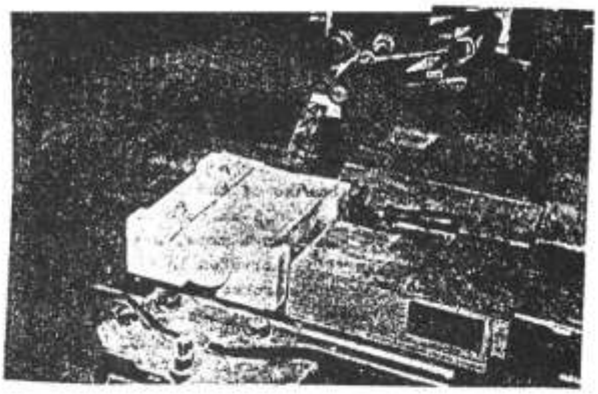


Figure 6. The test arrangement.

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The Wijkertunnel shell layer; evidence for a tsunami or a storm surge deposit?[1]

M.H. den Hartog, Section Engineering Geology, Faculty of Mining and Petroleum Engineering of the TU Delft, The Netherlands (January 1995).

A thin shell layer was found in the Netherlands during the building of the Wijkertunnel. The layer is a mixture of marine shells and salt-brackish water shells. At several locations old borehole logs show a similar layer. The date of the deposition of the layer is similar to the deposition time of the grey, micaceous, silty fine sand layer found in Scotland. There are two possible explanations for the type of event that deposited the shell layer: a storm surge or a tsunami wave. The fine sand layer found in Scotland is believed to be deposited by a tsunami wave, which was generated by the second Storegga submarine slide ±7000 years BP.

INTRODUCTION

This paper was written for a literature study, in the fourth year program of engineering geology, at the department of Mining and Petroleum engineering of the Technical University in Delft, the Netherlands.

During the building of the Wijkertunnel a shell layer was found in the building pit. This very heterogeneous layer was deposited by a high energy and low frequency event. There is still a lot of discussion about the type of event which deposited the layer. Some argue it was deposited during a storm, others think a tsunami is a better explanation for the layer. In Scotland the same type of discussion took place. There a grey, micaceous, silty fine sand layer was found. B.A. Haggart (1988) argued that the apparent confinement of the 7000 year old sand layer of eastern Scotland was better explained by a storm-surge than by the tsunami hypothesis, whereas Dawson et al. (1988) claimed that the great extent of the layer in its areas of occurrence favours the tsunami explanation. Nowadays it is believed that a tsunami wave is the most likely mode of deposition.

This paper describes the sand layer found in Scotland, the evidence for the tsunami wave hypothesis and the different opinions about the type of event that deposited the shell layer in the Netherlands.

Figure 1 The numbers refer to stes where the sand layer has been investigated.

SCOTLAND

The eastern coastline of Scotland is marked by a large number of raised shoreline features formed during and following the retreat of the Late Devensian ice sheet. The area is elevated above sea level due to glacio-isostatic recovery. Amongst the most prominent features are the carselands, raised estuarine sediments of Holocene age. The sedi-

[1] This paper was awarded the "best paper" price in the TUD course "Engineerig Geology workshop" of 1994/1995.

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ments of the carselands consist of a grey silty clay or clayey silt with lenses of sand, gravel and locally shells. These sediments are not uniform. The carseland clays penetrate small peat mosses inland and within these mosses form wedges of sediment. In the carseland sediments a persistent and widespread layer of grey, micaceous, silty fine sand has been identified over a large area in several hundred boreholes (fig.1). The silty fine sand layer generally shows little evidence of sorting either laterally or vertically. At most sites no sedimentary structure has been observed in this layer. The layer forms within the peat a separate wedge below the wedge of grey silty clay (fig.2).

Although not proved more than 74 cm, the layer is generally less than 15 cm in thickness. At several sites there is evidence of erosion of the underlying deposit.

The layer contains abundant diatoms (Paralia sulcata, Cocconeis scutellum, Diploneis and Hydalodiscus stellinger (Smith et al.,1985)) which are dominantly marine and marine to intertidal. There is a reduced occurrence of brackish diatoms which dominate the overlying and underlying carse clays.

Radiocarbon determinations of the peat located at either side of the layer, where the basal peat surface does not apparently show signs of erosion, range from 7140120 to 685075 yr. BP. Age differences between samples of peat taken at the same site above and below the layer are less than 300 years.

ORIGIN OF THE SAND LAYER

The landward tapering style of the sand layer implies a marine derivation, an origin supported by the diatom evidence. The stratigraphy and radiocarbon dates support the concept of a single event, whilst the pollen and radiocarbon dates indicate an event of relatively short duration. Certainly, a terrestrial origin, such as fluvial, is unlikely to have produced a single event so regionally extensive. An aeolian deposit would have been more variable in thickness. Rapid deposition is recognised by the absence of particle size gradation where the layer has been seen in section.

As Smith et al. (1985) observed, the presence of Paralia sulcata in large numbers amongst the diatom population indicates turbulent conditions during deposition.

Smith et al. (1985) considered that a storm surge may have been responsible for the layer, but raised doubts about this origin in view of the uniqueness of the deposit.

Storm surges are generally agents of erosion with the net movement of sediment seaward and deposition of coarse material within fine seafloor sediments. This is due to return currents which

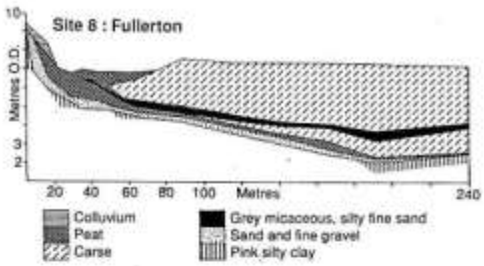

Figure 2 Section through the deposits at Fullerton.

tend to create storm deposits in shallow water areas with the movement of littoral material out to sea. In view of the doubts about a storm surge origin, Dawson et al. (1988) considered that the layer had been the product of a tsunami. Although tsunamis are generally viewed as destructive events, they have been known to deposit on land thin veneers of marine material, predominantly fine sand, derived from the local offshore environment (Wright and Mella, 1963). This is due to the translatory nature of a tsunami wave having net sediment transport landwards.

The sand layer in eastern Scotland has many of the characteristics of a tsunami deposit. Its extensive nature is similar to tsunami deposits. The lack of sorting in exposures of the layer can also be explained by tsunami waves, because due to the few waves involved there is little opportunity for sorting of the sediment carried ashore. A tsunami wave will often disturb the water column to a considerable depth, eroding sediment from the seabed at depths greater than a severe storm wave ; hence the transport of marine diatoms landward.

Dawson, Long and Smith (1988) believe that the fine sand deposit could have been laid down by a high magnitude tsunami caused by the Second Storegga Slide.

The Storegga Slide occurred on the Norwegian Continental Shelf edge at 63N 5E, 750 km northeast of Fraserburgh (fig.3). It is one of the largest submarine slides recorded in the world. This slide has been shown to comprise three events (Jansen et al., 1987). The first and largest slide of which involving 3880 km³ of material occurred before 30.000 years BP. A second slide occurred between 8.000 and 6.000 years BP whilst the third and smallest event probably occurred soon after the second slide. The combined volume of the second and third slides totals 1700 km³. The second slide

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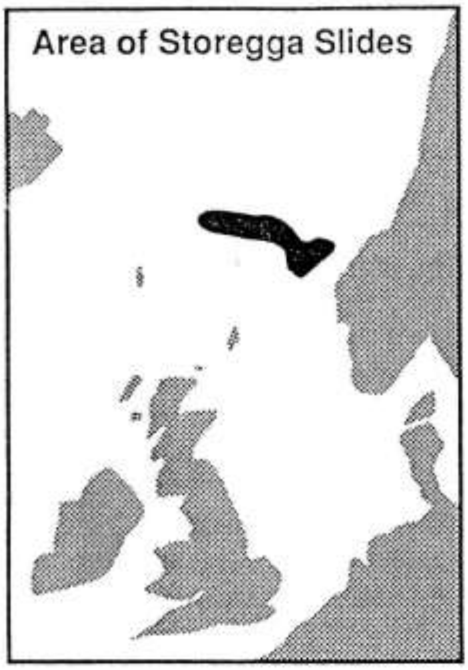

Figure 3 Location of Storegga slide.

covered a depth interval of 3500 m and had a total length of at least 800 km. Earthquakes possibly together with gas released from decomposition of gas hydrates are considered to be the most likely triggering mechanisms.

Computations of wave amplification effects reveal run-up heights for the Second Storegga Slide between 3 and 5 m in exposed areas along the eastern coast of Greenland, Iceland and Scotland and the western coast of Norway (1992).

An earthquake which initiated a submarine landslide and a tsunami wave occurred at Grand Banks, Newfoundland in 1929. The scale of this event (magnitude 7.2, volume of landslide 760 km^3, maximum wave height 12 m) is comparable to that at Storegga.

The Grand Banks earthquake and subsequent slide caused a tsunami which was extremely variable in its effects on the local coastlines. The nearby Sable Island experienced no tsunami, and Johnstone (1930) suggested that this was due to the island being protected by sand banks. The wave is recorded as having reached a maximum of more than 30 m high at Burin (Gregory, 1929) although Johnstone (1930) gave a value of only 12.2 m. The enormous increase in wave height within this area is thought to have been due to resonance in the V-shaped Burin inlet. It is interesting to compare this dramatic increase with the fact that the greatest height above the contemporary high water mark for the tsunami deposit in eastern Scotland occurs in the gullies around the Montrose Basin. Similar comparisons can be made when looking at the apparently strong directional component to the size and velocity of the tsunami wave of both the Grand Banks and Storegga events (if no evidence can be found in Scandinavia). In both events the direction of tsunami propagation was at right angles to the orientation of the slide.

NETHERLANDS

During the building of the "Wijkertunnel", a tunnel below the "Noordzeekanaal", the two building pits were a good opportunity to study the condition of the soils and the local geological evolution of the last 10.000 years. The geology of the pits can be seen in figure 4. The northern building pit was investigated by the 'Rijks Geologische Dienst' in Haarlem.

The basis of the two building pits is the 'dekzanden' (fig.4) at approximately 17 m below NAP. On top of the sand layer the basal peat is found. These peats were developed about 8000 years before present and are found in the building pits as a hard, compressed black peat about 20 cm in thickness with good preserved tree stumps and branches in it.

Thousand years later, approximately 7000 years before present, the west part of the Netherlands was characterised by an extensive lagoon complex that covered the basal peats. In this environment brackish clays (named : Klei van Velsen) were deposited. The following layer found in the northern building pit, has locally a very remarkable, almost 50 cm thick, shellrich layer at the base. Parts of the 'Klei van Velsen', the basal peats and even the Pleistocene sands are locally eroded. On top of this shellrich layer a layer was deposited with alternating sand- and clay layers from a few millimetres to a few centimetres.

N.W. Willemse (Rijks Geologische Dienst, 1994) : The shell layer consists of a graded bed of mainly autochthonous shells which lie at a depth of ± 15.80 m NAP erosively on top of the 'Klei van Velsen' (eastern side) and basal peat (western side). The shell-material consists mainly of Peringia ulvae (80%), Cerastoderma edule, Macoma balthica, Mytilus, Ostrea, Scrobicularia plana and Littorina. Doublets have been found of juvenile Cerastoderma edule and Macoma balthica which indicate an originally subtidal flat fauna of these shell debris. Lithologically it is a very heterogeneous layer of coarse shell-halfs and shell-remains in a matrix of Peringia ulvae, sand and mud (fig.5). In this layer,

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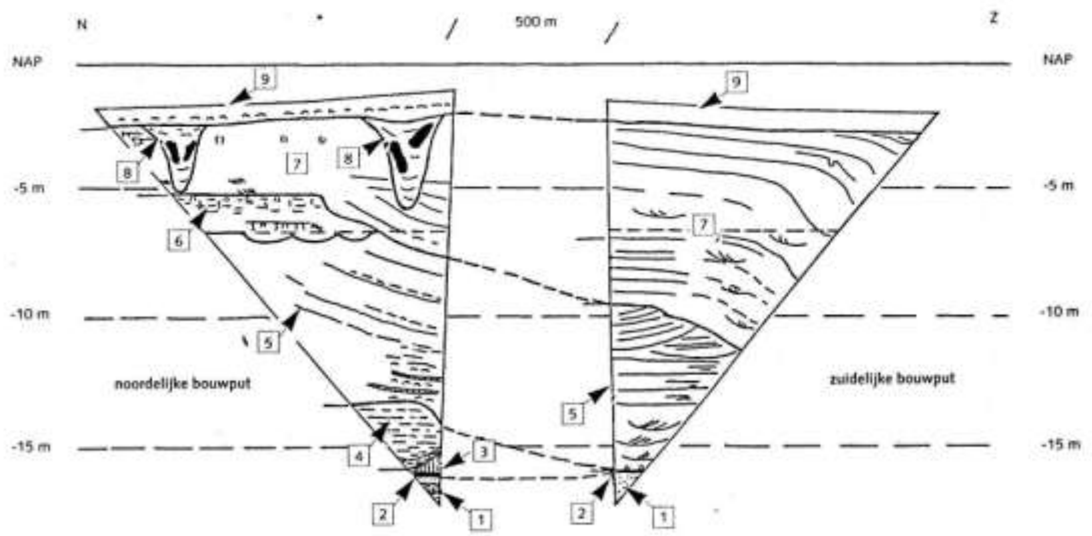

Figure 4 Geological section of the two building pits.

in the building pit, vertically as well as laterally, a gradation is visible. At the east side, the layer consists of a massive graded shell-bed with a thickness of 0.70 m. The layer consists of, in north-east direction dipping sets (± 10 cm) of shell-half's mainly piled up backwards in a finer matrix of Peringia ulvae which at the top of each set is graded in a layer with relatively more Peringia. At the eastside within this bed also enclosed parallel laminated sands with an equal set-thickness are found. More to the west the shellbed is alternated with an in thickness increasing silty/(fine) sandy clay layer which is slightly laminated. Here the shellbed lies unconformably on the basal peat.

There is still a lot of discussion about the way the shell layer was deposited. The layer was laid down during a high energetic and low frequency event. But was the shell layer deposited during a big storm or by a tsunami wave?

N.W. Willemse (1994) draws the (careful) conclusion that the shellbed in the Wijkertunnelpit is deposited in a lagoonal subsystem with a strong marine influence in association with supratidal saltmarshes.

Both for intertidal and supratidal accumulations an erosion/deposition mechanism is assumed in a shallow marine environment in connection with high energy and low frequency events, such as storm surges.

"The shell layer lies in between the lagoonal 'Velsenklei' and a more tidal-dominated deposit above it. The strongly bioturbated bed which is found as topfacies of the shell layer and which is associated with the shell layer is most likely intertidal (tidal flat). The gradation in the layer and the facies-association (parallel lamination, laminated clay), the fact that it is about an isolated deposit which lies unconformably on a lagoonal deposit and the dating of the shell layer at 6730±40 (^{14}C) years BP in relation to MSL, makes it, with the supposed sedimentary environment, acceptable that it is a inter/supratidal skeletal bank. A single trace oxidation within the shellbed which is observed sporadically could support this hypothesis, however it is unclear if it is not an artefact. The set structure in the massive bed which shows a north-east set direction and the intercalation of clays and fine sand layers seawards of the bank are for such a facies indicative sedimentological structures."

Th. de Groot (Rijks Geologische Dienst) thinks a tsunami wave is the most likely depositional medium.

The deposit is probably more extensive than thought when the layer was found. Several borehole logs in Noord-Holland and Zeeland show a similar shell layer. The layer is very heterogeneous and is a mixture of marine shells and salt/brackish-water shells. Some tree trunks were found which were rooted in the Pleistocene sand, but are broken off at the contact between the 'Velsenklei' and the shell layer. The event had to be strong and large enough to bring shells from a few kilometres from the coast onto land, and to erode locally the 'Velsenklei' and the basal peat.

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Figure 5 Profile sketch of the shell layer by N.Willemse.

The dating of the layer is more or less the same as the fine sand layer found in Scotland, ± 7000 years BP. This might be an indication that the tsunami generated by the Second Storegga Slide has also deposited this shell layer. But a lot more research needs to be carried out before more founded conclusions can be drawn.

The shell layer found in the Netherlands might support the tsunami hypothesis in Scotland and maybe investigations in adjacent areas (for example Norway) can show similar layers which never attracted special attention before.

CONCLUSIONS

There is still a lot of discussion about the mode of deposition of the shell layer. The shell layer was deposited by a high energy and low frequency event at more or less the same time the sand layer in Scotland was deposited, ± 7000 years BP. The sand layer in Scotland is believed to be deposited by a tsunami wave generated by the Second Storegga Slide. The uniqueness of the layer within the Holocene record mitigates against a storm surge origin. However a tsunami origin explains the transportation of marine sediments onto land and the regional presence of the layer with its highest contemporary altitudes occurring in narrow inlets. Still a lot of research needs to be carried out, but comparison of the shell layer with the sand layer in Scotland and with historical tsunami deposits found in other countries, might aid the determination of the type of event that took place ±7000 years ago.

ACKNOWLEDGEMENT

I would like to thank Th.A.M. de Groot, N.W.Willemse and D.Beets of the 'Rijks Geologische Dienst' who supported me in this literature study. The source of all the information about the shell layer found at the Wijkertunnel location is the 'Rijks Geologische Dienst' in Haarlem.

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Geotechnical aspects in soft soil shield tunnelling

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In densely populated areas more and more the underground will be chosen for the location of infrastructural works, storage rooms and parking places. Trenchless technology covers the field of shield tunnelling, pipe jacking and directional drilling; all techniques that have been developed to decrease the hindrance of work under construction for the regular life at the surface. The drilling techniques are highly mechanised and in the design of drilling machines and in tunnelling practice, calculations have to be made regarding torques working on the cutterwheel, friction on the shield, face stability, etc.

INTRODUCTION

At present, many infrastructural works are being built and planned in the west of the Netherlands. Examples are the Piet Hein Tunnel, the Wijkertunnel and the 'Tweede Heinenoordtunnel'. Even larger projects that are planned are the 'Noord/-Zuidlijn' in Amsterdam, the 'Hoge SnelheidsLijn' and the 'Betuwe Lijn' in which the 'Botlek Spoortunnel' is comprised. In the south west of the Netherlands the 'Westerschelde Oever Verbinding' is projected. Going underground seems to be the trend and politics is running warm for it as well.

In many cases trenchless technologies will be applied in order not to disturb activities at the surface or to avoid the necessity of breaking down sites with special historical or environmental value. In this respect the decision has recently been made in politics to drill a tunnel under 'Het Groene Hart' for the 'Hoge SnelheidsLijn'.

Today also for smaller projects such as crossings for pipelines, sewers and cables trenchless technologies are often chosen. Directional drilling and pipe jacking are widely available techniques and for sewers there is an option for relining the system.

Of the projects mentioned above the 'Tweede Heinenoordtunnel' and the 'Botlek Spoortunnel' form two study projects for the Netherlands as these are the first large diameter tunnels built by the shield method; the drilling of the 'Tweede Heinenoordtunnel' will commence by the beginning of 1997 and the drilling of the 'Botlek Spoortunnel' is planned for 1998.

In this article some geotechnical aspects are highlighted that are related to shield tunnelling. Many of them have a parallel to the pipe jacking technique. Both techniques will be briefly explained.

SHIELD TUNNELLING AND PIPE JACKING

Shield tunnelling is the construction of a tunnel in situ, using a machine that excavates the ground ahead of it and that leaves a tunnel tube behind. The shield moves itself forward by pushing against the section already constructed. Machines have been constructed for tunnels with an external diameter of some 5 meters to up to 14 meters.

Figure 1 *Pipe jacking.*

Pipe jacking is generally applied for tunnel diameters ranging from 25 cm to 3.5 m. Here the entire tunnel or pipe, with the machine in front, is pushed forward from the launching shaft. New lengths of pipe are fed into the shaft (figure 1). When the total length of pipe causes a friction that exceeds the thrust force of the jacks in the shaft intermediate jacking stations can be installed that travel along with the tunnel or pipe.

Roughly two types of shields can be distinguished in shield tunneling: the slurry shield and the EPB shield (Earth Pressure Balanced shield). These differ basically in how the stability of the excavation face is secured, which has a consequence for the way of transportation of the excavated soil to the surface.

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SLURRY SHIELD

The excavation and mixing chamber of the slurry shield is filled with a bentonite slurry (figure 2).

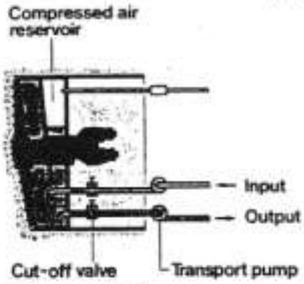

Figure 2 Slurry shield.

Bentonite is a montmorillonite clay mineral. Mixed with water to a slurry with a certain pH it has a stable cart house structure with a typical shear strength ,scosity, depending on the concentration of the slurry. By maintaining a pressure in the chamber that is slightly higher than then the pore water pressure at the face, the bentonite particles will infiltrate and block the pores at the face. Finally a consolidation phase of the bentonite towards the face takes place leaving a so called filter cake behind. The pressure in the chamber acting on this 'closed' face secures the soil stability.ideal medium

The transport of the excavated soil is normally by hydraulic transport: the stable bentonite slurry carries the soil particles and is pumped to the surface. Here the soil particles are separated from the slurry that is subsequently pumped back to the shield for reuse.

The slurry shield has initially been developed to deal with granular soils: pressurised air or water were not suitable to support the face and to control ground water influx in shield tunnelling. With a bentonite slurry it is possible to transfer a fluid pressure to the excavation face.

EPB SHIELD

With an EPB type shield the excavation and mixing chamber is filled by the excavated soil. A screw conveyor that reaches into the chamber extracts the excavated soil from the chamber (figure 3). The pressure of the remoulded soil in the chamber maintains the face stability. Control of the pressure is determined by the revolution speed of the screw conveyor and by a back pressure valve at the outlet of the chamber.

From the outlet the material is dropped on a conveyor belt, which takes care of the transport over the back-up train which houses the auxiliary equipment for the shield. At the end of the back-up train the soil is either dumped into a train for transport to the surface, or the conveyor belt reaches up to the surface. Several alternatives are possible, depending on the length of the tunnel, the advance speed of the shield, the availability of a dump site at the surface.

The soil often has to be conditioned to reach a certain level of plasticity and to decrease the permeability of the excavated soil. The plasticity is needed to improve the flow characteristics while the low permeability is necessary to avoid the influx of water into the tunnel via the screw conveyor. Some soils, such as soft clays, may not require any treatment to reach the desired characteristics. Soil may be treated with bentonite, polymers or foams. The foams that have been developed have the advantage of being biodegradable: the excavated soil can easily be disposed of without any treatment or additional costs.

Regarding the soil conditioning the mixing process in EPB shield tunnelling is much more important than in slurry shield tunnelling. The excavated soil should have a certain homogeneity when leaving the chamber. The mixing is done by the cutterwheel, equipped with possible additional bars at the back, or by extra installed mixers in the chamber.

The EPB shield is of Japanese origin. The application was mainly in clay, which is common in the urbanised areas. The use of foam has widened the range of soil types in which the EPB shield can be applied (figure 3).

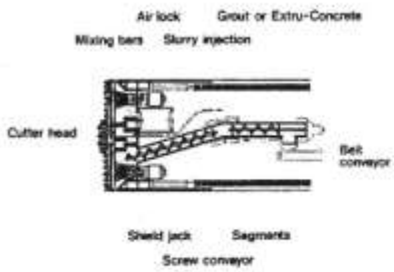

Figure 3 EPB shield.

FACE STABILITY

In shield tunnelling there is a large free excavation surface: the face. When the face is stable no deformation of the ground will occur: there is no settlement or heave at the surface. To maintain face stability ideally the pressure in the chamber is equal to the horizontal ground pressure. Too low a pressure in the chamber will cause active failure of the face while a too high chamber pressure may induce passive failure of the face.

The chamber pressure in the slurry shield is

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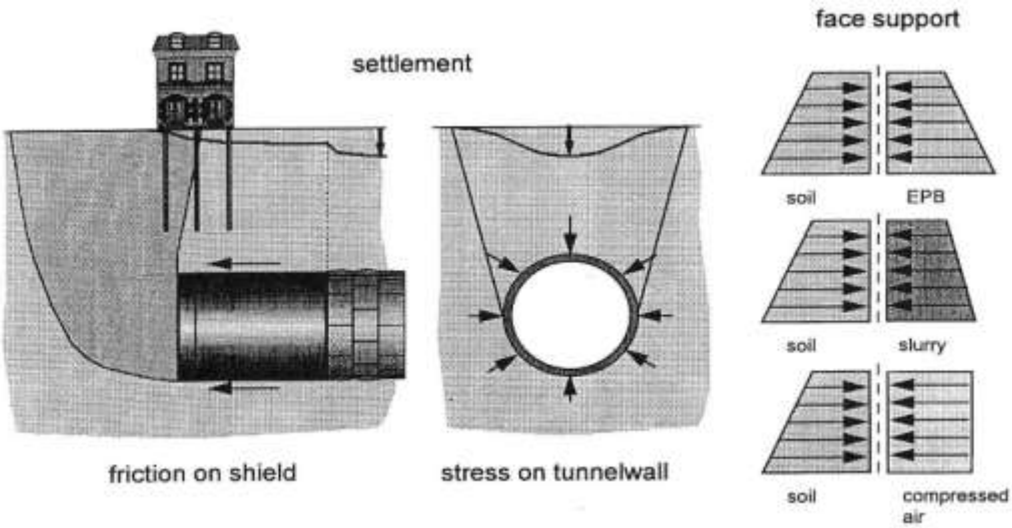

***Figure 4** Face stability.*

maintained by the pressure of the bentonite slurry. Controlling this pressure by valves and the rotation speed of the slurry pumps is difficult and therefore involves a certain risk. A German patented way of controlling the pressure is by controlling the pressure of an 'air vessel' above the slurry. The slurry shield intrinsically does not offer optimal stability when the density of the slurry is lower than the density which causes a too low pressure gradient along the height of the tunnel is (figure 4).

In the search for optimising the face stability in shield tunnelling and not wanting to apply the German patented way of controlling the pressure of the bentonite slurry in the chamber, the alternative was born to support the face by the excavated soil itself. This has the advantage of a chamber filled with a substance of a density equal to the density at the face. Hence, the pressure gradient is equal as well.

However, to create a natural pressure distribution of the excavated soil in the chamber it should behave as a liquid. This is not to difficault to realize for soft soils. Granular soils, however, will have to be conditioned to reach proper flow characteristics.

SELECTION CRITERIA

As stated above, the slurry shield has been developed for granular soils. The mechanism of infiltra tion of the bentonite slurry into the pores, the blocking of these and the consolidation of the adjacent layer to form the filtercake gives optimum results for these soils. Sofar, granular soils refer to sandy soils (figure 5). Gravels and pebbles induce problems because of their higher permeability and the larger pore size. Additional additives, such as sawdust may be necessary. Leaving some sand in the recycled slurry to block the pores is also mentioned as a solution.

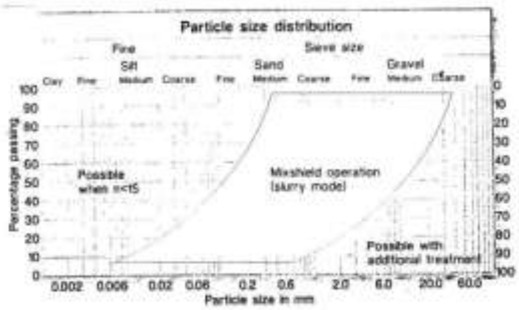

***Figure 5** Grain size distribution of soil suitable for slurry shield tunnelling.*

Soils with a high fines content or cohesive soils give problems at the separation plant: the fines cannot easily be separated from the bentonite particles in the slurry. For an effective separation the plant becomes extensive and expensive and the separation time gets long.

The EPB shield tunnelling method is especially suitable for clayey and silty soils and granular soils with a high content of fines (figure 6). As the excavation and the transportation takes place in a 'dry' environment the flow characteristics of the soil are important. The better the characteristics the less treatment is required and the less important a proper mixing of the excavated soil in the chamber becomes. Recently the range of soils for which the

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EPB shield method can be applied is expanding because of the development of new additives, like foams and polymers.

For the selection of the shield type, the costs of the slurry treatment plant in case of a slurry shield is an important aspect. For the EPB method the conditioning of the soil may lead to high operational costs. First of all the applicability of the technique with respect to the soil profile should be determined. In geotechnical terms the parameters are:

- soil type: grain size and grain size distribution
- water content
- ground water pressure

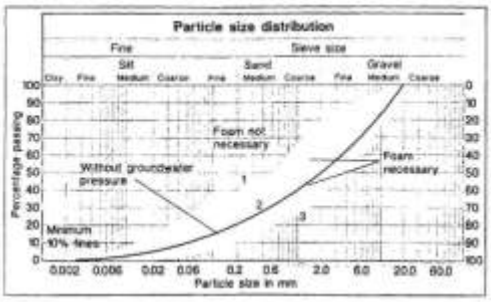

Figure 6 Grain size distribution of soil suitable for EPB shield tunelling.

CUTTING FORCES

The excavation of the ground in shield tunnelling is generally done by a cutterwheel consisting of a hub in the centre from which spokes run radially outward.

In some cases the spokes are surrounded and connected at the circumference by a rim. At the spokes the cutting elements are mounted that excavate the soil face ahead by the rotation of the cutterwheel and the progress of the entire shield. In the design of tunnelling machines it is important to know the maximum torque to occur on the cutterwheel.

One of the torques that play a part is the torque required to move the cutting elements through the face. This torque can be found by calculating the cutting forces for the individual cutting elements on the cutterwheel. The summation of the individual products of the tangential component of the cutting force and the distance to the axis of rotation provides the cutting torque.

Several theories, originally developed for the dredging industry, are available on the cutting of sands and clays. During the cutting process of the cutterhead of a cutter suction dredge the soil is loosened from a fresh soil mass in a watery environment. When cutting sand dilatation takes place in front of the cutting element which causes water underpressures to occur in the cut sand chip. Water can flow into this chip from the uncut sand and from the free surface. The cutting process in a slurry shield takes place in a bentonite environment. Like in the former case dilatation takes place and water underpressures occur, however the resistance for the water (or bentonite) to flow towards the lower pressure is higher. Firstly, bentonite has infiltrated into the face and so into part of the chip or even into the entire chip, depending on the cutting depth. Secondly, the free surfaces are covered by the filtercake of bentonite particles. Consequently, the water underpressures will remain lower. These water underpressures are the cause of higher cutting forces. When the water underpressure reaches vapor pressure cavitation will occur.

Shortly, to apply cutting theories from dredging to the excavation process of a slurry shield new boundary values have to be stated clearly.

The following parameters concerning the ground conditions must be known for calculations on the

cutting forces:
- depth below surface and soil profile
- ground water pressure
- density of ground water
- density of the soil
- angle of internal friction
- cohesion
- adhesion
- permeability
- porosity and maximum porosity
- friction angle soil to steel
- characteristics of the bentonite slurry and the bentonite pressure.

Besides the configuration and shape of the cutting elements, the rotation speed of the cutterhead and the progress speed of the machine belong to the input values. Looking to the excavation of the soil in an EPB shield, not only the properties of cut soil have been changed depending on the type of soil conditioning. Also the chip experiences a high friction when travelling along the cutting element because the chamber is not filled by a slurry but by the soil itself.

SHIELD FRICTION

Another important part in the design of a tunnelling machine is the dimensioning of the jacks with which the machine pushes itself forward. Forces involve amongst others the axial component of the cutting forces, the pressure in the chamber and the friction of the surrounding soil along the outer circumference of the shield.

Generally some overcutting of the profile of the machine is done, i.e. the cut diameter is slightly larger than the diameter of the machine. This is done to reduce the friction of the shield in the ground to a certain extent. The friction cannot be reduced too much as in that case the reaction torque resulting from the cutterwheel will cause the shield to rotate itself. After the overcutting relaxation of the soil towards the shield takes place, the result of which upon the resulting stress on the shield is hard to predict. Also will the bentonite fill the space between the soil and the shield, having a lubricating effect. This implies that the regular values for the friction angle between a certain soil type and steel cannot be used.

The overcutting can be done selectively with the position of the cutterwheel. This is to be able to excavate some extra space for initiating and controlling curves in the planned track. Both for friction purposes and for the manoeuvrability of the machine often the diameter of the machine decreases towards the tail. This makes predictions on the shield friction in the ground even more complex.

The geotechnical parameters for calculations on the shield friction are:
- depth below surface and soil profile
- ground water pressure
- density of ground water
- density of the soil
- angle of internal friction and cohesion
- characteristics of the bentonite slurry

TRANSPORT AND TREATMENT

The excavated soil can be transported by hydraulic transport or in a 'dry' state by conveyor belts, train wagons or others.

In hydraulic transport the prevention of settlement of soil in the pipe, i.e. blocking of the pipe, is important. The power depends on the pipeline resistance of the slurry in the pipelines and on the static head. Towards the face the slurry is clean and has a low density. From the face to the surface the slurry is loaded with the excavated soil and has a high density. The pipeline resistance depends on the average grain size of the soil, the roughness of the pipe wall and the slurry velocity.

At the surface the slurry is fed to the separation plant or slurry treatment plant. Here the excavated soil is separated from the bentonite slurry that is used again. For coarse soils this separation is relatively simple. If the soil contains much fines, the separation plant gets more complex because the fines are hard to separate from the bentonite particles. The less the measure of separation the faster the bentonite gets contaminated and the fewer times the bentonite can be recycled. So for the separation it is not only the average grain size that counts but also the grain size distribution that plays a very important part. This way of transport and the slurry treatment are inherent to the slurry shield tunnelling method.

The excavated soil from an EPB machine is often transported dry when leaving the screw conveyor. This means removal to the surface takes place by conveyor belt or by train wagons or others. When foam is used a defoamer may be sprayed on the soil at the beginning of the conveyor belt. No further after treatment is said to be necessary before disposal.

Soil treatment, both separation in the case of a slurry shield and conditioning and defoaming in the case of an EPB shield, is expensive. Estimations on the consumption of foaming and defoaming agents per quantity of soil have to be made. Only little is known about this subject. However parameters that are important in the predictions can be given by:
- density of the soil
- angle of internal friction and cohesion

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- moisture content, plasticity index
- shear strength
- grain shape and size distribution
- bulking factor

It is clear that there is an interesting link between engineering geology and mechanical engineering in the field of trenchless technologies. The soil and its properties determine to a large extend the requirements of a tunnelling machine, such as cutterwheel torque, thrust force, transport and mixing phenomena. Many soil parameters are being used in the design phase of a tunnelling machine and during the tunnelling project itself, for extensive calculations. Soil investigation and soil testing is important prior to the design of a tunnelling machine.

A lot of work is still to be done in finding and validating calculation models on the various (geo)-technical aspects of shield tunnelling.

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"$100,000,000 was the damage: whither research in the 1992 Roermond Earthquake?"

P.M. Maurenbrecher and A. den Outer, Delft University of technology, Faculty of Applied Earth Sciences, Delft.

Originally the above damage for the talk was advertised in guilders but Berz (1994) puts the figure in terms of *f* 170 m. If Germany is included the total comes to DM 250 million. Less than one percent of that amount would be needed to carry out a Ph.D. research project of 4 years to make an inventory of the actual damage so that architects and structural engineers in the Netherlands can know what aspects of their buildings are vulnerable to earthquake loading damage.

The cost of such research is minuscule not only in terms of percentage or as a research project in itself. The reason for this is that the information exists on paper: in the vaults of the Municipality of Roermond consisting of about twenty to thirty large lever-arch files. These files contain forms filled in by surveyors to estimate damage to houses so as to decide which households would be eligible for repair grants from the "Rampen Fonds" (Disaster Fund) set up by the government following the earthquake. The difference in damage exceeding *f* 5000 would qualify for grants.

WHERE ARE THE PRIORITIES FOR RESEARCH ON THE ROERMOND EARTHQUAKE?

Initially in 1992 with this unique event fresh in everyone's mind and having received considerable publicity on television, radio and the press there were many potential researchers keen to get on the earthquake/seismology bandwagon. Evidence for this is the special volume giving papers presented at a symposium held nine months after the earthquake in Veldhoven. A total of 40 papers were published subsequently in a special issue of "Geologie en Mijnbouw," Vol 73, Nos. 2-4 1994/1995. Most of the papers were seismological in content and only a dozen papers were concerned with damage to structures. The only paper by a structural engineer (Bouwkamp, 1994) is not from the Netherlands but from Germany. He did view a number of damaged buildings, which he describes in his paper, and then, gave a general presentation on the causes of damage to buildings in terms of their design and construction under earthquake loading. Quite rightly he states "*Based on geological site conditions, zoning provisions are absolutely essential. The observed slope failures of the Maas embankments as well as the observed soil liquefaction in the flat lands along the Maas river, clearly illustrate the danger of building along or near the Maas, or for that matter in the flat lands along any major river in western Europe (e.g. Rhine), where such river passes through a seismic zone.*"

RESEARCH DESPITE LACK OF FUNDS: WHAT IS POSSIBLE?

This research has been pursued by the Engineering Geology Group at the Faculty of Applied Earth Sciences, TU Delft.

The research is financed by the "primary money stream," a lump sum given by the Ministry of Science and Education to allow for basis research financing. The sum is distributed to researchers in Delft in proportion to the number of publications they produce. The sum is normally insufficient to

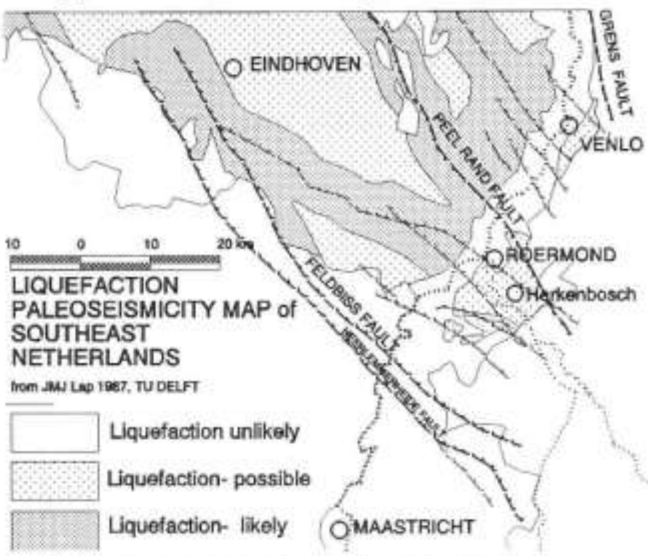

***Figure 1** Liquefaction Zoning Map of the South of the Netherlands by Lap (1987)*

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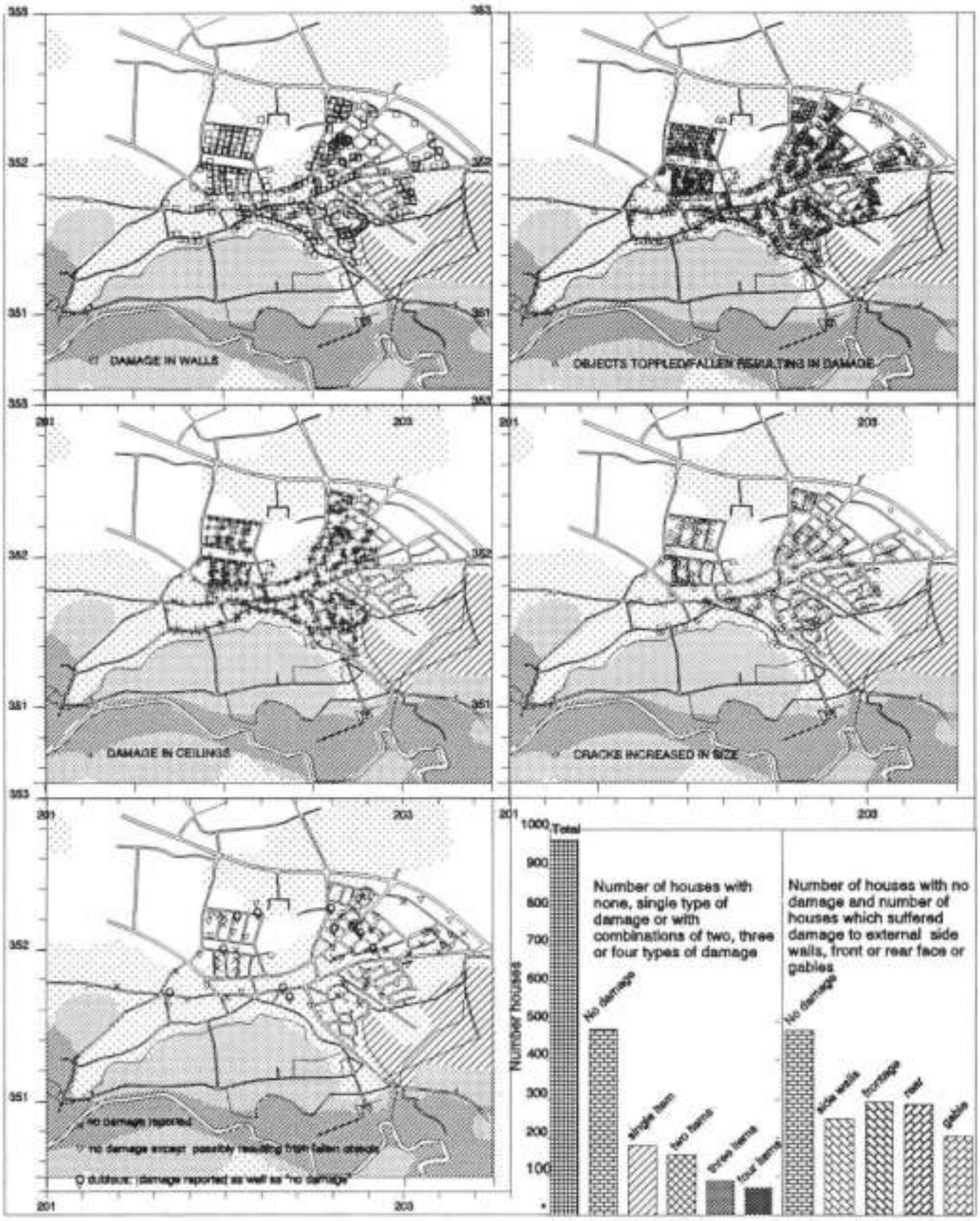

from Geology Map HerkenboschNetherlands Geological Survey map 58 IV Roermond 1933

HOLOCENE				PLEISTOCENE
Aeolian sand often dune complex. Grading fine little medium or coarse.	Fluvial clay; fine sandy clay or clayey fine sand, thicknesses exceed 0.5 m.	Marsh peats forming over fluvial sands & lower terraces; peat formation interrupted due to drainage or exploitation. Organic sand with peat crusts	Fluvial sand; predominant-ly fine sands sometimes slightly clay-ey or with small clay lenses	Lower terraces of Maas and Roer rivers. Horizontal layers fine sands could have coarser grading and contain gravel bank deposits

Figure 2 Distribution of Damage Superposed on geological Map 58 IV Roermond 1993.

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finance research projects such as the for the Roermond Earthquake. As long as expensive fieldwork site investigation methods and laboratory

testing is avoided costs can be met in this way.

This has, up to now, been possible as the research for the zoning maps required assembling existing ground data.

An early example of a zoning map is that produced by Lap (1987) and shown in Figure 1 as an English version of that given in the Prepal publication (1994). Lap's map has been produced on a relative macro-scale.

Presented as poster at Veldhoven and later published in "Soil Dynamics and Earthquake Engineering VII", a paper by ourselves and De Vries (1995) attempted to correlate damage to the geology (Figure 2). The correlation was not really possible as the geological "resolution" was to coarse- based on a map of 1933. The recorded damage seemed to be fairly evenly spread. What the survey did highlight, however, was that some houses of the same design as their neighbours, were not damaged. We reassessed the data and produced maps (Den Outer & Maurenbrecher, 1994) showing the distribution of degree of damage using the MSK, MMS, & JMS classification (Figure 3). Though this procedure is not considered very scientific by seismologists (they use the MSK, MMS & JMS system to plot the regional attenuation of earthquake intensity). Anomalies remained in that why were some houses of the same design and construction, more badly damaged than their neighbours?

The stage has now been reached that further investigation is needed to fill in the gaps of knowledge. Ard den Outer who has been the "brain" behind most of the research in the last three years outlined briefly some of the research that has been done, mainly by students as their final year M.Sc. project.

One gap in information was geology. A map has been produced by De Vries (1996) based on predominantly information from the RGD and site investigation companies' archives. The earlier geology map superimposed on the damage survey maps in Figure 2 should, ideally, be replaced by the up-dated map.

Another project carried out by Manzano, (1996) focused on the liquefaction phenomena that occurred during the Roermond earthquake, sand-vents just outside the municipality of Herkenbosch. Back calculation of liquefaction potential, using a method based on Cone Penetration Testing developed by Olsen (1988) showed that at least 0.29g (= 2.8 m/s^2) of ground acceleration was needed to cause the sand-vents; much higher than originally assessed by the seismologists. Although these results have to be investigated for the sensitivity for those geotechnical parameters that had to be estimated, it has become clear that to predict proper ground accelerations in an area, geotechnical investigations, in addition to traditional seismological methods, are essential.

A numerical study of the ground response, using the program SHAKE91 (Schnabel et al, 1972), during the Roermond earthquake by Van Daalen (1996) showed that only the upper 30 to 40 ft have a significant influence on the decrease in damping in the ground and the related "amplification effect." Any local variations of the macro-seismic intensity

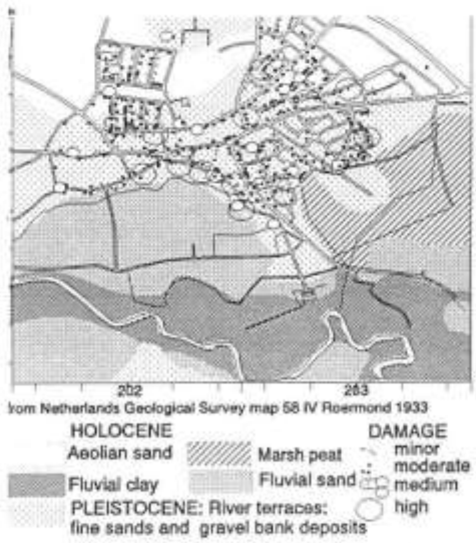

***Figure 3** Simplified Detailed MMS-Intensity Assessment for the Herkenbosch Area.*

are therefore thought to origin in this range of depths, while more regional variations are probably resulting from changes in total thickness of the overburden or due to a particularly unfavourable location relative to the tectonic structure.

The most recent study by Spruit (1997) is still carried out at this moment and investigates the local soil type and thickness variation by an Electro-Magnetic (EM) survey. This results have to be compared with the information on soil variation from the relative small number of borehole and penetration test locations in the area. Geostatistical analysis has to show whether the observed variation for the high resolution EM-survey can be properly described by the low resolution down-hole observations.

BUT WHAT HAS HAPPENED TO THE BAND WAGON OF 1992?

It appears to have been ditched once the Veldhoven symposium was held. Yet the date of the Veldhoven meeting was only nine months after the earthquake, hardly much time to allow for research. In truth, a submission had been made for one of the EU-theme projects. Its north-European flavour probably meant it was doomed from the start.

To try and keep an active group together proved increasingly futile from the limited resources within the Engineering geology group. Interest has been minor or non-existent from institutions who one would expect to play a leading role: VROM (Ministry of Housing, Planning and Environment), TNO-Bouw, the faculties of building sciences at TU Delft or TU Eindhoven. Similarly little interest has been shown by the structural engineering departments at Delft or Eindhoven despite some communication with student researchers. A few tentative attempts were made to interest them of the possibilities to carry out research. The reason co-operation is sought is because of that huge archive lying idle in the Roermond municipality vaults. The forms are filled in by hand in almost illegible handwriting. For some one familiar with building terminology it should be quite simple to make out the handwriting. Registration of the information in a proper data base should show where weaknesses existed in design and construction of housing and indicate, for safety, damage mitigation and insurance purposes what extra strengthening would be required in existing and future buildings.

WHAT CHANGES ARE REQUIRED TO PUT EARTHQUAKE ENGINEERING ON A PROPER FOOTING IN THE NETHERLANDS?:

CHANGE OF ATTITUDE? INTENSIVE LOBBYING? NETHERLANDS EARTHQUAKE SOCIETY?

The following programme was downloaded from the home pages of the Institution of Civil Engineers (ICE on-line: http://www.ice.org.uk/)

Evening meetings of the Society for Earthquake and Civil, Engineering Dynamics

- Introduced by Dr K Pilakoutas et al, 29 JAN 17:30 "Repair and strengthening of structures following an earthquake"
- Introduced by R Yeung et al, 26 FEB, 17:30 "Alternative methods for blast analysis on structures"
- Introduced by Dr T Blakeborough and Dr J Bommer 26 MAR, 17:30, "Field observations of earthquakes
- Introduced by Dr T Wyatt, Professor R Severn et al 23 APR, 14:00 "Passing on experience - a master class" AGM at 17:00
- Presented by Professor R Severn 21 MAY,17:00 "Structural response prediction using experimental data", Mallett Milne Lecture

Should not the Netherlands also have a Society for Earthquake and Civil Engineering Dynamics? Is the Netherlands less prone to earthquakes than the UK? I leave you to ponder these questions.

DORMANT ARCHIVE OF THE DISASTER FUND: WHAT TO DO?

One gaping void remains in the research which can easily be financed through the primary funding: compilation of the disaster fund archive. Despite Bouwkamp emphasising that zonation is important, he also makes it abundantly clear that design and construction of buildings and structures can define their susceptibility to earthquake loading.

The "damage" surveys carried out by the engineering geology group were based on a hurried and not all to comprehensive questionnaire. The disaster fund surveys go a stage further. The actual damage is recorded and the degree of damage is given in terms of the repair costs.

In most cases of damage only the repair costs will be used in the Ph.D. study by den Outer, for the correlation with geology, as specialist knowledge to make a more detailed interpretation of the damage reports is not available. Together with construction records a great wealth of information remains to be unleashed from the Municipality of Roermond and the Municipality of Roerdalen (Meelick, Herkenbosch and Vlodrop) situated to the southeast of Roermond.

Possibly Prof. David Price had the last word shortly after the Roermond earthquake: "There is sufficient (research) work for decades to come; we must ensure that we provide the data for that research".

ADDITIONAL NOTE

April 13th 1997 the 5 year seismic pause after the Roermond earthquake was remembered in the area near Herkenbosch, by a local television coverage of past experiences and recent accomplishments on "How to prepare for a future earthquake ?" It is unfortunate that the authors reluctantly had to comment that the earthquake engineering research is limited to the Engineering Geological Group Delft only.

The recurrence time of significant flooding caused by the main rivers is said to be less than 300 years, and is a thread taken seriously at this time. It should be needless to say that the Roermond earthquake

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could easily have caused more casualties, had it occurred at a less fortunate time, i.e. day-time. An earthquake of magnitude 5 of higher has a recurrence time of 105 years (de Crook, 1994) and no actions are initiated. It can only be hoped that the recent developments in case of induced earthquakes in the northern part of the Netherlands will result in more serious plans to prevent future damage or worse from any type of earthquake.

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Offshore Edition, 2003

Kunstmatige eilanden in de Kaspische zee

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Inleiding
In het noordelijk deel van de Kaspische zee, waar de waterdiepte maximaal circa 8 meter bedraagt, wordt door een consortium van oliemaatschappijen, Agip KCO, een aantal kunstmatige eilanden aangelegd voor oliewinning. Het totale project is rond 1995 in gang gezet. De Kaspische zee wordt gezien als belangrijk natuurgebied, waar strenge milieueisen gelden.

Op dit moment is één eiland in gebruik voor exploratieboringen en zijn de twee volgende onder constructie. Witteveen+Bos levert de civiele ontwerpen en voert ook toezicht. Hiertoe is een kantoor in Kazakstan geopend (in Aktau).

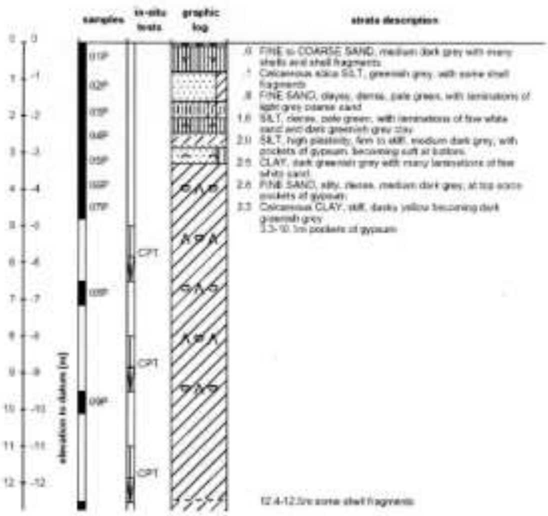
In dit artikel wordt een beschrijving gegeven van een deel van de ontwerpactiviteiten voor het eerste eiland.

Bodemgesteldheid
Voor drie "hoofd locaties' is in 1997 verkennend grondonderzoek uitgevoerd. Het onderzoek is uitgevoerd door Fugro Engineers BV en bestond uit sonderingen, boringen en verschillende laboratorium tests. In 2000 is aanvullend onderzoek uitgevoerd, op of rond de locaties waar de eilanden waren gepland. Dit onderzoek is eveneens door Fugro Engineers uitgevoerd.

Uit het grondonderzoek bleek dat de bodem van de Kaspische zee als volgt is te karak-teriseren.
- Waterdiepte circa 2 – 5 meter;
- Toplagen bestaande uit jonge, mariene afzettingen, bestaande uit klei, silt en zand. De dikte van de toplagen is rond 3 à 4 meter;
  Basis, vooral bestaande uit stijve klei met een laagdikte van 40 à 50 meter.

De toplagen zijn vrijwel alle overgeconsolideerd. Dit is te verklaren door de jaarlijkse aanwezigheid van grote ijsmassa's in dit deel van de Kaspische zee. De ijsmassa's lopen aan de grond, waardoor deze wordt verstoord en een heterogene toplaag ontstaat. Door het gewicht van het ijs zijn de grondlagen overgeconsolideerd geraakt.

Ontwerp KE-F eiland
Op de zogenaamde KE F locatie is het eerste eiland gebouwd. De waterdiepte bedraagt circa 4 m. De –lokale bodemopbouw is als volgt te beschrijven:
- zand met veel schelpen, dikte 0,4 m;
- slappe klei, dikte 1 m;
- vast, siltig zand, dikte 1,7 m;
- stijve tot harde klei.



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De afmetingen van het eiland zijn circa 100 ´ 160 m². De bovenzijde van het eiland ligt op 4 m boven gemiddeld waterniveau (= ORL, lokaal referentie niveau). Het eiland wordt aan drie zijden begrensd door een 'dynamic slope', bestaande uit een steen-bestorting. De vierde zijde is uitgevoerd in stalen damwand. Deze damwand fungeert als kade.

Het geotechnisch ontwerp van het eiland was vooral gericht op de kademuur, de stabiliteit van de taluds, zettingen als gevolg van de bouw van het eiland, fundering van diverse gebouwen op het eiland en de verticale deformaties van de boorstelling.

Lime stone

Wat een lastig aspect was, is dat het eiland gemaakt is van 'lime stone' –kalk steen uit een naburige groeve. De eigenschappen van dit gesteente waren slechts bij benadering bekend. Vooral de verwachte cementatie (na verdichting) was lastig te voorspellen. Op basis van enkele proeven zijn de geotechnische eigenschappen van het materiaal vastgesteld.

Vanwege de cementatie kon worden uitgegaan van zowel een hoge wrijvingshoek als van een hoge cohesie. De hoge cohesie is echter alleen aanwezig bij geringe deformaties. Bij relatief grote deformaties zal de binding door cementatie verdwijnen. Gerekend is met een wrijvingshoek van 35° en een cohesie van 20 kPa, voor de –goed verdichte laag boven de gemiddelde waterstand.

Damwandontwerp

Belangrijke randvoorwaarden voor de kade waren sterk wisselende waterstanden door 'upsurges' en 'downsurges', als gevolg van wisselende windrichtingen en stromingen in het bekken en relatief grote kraanbelastingen op de kade (150 ton). De kraanbelasting bleek maatgevend voor de verankering te zijn,

Grote ijsbelastingen welke op de constructie werken zijn voor het ontwerp van de verankering niet als maatgevend beschouwd, aangezien deze belasting tegen de grond indrukt (welke ook bevroren is). Voor het buigend moment bleek de ijsbelasting wel maatgevend te zijn.

Voor het damwandontwerp is rekening gehouden met het effect van verdichting door de actieve en neutrale gronddrukcoëfficiënten met een factor drie te vermenigvuldigen.
De bovenzijde van de damwand bevindt zich op 3,3 m boven ORL water lengte van de damwanden is 15 meter, waarbij de voet van de damwanden circa 4,5 meter in de stijve klei staat.

Door de verdichting van de aanvulling wordt de verankering relatief zwaar belast. De uiteindelijke verankering bestaat uit 3" ankers (ø76 mm), hart op hart 2,4 m. De ankers zijn verbonden aan een 4 m hoge ankerwand op 16 m achter de voorwand.

Zettingen

Er waren relatief weinig samen-drukkings-proeven uitgevoerd, waardoor de zettingen slechts met een beperkte zekerheid waren te voorspellen. De berekende zettingen van het eiland lagen in de orde van 0,1 à 0,15 m.

De vereiste nauwkeurigheid van de voorspelling van de deformaties door het gewicht van de boorstelling ligt in een heel andere orde van grootte; verschilzettingen van maximaal enkele centimeters werden acceptabel geacht. De maximaal berekende zettingen bedragen 63 mm, met verschilzettingen van 43 mm.

Voor de predictie van deze zettingen is gebruik gemaakt van het eindige elementen pakket Plaxis. De fundering van de boorstelling bestaat uit twee parallelle balken met een lengte van circa 20 m. Vanwege de grote belastingen (tot 500 kN/m) is de spreiding in de bodem in twee richtingen gemodelleerd. Eerst is de spreiding in dwarsrichting bepaald, waarna de gereduceerde belasting is ingevoerd in het model waarbij de langsspreiding is gemodelleerd.

Voorzieningen

Vanwege de strenge milieueisen is op 1 meter boven de gemiddelde waterstand een vloeistof-dicht folie aangebracht. De folie levert voor sommige constructies

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lastige details op. Elke doorvoer in de folie verdient speciale aandacht. Dit was bijvoorbeeld de reden dat er geen paalfunderingen op het eiland zijn toegepast en dat de paalfunderingen welke noodzakelijk waren buiten het eiland zijn gerealiseerd. Dit geldt bijvoorbeeld voor de 'flare booms, waarmee overtollig gas wordt afgefakkeld. Verder zijn alle constructies op staal gefundeerd, soms op grote funderingsvoeten.

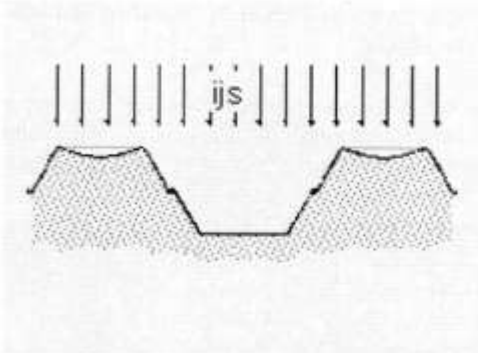
Op het eiland worden opslagloodsen in de vorm van grote tenten gebruikt. Deze tenten hebben afmetingen van bijvoorbeeld 20 ' 50 m². De funderingsbelastingen zijn aanzienlijk. Vooral een horizontale belasting, in combinatie met een opwaartse kracht leidt tot grote, zware funderingsblokken, bijvoorbeeld 1,2 ' 0,6 ' 4,5 m³ per steunpunt.

De eerder genoemde 'flare booms' zijn op een paalfundering buiten het eiland geplaatst. Voor deze fundering was niet zozeer de belasting uit de 'boom' maatgevend, maar meer de horizontale ijsbelasting.

Winter 2001-2002

Tijdens de winter van 2001-2002 waren de ijscondities tamelijk extreem. Naar aanleiding van de ervaringen met het gedrag van het eiland zijn verschillende aanpassingen en studies verricht. Op de hoeken van het eiland zijn de taluds flauwer gemaakt door extra steen te storten.

De damwanden hebben de winter goed doorstaan, echter is er een verschijnsel opgetreden waar in de Nederlandse praktijk nooit rekening mee wordt gehouden en in het Engels 'denting' wordt genoemd: Het vervormen van het damwandprofiel.



Het mechanisme is met behulp van een raamwerkprogramma nagerekend, waarbij inderdaad plastische vervormingen konden worden gemodelleerd.

Besluit

De opgedane ervaringen zijn vastgelegd in een 'design-report', waarin alle ontwerpnotities en -rapporten zijn vastgelegd, samen met de 'as built' tekeningen en een evaluatie van de uitvoering. Deze kennis zal zeker worden ingezet bij de ontwerpen van de volgende constructies in de Kaspische zee.

Environmental Edition, 2004

Environmental Friendly Roads in Bhutan: Providing access to rural communities while protecting the environment

Hendrik Visser (Civil Engineer Delft University of Technology)

Bhutan

The Kingdom of Bhutan, or the land of the thunder dragon, has an area of approximately 38,394 square kilometres. It has borders with the Tibetan autonomous region of China in the north and India in the east, west and south. Geographically Bhutan lies within the latitudes 26°45' N and 28°10'N and longitudes 88°45'E and 92°10' E.

The climatic conditions vary due to the mountainous nature of the country. The country is subject to the monsoon rain in summer, with relatively dry winters. About 73 percent of the land area is covered by forests of temperate and sub-tropical species that are a natural habitat of a diversity of flora and fauna. The country has one of the richest biodiversity in the world with about 3,281 plant species per 10,000 square kilometres and has been declared one of the ten global biodiversity 'hotspots'.

Bhutan is one of the least populated countries in South Asia. Most of the population is concentrated in the valleys, while large areas at higher altitudes in the north of the country are virtually empty except for nomadic herders. The population was estimated to be 699,000 in 2001, with more than 40 percent of the people below 15 years.

The population is probably growing at between 2.5-3.5 percent per year, putting heavy pressure on natural resources in the villages and resulting in a high rate of

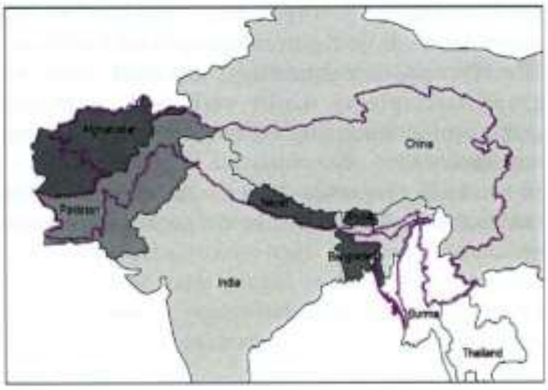


Figure 1: Countries within the Hindu-Kush Himalayan

migration from rural to urban areas. The current rate of increase is, however, expected to decline to 1.6 percent per annum in 2011 and 1.3 percent in 2016.

Approximately 80 percent of the population lives in villages in an extended family system. The average household is variously estimated to comprise between 6-8 members, with an average of 43 houses per village. There are approximately 80,000 landholdings in rural Bhutan. About 60 percent of farmers own less than 2 hectares, which is usually insufficient to feed the average size family. Only 7-8 percent of the land can be cultivated and the population pressure on land and other

figure 2: equipment use

figure 3: controlled blasting

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natural resources has become a real challenge. The livelihood of the rural population has improved significantly during the past 20 years; most villages have improved access to health, educational and agricultural extension services but some 30 percent of rural households may remain vulnerable, with consumption levels close to the poverty line.

Rural-urban migration is a serious problem. A majority of Bhutanese live in rural areas but about 20 percent of the population now lives in urban areas. Centres like Thimphu, the capital with a population of almost 50,000, and Phuentsholing, the most important border post for goods flowing to/from Bhutan, have been growing very fast recently.

Economic development through rural access

One of the keys to economic development of the country is the provision of an efficient and cost-effective transport system for freight and passengers. This is particularly important in the context of Bhutan's difficult physical environment. The country is land-locked and almost entirely located in the various mountain ranges of the Himalayas (see figure 1), which makes it highly dependent on transport for its international trade, and makes the provision of domestic road transport links difficult, especially in the north of the country.

Because of the mountainous terrain, with altitudes between 200 to 7500 meter, the area of land suitable for agriculture is very limited and the population is distributed in remote scattered settlements. The above factors make the construction and maintenance of roads and the delivery of health and education services extremely difficult and costly.

The Royal Government of Bhutan's (RGoB) series of Five Year Plans (FYP) have since 1961 therefore placed strong emphasis on improving the country's transport infrastructure, and one of the most important results is a road network that has now been extended to cover all of the main towns and valleys.

Bhutan has about 3,800 km of road network at present, but many rural communities are still cut off from the road network and depend on animal and head-load transport. In about one-third of all Geogs (Blocks), no part of the Geog is connected to feeder roads, and in another one-third only few parts are connected to them. In this situation, farmers remain dependent on subsistence agriculture, with no access to markets or to the education and health care services that have been provided, and there is a need to expand the national road network to enhance their lives.

At the same time, travel on the National Highways remains slow and costly, and existing roads may have to be upgraded or improved. Maintenance of the existing infrastructure is an equally important component of the development and management of the road system. Priorities for the next 20 years therefore include both the further development of the national trunk roads and of the network of district and feeder roads, and the expansion of access to rural areas and to vulnerable groups through power tiller and farm roads.

The Middle Path

Bhutan has realized the possible problems that could be caused by uncontrolled economic development. It has also recognized the importance of sustainable development with its fragile mountain ecosystem and extremely rich bio-diversity. Accordingly, Bhutan has chosen the "middle path" where economic development should not take place at the expense of natural resources. The Environment Assessment Act, 2002 approved by the National Assembly, ensures that environment assessments are conducted for all development activities that have potentially significant environmental impact.

Road projects are one of the largest land users in the country. Inadequate road construction techniques have a significant impact on the environment and on sustainability. Road projects can therefore only commence upon receipt of an environmental clearance from National Environment Commission. The implementation of the recommendations of the Environment Assessment are monitored at site and penalties imposed wherever lapses have been noted. The Ninth Plan therefore also states that all road construction activities will be required to adopt environmentally sound techniques and conform to the Environmental Code of Practice for Highways and Roads.

The EFRC Support Project

In the end of 1999 SNV Bhutan started its support to the Department of Roads for the implementation of a World Bank credit for the construction of 122 km of feeder roads. Since environmental friendly road construction (EFRC) techniques were still absent in the country the project focussed on the development of such techniques and on creating an enabling environment for EFRC. In 2002 SNV Bhutan, with support from the Sustainable Development Secretariat (bi-lateral funding from The Netherlands), decided to continue it's support to further develop and introduce the EFRC concept and to further strengthen the capacity of the Department of Roads and other actors involved. For this purpose the EFRC Support Project

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started in March 2003, which brings all relevant actors together. The eight components of the project cover the following areas:

1. Rural Access Project (RAP) implementation support (World Bank credit);
2. EFRC concept further development within the RAP;
3. EFRC adoption by Department of Roads;
4. Sustainable EFRC policy development;
5. Introduction EFRC concept to other Road Agencies (mainly farm and forest roads);
6. Capacity building for Districts and communities;
7. Capacity building Private Construction Sector (mainly contractors and branch organisations);
8. Dissemination of EFRC to Technical Training and Education Institutes.

The EFRC Support Project will end in December 2005. Already at this moment there is a high demand for the (continued) services of SNV in the road sector, from as well RGoB as the international donor community, especially on district capacity strengthening for infrastructure development (EFRC) and for integrated rural accessibility planning combined with rural enterprise development and marketing support.

SNV Netherlands Development Organisation

SNV is a provider of advisory services to local organisations in developing countries and also supports knowledge development and dissemination. Through these activities SNV aims to contribute to reduction of poverty, increased equity and improved governance. SNV works from a broader understanding of poverty issues and patterns since specific poverty issues are often embedded in wider patterns of imbalances in power, in the right to claim support and in access to (economic) opportunities, on the basis of gender, cultural and/or socio-economic status.
SNV provides services for diagnosis, organisational development, partnership building and institutional change. SNV provides these services mainly in the thematic areas of natural resource management, private sector development and local governance. To effectively assist its clients, SNV provides for a mix of technical-thematic and change expertise.

The support from SNV Bhutan in the EFRC Support Project is focused on strengthening the organisational and institutional capabilities of the Department of Roads and other actors involved in road construction. SNV provides a variety of technical assistance which includes expertise for surveying, design, supervision and execution of the road construction projects. The present Technical Assistance team comprises of two international road engineering experts, five national road engineers, one environmental specialist, one construction development specialist and one community maintenance specialist. There is also a variety of short term national and international experts available in the areas of quality assurance, bio-engineering, engineering geology, curriculum development, equipment management, transport economy, community organisation development and construction sector development.

The main objective of SNV's technical assistance is to ensure that in future all road construction in Bhutan can be environmental friendly. The aim of the cooperation between all the partners is ultimately to create a self-sustaining Bhutanese capacity for environmental friendly road construction.

Environmental friendly roads

For the introduction of EFRC the normal project cycle for building the roads was analysed and adapted to include environmental friendly features. First, it was important to develop a realistic implementation plan for the road project, as well as a proper monitoring and evaluation system. A main focus at the start of the project was also to develop an Environmental Code of Practice, which provides environmental guidelines for all the stages of the road project cycles.

Environmental friendly design takes into account specific site conditions such as wet areas, rocks, paddy fields and areas with high environmental and cultural value, to prevent and mitigate as much as possible environmental damage. Before the first excavator appears, these and other important site conditions are identified with a "desk study" of possible road alignments made by the design engineer in consultation with other team members. The desk study uses accurate maps based on aerial photographs and identified geologically-hazardous and culturally-vulnerable areas to plot out potential alignments between the villages to be connected.

After the desk study is completed, a feasibility study and environmental impact and social assessments are made. The proposed road alignments must be verified in the field from an economic, technical, social and environmental point of view. For this purpose the Department of Roads survey team cooperates with the Environmental Assessment team and when found necessary with a team of the Department of Mines and Geology to find the best possible alignment. The National Environmental Commission Secretariat then reviews the EIA report for issuing the environmental clearance. It is only after this process has been completed that the actual work plan for the building phase begins.

In order to make an optimal environmental friendly design, detailed survey data must be collected about

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figure 4: cross drainage

forest cover, soil classification, slope steepness, water sources, cultural heritage, and other factors. For this purpose, survey teams have been trained to collect information and make assessments. By building the roads away from geologically-hazardous and culturally-vulnerable areas, many problems can be avoided in the later stages of construction and it extends the life of the road. The design engineer needs therefore to find the optimal balance in the design between prevention of environmental damage and investment and maintenance costs over the life-time of the road.

The right balance

Environmental friendly design aims to minimize where possible cuts into mountain slopes. Especially where the slopes are fragile and prone to landslides, part of the road width will be made in fill, so retaining walls are necessary. Box cut designs are avoided in order to reduce the volume of cut and the height of cut. Based on experiences from executed projects, it is estimated that the volume of cut decreases by roughly 4 times if box cuts are avoided in the design. It is also estimated that by shifting the road centre line to the valley side by 1 meter, a reduction of cut volume by 35 % and a shift by 2 meters a reduction of cut volume by 65% can be achieved. The design engineers must find a balance between less cutting in the slopes and higher initial investments for retaining walls.

Cutting of trees is kept to a minimum and only within the road corridor. The previously used bulldozers are now replaced by excavators, which allows for the loading of tippers and the transport of excavated surplus material and debris to selected spoil disposal sites (see figure 2). Excavated materials are now also segregated in order to optimise re-use. Excavation is done in benches starting from the top batter by the smaller excavator and subsequently excavation to the final level by the medium sized excavator.

Barriers constructed out of logs or boulders at about 10-15 meter below the road are used to catch materials falling down or in some cases to allow for controlled dumping of excavated materials. Trenches, excavated by excavator, are also used for this purpose. These features are especially important to protect the valley side vegetation and to create spoil disposals near the excavation site. The latter is important since transport of excavated materials is relatively expensive.

Controlled blasting is adopted in order to minimize damages to the down hill environment and to prevent destabilization of the remaining slopes (see figure 3). Marshy areas are drained before excavation is allowed through the construction of French drains and other water management structures such as catch drains.

Permanent structures and the base course are now taken up with the formation-cutting works in one contract. This provides an incentive for the contractor to ensure that excavated useful materials like boulders are kept aside and are used in the permanent works. It also ensures that proper water drainage is available already during the first monsoon, which decreases the risk of erosion and landslides (see figure 4). A new design for retaining walls, with gabion boxes, has been introduced. These walls are permeable and are flexible enough to adapt to small slope movements.

figure 5: bio-engineering

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Bioengineering works are carried out in conjunction with the construction of support structures like breast- and retaining walls. This use of selected vegetation to stabilize and protect slopes against erosion in the monsoon has proven to be very successful (see figure 5).

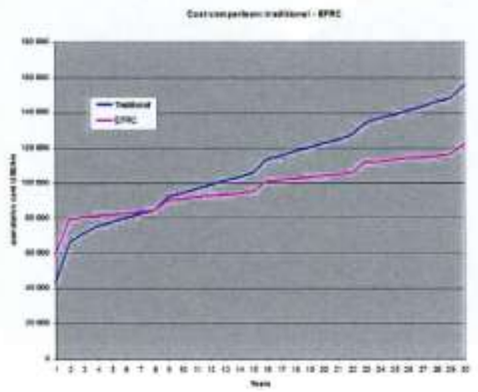
Quality Assurance

Quality Assurance has got a high priority in the Environmental Friendly Road Construction Project. In all stages of the project cycle from design to maintenance the quality will be enforced through Quality Assurance Plans in which all activities for process and product control are planned and described. The contractors are trained in the production and execution of Quality Assurance Plans to enable them to carry out the works in accordance to the technical specifications. The clients (like the Department of Roads) are trained to control the contractors and to ensure the quality of the design and of the bidding documents including the technical specifications. Besides the Standard and Quality Control Authority is supported to carry out audits and trainings in the field of quality assurance

Economic feasibility of EFRC

The initial investments in the first (two) year(s) of construction in EFRC roads are higher than for the traditional roads constructed in the past. The increase in investment goes, however, hand in hand with an improved road standard and road quality. The maintenance and monsoon restoration costs are therefore substantially lower over the total life time of the road. As shown in the figure below the EFRC roads become already economically feasible after about eight years.

The improved quality of the roads also leads to lower vehicle movement costs, which has a significant positive impact on the economic benefits of the roads. Other indirect (economic) benefits are: fewer road blockages, fewer damages to flora and fauna and fewer damages to (private) properties and cultural heritage sites.



Because on the relatively low economic rate of return (mainly caused by the low population density), investments in single roads will remain difficult to justify. RGoB with support from the World Bank has prepared a Transport Sector Note in which a more integrated approach to road investments is proposed. The development of Transport Plans at district and national level, where different road categories (and transport means) are combined in transport networks is promoted. The development of transport networks with an increased in service area, combined with additional sector interventions (like agriculture, forestry, health, and education) is essential to justify the continued investment in road construction in Bhutan.

SNV

For more information please refer to the EFRC Support Project, SNV Bhutan Technical Assistance team, Mr. Hendrik Visser, P.O. Box 815, Thimphu, Bhutan, hendrik@druknet.bt

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Bodemdaling door aardgaswinning in het Waddenzeegebied

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Inleiding

De Nederlandse Aardolie Maatschappij B.V. (NAM) is van plan aardgas te gaan winnen uit zes gasvelden waarvan er vijf geheel of gedeeltelijk onder de Waddenzee liggen en één ten dele in het Lauwersmeergebied. Deze velden zijn zo'n tien jaar geleden aangeboord vanaf de locaties Moddergat, Lauwersoog en Vierhuizen. Het streven is om in 2007 met de winning te beginnen. Voor zowel de voorbereidende werkzaamheden als voor de winning zelf zijn verschillende vergunningen nodig. Ter voorbereiding op de besluitvorming hierover is een milieueffect-rapportage uitgevoerd (NAM, 2006). Daarin is onder meer uitgebreid onderzocht wat de mogelijke effecten zijn van de bodemdaling die de gaswinning gaat veroorzaken in een deel van de Waddenzee en in het Lauwersmeergebied. In dit artikel wordt de bodemdalingmodellering die ten grondslag ligt aan de effectbeoordeling toegelicht. Ook wordt kort ingegaan op de monitoring van de bodemdaling door aardgaswinning

Bodemdaling door gaswinning

De winning van aardgas veroorzaakt in het algemeen een vermindering van de poriëndruk in de gasvoerende gesteentelaag. Daarbij wordt het gesteente langzaam iets samengedrukt onder het gewicht van de bovenliggende lagen. De mate van deze zogenaamde compactie hangt af van verschillende factoren zoals de materiaaleigenschappen van het reservoirgesteente, de grootte van de drukdaling en de dikte van het depleterende reservoir (Fig. 1). De mate waarin de compactie waarneembaar is als bodemdaling op maaiveldniveau is onder meer afhankelijk van de diepte en omvang van het depleterende gasveld en van de eigenschappen van de bovenliggende formaties. Bij een zeer groot gasveld als Groningen zal de bodemdaling boven het centrum van het veld vrijwel gelijk zijn aan de compactie op reservoir diepte. Bij kleinere velden, zoals de meeste velden in Friesland, zal de bodemdaling aan het aardoppervlak slechts een fractie van de compactie van het reservoirgesteente bedragen. De bodemdalingschotel van een veld beslaat een groter oppervlak dan het veld zelf. Indien gasvelden dicht bij elkaar liggen kunnen de afzonderlijke bodemdaling-schotels elkaar overlappen, wat zal leiden tot een grotere totale bodemdaling.

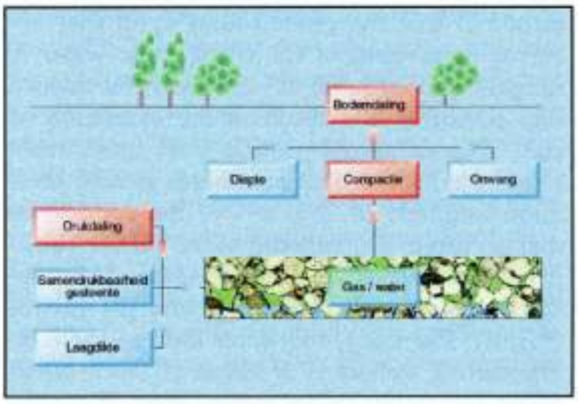

Fig. 1. Drukdaling in het reservoir resulteert in bodemdaling. Factoren van invloed op proces

Bodemdalingmodel

Voor bodemdalingprognoses worden binnen de NAM in het algemeen semi-analytische modelberekeningen uitgevoerd, gebaseerd op het werk van Geertsma en Opstal (Geertsma, 1972; van Opstal, 1973). Daarnaast wordt ook een Eindige Elementen programma (Geomec) gebruikt voor velden zoals Ameland, waarvan het reservoir zeer heterogeen en de geologische structuur gecompliceerd is. Geomec is een door Shell en TNO ontwikkeld programma op basis van het eindige elementen-pakket DIANA (de Witte & Kikstra, 2005).

In 2000 zijn zowel met het semi-analytische als met het Eindige Elementen programma bodemdaling-berekeningen uitgevoerd voor het Groningenveld en zijn de resultaten vervolgens met elkaar vergeleken. Beide modellen bleken vergelijkbare resultaten te geven en de gemeten bodemdaling goed te kunnen volgen. In het semi-analytische model kan een groot aantal velden tegelijk in één berekening worden meegenomen, wat met het Eindige Elementen programma vanwege computertechnische beperkingen nog niet mogelijk is. Daarom zijn de bodemdaling-berekeningen voor de meeste velden in Noord-Nederland uitgevoerd met behulp van het semi-analytische programma. Hierbij worden alle bovenliggende lagen als één uniforme laag beschouwd, die elastisch deformeert. De Poisson ratio (in dit geval met waarde 0,25) is daarbij de enige gesteentemechanische parameter die van invloed is op de mate waarin compactie in bodemdaling wordt vertaald. Het Rigid Basement is een, op geologische gronden veronderstelde, niet deformerende onderlaag, welke in het geval van velden in Noord-Nederland zoals het Groningengasveld, maar ook de betreffende Waddenzeevelden, op 5 km diepte wordt aangenomen. Callibratie van het model met de waterpasmetingen in de tijd tonen aan dat dit de optimale waarde voor de diepte van het Rigid Basement is.

Compactiemodel

Om een prognose van de te verwachten bodemdaling te maken moet een model voor het compactiegedrag aangenomen worden. Bepalende factoren voor het compactiegedrag zijn o.a. cementatie (mate waarin zandkorrels aan elkaar gekit zijn), porositeit en de microstructuur van het gesteente. Tot nu toe is er geen model beschikbaar dat een direct kwantitatief verband geeft tussen deze parameters en het compactiegedrag. Derhalve wordt gebruik gemaakt van een empirisch model, dat is afgeleid uit macroscopische waarnemingen, zoals veldmetingen en compactie experimenten in het laboratorium.

Tot voor kort werd voor de modellering van de compactie van de zandsteen van de Rotliegendes formatie, waaruit het gas in Noord-Nederland voornamelijk wordt geproduceerd, het compactiegedrag beschreven door een model waarbij de compactie zuiver lineair met de drukdaling verloopt. Uitgebreide inversiestudies, waarbij gebruik is gemaakt van de resultaten van bodemdalingmetingen over de gasvelden in Noord-Nederland, tonen echter aan dat het compactiegedrag nog beter kan worden beschreven wanneer wordt aangenomen dat er enige tijd overheen gaat voordat de compactiecoëfficiënt een constante eindwaarde bereikt. In het bodemdalingmodel, waar het compactiemodel onderdeel van is, wordt de compactie daarom als volgt beschreven. Initieel verloopt de compactie lineair met drukdaling. Vanaf een bepaalde reservoirdruk, het transitiepunt (Tp), wordt een hogere, maar weer constante, compactie-coëfficiënt aangenomen. Zowel voor als na het transitiepunt wordt het compactiegedrag beschreven door:

$$\Delta H = C_m * H_0 * \Delta P$$

ΔH: Compactie (m)
C_m: Uniaxiale compactiecoëfficiënt (bar^{-1})
H_0: Initiële reservoirdikte (m)
ΔP: Drukdaling (bar)

Voor het beschrijven van het compactiegedrag van het reservoir moeten de volgende waarden worden bepaald:

C_{mpre}: Initiële compactiecoëfficiënt (bar^{-1})
C_{mpost}: Uiteindelijke compactiecoëfficiënt (bar^{-1})
Tp: Transitiepunt (bar)

Gebaseerd op geologische modellen en reservoirsimulatie zijn bodemdalingprognoses opgesteld voor de velden Nes, Moddergat, Lauwersoog-C, -West en -Oost en Vierhuizen-Oost.

Zoals uit het bovenstaande blijkt is de compressibiliteit (C_m) van het reservoirgesteente een belangrijke parameter in de modellen voor bodemdaling. Deze parameter beschrijft de compactie van het reservoir als gevolg van de spannings-veranderingen die optreden door drukdaling in het poriën-systeem ten gevolge van de winning van het gas.

C_m kan op verschillende manieren worden bepaald:

Op reservoirgesteente uit velden waarvan het compactiegedrag nog niet goed bekend is, kunnen laboratorium-experimenten worden uitgevoerd. In het Shell-laboratorium in Rijswijk en bij Sintef Petroleum Research in Trondheim wordt in opdracht van de NAM al vele jaren experimenteel onderzoek verricht naar het compactie-gedrag van reservoir-gesteente. Ook worden compactieproeven op kunstmatig gemaakt zandsteen uitgevoerd. Het hierboven beschreven bilineaire compactiegedrag volgt niet alleen uit metingen in het veld, maar is ook waargenomen in compactieexperimenten in het laboratorium op artificiële "Groningen" zandsteen. Compactieparameters die enkel en alleen gebaseerd zijn op laboratoriumexperimenten op kernmonsters vertonen vooralsnog een aanzienlijke onzekerheid en verder wetenschappelijk onderzoek is gaande om de een

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methode te ontwikkelen om deze onzekerheid te verkleinen (door bijv. de gemeten waarden te corrigeren voor de effecten van kernschade).

Boven velden waar al bodemdaling is gemeten, kunnen de parameters van het compactiemodel nauwkeuriger worden bepaald met behulp van inversie van de waterpasmetingen. Hierdoor kan de onzekerheid in de compactie coëfficiënt worden verkleind. Dergelijke inversies zijn voor een groot aantal gasvelden in Noord Nederland uitgevoerd. Zoals verwacht vallen de door inversie bepaalde compactiecoëfficiënten van het gesteente binnen de range van waarden die door middel van experimenten op zandsteenkernen uit de Rotliegendes Formatie in laboratoria zijn bepaald.

In-situ compactiemetingen in Groningen die in de diepe compactiemonitoringsputten in de Rotliegendes formatie zijn uitgevoerd, geven aan dat sinds 1983, toen metingen met Schlumberger's FSMT compactie tool zijn gestart, de compactie lineair verloopt met de drukdaling in het reservoir. De compactiecoëfficiënten die uit deze in-situ metingen zijn afgeleid, komen overeen met de waarden die met behulp van inversie zijn bepaald. Verder geeft de inversie aan dat het transitiepunt (Tp) voor het Groningen gasreservoir al voor 1983 is opgetreden. De overeenkomst tussen het compactiegedrag bepaald met behulp van in-situ metingen en door middel van inversie voor het Groningenreservoir geeft vertrouwen in de door inversie bepaalde compactiecoëfficiënten. (Mobach & Gussinklo, 1994; NAM, 2005)

Vanwege geologische analogie is voor de Waddenzeevelden Lauwersoog en Vierhuizen hetzelfde compactiemodel gebruikt als voor het Munnekezijlveld. Dit veld is al sinds 1994 in productie en dientengevolge konden de compactieparameters door middel van inversie worden bepaald. Het compactiemodel is in dit geval vrijwel gelijk aan het voor het Groningenveld gebruikte model.

De geologische situering van de Waddenzeevelden Nes en Moddergat, gelegen aan de westzijde van de Hantumbreuk, is analoog aan die van het gasveld Anjum. Voor deze Wad-

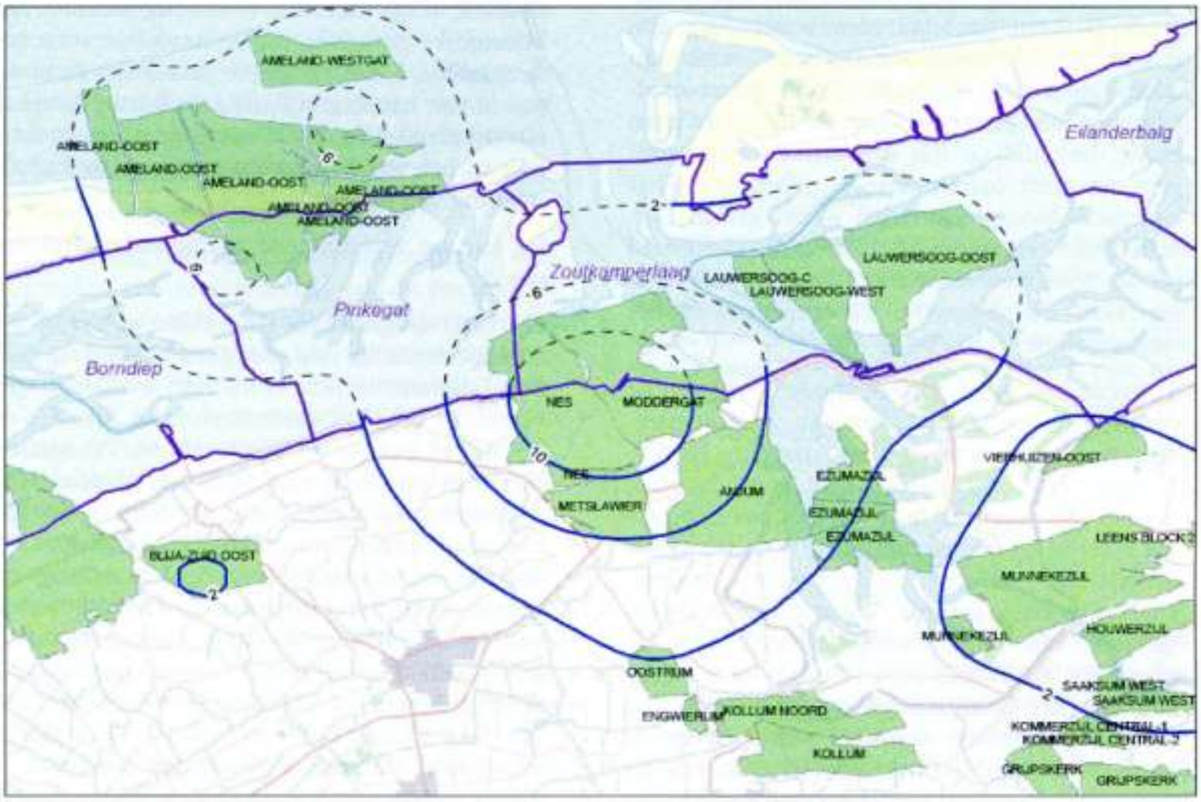

Fig. 2. Verwachte cumulatieve bodemdaling (in cm) vanaf 2007 (start productie vanaf de locatie Moddergat) tot 2040 ten gevolge van de gasproductie uit de nieuwe gasvelden in combinatie met naburige reeds producerende velden.

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denzeevelden is daarom het compactiemodel van Anjum gebruikt, dat eveneens door middel van inversie is gekalibreerd aan de resultaten van waterpassingen die al voor de start van de productie in 1997 en vervolgens regelmatig over dit gebied zijn uitgevoerd.

Bodemdalingprognose Waddenzee

Modellering van de bodemdaling ten gevolge van de gaswinning uit de velden Nes, Moddergat, Lauwersoog-C, -West, -Oost en Vierhuizen-Oost is opgezet met bovenbeschreven methode. Hiermee zijn de bodemdalingcontouren in de tijd berekend, gebaseerd op het voorgenomen gasproductiescenario. Fig. 2 toont de totale bodemdaling (inclusief reeds ontwikkelde omliggende velden) die te verwachten is tussen start van de productie van de nieuwe velden in 2007 en de beëindiging van de winning uit de betreffende velden in 2040.

In tegenstelling tot de op land geldende criteria voor bodemdaling (mogelijke effecten op de waterhuishouding), is voor de Waddenzee niet zozeer de uiteindelijk bodemdaling van belang, maar is de bodemdaling-snelheid de belangrijkste parameter. Wanneer de bodemdalingsnelheid beneden een bepaalde zogenaamde natuurgrens blijft, zullen de effecten van de bodemdaling in de Waddenzee namelijk worden gecompenseerd door natuurlijke sedimentatie. Wanneer rekening wordt gehouden met autonome bodemdaling en zeespiegel-stijging blijft er een gebruiksruimte over voor bodemdaling door gaswinning. Zaak is dat het productieprofiel van de nieuwe velden zodanig wordt gekozen dat de resulterende bodemdalingsnelheid, zoals geprognosticeerd met het hierboven beschreven bodemdalingmodel, deze gebruikruimte niet zal overschrijden. Fig. 3 toont de bodemdalingsnelheid in het komberginggebied Zoutkamperlaag veroorzaakt door de voorgenomen productie van de reeds producerende velden en de nieuwe velden. Zowel voor de te toekomstige zeespiegelstijging als voor de natuurgrens worden verschillende scenario's gehanteerd. In de betreffende figuur zijn twee scenario's voor de natuurgrens aangegeven. Voor het bepalen van het uiteindelijke productiescenario is met de meest conservatieve natuurgrens (Scenario 2) rekening gehouden. Ook voor de zeespiegelstijging is rekening gehouden met het meest conservatieve scenario, te weten 85 cm/eeuw (vanaf 2010).

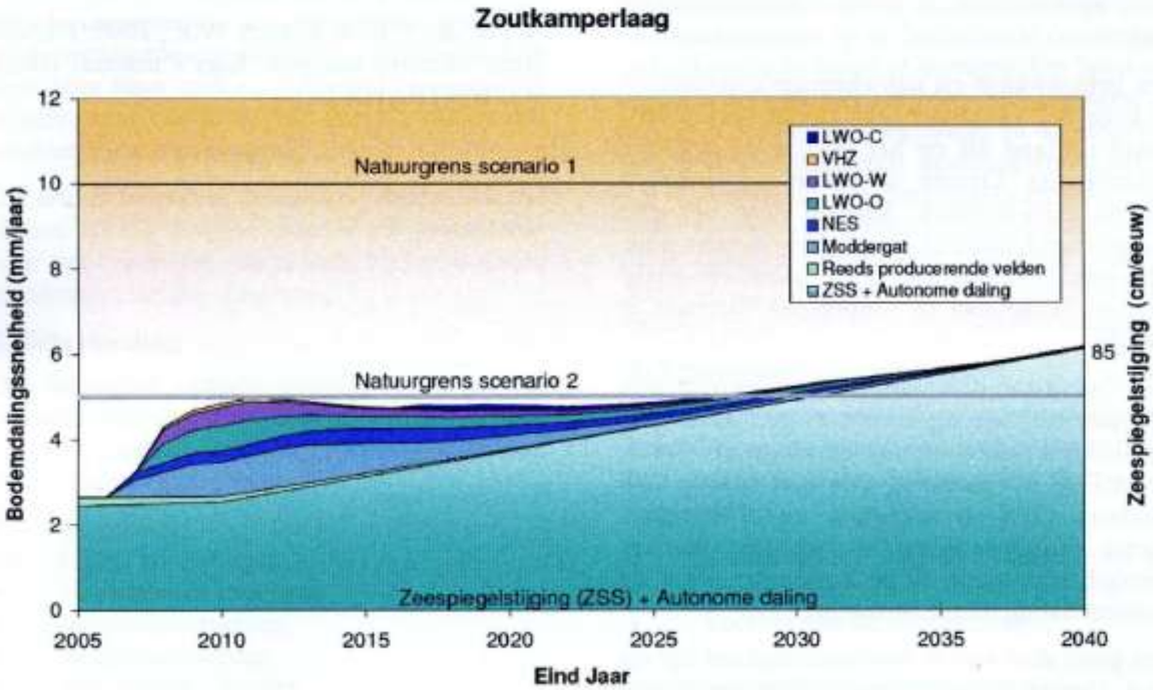

Fig. 3. Zesjaarlijks voortschrijdend gemiddelde van de bodemdalingsnelheid in het kombergingsgebied Zoutkamperlaag (mm/jaar) bij toepassing van het meest conservatieve zeespiegelstijgingscenario (85 cm/eeuw).

December 2006

Om te borgen dat de bodemdalingsnelheid veroorzaakt door gaswinning de natuurgrenzen niet zal overschrijden, zal het zogenaamde "hand-aan-de-kraan" principe worden toegepast. Modelberekeningen hebben aangetoond, dat gasproductie volgens het voorgenomen (aangepaste) productieprofiel zal leiden tot een verloop van de bodemdalingsnelheid dat past binnen de gebruiks-ruimte voor gaswinning zoals aangegeven door het bevoegde gezag. Omdat er aan de model-resultaten, zoals bij elk model, onzekerheden verbonden zijn, wordt voorzien in regelmatige metingen van de bodemdalingsnelheid, zodat bij dreigende overschrijding op basis van een geactualiseerde bodemdalinganalyse tijdig kan worden ingegrepen door middel van tempering van de productie.

Meetmethode bodemdaling

Om de door gaswinning veroorzaakte bodemdaling te kunnen bepalen zullen de hoogteverschillen tussen de in het gebied aanwezige meetpunten van het NAP netwerk regelmatig worden gemeten. In gebieden waar de puntdichtheid van het historische NAP-netwerk onvoldoende is, zullen extra meetpunten worden bijgeplaatst. Dit laatste is met name het geval op het Wad en in het Lauwersmeergebied.

Een betrouwbare en nauwkeurige meetmethode voor het vaststellen van hoogteverschillen, zowel op land als op het wad is de optische waterpassing. Gezien de snelle ontwikkeling van de GPS-techniek, kunnen deze optische waterpassing wellicht worden vervangen door GPS-hoogtemeting indien de nauwkeurigheid daarvan acceptabel is. Onderzoek hiernaar is gaande.

Het "hand-aan-de-kraanprincipe" vereist een adequate respons op eventueel geconstateerde afwijkingen tussen meting en prognose. Daarom zal de bodemdaling ook met behulp van continue GPS- trendmeting en/of hoogfrequente waterpassingen nauwkeurig worden gevolgd, met name in de komcentra waar de grootste daling wordt verwacht.

Een groot deel van de bodemdalingkom ligt op land. Omdat het verstorende effect van waterpasmetingen op land zeer gering is en de metingen tot 20 keer sneller uitgevoerd kunnen worden dan op het wad, zal bij geconstateerde trendafwijkingen de nadruk aanvankelijk liggen op het volgen van het bodemdalinggedrag over een aantal kritische landtracés. Afhankelijk van de resultaten van deze metingen kunnen verdere verichting van het landnetwerk en extra metingen op het wad noodzakelijk blijken.

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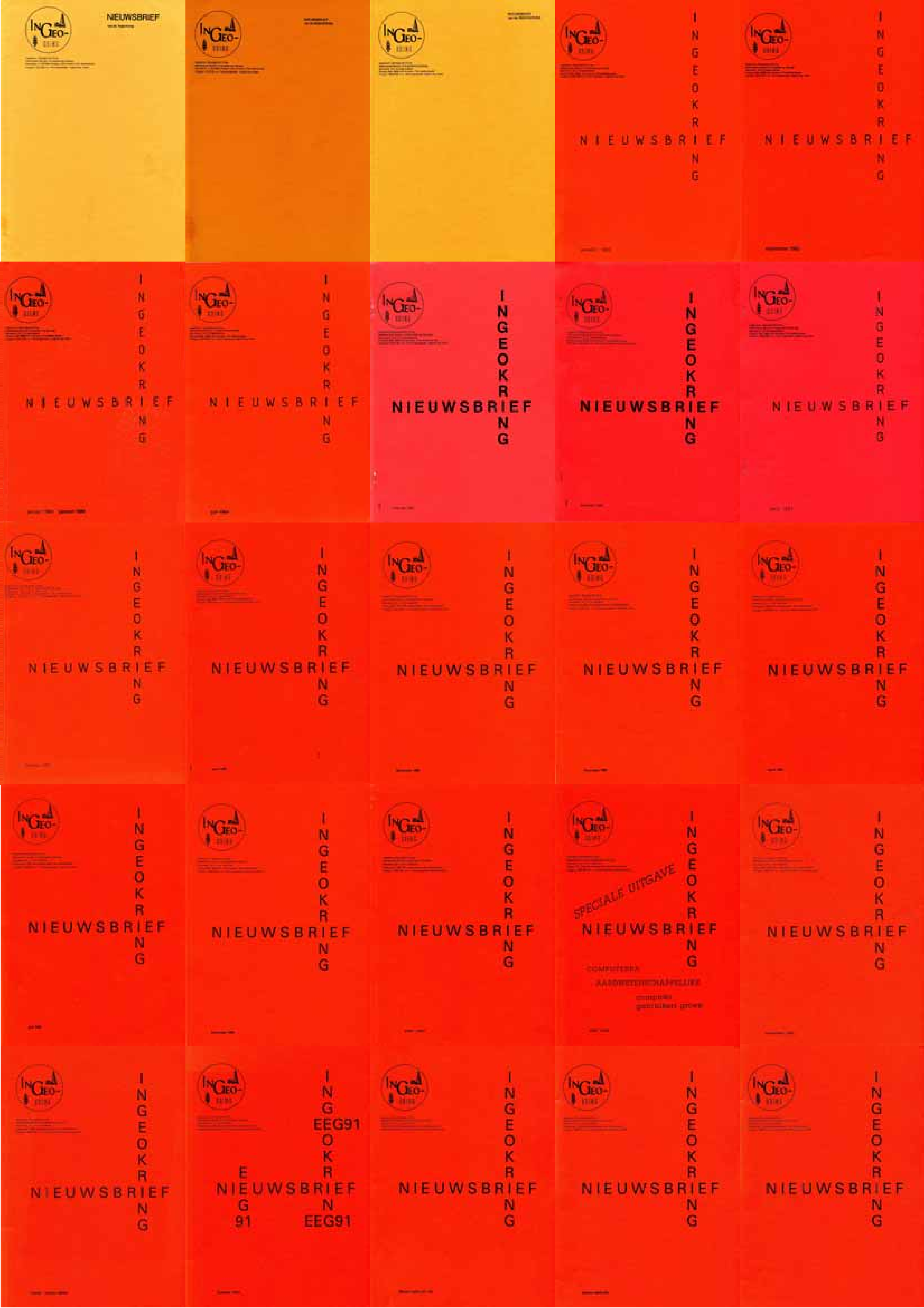
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