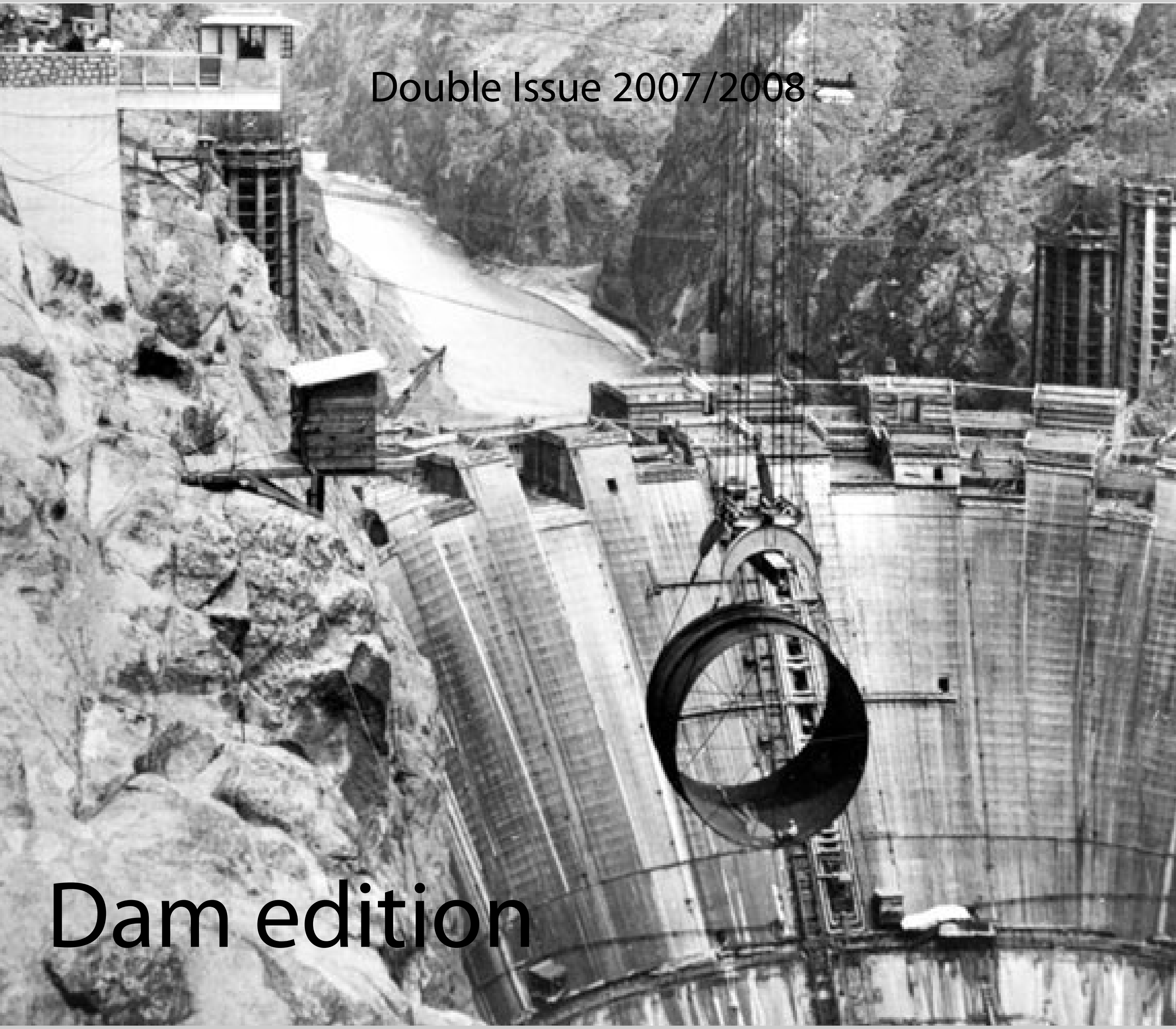
News letter

nl-iaeg ingeo kring

Double Issue 2007/2008



Dam edition

Guest-editor Michiel Maurenbrecher: Analysing the analysis of the Malpasset arch dam failure of 1959 - Estimating 'efficiency-permeability' of upstream cut-off membranes for dams

Water at the desert fringe: The Marib dams and irrigation schemes in the Republic of Yemen - Dam and canal design on soluble rock, Ethiopia - Dams in the Western USA - Verification of two-dimensional numerical earthquake site effects on a dam site, Costa Rica - Moraine dams and glacier lake outburst floods (GLOF), Nepal - Thesis abstracts - 10th IAEG Congress, Nottingham - The activities of the Netherlands' National Committee on Large Dams (NETHCOLD) - GeoGolf - Book review - Piping phenomenon in earth dams, Cuba - Professor's Column: Ode to Ground - Engineering geologist abroad - Excursion to Hubertustunnel - In the spotlights: De Ondergrondse - Excursion to GeoCentrifuge - 11th ISRM Congress, Lisbon

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 € 16,-, student membership fee is € 8,-.

Cover photo:
Hoover Dam under construction (1931-1936)
(source: www.eng.auburn.edu/users/zechwes/pictures.html).

Guidelines for authors of articles for the Newsletter and information about advertising in the Newsletter can be found at the inside of the back cover.

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From the Chairman

Joost van der Schrier, Chairman Ingeokring

Dear Ingeokring member,

It is my pleasure to introduce to you the 2007/2008 double issue of the Newsletter.

Receiving a Newsletter is a little bit like the changing of the seasons: you can feel it in the air, you know it comes, and suddenly it is there. Laying on your doormat or in your mailbox and challenging you to pick it up and read it. And I promise: you will not regret it.

The editorial staff and authors succeeded in filling this issue with a selection of articles relating to a ‘hot, alive and astonishing’ topic entitled ‘dams’. The design, construction and operation of dams touch the core of engineering geology. All dams, either huge or small, are founded on rock and/or soil, and almost always under challenging conditions considered from either a technical, geographical or geological point of view.

Evidently, the integral knowledge of the properties of the geo-materials involved, being rock and/or soil, their (in-situ) mass behaviour and their genesis with all its facets, are required as well as is the ‘affiliation with the (structural) behaviour’ of the dam. All of this is anchored within engineering geology. But, the field of expertise which is engineering geology covers more than only this: it is a typical multidisciplinary science.

In view of the above, I am very happy to note that the Ingeokring, the Dutch section of the IAEG (International Association for Engineering Geology and the environment), is a ‘vibrant and healthy’ society that includes members of different backgrounds and is active in different fields of expertise. As such, the Ingeokring proofs to be a powerful platform for interacting and sharining of expertise and knowledge in the fields of engineering geology and its overlapping or adjacent disciplines. So, if you are not a member yet, we kindly invite you to consider the membership and join us.

I would like to thank all the Newsletter authors, sponsors and editorial staff for making this issue of the Newsletter possible and I trust you will enjoy reading it as much as I did.

Hope to meet you soon at one of the coming Ingeokring activities.

Editorial

Michiel Maurenbrecher, guest editor

"Holland....lies so low its people are only saved by being damned!"

Up the Rhine , Thomas Hood (1799-1845)

This must be the first issue of the Ingeokring Newsletter that deals with dams. Well, almost: I wrote an article on a presentation given by Charles Dufour on invitation of the Mijnbouwkundige Vereeniging on irrigation schemes in Yemen mentioning the old and new Marib Dams (*Water resources assessment in the Republic of Yemen,* F.C. Dufour (reporter P.M. Maurenbrecher), Ingeokring Nieuwsbrief, lente-zomer editie, 1991, pp. 41-43). For this Newsletter issue, Charles has written a more specific paper on the old and new dams. Perusing further through my complete collection of Newsletters dating from 1977, there seem to be two more articles on dams. One is about the Ribarroja Dam, written by ITC student Carlos Caranza Torres (*Ribarroja Dam excursion,* C.M. Caranza Torres, Ingeokring Nieuwsbrief, herfst editie, 1993, pp. 19-22). The problems associated with this dam, situated close to the ITC and TU Delft engineering geology students' common fieldwork area in Spain, can also be read in Peter Verhoef's dam experience and case history in the Afar District, Ethiopia which makes riveting reading: damnation lurks for those dam engineers lacking knowledge of geological materials. The other article is about the Canelles Dam and can be found in the summer 2001 issue of this Newsletter (*Excursion to the Canalles Dam,* R.M. Schmitz & S. Gyaltsen, Ingeokring Newsletter No. 9, summer 2001, pp. 13-16).

Because there are insufficient steep valleys to flood in the Netherlands, it seemed for a time that engineers and engineering geologists are not much involved with dams except the type Thomas Hood refers to. The theme 'dams' came up when Dr. Mario Alvarez Grima sent a paper on piping in dams to the Ingeokring Newsletter. That was the one theme not mentioned by the editors in their 'invitation' to me at my TU Delft farewell symposium of June 2006 to be guest editor of one of the Newsletter issues. Increasingly, dams seem to be damned, especially by environmentalists. Our new Ingeokring Newsletter editor Erik Schoute, whilst working for Boskalis in the USA, went on two road trips in the Western USA and writes about the dams he visited. Two dams, Hoover Dam and Glen Canyon Dam, were also visited on the DIG Colorado study tour with Prof. Keith Turner in 2000. I remember the engineer from the USBR complaining about the increasingly stronger lobby from the Sierra Club wanting to demolish these dams so as to give the Colorado river its old confluence back. Dams are damned structures but possibly the necessary evil to save humanity from thirst, hunger, decease and wasteful use of fossil fuels. The Sierra Club is not alone: look at the advertisement by the WWF depicting the Hoover Dam as 'Hoover Damned' gravestone in their recent advertisements. Despite this negative publicity, dams are 'Megastructures' as Discovery Channel viewers know by now. There are surprisingly plenty of Dutch engineers and engineering geologists who have been and are involved with dams, of which one is featured on Discovery Channel: 'The Enclosure Dam' in the Netherlands, still the longest in the world. Another equally grand dam crosses the Suriname River: the Afobaka Dam, a 54 m high and 2 km long rock fill dam with a concrete gravity spillway-sluice gate and power station complex. The dam was designed by W.J. van Blommestein, after which the huge reservoir behind the dam is named (size of Province of Utrecht). Trying to obtain technical information on its design through the TU Delft library yielded nothing, while Google searches painted a picture of the Afobaka dam as another 'Thomas Hood damned' dam: the reservoir being choked with water hyacinth, piranha fish and the power generated by the dam solely provided for the benefit of the aluminium industry (the main money earner) of Suriname.

This Newsletter has become fairly sizeable, its size could have been bigger as I discovered more and more Dutch engineers involved in this industry. There has been a visit of the new student chapter of Geo-Engineering to Madrid. During this visit, Gerard Arends (Ondergronds Bouwen) owned up he had been involved in a dam project during his internship. Very recently the section head of Geo-Engineering, Frits van Tol, also mentioned he did his student internship on a concrete arch dam (170 m) north of Madrid. Enquiring about Van Blommestein through Emeritus Prof. Arnold Verruijt led me to another emeritus professor, Jan Kop. He has actually worked under Van Blommestein in Bangladesh and had also visited the Afobaka Dam in Suriname (assuring me that one can see the design features on posters/drawings hanging on the wall, presumably at the power station).

Yet what I found a disappointment at TU Delft was the lack of literature on dams. Not only had the search on Afobaka Dam led to nothing, years ago I had a similar search for a paper on Scotland's Monar Dam, an arch dam on jointed rock foundation. This paper (*Stability of the Foundations of Monar Dam,* D.J. Henkel, J.L. Knill, D.G. Lloyd & A.W. Skempton, pp. 425-441, ICOLD Congress Proceedings, 1963) was published as part of the series of ICOLD congress proceedings. I found it hard to accept that TU Delft library did not have the ICOLD proceedings. Well, they do! It seems since the Netherlands became a member of ICOLD in 1966, TU Delft only has the proceedings in the library from that year onwards. For my personal introduction to ICOLD, you have to read my short paper in this Newsletter on efficiency of upstream membrane dams, a study I did when working in London in 1973. Eventually, a copy of the Monar Dam article came from Haskoning in Nijmegen, from ir. J. van Duivendijk, who is closely associated with both TU Delft and Haskoning and writes about the role of the Netherlands in ICOLD (NETHCOLD). Monar Dam is interesting as it discusses foundations of a dam in connection with a jointed rock mass. The paper contains a stereographic plot of the poles of the discontinuities, but further analysis on the stability appears to be based on two-dimensional models. Searching for literature on the use of stereographic methods in foundations led me to the Malpasset Dam failure analyses. Back in the 1960's the stability was analysed using stereographic plots preceded by analyses using 3D Cartesian coordinates. At the discovery of what I now call the 'Londe papers' I challenged students to tell me how the analysis was done, with little result. I will not state how long it took to decipher the papers. Some students (old students) will know. This Newsletter's theme was, I admit, especially in my newly retirement status an inducement (in Dutch: 'een stok achter de deur') to present the analysis from Londe who claims, quite rightly, Malpasset Dam was the start of modern rock mechanics.

At the 11th ISRM Congress in Lisbon, a paper on earthquakes and dams, which is reproduced in this issue, was presented by Carolina Sigarán-Loria and Robert Hack. Robrecht Schmitz reports briefly on the conference as does Leon van Paassen on a conference a year earlier, of the IAEG in Nottingham. One paper that will be published in the next Newsletter issue is about dams in Bhutan, to be written by Niek Rengers and Wim Verwaal. There is though a very interesting case history on a frozen moraine dam in the Himalayas by Senta Modder, who studied geology at the VU University Amsterdam and engineering geology at TU Delft. The Bhutan case histories would compliment Senta's paper. Niek visited a natural dam caused by a landslide and Willem did investigations for man made dams.

Once into dams one is hooked into these structures. If you travel up the Maas instead of the Rhine and then follow a tributary called la Gileppe to the watershed between the Maas and Rhine catchments in the Ardennes you come to the highest dam in the world, named after the tributary. Highest dam? In Belgium? Well, it was so in the 19th century when it was built (40 m) and heightened to 66 m more recently. Today dams almost ten times higher are on the planning boards, and dams of over 300 m (Nurek, Russia) already exist. 'Werk aan de winkel' for engineering geologists.

My thanks to the editorial team, Gerhard Wibbens, Wiebke Tegtmeier, Erik Schoute, Paulien Kouwenberg and Jacco Haasnoot, for showing all that patience and my apologies to the members of Ingeokring for the delayed publication. The meetings were great and I got to know a few restaurants in Delft and Rotterdam. The layout and content increases in quality with each issue. A suggestion for the next issue is to celebrate 30 years of Newsletters!

Ingeokring Jaarverslag 2006

Marcel Remijn, secretaris Ingeokring

Missie van de Ingeokring

De Ingeokring is onderdeel van het KNGMG en bestaat uit leden die zich professioneel en vanuit interesse bezig houden met het vakgebied Ingenieursgeologie. De Ingeokring fungeert tevens als Nederlandse afdeling van de International Association for Engineering Geology and the environment (IAEG).

De Ingeokring heeft een bestuur wat naast de gewone bestuursleden bestaat uit een bestuurslid van het Dispuut Ingenieurs Geologie (DIG) van de Technische Universiteit Delft en een Nederlands lid van de International Society for Rock Mechanics (ISRM). Het bestuur organiseert lezingen, excursies en vergaderingen ten einde elkaar te informeren over vaktechnische en relevante nationale en internationale ontwikkelingen binnen het vakgebied van de Ingenieursgeologie. Tevens zet de Ingeokring zich in om ontwikkelingen van Ingenieursgeologie in Nederland te stimuleren op het gebied van onderwijs, onderzoek en samenwerking tussen verschillende instituten, universiteiten en bedrijven.

Tot de Ingeokring kunnen toetreden:

- Leden en buitengewoon leden van het KNGMG en alle leden van de met het KNGMG samenwerkende verenigingen
- Personen met interesse in Ingenieursgeologie

Bestuur en leden

Op 1 januari 2006 had de Ingeokring 176 leden en op 31 december 2006 171 leden. De aanmeldingen en opzeggingen over 2006 zijn in evenwicht, de daling komt vooral door het opschonen van het ledenbestand.

De samenstelling van het bestuur bij aanvang van 2006 was:

Chris Bremmer	voorzitter
Marcel Remijn	secretaris
Leon van Paassen	penningmeester
Peter Verhoef	bestuurslid
Marco Huisman	vertegenwoordiger ISRM
Jacco Haasnoot	voorzitter Newsletter redactie
Roeland van Hof	vertegenwoordiger DIG

In oktober 2006 heeft Elles Bader de taken van Roeland van Hof overgenomen als vertegenwoordiger van het DIG. Met ingang van het studiejaar 2006-2007 is het DIG opgegaan in studievereniging 'De Ondergrondse' waarin studenten van diverse gelieerde richtingen verenigd zijn. Een vertegenwoordiger van 'De Ondergrondse' zal deel uit blijven maken van het Ingeokring bestuur.

Jaarvergadering

De jaarvergadering van de Ingeokring werd gehouden op 26 april 2006 bij de TU Delft. Tijdens de jaarvergadering werden de secretariële stukken door de leden goedgekeurd. De financiële stukken werden op 22 augustus aan de leden gepresenteerd en goedgekeurd. Tevens is de 'Afstudeerprijs 2004/2005' aan Bart van Knapen uitgereikt voor zijn scriptie getiteld: *'A method to automate the identification and characterisation of rock mass discontinuity sets using 3D terrestrial laser scanning data'*.

Activiteiten

De Ingeokring heeft in 2006 verscheidene excursies en lezingen georganiseerd waarbij de opkomst gevarieerd was:

26-04-2006:	Jaarvergadering 2005 en uitreiking 'Afstudeerprijs 2004/2005'
16-06-2006:	Symposium ter ere van het afscheid van Michiel Maurenbrecher
22-08-2006:	Presentatie financieel jaarverslag voorafgegaan door een lezing van L. van Paassen en P. Verhoef over 'Cementation'
15-09-2006:	Bezoek aan MTI met lezing en rondleiding over de werf van IHC Kinderdijk
19-10-2006:	Lezing over 'Remote Sensing' bij TNO Utrecht
14-12-2006:	Kerstborrel gecombineerd met een lezing door Salle Kroonenberg in 'Het Noorden'

Newsletter

In 2006 kwam de Newsletter eenmaal uit. Het thema was 'Bodemdaling'. De Newsletter verscheen in december 2006, maar werd pas in maart 2007 verstuurd door problemen bij de drukker.

Ingenieursgeologie opleidingen

De master opleiding Ingenieursgeologie aan de TU Delft is samengevoegd met een aantal andere master opleidingen en maakt nu deel uit van de sectie Geo-Engineering. Mede door deze samenwerking is ook het DIG gaan samenwerken met andere disputen en is opgegaan in de studievereniging 'De Ondergrondse'. De samenwerking met ITC is beëindigd. Twee nieuwe universitair docenten zijn aangetrokken, Joep Storms en Bob Hoogendoorn, beide gepromoveerd bij Applied Earth Sciences.

Bij ITC is eenzelfde tendens ingezet: de opleiding Engineering Geology is hernoemd in Geo-Engineering en naast Geohazards, Earth Resources Exploration en Earth Science Data Provision , een van de 4 specialisaties binnen de master Applied Earth Sciences (AES). De master AES heeft 30 studenten, waarvan 7 de specialisatie Geo-Engineering volgen. Daarnaast werkt ITC aan samenwerkingsverbanden met Universiteit Twente.

Website Ingeokring

De internetsite www.ingeokring.nl is in 2006 regelmatig voorzien van aankondigingen van de Ingeokring en internationale activiteiten. Tevens zijn de laatste Newsletters er digitaal te downloaden. De website wordt op dit moment beheerd door Leon van Paassen.

Afsluiting

De Ingeokring hoopt dat U door dit jaarverslag een goed beeld heeft gekregen van de door de Ingeokring georganiseerde activiteiten in 2006.

Voor vragen of opmerkingen kunt U contact opnemen met de secretaris van de Ingeokring.

Analysing the analysis of the Malpasset arch dam failure of 1959

Michiel Maurenbrecher, Delft University of Technology, Section Geo-Engineering

Introduction

At dusk on December 2 1959, engineers decided after heavy rains and an impending overflow of the reservoir to open the bottom valve outlet of the Malpasset concrete arch dam. Sixty metres of water pressure were then unleashed to try and lower the water levels in the reservoir. Three hours later the dam failed when the foundation of gneissic-schistose rock beneath the left abutment slid along a wedge. The reservoir water rushed towards the sea as almost sixty metres of flood water gushed through the completely obliterated left abutment portion of the dam. For eleven kilometres the water tore up farms, river banks and infrastructure as it headed towards the estuary of the Reyran River at Fréjus into the Mediterranean. 423 people died as a result of this disaster as well as flooding and destroying property and infrastructure along the path of the unleashed torrent.

The design engineer, André Coyne of the well-established engineering partnership of *Coyne et Bellier* based in Paris, was nearing the end of his professional career and had over one hundred successful dams constructed. Despite the calamity and the first arch dam ever to fail, his immediate reaction was to try and establish the cause of the failure (Bellier, 1977).

By 1969, ten years later, publications on the method of analysis Coyne and Bellier had established started to appear in the international journals, starting with two papers in the *Journal of Soil Mechanics and Foundation Engineering* of the ASCE (American Society of Civil Engineers). These were by Pierre Londe, Gaston Vigier and Raymond Vormeringer (1969 and 1970). Subsequently, Pierre Londe published a number of papers in the *Quarterly Journal of Engineering Geology* (1973) and in *Engineering Geology* (1987). Both these publications were milestones: The QJEG providing an introduction to rock mechanics as is still practiced today, notably the paper in that issue on slope wedge analysis by Hoek, Bray and Boyd. The Elsevier *Engineering Geology* publication was based on a meeting held to discuss dam failures.

The milestone QJEG publication re-appeared as part of Hoek and Bray's *Rock Slope Engineering* almost the same year and re-appeared recently in a new 4th edition version by Wyllie & Mah (2004). The work by Londe, however, seems to have been put on the sidelines despite his international reputation, especially in the International Society of Rock Mechanics. The Malpasset dam failure could be recognised as the starting point of the discipline of rock mechanics in Civil Engineering (Londe, 1973). Londe analysed the dam failure in three dimensions using simple mechanics on a three-dimensional wedge that was believed to exist beneath the left abutment (note: convention in dams is to always look downstream in describing which half of the dam one is dealing with). The method is not dissimilar to the 'Block Theory' developed by Richard Goodman and Gen-Hua Shi (1985). Despite referring to Londe and Malpasset in their books (see also Goodman (1989)) they make no attempt at explaining the work done by Londe, Vigier and Vormeringer. Goodman and Shi offer one tantalizing clue, though, that their block theory should be used as the initial step in the process of analysing stability before continuing with the method of Londe, Vigier and Vormeringer: *"It is beyond our present purpose to describe the solution of these parameters* (shown in the plot) *that determine the degree of safety of the wedge. We wish to point out, however, that such an analysis can only be run after a particular tetrahedral block has been singled out. Block theory is not a substitute for the limiting equilibrium analysis but, rather, a necessary prerequisite since it will allow you to determine which block to analyze"*. Hence the purpose of this paper is to describe the solution.

Starting, then, with Block Theory

Block theory requires plotting stereographic equal angle projection circles of the wedge planes beneath the dam and the plane of the slope (excavation pyramid). Note: many terms exist for 'wedge' such as 'tetrahedral block' as mentioned above or, used subsequently in Goodman & Shi (1985) to explain their block theory: 'joint pyramid'. With three planes there are a possible eight wedges and in combination with an open slope face a 'removable' wedge can be defined. Removability means the wedge can move from the rock mass by sliding on one plane, two planes or no planes, the latter situation either by 'popping' or falling out. The wedge, though, will only move if the resultant force on the wedge acts outside a boundary or envelope defined by the geometry of the wedge and slope (region known as a 'space pyramid') and the angle of shear resistance of the wedge discontinuities. Londe did not carry out this first step as the field evidence after the failure defined the wedge surfaces and the wedge configuration from which the wedge slid or lifted. First though the geometry of the wedge has to be obtained. Surprisingly, one has to resort to a number of techniques to discover what parameters were used to analyse the stability of the dam! The following parameters were determined (after substantial 'forensic research' of the suite of 'Londe papers'):

Plane 1: upstream face of wedge 'P1', dip 45°, dip direction 270°

Plane 2: downstream face of wedge 'P2', dip 40°, dip direction 013°

Plane 3: toe face of wedge 'P3', dip 0°, dip direction horizontal

The wedge in relation to the dam is shown on contour maps before and after failure in Figures 1 and 2. P1 and P2 are faults. P3 was an induced 'crack' (Londe, 1973).

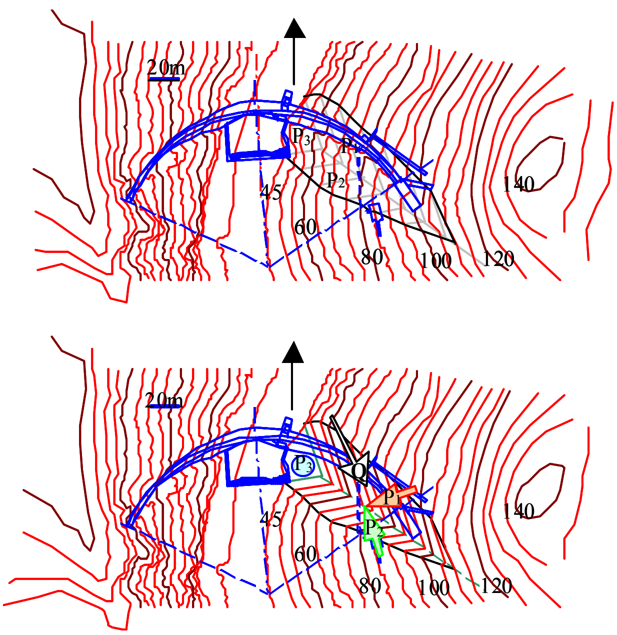

Fig. 1 Contour map prior to failure showing the plan of the dam and the underlying wedge causing failure. Contour map based on Londe (1987), wedge based on Londe, Vigier and Vormeringer (1970).

For the Block Theory to establish removability, a slope (EP) is introduced based on the contour map in Figure 1, this is dip 33° with orientation 280°.

In Figure 3 the great circles are plotted of planes 1, 2 and 3 and the slope. These are plotted on an equal angle projection, in this instance using both the 'upper focal point' and the 'lower focal point'. In the upper focal point projection the space within the reference circle (which coincides with the P3 great circle) is the 'lower hemisphere' and the 'outer hemisphere' is the space outside the reference circle. The three planes result in eight spherical triangles representing eight possible wedges or 'Joint Pyramids'. Using Shi's theo-

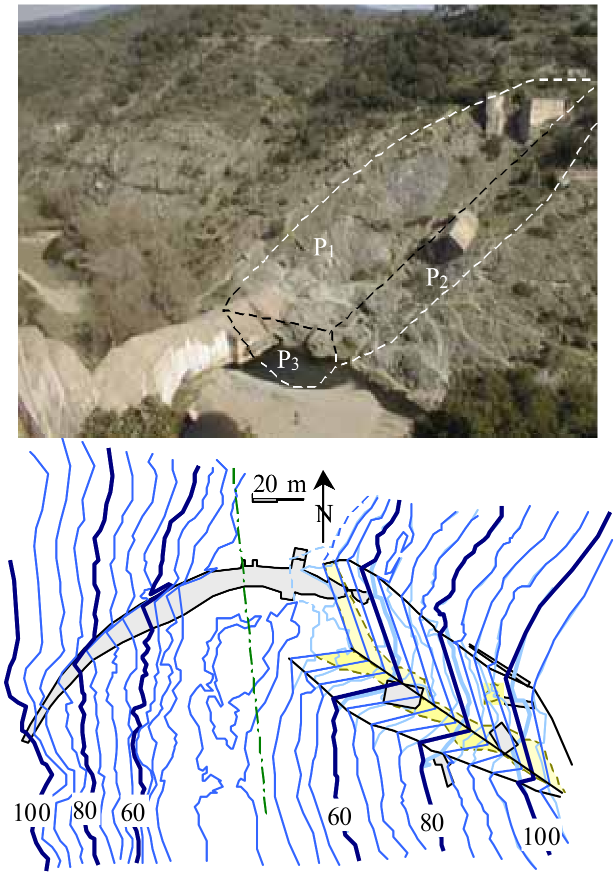

Fig. 2 Photograph and contour map of Malpasset Dam site after failure showing tie-in between wedge contours and ground contours; shaded portion is wedge. Wedge contours below survey contours (ground surface) indicating P3 either at a higher elevation or probably P3 consists of two or more planes (photo source: en.structurae.de/photos/index.cfm?JS=7596; photo used with permission of photographer Alain de la Forest (alaindelaforest@yahoo.fr)).

rem (Goodman, 1989) the only wedge that does not intercept the slope great circle is JP 000. This is the wedge which is potentially removable and represents the wedge which was used to solve the stability of the dam foundations. There are two further wedges which are potentially removable should the orientation of the slope change and these are shown in Figure 3: JP 001 if the slope dips to 295° and JP 010 for a slope facing towards 265°. JP 000 means that the wedge planes are 'upper half spaces' of the discontinuities forming the planes. JP 001 means planes 1 and 2 are still 'upper half spaces' and plane 3 is a 'lower half space'. In this instance plane 3 would be at higher elevation leaving an overhanging roof, the 'upper half space' in the rock mass. In the case of JP 010 planes 1 and 3 form 'upper half spaces' and 2 a 'lower half space'. All three cases should be analysed though JP 010 is unlikely as the forces exerted on this wedge would probably ensure it remains in place.

The wedge would be stable if the resultant force on the

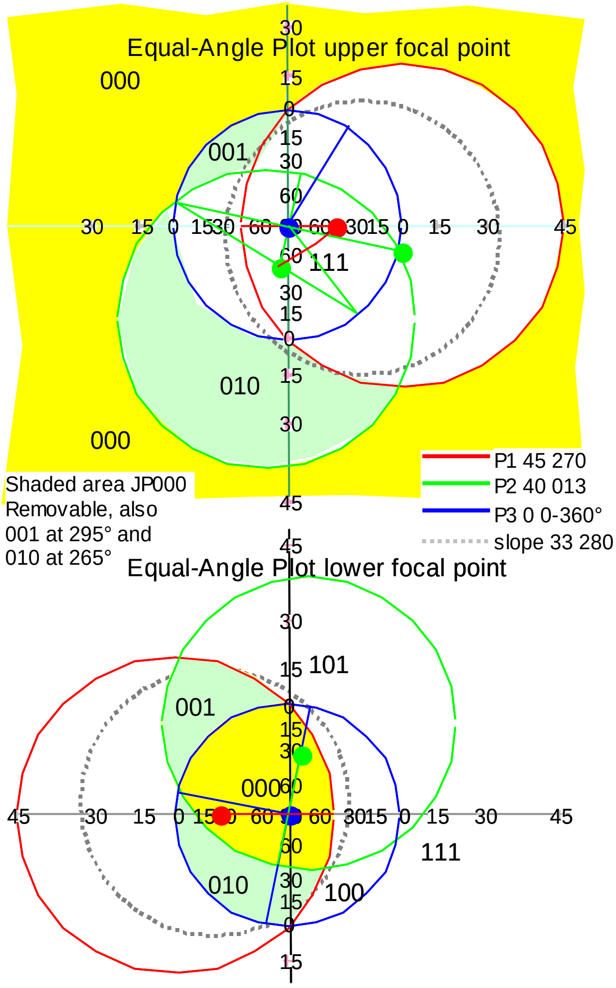

Fig. 3 Plot of great circles P1, P2, P3 and slope on extended equal angle net confirming that the Londe wedge is removable. Inner circle is lower hemisphere and outer area upper hemisphere.

wedge occurs within a zone defined by a spherical triangle with its intersection points at the poles of P1, P2 and P3. These are plotted in Figure 4 for the upper focal point projection (continuing from Figure 3).

The plot is further extended by connecting pole 1 with the intersection points of planes 1 & 2 and with planes 1 & 3. This is done for pole 2 with 1 & 2 and 2 & 3 and pole 3 with 2 & 3 and 3 & 2. The plots were all formed by generating the arcs using spherical trigonometry equations on a spreadsheet. Normally these are plotted by hand using an appropriate stereo net. The techniques used on a spreadsheet require a separate publication making use of spherical trigonometry and, in this instance, projection equations for an equal angle net (the x–scale and y-scale provide the projection equations to indicate the spacing in terms of α). The Londe plots were all done by hand, probably using tracing paper placed over an extended equal angle stereo net of which an example is given in Goodman (1989). The advantage of a spreadsheet is that different values can be inserted to test

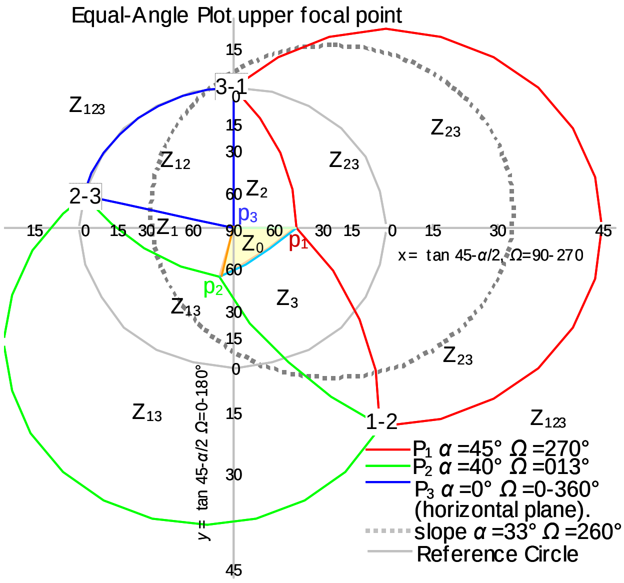

Fig. 4 Continuation of Figure 3, using a spreadsheet connecting poles of P1, P2 and P3 (p1, p2 & p3) with intersection points of P1-P2, P2-P3 and P3-P1 (1-2, 2-3 & 3-1) Londe rupture modes Z_0 through Z_{123} shown. A force in a direction within zone Z_0 is considered as safe.

the sensitivity of slope angle and direction, as well as difference in dip and dip direction for the discontinuity planes.

Forces on a wedge: influence of friction

Stereographic projections are three-dimensional representations of angles. Forces have a direction but also a magnitude. One simplification making allowances for 'magnitude' in stereographic methods is to represent magnitude by use of the internal friction angle from the Mohr-Coulomb equation. Simply, this states that shear resistance is the product of the normal force acting on a surface times the tangent of the friction angle. Hence, if the direction of a force acting on a plane is known there could be a component parallel to the plane and a component normal to the plane 'mobilising' the frictional shear strength. Consider a force F acting at an angle δ from the perpendicular to a plane. By resolving this force into a component along the plane, $F \sin\delta$, and a component normal to the plane $F \cos\delta$, and substituting this into the Mohr-Coulomb equation the mobilised shear resistance is $F \cos\delta \tan\varphi$. If $F \cos\delta \tan\varphi$ is greater than $F \sin\delta$ no sliding will occur. This relationship simplifies to $\tan\varphi > \tan\delta$, and even further to $\varphi > \delta$, leaving only angles to deal with.

The safe zone defined by the spherical triangle p1, p2 and p3 is expanded by plotting the 'friction cones' at p1, p2 and p3. If the force direction falls within the cone ($\delta < \varphi$) sufficient shear resistance is mobilised to prevent movement. In Figure 5 the friction cones have been added and connected to

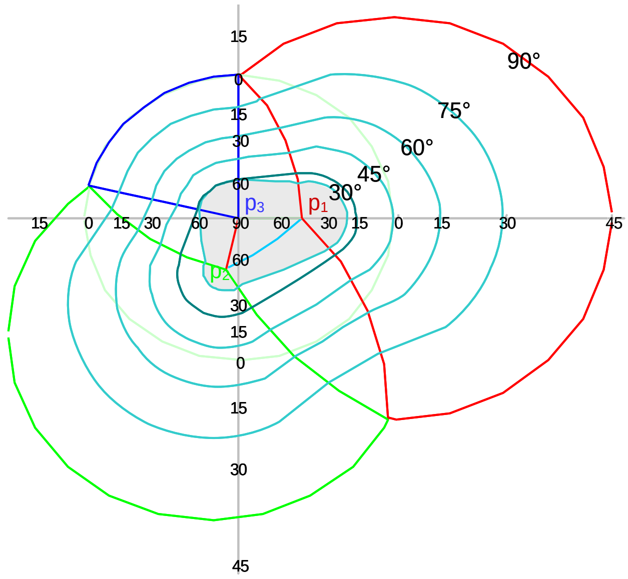

Fig. 5 Friction 'cones' added to show increased safe zone. Values correspond to Londe (1973) and are 25°, 15° and 30° for shaded zone. Friction angle iso-lines plotted for 30° through 90°.

each other at the end between the poles as the force changes direction it acts less on one plane and may start to act on two. Londe *et al.* (1969) and Londe (1973) describe the various types of sliding on one plane, on two planes along the line of intersection and possible dislodgement without sliding (or if in the opposite direction, where the wedge would be pushed into the rock mass, compression only). This results in seven modes of movement. Much of these two papers are devoted to this aspect. Goodman (1989) projection lines show where these zones can be found as well, but he has not defined them as such. The essential is if the resultant force falls in a zone that can cause movement or remain stable: a 'safe zone' and an 'unsafe zone'. The Goodman approach and that of Londe are as good as identical. Goodman does have a preference for using the 'lower focal point' whereas Londe has used the 'upper focal point', hence in Figure 3 both projections are given. In Figure 4 the 'upper focal point' is used so that direct comparison can be made with the original Londe publications.

Further friction angle isolines have been added as was done by Londe showing that the stable zone increases substantially with increase in friction up to the great circle of planes 1, 2 and 3 for φ=90°.

Plotting forces

Two types of forces are dealt with: the forces exerted by the dam on the foundation (including the dead weight of the wedge) and water pressure (seepage) forces acting perpendicular to the wedge planes. The forces from the dam are the weight of the dam and the hydraulic forces of the reservoir acting on the dam. The magnitude and direction of these forces are provided in the 1970 paper but do not specifically refer to the Malpasset Dam.

The total weight, including the weight of the portion of the dam resting on the rock volume, is W=111 000 ton. The thrust of the dam is horizontal: Q=84 000 ton. The water forces corresponding to full hydrostatic head are: (1) U_{1T}=85 000 ton; (2) U_{2T}=62 000 tons; and (3) U_{3T}=25 500 ton.

The above paragraph is taken verbatim from the paper. Q is shown in a direction 150° using the wedge diagram (replotted in Figure 1). The 'dip' of the resultant W with Q would be: $\tan^{-1}$(111 000/84 000)=53°.

When this is plotted in Figure 5 (150°/53°), the resultant is in the 'safe zone'. If the friction is reduced to zero, sliding of the wedge would occur on plane 3 only (zone Z_3). As with the orientation of the planes, the Londe papers do not provide these values; the vector does correspond well with the projections (1970 and 1973 papers).

The final step is to examine the influence of U_{1T}, U_{2T} and U_{3T} on the direction of F vector (point f). At this stage the final scenario of the analysis has been reached and to give Londe and his co-authors credit, the approach then and even today 30 years later has remained unique. A paper by Karaca & Goodman (1993) does indicate a similar approach showing the rotation of f under influence of the build-up of water pressures. Though they do not refer to any of the Londe papers, they could possibly have unwittingly presented the cause of the Malpasset Dam failure in their paper! A hint to this is given in the opening sentence of this paper.

Plotting water forces

To understand the Londe approach, the first paper (1969) presents a figure (reproduced here in Figure 6 for the water pressure force vectors on planes P1, P2 and P3). The amount of rotation of the force F when combined with forces U_{1T}, U_{2T} and U_{3T} individually is calculated for different percentages of U_{1T}, U_{2T} and U_{3T}. The resulting zone constructed on the stereograph will show that not the full percentage of the water force is necessary to cause the resultant to occur in the

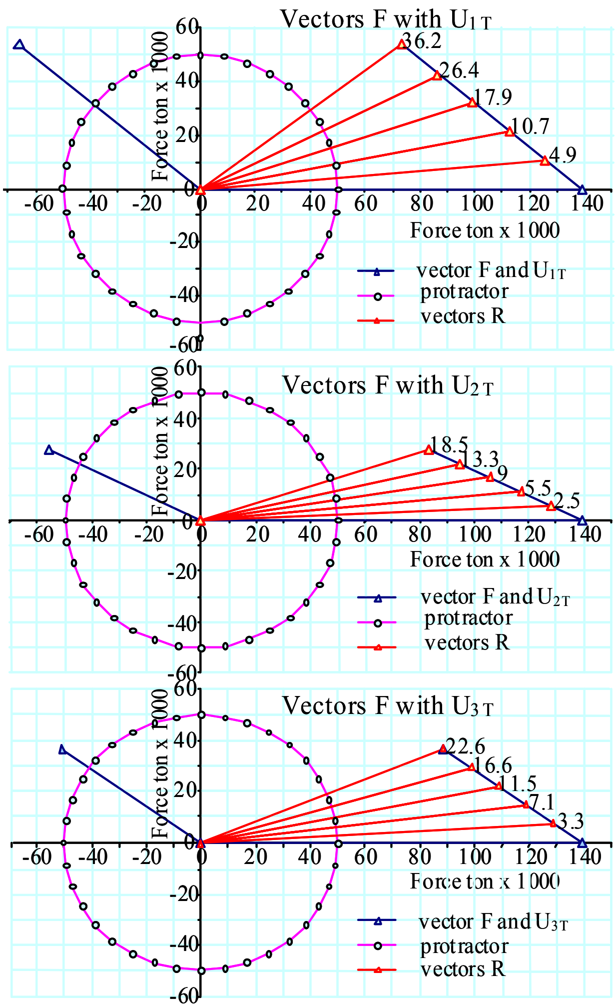

Fig. 6 Influence of water pressure forces on wedge planes P1, P2 and P3 with force F from the dam's weight, weight of the foundation wedge and the hydraulic force of the reservoir. Not surprisingly, plane P1 has the most influence on rotation of the resultant force R (note that rotations are from F in common great circle planes F & U_{1T}, F & U_{2T} and F & U_{3T}. The rotation angles are given in degrees (r values) at the ends of the R vectors.

'unsafe' zone. These values are plotted in Figure 7 and plotted along the great circles common to f and the poles p_{1U}, p_{2U} and p_{3U}. These poles are located on the upper hemisphere of the stereograph (the outer portion) as they represent the directions of the water forces u_1, u_2 and u_3 (perpendicular to the planes P1, P2 and P3 in an upward direction as if to lift the underside of the wedge or, strictly, acting on the upper half spaces of the wedge discontinuities).

The water pressures do not, of course, build up individually on the wedge planes but build up gradually on one plane and possibly more rapidly on another depending on their permeability and drainage paths. In Figure 8, graphical methods were used to complete the unsafe zone for water pressures. The total vector positions (100% build-up) are combined by rotating f-u_{1T} in a direction u_2 to produce f-u_{12T} and then rotating f-u_{12T} in a direction u_3 to produce the point position f-u_{123T}. In this way a zone is created, appropriately shaded indicating the range of positions where a resultant F with U(R) would cause the wedge to be unsafe and show which mode of movement would occur.

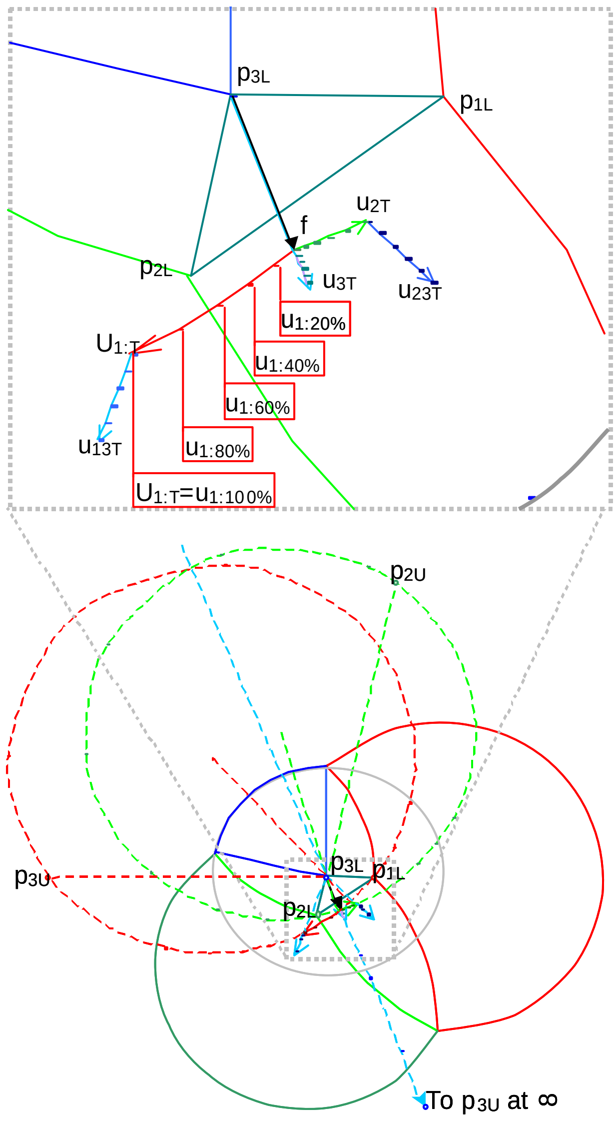

Fig. 7 Plotting influence of water pressures. First stage: f to u_{1T}, f to u_{2T} and f to u_{3T} towards poles p_{1u}, p_{2u} and p_{3u} (upper hemisphere, the directions of the water forces on P1, P2 and P3 respectively). Great circles are shown. The 'distances' f to u_{1T}, f to u_{2T} and f to u_{3T} are equal to the angles determined in Figure 6. Intermediate values of 20%, 40%, 60% and 80% are shown. Second stage is plotted for f & u_{1T} with u_{3T} (point u_{13T}) and f & u_{2T} with u_{3T} (point u_{23T}) showing graduations again for intermediate pressures 20 through 80%. Completion of second stage and final third stage to point u_{123T} is shown in Figure 8, this time solely by graphical techniques of intersecting great circles.

Fig. 8 Completion of plot to define safe (light blue) and danger zone (light yellow) for water pressures superimposed on friction iso-lines kinematics of possible movement.

Last word on the cause of the Malpasset Dam failure?

This article hopefully explains the analysis and model used to examine the cause of failure of the Malpasset Dam. It not only condenses the original suite of papers by Londe but compiles pertinent aspects of the Londe suite essential to the analysis. The analysis is, however, not 'the last word' with regard to exploring the cause of failure. Londe *et al.* (1970) also looked at moment stability of the wedge as the forces developed on the wedge do not pass through a common point. This analysis showed that the wedge was not safe. Further analysis was carried out by Wittke & Leonards (1987) using finite elements resulting in another explanation as to the cause of failure though broadly it is still the discontinuities and water pressure build up that caused failure. What none of these analyses considered was what could have been the influence of the water spout under 60 m of water head when the bottom valve outlet was opened that fateful evening 48 years ago. Dubbed for the time being as the 'Karaca-Goodman effect' is the rise in discontinuity water pressures as a result of an impinging water jet. It is not the momentum of the water splashing against rock blocks which causes them to dislodge but *the rise in discontinuity water pressures the water splashing induces*. Could this be the trigger that initiated instability of the left abutment wedge at 9 pm that night of December 2, 1959?

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Vilufushi, Maldives. Destroyed by the Tsunami...

... Restored by Boskalis

Professor's Column: Ode to ground

Prof.dr.ir. Frans B.J. Barends, GeoDelft/TUDelft

When speaking about water in the Netherlands, everyone is informed: the eternal battle against the sea, controlling large rivers under intensifying rain showers, sea level rise, as well as the importance of water for drinking quality, transport and recreation.

When speaking about air, there are similar appealing subjects: sailing and surfing, Schiphol airport, daily weather forecasts, fine dust and wind energy.

However, when speaking about ground the topics are less vivid: landscape, agriculture and spatial planning. Water we drink, air we breath, and from soil you become dirty. The unrealised fact is that almost all our comfort is based upon ground that we possess: water and air come from elsewhere. With ground, we control the sea and rivers by making dams, dikes and dunes. With massive earthen dams we control nature, influence the hydrological cycle and create energy. On ground we have built impressive infrastructures: cities, roads, railways, tunnels, bridges, pipelines, and we keep on doing so at an increasing scale. We are experts in engineering with ground. Let's expand on the significance of ground for our existence, with focus on the Netherlands.

Ground is protection. For at least ten centuries, the Dutch have built dikes and polders to withstand surging seas and rivers. Originally, by hand, all groups of society contributed as shown by names like Thief's-dike and Children's-dike, later by wind energy produced by thousands of windmills and thereafter steam and electricity took over, coping with an ever-increasing scale of lowland development. The social organisation by water boards, at a level of municipal authority, is unique in the world and paved the way towards an excellent management system for optimum safeguarding against flooding. However, reclaimed lands (particularly marshlands) shrink and compact. As a result, ground is subsiding. Along the coast at a maximum rate of 10 cm per century, in polders at an average rate of 25 cm per century, and in some peaty areas even at 150 cm per century, and more. It invokes a permanent concern, but at this rate the safeguarding against floods is technically manageable, even with the expected sea level rise . Since 1996 all protection works are controlled by law each five years and in general the result is that about 50% is sufficient, 30% is uncertain (lack of data) and 20% must be improved. The government spends about 1 billion euro annually to maintain sufficient quality of protection. The public, now being informed in exact numbers about the status quo, complains about the lack of safety. Only 50% is guaranteed. But at present they are safer than in the past. Improvements take years and we should not forget that a life without risk is unrealistic. Our ancestors knew better and survived.

Ground is support. Amsterdam is founded on hundreds of thousands of piles, tourists are told. Indeed, all Western Netherlands' structures are supported in smart ways, exhausting the available bearing capacity of the ground which is only better at the level of glacial sand layers at depths of more than 10 metres. The early foundations of monumental buildings consist of thin wooden friction piles of 6 metres length at most, sometimes bundled in hoods ("huien"). During the 17th century, pile-driving machines (by hand) allowed to install longer wooden piles reaching solid sand layers. At present, a great variety of foundation methods are applied reaching depths of 50 metres and more. Now, the challenge is to build in complex situations with minimum hindrance to the adjacent environment. The construction of the north-south metro line in Amsterdam is a perfect example. Also in the past smart solutions were applied. Around 1650 a strong southwestern storm blew the bell tower of the Laurens Church in Rotterdam lean, against the nave. The tower founded on short friction piles had become top-heavy by the last build-up in fine Italian marble. Lead by the city's engineer, a new foundation of long timber piles was driven around the basement and the heavy tower was adjusted and put on the new foundation using iron chains and horsepower without a scratch, an incredible achievement. How exactly is not described. If we look at Rotterdam, to the new city centre with tallest buildings, the Kop van Zuid, the Van Brienenoord- and Erasmus Bridge, the enormous harbour and the many tunnels, recalling the local very soft soil conditions and high groundwater levels one may see impressive demonstrations of sophisticated engineering, however invisible.

Ground is mobility. In the West of the Netherlands road and railway infrastructure is built on sand embankments. Without such support, no straight track is possible. Building roads on weak grounds that hardly settle is not an easy matter, mainly due to the heterogeneity in the underground. Ground is not transparent. Modern technologies like acoustic, seismic and radar reconnaissance are limited. Mobility needs a firm basis; with proper roads we can move around. The underground transportation infrastructure is impressive.

More than 50 km of short tunnels exist, mostly crossings in complex situations, under rivers and canals, through dikes and in dense cities, made under extremely difficult soil conditions. Since tunnels may provide a short connection, their flood resistance is of special concern. Tunnel-boring methods have only been applied since the last decennium, because current techniques were too risky. Within ten years Dutch tunnelling techniques conquered a top position in the international state of the art. The underground is full with cables and pipelines; every house directly connects to underground water, gas, sewer, electricity and communication lines, the length of which reaches many thousands of kilometres. Main pipes are pushed or conducted under waterways and other obstacles by directional drilling. The next step is underground litter transport, underground supply in city centres and harbours, and large multiple-cable tunnels.

Ground is business. The mainports in Rotterdam and Amsterdam, as well as Vlissingen and Delfzijl, and many marine and yacht harbours have been built at borders of land and water. Quays, wharfs and jetties are suitable for the largest ships down to the smallest pleasure yachts. Schiphol airport sets high demands to its runways, constructed on the clayey bottom of a former lake. Mainports of Europe are cradles of industriousness, and their capacity is reaching limits. New ways of expansion are required on the over-occupied ground: expanding roads and high speed- (HSL) and cargo railways (Betuwe-line), new lands at the coast (after Stive: "an anthropogenic massive sand dune just offshore, as a sand engine creating coastland with time by natural morphological processes"), new harbour territory (Maasvlakte II), reviving wetlands in combination with living, working and nature (IJssel lake). These are great challenges with a distinct role for the engineer.

Ground is home. More and more we use the underground space, the 3rd dimension. By moving less pleasant activities and functions underground, more space becomes available for activities mostly wanted on the surface such as living, recreation, agriculture and none the least, giving space to and enjoying nature. In the underground we may put all parking garages, shopping centres, libraries, prisons, greenhouses, public and cargo city transport, various industries such as water treatment plants, waste storage, and distribution systems. The underground is insensitive to climate and daily weather. The ground is a safe home to many activities.

Ground is promising. The bio-chemical potential of the underground is immense and yet unexplored. In a single kilogram of soil one may find a thousand billion bacteria with all kinds of roles: decomposition of organic matter, nutrient cycling, involved in almost every chemical transformation in the soil. They can be active or inactive depending on availability of suitable nutrients. What if we can make them work for us! Recent developments in this field at GeoDelft have shown that natural forces of micro-life can be directed by bio-stimulation, bio-augmentation and bioremediation to alter soil conditions: making sandstone in just a few days, sealing-off leaky holes in soil layers without knowing a priori where they are located, creating specific slurries for settlement compensation, stabilising peat (not anymore exposed to oxidation), and cleansing polluted areas. The beauty of this approach is that we use nature itself to perform the job. Another striking example is geothermal energy. More that 99% of the earth is hotter than 1000 ºC, less than 0.1% is colder than 100 ºC. The underground becomes warmer with depth at 3 ºC per 100 metres. Imagine: we live on an immense stove. A large-scale solution for the present-day energy problem is right under our feet, reachable by modern and future smart drilling techniques from the oil industry. Moreover, knowing that more than 90% of our energy is won out of coal, gas and oil, and that over 35% of energy use is spent on in-house climate control, it is obvious that geothermal energy is just what we must develop. Fossil fuels become extinct (price tripled since 2000). Growing dependence on energy import is a bad policy. CO_2-emission can be reduced by 20 to 30 percent, meeting the Kyoto declaration. Oil is in fact an irreplaceable ore, too important to burn. Finally, the oil industry may just stay in the energy market and expand to geothermal energy. Indeed, ground is promising.

Moraine dams and glacier lake outburst floods (GLOF)

Senta Modder, DG Water, Ministerie Verkeer en Waterstaat

Increased melting of glaciers due to climate change

Climate change has its impacts on biodiversity, freshwater resources and local livelihoods. A rising average global temperature is threatening fragile ecosystems like glaciers. Seventy percent of the world's freshwater is frozen in glaciers. Glacier melt buffers other ecosystems against climate variability. Very often it provides the only source of water for humans and biodiversity during dry seasons (IPCC, 2007). The Himalayas have the largest concentration of glaciers outside the polar caps. With glacier coverage of 33 000 km^2, the region is aptly called the 'Water Tower of Asia' as it provides around 8.6 million m^3 of water annually. These Himalayan glaciers feed seven of Asia's great rivers and ensure a year round water supply to millions of people. Climate change has impacted the glacial ecosystem tremendously. 67% of the glaciers in the Himalayas are retreating at a startling rate and the major causal factor has been identified as climate change (UNEP & ICIMOD, 2002).

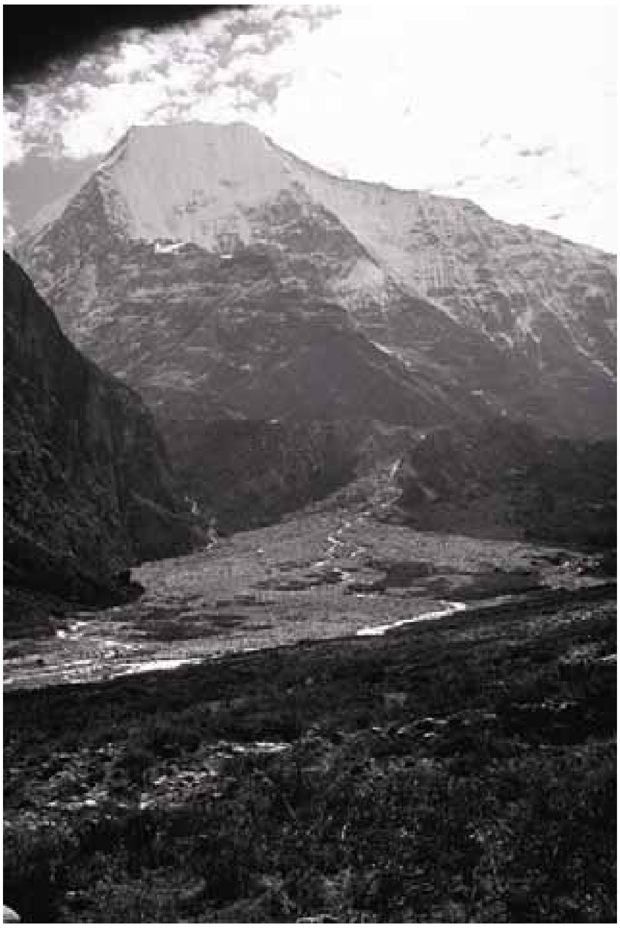

Fig. 1 Small GLOF at Chubung Glacier (Nepal) in 1991 (Modder & Van Olden, 1995).

Increased risk of GLOF's due to melting glaciers

Many mountain glaciers in the world built up a prominent end-moraine during the Little Ice Age (Neo-Glacial) which lasted until approximately 1850. Since then, the majority of mountain glaciers in the world has been thinning and retreating upon a gradual change in climate. During retreat, a basin is usually formed between the thinning and receding ice-front and the end moraines. If the morainic dam is relatively impervious, a lake may form (Mool & Kadota, 1993). The material of an end moraine consists of unconsolidated moraine material and/or ice. Moraine material in general is composed of a diamicton, a heterogeneous mixture of boulders, gravel, sand and silt. Opposed to most man-made dams, which are constructed of reasonably erosion resistant material, the materials of a natural dam are generally very susceptible to erosion by piping and fluvial overflow. Glacial lake outburst floods (GLOF's) are catastrophic discharges of water resulting primarily from melting glaciers. Principally, a moraine dam may break by the action of some external trigger or self-destruction. A huge displacement wave generated by rockslide or a snow/ice avalanche from the glacier terminus into the lake may cause the water to top the moraines and create a large breach that eventually causes dam failure (Ives, 1986). Earthquakes may also be one of the factors triggering dam break depending upon magnitude, location and characteristics. Self-destruction is a result of the failure of the dam slope and seepage from the natural drainage network of the dam or melting of internal ice cores. GLOF waves comprise water mixed with morainic materials and cause devastation for downstream riparian communities, hydropower stations and other infrastructure. In South Asia, particularly in the Himalayan region, it has been observed that the frequency of the occurrence of GLOF events has increased in the second half of the 20th century. GLOF's have cost lives, property and infrastructure in India, Nepal and China.

Case study in Tsho Rolpa, Nepal Himalayas

In 1995 I conducted a M.Sc. thesis in the Nepal Himalayas with my fellow student Quirijn van Olden. Subject was an engineering-geomorphological analysis of a moraine dam at Tsho Rolpa. We were invited by the Nepalese government and signed a Memorandum of Understanding with the Water & Energy Commission Secretariat of Nepal. After a small GLOF at Chubung Glacier in Rolwaling Valley, people were afraid that the Tsho Rolpa lake would also burst. Tsho Rolpa is located at an altitude of 4 548 m, is about 3.5 km long by 0.5 km wide and is up to 135 m deep. The lake holds about 80 million m^3 of water. We had two fieldwork periods of a month, one before and one after the monsoon. For our M.Sc. thesis, the Tsho Rolpa end moraine complex was analysed in terms of geomorphology, geotechnical properties of (surface) materials and (present-day) activity of geomorphological processes. The first week we spent making an appropriate topographical map. Then we proceeded to make a geomorphological map with profiles and an activity map. The (mass movement related to) melting of dead ice was identified as the geomorphologically most important factor. Then we prepared a geotechnical map and detailed geotechnical descriptions of the units in the geotechnical

Fig. 2 Overview of Tsho Rolpa (Modder & Van Olden, 1995).

map, using field observations (scan lines, checklists) and laboratory tests. The following genetic material units have been recognised: present-day fluvial deposits, dead-ice melting induced deposits, colluvial deposits, glacial-fluvial deposits and glacial deposits. The glacial deposits, especially the so called ablation deposits, form the skeleton of the moraine dam and these units will be reached sooner or later when excavating. Ablation deposits can be described as an unsorted, well-graded, unstratified, non-oriented mass, consisting predominantly of subangular cobbles and boulders, lacking substantial amounts of fine and medium gravel, sand and fines (4%). In the end we used electromagnetic equipment to detect the dead-ice core bodies within the

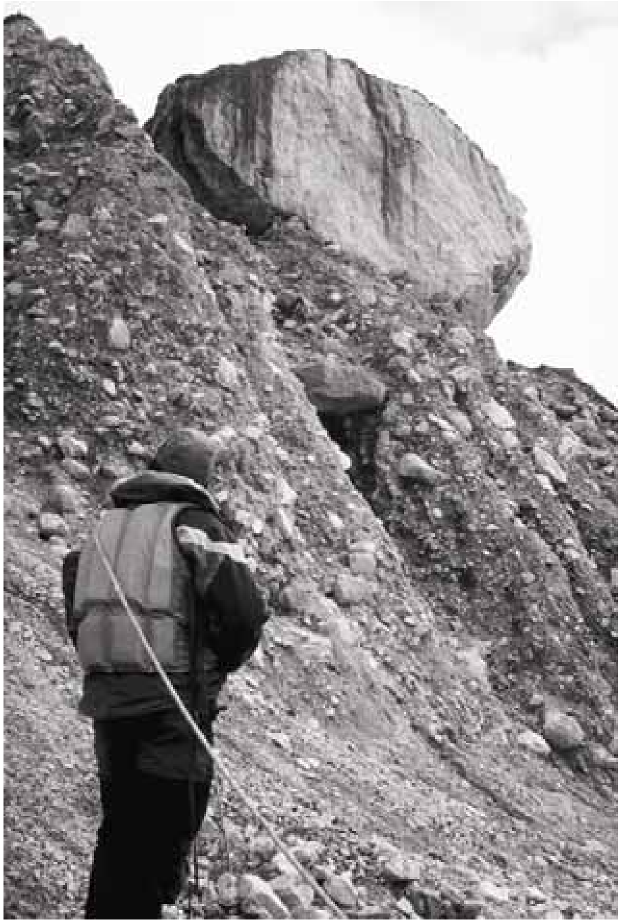

Fig. 3 Material of the moraine dam (Modder & Van Olden, 1995).

moraine complex. It was not possible to predict exact depth and size of dead-ice cores. Finally we translated the geomorphological and geotechnical information into useful recommendations for engineering applications. The main conclusion was that the melting of dead-ice in the subsurface is generating active mass movement on the inner slope of the dam and a lowering and retreat of the local water divide, the rim of the dam. One important consequence is the reduction in height difference between the lake and the dam, the so called freeboard. Therefore, we recommended lowering

Fig. 4 Slope instability at inner side moraine dam (Modder & Van Olden, 1995).

the lake level in order to prevent future GLOF. The second conclusion was that the spring zone present below the northwestern terminal moraine may be an important clue for the future, as by then dead-ice is assumed to be absent in the subsurface. This makes the location more suitable for possible future excavation processes.

Installing siphons at Tsho Rolpa

As a start of a remediation strategy Wavin Overseas B.V. Holland funded and installed a trial siphon over part of the northwestern terminal moraine in May 1995. This consisted of a triple-inlet pipe from the lake connecting to a single pipe (all 160 mm O.D.) whose discharge outlet was located about 80 m down the outer flank of the moraine.

Alternative technical solutions to reduce GLOF hazard at Tsho Rolpa

In March 1996, a proposal to the Dutch government for the long term remediation of Tsho Rolpa was prepared on behalf of Nepal by John Reynolds from the UK. He suggested to install more siphons at the northwestern terminal moraine. Jan Rupke from the University of Amsterdam was asked for a second opinion by the Ministry of Foreign Affairs of the Netherlands. Therefore, Jan Rupke and I made a technical paper (Rupke & Modder, 1996). Conclusion was that installation of more siphons had several technical disadvantages. The main disadvantage is the fact that the lake bottom topography is very shallow at this location. Because of friction losses due to the long pipe-inlet, this location is not suitable for siphoning. According to technical calculations by Wavin, the suction head of the siphon system is reduced to 0.4 m when the horizontal distance from pipe-inlet to highest point of the rim is 100 m. Another disadvantage of the use of siphons is the resulting fluctuating lake level. A siphon system is not capable of releasing the summer influx (up to 16 m^3/s). The resulting fluctuating lake level has a negative influence on the stability of the dam due to changing ground temperatures and changing pore pressures. Another problem is that this shallow part of the lake has no connection to the deeper parts of the lake because it consists of several isolated holes. A 12 m lowering cannot be achieved here without excavating a long channel through the lake bottom. Considering these disadvantages we proposed the excavation of an overflow channel without using siphons. The main advantage is that a continuous, quick and permanent lowering of the lake level is achieved.

Mitigation measures at Tsho Rolpa

A revised proposal for remediation of Tsho Rolpa (by excavating an open channel!) was submitted by John Reynolds in December 1997. Funding worth $2.9 million was granted in March 1998 by the Dutch government. Preparation for mitigation measures could start (Reynolds, 1999).

The geophysical studies of Tsho Rolpa moraine dams were conducted quite intensively during 1999 and 2000. This gave quite good insight into the internal structure of the dam, particularly the ice core. While the dam continues to degrade gradually, rapid and disastrous degradation has not been observed in recent years. The melting of the ice cores is still closely being monitored. The implementation of the mitigation measures started in April 1999 and the draw-down of the lake was completed in June 2000. The project essentially includes construction of an open channel, trapezoidal in cross-section, 6.4 m wide at the bottom and designed to cater 14 m^3/s of flow with a maximum capacity of 35 m^3/s and achieve at least 3 m lowering of lake water level. A gate structure is designed to regulate the flow if necessary (e.g. during maintenance) and is otherwise left open year round. The open channel passes through Horseshoe Lake, roughly along the alignment of test siphons from May 1995. In 2004 a micro hydro-electric plant with a capacity of 15 kW was established to supply electricity to the operator house built at the project site and manned by 3 Department staff members year round. The plant taps water flowing through the sluice gate (Shrestha, 2007).

Fig. 5 Mitigation works at Tsho Rolpa in 1999 (Shrestha, 2007).

Fig. 6 Mitigation works finished at Tsho Rolpa (Shrestha, 2007).

Fig. 7 Open channel finished in 2000 (Shrestha, 2007).

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In the spotlights: De Ondergrondse

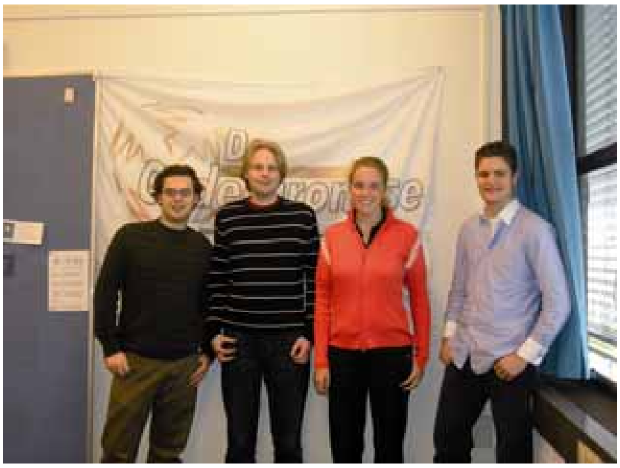
Sanne Brinkman, Commissioner of De Ondergrondse (2006-2007)

For years, the 'Dispuut Ingenieurs Geologie' (DIG) has been representing Engineering Geology students. As from October 2006, this dispute has merged with its equivalent for Underground Construction. This led to a new student chapter named 'De Ondergrondse - Dispuut voor Geo-Engineering'. All students doing their M.Sc. (Master of Science) within the section Geo-Engineering at TU Delft are members of De Ondergrondse. The section Geo-Engineering consists of Geomechanics, Geotechnics, Underground Construction and, of course, Engineering Geology. This merge also implies that De Ondergrondse will have considerably more members than the DIG used to have.

Visit to a geotechnical laboratory during the Madrid study trip. Third from left, this Newsletter's guest editor Michiel Maurenbrecher can be seen.

The question is now: What are the consequences of the merge regarding the activities of De Ondergrondse? Most of the activities previously organised by the DIG will still take place. These are, for example, lectures within the area of Geo-Engineering, company visits, the traditional Christmas Dinner, and many excursions. Regarding the excursions of last year, the trip to Madrid can definitely be considered one of the highlights. 28 students and professors spent four days in the capitol of Spain to visit projects and, of course, enjoy the Spanish 'culture'!

Further, an excursion to the Noord-Zuidlijn took place and the centrifuge of GeoDelft was visited. The latter is one of the many examples of the cooperation between De Ondergrondse and Ingeokring. De Ondergrondse is, besides all other activities, also involved in the production of this Ingeokring Newsletter. Furthermore, De Ondergrondse is involved in the evaluation of the various courses of Geo-



2006-2007 Board of De Ondergrondse: Ivo van Kempen, Jan Mulder, Elles Bader and Sanne Brinkman.

Engineering. Questionnaires are used to assess the students' opinion about specific courses and a discussion with professors follows in order to improve the courses. The results of this course evaluation will also be made available on BlackBoard to both lecturers and students.

The larger target group of De Ondergrondse does not only mean that the size of the activities will, in general, be larger but also leads to an increased number of different activities. Once a year, there will be an annual meeting (ALV), during which the board will be changed, an overview of the past years' activities is presented and insight in the financial status of De Ondergrondse can be obtained.

Regarding the organization of the Dispuut, the 2006-2007 Board consisting of Elles, Jan, Ivo and Sanne have experienced the period as board of De Ondergrondse as great fun,

2007-2008 Board of De Ondergrondse: Auke Lubach, Daan de Clippelaar, Johan Haan, Paul Gerrits and Paulien Kouwenberg.

above all. The new board consists of Paulien, Johan, Paul, Auke and Daan. Paulien, Johan and Paul are all studying Engineering Geology and will therefore be in touch with the Ingeokring.

Finally, if you want to stay up-to-date, please visit our website at www.ondergrondse.nl. The latest news within the subject of Geo-Engineering and reports of activities can also be read in the newsletter of De Ondergrondse, which appears four times per year. If you wish to receive the Ondergrondse newsletter or want to contact us, please send an email to: info@ondergrondse.nl.

Wishing you, on behalf of the board, a cordially Glück Auf!

Dam and canal design on soluble rock (West Gode Irrigation Project, Ethiopia)

Dr. Peter N.W. Verhoef, Royal Boskalis Westminster nv (formerly Delft University of Technology, Section Engineering Geology)

Prologue

In December 1985 ir. Jaap Oosterman, a DHV engineer who was managing a NEDECO project for an irrigation scheme in Ethiopia, visited Prof. David Price. David called me to his office to join this meeting in which Jaap shortly explained the project. In the Ogaden desert, along the river Wabi Shebelle near Gode, an irrigation scheme was planned. It involved some 20 000 hectares of land to be irrigated, a major project for the Somali province of Ethiopia. It was a gravity irrigation project, one of the first of its kind. Current irrigation was normally done using pumps, where the fuel needed for the pumps would cost as much as the harvest of the land would gain. The team of Jaap Oosterman was doing the design of the project, which had to be finished within a time span of 8 months, including all field work needed. One of the key factors was the design of a small weir. This diversion dam was going to be constructed on a horizontal platform of limestone, which acted as a barrier causing the river Wabi Shebelle to flow around this rock. The main reason of the visit was to discuss the foundation of the dam. Since the foundation was on rock, Jaap sought the advice of David Price, the 'éminence grise' of Dutch Engineering Geology.

The idea was to build the dam on this rock platform in the dry, while the river could continue to flow uninterrupted by the construction process. When finished, the river would be dammed and the water diverted to the weir to enter the main irrigation canal. This looked like a good idea.

But then Jaap started to sum up some of his worries. He mentioned big slabs of limestone found downstream of the rock platform. He was worrying about these. We talked about erosion processes that might have caused dislocation of slabs from the rock outcrop. But then Jaap mentioned there was gypsum rock in the subsurface. At this point David Price started to laugh and said with his typical thunderous bas-baritone voice: "Gypsum?...forget the whole project!". With gypsum present in the rock mass under the dam there would be a high chance of problems caused by the solubility of this rock. He mentioned examples of dams on gypsum rock foundation that had failed when reservoirs were filled for the first time (Flores Calcano & Rodrigues Alzura, 1967). He was pessimistic and said that a thorough investigation was needed to find out the geological conditions of the dam site.

It was decided at the meeting that I would go to Ethiopia in January 1986 to do field work and prepare a site investigation program for the dam. Jaap had scheduled a visit to the site of three days, but David and I insisted that two weeks of fieldwork were needed for me to be able to do the work.

So I went off to Addis Abeba in January, where I was to join other team members to travel to the Ogaden. There we found that the old DC-4 Dakota plane that should fly us to Gode had broken down. We had to wait a few days, which meant that I had only one week to spend in the field. During the week preceding the site visit, I visited the Ministry of Mines where I could study 1:20 000 aerial photographs of the site and a hydrogeological report on the Ogaden area (NWRC, 1972). This allowed a proper preparation of the fieldwork. A visit to the Technological University in Addis Abeba was made, to acquire information on the possibilities for geophysical site investigation and laboratory testing of rock in Ethiopia.

Our party, consisting of Jaap Oosterman, other Ethiopian and Dutch engineers and myself, left next weekend for Gode. This was no straightforward trip. First by plane to Deri Dawa. We stayed overnight in this town and bought food at the local market place to bring with us. The next day we flew in the Dakota to Gode. The most interesting flight I have had up to now! The doors could not be locked anymore and were secured by hemp rope. The small plane slowly rose and flew at low height over the mountains, rocking and jumping. One of the few times I saw many people making use of paper vomiting bags… After a few hours we arrived at Gode. From there the party went by Toyota Land Cruiser to the camp site, some 40 km to the west. We made a stop along the road to examine a basalt outcrop. One of the nice things of being a geologist is that you can explain people a lot about their own rocks, even when you yourself are a newcomer under the African sky!

1. Geology

Ethiopia is situated at the Afar triangle, the well-known triple point of the African and Red Sea rift systems (Figure 1). Therefore one of the first things to think of is seismic risk related to movements along the fault system of the rift. Existing hazard maps show that the Ogaden occurs far south of the mountain ranges along the rift system and Gode is situated in a low seismic risk zone.

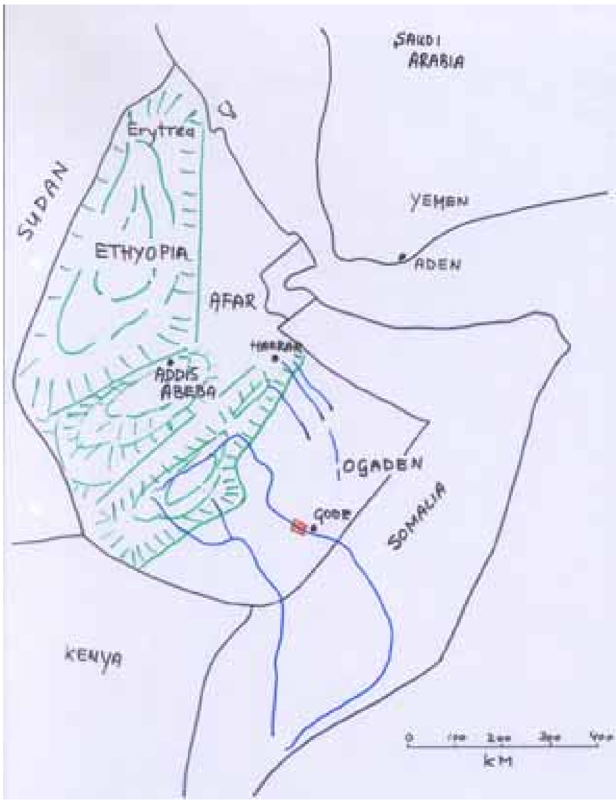


Fig. 1 Location of the Ogaden and Gode.

The geological map of Ethiopia reveals that the Ogaden is underlain by Cretaceous gypsiferous rocks (Figure 2). On the map one can see that the northern rivers entering the Ogaden desert disappear in the underground, attesting to the effect of dissolution of gypsum (Figure 2). At a distance of 40 km from the proposed diversion dam site near Gode, a deep borehole has been made which shows that the gypsiferous rock formation is at least 250 m thick. The cross section shows that basaltic rock, related to African Rift volcanism, has intruded into the gypsiferous Cretaceous rocks (Figure 3).

When we examine the local geological map west of Gode (Figure 4), we see that the basement of gypsiferous rock is covered by recent wadi deposits, with the wadi streams flowing into the direction of the Wabi Shebelle. At present the Ogaden is a plateau gently dipping into the direction of the Indian Ocean (Figures 2 and 3). The Wabi Shebelle itself has incised in this plateau, it flows 5 to 6 m below plateau level (Photo 1). The Wabi Shebelle flows in a broad valley filled with alluvial clay deposits. This clay apparently seals the river from the surrounding soluble gypsiferous rock, which explains that this river does not disappear into the underground and can reach the Indian Ocean (Figure 2).

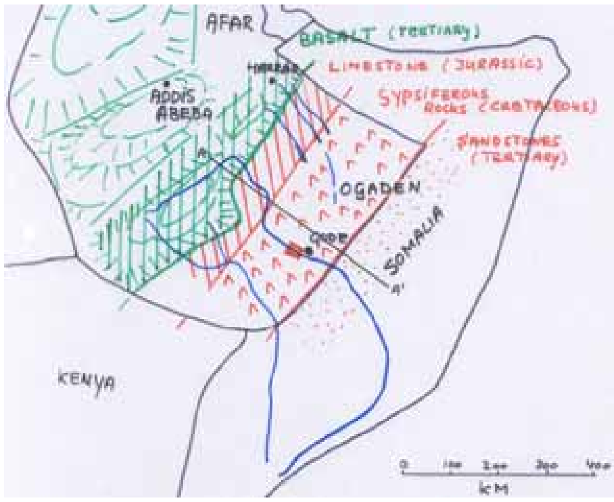


Fig. 2 Geological map of Ethiopia (note that the Ogaden desert coincides with the presence of gypsum rock at the surface).

The alluvial deposits are reported to reach a thickness of at least 30 m (NWRC, 1972). In the study area the alluvial deposits have a width of 12 to 13 km. The deposits consist of clays, sandy silts and gravels rich in ferromagnesian minerals resulting from the weathering of volcanic source rocks situated in the mountains far to the northeast (Figure 4, green horizontal hatching). The recent local deposits are alluvial wadi deposits and debris cones from intermittent rivers and wind-blown silt deposits often redeposited by flood flows during rainstorms (Figure 4, red hatching).

The main gypsiferous formation (Neocomian, Lower Cretaceous) occupies a large part of the Ogaden (Figure 4, white). It is made up of alternating marls, more or less gypsiferous clays, thick beds of massive gypsum, dolomites and very

Photo 1 Wabi Shebelle near Gode.

thick rock salt layers. Thickness near Gode surmounts 250 m. On top of the gypsiferous formation a dolomitic-calcareous layer of about 30 m thick occurs, the Mustahil Limestone. This layer forms a very distinct resistant cover over the main gypsum, and represents a major geomorphologic feature of the Ogaden (Figure 4, blue coloured rock in the south). The

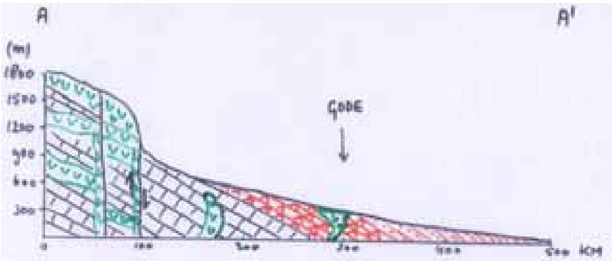


Fig. 3 Section AA' of geological map in Fig. 2. Basement rock is Jurassic limestone overlain by gypsum rock and sandstones, which are invaded by dolerite and basalt related to the African Rift system.

very rich fossil content points to a Barremian-Albian age (Lower Cretaceous). The Mustahil Limestone is covered by gypsiferous marly layers. Dolerite dykes of olivine basalt composition cut through the sedimentary cover all over the Ogaden Basin (Figure 4, green colour). Occurrences of these

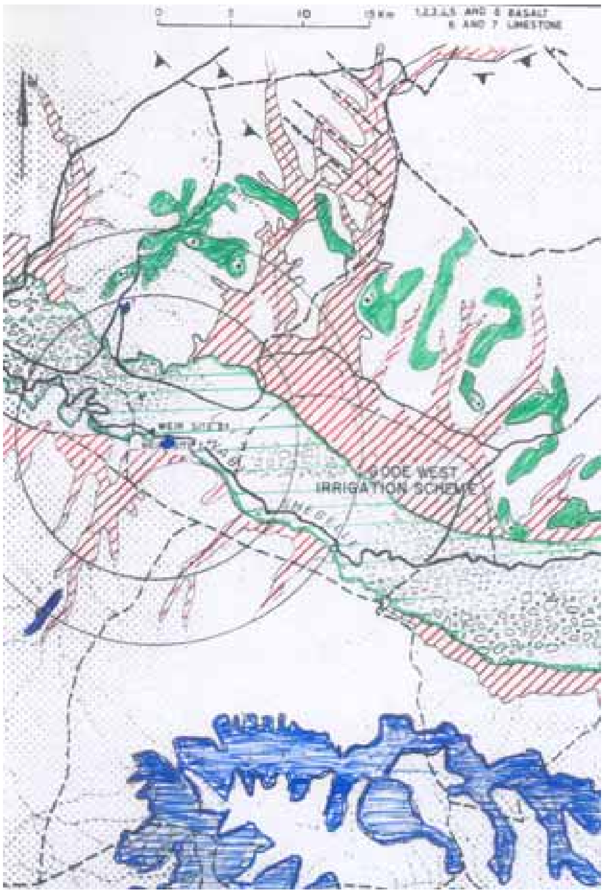


Fig. 4 Geological map west of Gode (white: gypsiferous rock series, blue: Mustahil Limestone, green: dolerite and basalt, red hatch: recent wadi deposits).

dolerites are within 15 km of the building site of the dam. Examination in the field has shown that the dolerite occurs mainly as rounded core stones left over from ancient weathering episodes. The core stones itself, however, have only a very thin weathered skin of brown coloured iron oxides. They are strong, dense and fresh and probably suitable for construction purposes.

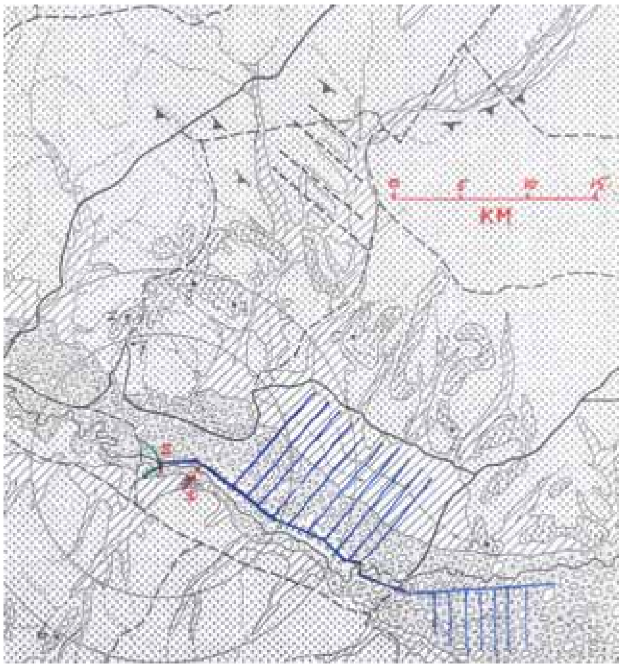


Fig. 5 Outline of irrigation project (I and II: location of dam site I and II).

2. The irrigation scheme layout

Figure 5 gives schematically the layout of the irrigation scheme. The proposed dam site occurs at location I. As you can see, the design was changed (why will become clear in the following) and the main canal starts at dam site II. First an area to the north of Wabi Shebelle is irrigated. This area is called West Gode. Then the irrigation canal crosses the river by a siphon to irrigate an area south of the river.

3. Dam site I

The site of the weir structure has been chosen at a locality where limestone rock forms a natural barrier for the Wabi Shebelle (Photo 2). On the 1:250 000 geological map this rock is indicated as belonging to the Mustahil Limestone (Figure 4).

When I examined this outcrop in 40 degrees Celsius heat (it was forbidden to swim in the river, since the local crocodile was really dangerous), I found that the limestone rock is

Photo 2 Dam site I, on this limestone rock the dam was planned to be constructed in the dry.

capped with a reddish-yellow dolomite layer of about 1 m thickness, which appears to be very resistant to river erosion. Below this dolomite layer, a thinly bedded limestone sequence occurs with a thickness of at least 2.70 m (measured up to the water line). The limestone rock has a more or less horizontal position. Striking, however, is that the limestone beds tend to dip water-inward at many places where the limestone borders the river (see the dip signs in the sketch map of Figure 6). This indicates that somewhere below the waterline the river erodes a soft layer, undercutting the limestone. On the left bank of the river the limestone and dolomite are covered by recent sandy silt deposits, with some clay intercalations. Cross-bedding in the sandy silt indicates that these have been laid down by sheet wash on the riverbank slope during heavy rainfall periods. On the right bank, however, on top of the yellow dolomite two beds of gypsiferous formation occur, with a total thickness of 3 m (maybe these beds have been eroded away by the river on the left bank in the past, before deposition of the recent sheet wash deposits). The lower of the two gypsiferous beds is about 1.25 m thick and consists of thinly bedded distorted limestone layers and pebbles and loose blocks of limestone cemented by fine, crystalline gypsum. On top of this, there is a 1.75 m thick layer of gypsiferous gravel and marly sand. It has been observed that this gypsiferous bed can be easily eroded by the river and that dissolution occurs. On top of the gypsum beds about 2 m of recent sandy silt sheet wash deposits occur (Figure 6, cross section AA').

I wrote the following description in my report concerning the engineering geological situation of this site:

"The foundation rock for the diversion dam consists of strong dolomite and limestone. Thickness of the foundation rock is at least about 4 meters, drilling will have to provide information on depth and nature of the underlying materials. It is expected that below the limestone, layers of clay alternating with gypsum occur, and below that massive gypsum and gypsiferous dolomite may occur.

Both the limestone foundation and the underlying rock are suspect of having dissolution cavities or open discontinuities, since both gypsum and carbonate are solvable. Field observations have not shown obvious dissolution features in the limestone, apart from features resulting from the river erosion. The gypsum layers above the limestone, however, indicate recent dissolution features; open piping-like holes at least 1.5 meters deep into the formation occur along the riverside."

In the field I gathered information on the rock discontinuities along scan lines. In the limestone two sets of subvertical discontinuities occur; a set (J1) which trends in NE-SW direction and is persistent (>20 m) and a set (J2) which trends in NW-SE direction. On the dolomite surface these subvertical joints have a very wide aperture (up to several tens of centimetres), due to erosion by the river. In the underlying platy limestone the aperture of the joints closes to very tight. The third set of discontinuities that exists is parallel to the subhorizontal bedding. In the platy limestone beds also a scan line measurement has been carried out. The joints and bedding planes are being opened up by tree roots, some are very wide, some are narrow and filled with stiff clay. Most discontinuities are tight.

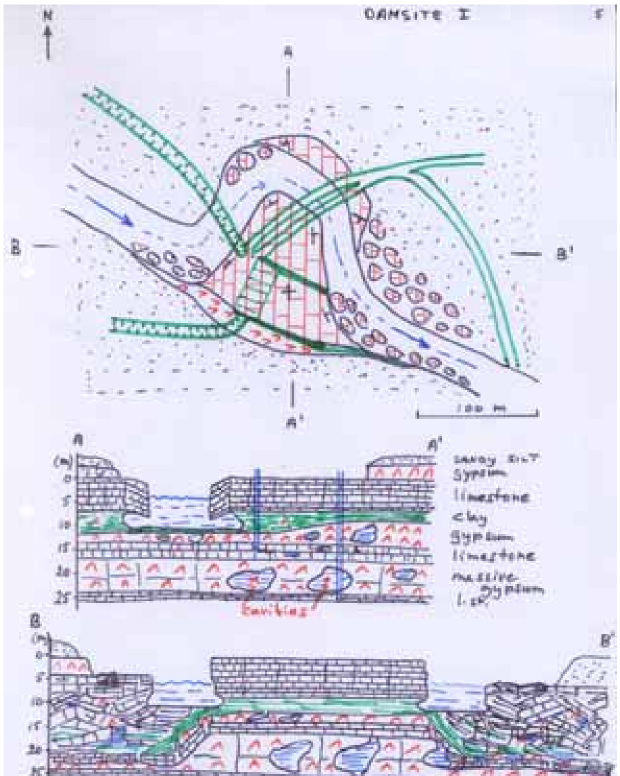


Fig. 6 Sketch of the design of the dam on site I, with cross sections made after the results of the borehole investigation became available.

The aim of the field work was to prepare the site investigation that would follow. The observations made it evident that more information of the discontinuities should come from the boreholes. Since the two sets of joints are subvertical, inclined boreholes are necessary to obtain information on the nature of these. So a borehole program was set up and it was specifically stressed that inclined drilling had to be done. Also borehole packer tests were requested, to determine rock mass permeability.

My main assignment was to assemble the necessary parameters to define the foundation of the dam. I did not expect problems concerning the bearing capacity of the foundation rock. The crucial hazard was the possibility of leakage pathways, either already existing or induced by the changed hydraulic head after construction of the dam. The possibility of piping erosion due to dissolution of gypsum present in the foundation rock should be considered. Possibly a grouting curtain would have to be constructed below the dam to prevent such piping erosion, or to decrease the permeability of the foundation rock. A grouting blanket to seal the pervious dolomite layer might also be necessary. So I summed up my requirements for the ensuing site investigation:

"The data coming from the further site investigation and laboratory tests will be necessary to assess:

- the nature of the foundation rock mass
- the bearing capacity of the rock mass
- permeability of the rock mass
- possibility of piping erosion/dissolution problems
- necessity of grouting
- rippability of the rock
- abrasion resistance of the limestone rock
- type of concrete to be used on site (considering the possibility of reaction of concrete with sulphate-rich water or with dolomite)

Certainly for construction of the dam the gypsum layers on top of the dolomite will have to be removed. This can easily be done by ripping.

The quality of the further site investigation will determine the firmness with which answers can be given to the above mentioned questions concerning the diversion dam construction on this site."

Photo 3 Dam site II with drill rig at position of borehole 25 (Figure 8).

4. The site investigation in April and May 1986

After the preparatory fieldwork, I wrote the requirements for the fieldwork needed on the dam site. The locality was chosen in the expectance of finding a sequence of limestone and underlying gypsiferous stiff clay to found the dam upon. Since also massive gypsum was expected below the site, deep boreholes were prescribed. The cross sections of Figure 6 show the situation that was found after drilling these boreholes. The limestone is at places highly fissured parallel to the bedding and has low RQD values. In one borehole a cavity, of which the bottom was not reached, was encountered (probably in gypsum) at 18.5-20.0 m depth. At this site also geophysical surveying has been performed. Due to the complicated subsurface conditions at the dam site the seismic refraction measurements were unsuccessful. Electric resistivity soundings along flushing canal and desilting basin gave better results, but their interpretation awaited confirmation by drilling (Ruiter, 1986).

After the execution of 6 boreholes it was decided to abandon the site because the quality of the underlying limestone and gypsum was such that an expensive grouting program would be surely necessary to improve the foundation of the diversion dam. The main advantage of the site, from a construction point of view, was the possibility to build the dam without any necessity to divert the river during construction. Now a new site had to be selected, lacking this advantage.

I was phoned at that time and told that the cavities below the limestone layer were so large that when the driller hit the rock with a steel bar, one could hear a hollow sound from below!

5. A new site for the dam: dam site II

A new site was selected one kilometre upstream, with the intention of founding the dam on alluvial stiff clay, which was reported by the Wabi Shebelle Survey to have a thickness of at least 30 m at that location (Figure 5). A comprehensive drilling program was set up with this in mind (Photo 3). An electrical resistivity survey along the proposed dam site was performed as well, by an engineering geophysics student from Delft, Jacob Ruiter, who stayed for several months at the project site (Ruiter, 1986). Unexpectedly, already the first borehole (nr. 1) hit gypsum rock at the unexpected depth of 7.5 m. The following two boreholes (nr. 5 and 7), found the rock surface at depths of 23 and 30 m, respectively (Figure 7). At this stage I was called in, since again problematic rock was involved at the site. From June 19 until June 26 a field visit was made.

It was expected that two days were needed to evaluate the situation at dam site II. Unfortunately the drilling crew had experienced difficulties. Rock coring had become impossible due to lost wire-line rods in borehole 7. Up to now, only information of the above mentioned 3 boreholes was available. Drilling equipment was excellent: a brand new Nordmeyer DSB 1/3.5 trailer-mounted rig. But it would take 3 months before the replacement rotary core barrels would reach the site from Germany.

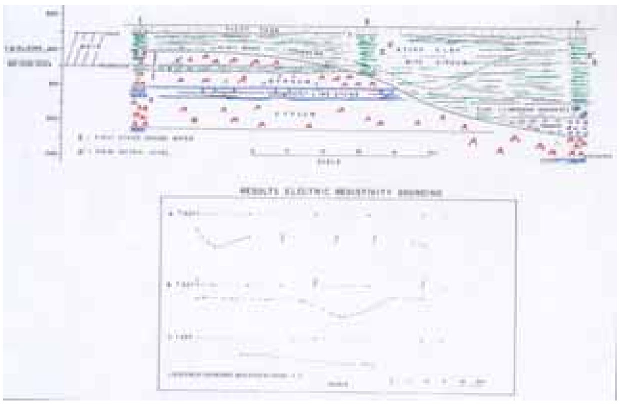


Fig. 7 Interpretation using the first three boreholes and the electrical resistivity survey made at dam site II.

In the meantime I had developed quite a good idea of what could be expected in the subsurface at this new dam location. I envisaged that when the river first encountered this gypsiferous rock, extensive dissolution must have occurred at the river bed. Steep gorges as you see in some limestone terrains must have developed. A karstic moonscape, now filled with alluvial clay deposits. A plan was drawn up to spread the boreholes to a wider area. Fortunately we could still drill up to the rock head, using the shell and auger equipment which was still functioning.

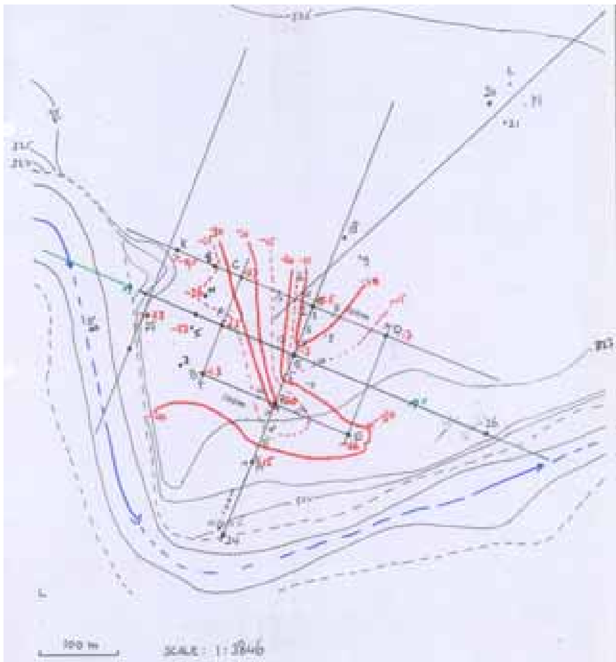


Fig. 8 After the results of the auger drilling through the clay soil became available (red numbers give depth to rock), a depth contour map of gypsum rockhead could be made. This map shows the outline of a steep gorge in the gypsum rock in the subsurface.

Somewhere in August we could examine the results of the drilling exercise. Figure 8 shows just as I expected the outlines of a meandering river valley with steep slopes. The first design had placed the dam structure right on top of a subsurface gypsum rock hill!

The new information now allowed us to define the best position for the dam. This should be a position where there would be a thick layer of alluvial clay separating the foundation of the dam from the gypsiferous rock. In Figure 10a the design of the diversion weir is given. It is shown that the weir is used to divert the water of the river via a desilting basin to the main canal. Also the dikes that are constructed once the dam is finished are indicated on the drawing. Plateau level is 327 m at the site, so the dam is constructed in excavation. The simplified picture of the design is shown in Figure 10b.

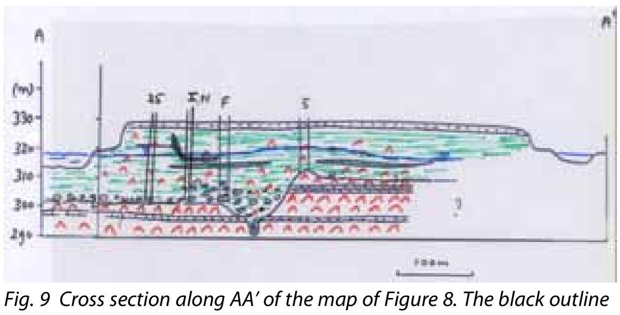


Fig. 9 Cross section along AA' of the map of Figure 8. The black outline shows the location chosen for dam construction.

The location of the dam is along the line of boreholes JIFHG in Figure 8. There, the depth of the gypsiferous rock is at -22 m or more, which is at about 300 m as can be seen in Figure 9. On the cross section in this figure the outline of the dam is shown in profile. Borehole H has been continuously sampled by large diameter Shelby tube. This was done to ensure that any permeable layer within the clay would be detected. In several of the boreholes, permeability tests (falling head) were performed in the clay and piezometers were placed.

Photo 4 120 mm tube core of stiff clay with gypsum veins and nodules (borehole H).

One of the more hazardous findings was that the stiff clay regularly contained gypsum nodules, at some places up to 40 volume percent (Photo 4). This was disquieting, since one of the major questions for the foundation stability of the dam was at this time formulated as: "What would be the effect of the rebound of the soil after excavation in the clay?" This question was related to the fact that the dam would be constructed in excavation. Since the weight of the soil mass removed in the excavation would be higher than the weight of the diversion weir constructed in it, rebound fractures might form in the clay mass and together with any dissolution of gypsum this would have a deteriorating effect on the integrity of the clay directly below the dam structure.

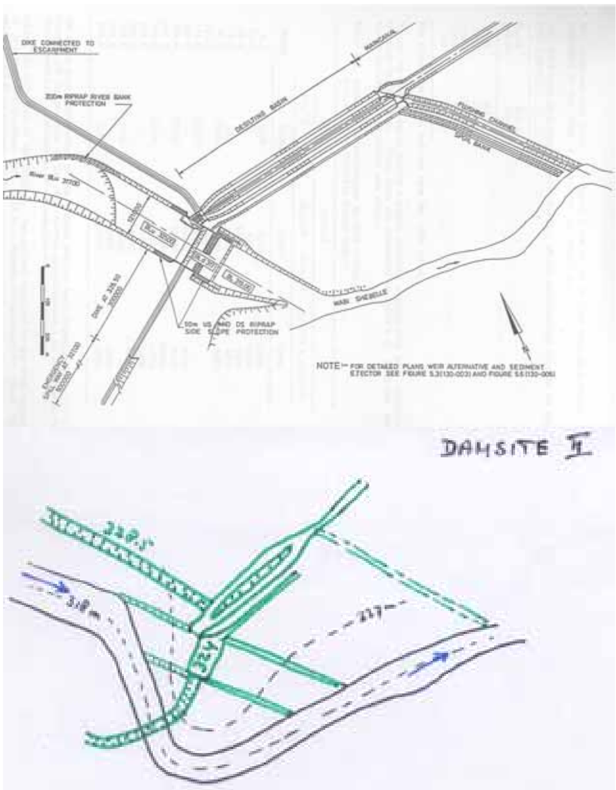


Fig. 10a: design of the diversion weir (top); 10b: sketch map (bottom).

6. Helicopter Survey

My second visit in June gave me extra opportunities to get familiar with the local geology. As I had difficulties to convince people of the necessity of inclined drilling and the performance of packer permeability tests after my first visit in January, in June the client had become very aware of the major problem which the project was facing: the solubility

Photo 5 Sinkholes in the silt cover of the plateau in the West Gode irrigation area.

of the gypsum bedrock! A helicopter was now available to perform an extensive survey over the area. General geological surveying was performed and suitable localities for the construction materials needed for the project were sought for. The work during this week was carried out together with the site geologist, Ato Getu Tilahun.

Three days were needed to survey the area and to find suitable locations for construction materials. On the geological map (Figure 4) the localities visited are given (Nos. 1-8); at each locality a walk-over survey lasting one to two hours was made. Sufficient basalt of good quality is present at site 8. The basalt forms a ridge, estimated to be possibly 500 m long (striking NWW-SEE). If it would be decided to open up a quarry there I advised to put down exploratory boreholes to check for the degree of weathering of the olivine basalt, since other parts of the basalt nearby (sites 2 and 5) are extensively affected by secondary alteration and serious weathering. Site 8 is about 14 km from the dam site.

The best source for sand and gravel is the wadi south of dam site II. The wadis north of the Wabi Shebelle are unsorted with a large amount of fines. In the wadi south of dam site II the fines are washed out at many places. Up to 10 km south of the dam site, the wadi has localities with large accumulations of sand and gravel. The composition is mainly limestone and chert (amorphous quartz). Also some contamination with gypsum is likely to be present. Chert is known to cause alkali-aggregate reactions in concrete, causing cracking after about 10 years. However, since it is clear that sulphate resistant cement has to be used for construction of the dam (to protect the concrete from aggressive sulphate-rich groundwater), alkali-aggregate reactions will be prevented.

Photo 6 Sinkhole about 40 cm deep, developed after rainy period of two weeks.

The proposed irrigation area of West Gode was surveyed from the air and photographs were taken. Especially in the area north of the alluvial deposits (see geological map, Figure 4), where overland flow occurs during the rainy season, many sinkholes have developed (Photo 5). The sinkholes have apparently been formed where gypsum is present in the subsoil. The gypsum dissolves, forming cavities that subsequently collapse. Size of these holes is mostly 0.5-1.0 m (Photo 6). Diameters up to 3 m have been observed. The holes may be very deep (in the order of one to three metres), which can best be observed in January (as was told by the local Somali herdsmen), when the soil is completely dried out and maximum shrinkage has occurred. In the central area of West Gode a bush zone occurs, which is excluded from irrigation. Within the bush the concentration of holes is less but the sinkholes are larger (up to 3 m). This is probably due to the vegetation; the groundwater does not reach the underlying gypsum in such an amount as in the adjacent area next to the bush.

Photo 7 Cave in gypsum rock below a limestone roof of about 2 m thick. This situation serves as a model for the situation at dam site I (Figure 6).

Another interesting observation was made at site 7 (Figure 4). From the air this seemed to be a quarry in limestone. It turned out, however, to be a limestone layer about 2 m thick overlying a thick, massive gypsum layer. An entrance into a cave was present. The cave, developed in the geological past by the dissolution of gypsum, was 50 m long, 20 m wide and 5 m high. Apparently the limestone layer could span a cavern of this size. Adjacent to the cave a collapsed cavern is present (Photo 7).

Photo 8 Vast area with huge limestone slabs downstream of dam site I. Probably this rock is the remnant of a collapsed roof above a dissolved gypsum layer (Figure 6). Somali pastors guard their herd.

After I had seen the collapsed roof of the cave, it became completely clear to me what situation I had encountered at dam site I. I concluded that the limestone plate abutting the river was in fact a remnant roof rock above an elsewhere completely dissolved and collapsed gypsum layer as illustrated in Figure 6. This explained the loose large slabs of limestone lying upstream and downstream of the rock (Photo 8).

The helicopter survey, which lasted three days, allowed a better understanding of the local geology. I could now draw a sensible cross-section through the area, which is given in

Figure 11. The interpretation of the limestone at dam site I as being Mustahil Limestone on the geological map was certainly wrong. It contains hardly any fossils and is a limestone and dolomite intercalated in the gypsiferous rock formation.

Everywhere in the area there is evidence of the high dissolution capacity of gypsum, which forms the main geotechnical problem of the Gode Irrigation Project.

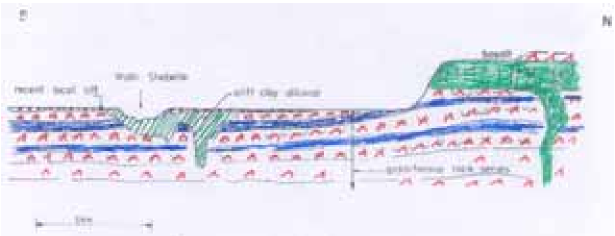


Fig. 11 NS geological cross section through the valley of the Wabi Shebelle.

7. The engineering geological situation

Figure 9 gives schematically an interpretation of the geological situation at dam site II. The alluvial sequence is silty sand underlain by stiff clay with on average 10-20% of gypsum nodules. At a lower level pebbles of limestone are present in this clay, and the lowest layer consists of limestone gravel. The bedrock consists of a sequence of gypsum with clay and limestone intercalations. The top 15 m along the rock-alluvium surface may be badly weathered gypsum and limestone.

If the diversion dam would be founded upon stiff clay, a layer of about 10-15 m of clay is sealing-off the probably highly permeable zones below that. This situation seems to be acceptable. What has to be considered to assess the risk of this situation are the possible water transporting zones in the clay and their permeability. A second consideration should be the possible dissolution of gypsum crystals in the clay and its effect on settlement of the structure and maybe even on leakage.

If the dam or dykes would be founded upon the gypsum rock series, such as below borehole 1 and 5 (Figure 8 and 9), then permeable zones should be detected. Dissolution of gypsum may lead to excessive settlement of the structures. Concentration of dissolution in certain zones results in tubular water pathways (piping) below the structures with consequently excessive leakage.

In Figure 12 the implications of such a situation are illustrated. Two measures to decrease the hydraulic head are indicated: a blanket in front of the dam (top drawing) and a grout curtain (middle and lower drawing). Both a blanket in front of the dam and a grouting curtain ensure a reduction in hydraulic velocities and uplift pressures under the dam. Grouting is common practice in dam construction. It is used to improve bad rock conditions in the foundation (sealing of fissures to improve bearing capacity and to reduce seepage). In general, if the rock is reasonably sound and no improvement of the bearing capacity is necessary, then a sealing blanket fulfils the function of reducing seepage velocities. In the case of gypsum rock, a classic curtain design would probably even be hazardous. If the grouting has been done by injection through boreholes, there is always a chance that water comes directly in contact with gypsum and starts dissolving, right under the dam structure itself (Figure 12).

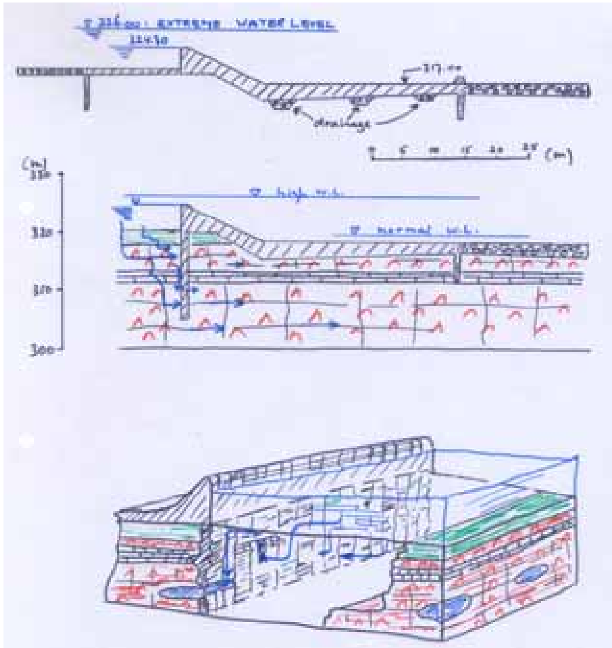


Fig. 12 Sketch of the design of the diversion weir and hypothetical drawings of the groundwater flow paths in the case of a design with a grout curtain. The disadvantage of a grout curtain is the possibility of dissolution taking place right at the dam structure itself.

At dam site II it seems that the problem of dissolution of gypsum might be controllable since the foundation of the dam will be on stiff clay which seals off the fresh water from the gypsum rock below (Figure 13). But considering the entire project, major difficulties can be envisaged. Fresh water will flow in canals over large stretches of area directly underlain by gypsum rock. This requires an analysis of the impact of this problem on the project.

8. The engineering of hydraulic structures on gypsiferous rock: dissolution problems

The Ogaden near Gode is underlain by a sub-horizontal sequence of gypsiferous rock, which at Gode has a thickness of at least 250 m. In the irrigation area on top of the gypsiferous rock there may be alluvial sediments of varying thickness, deposited by the Wabi Shebelle. These are mainly stiff red clays that contain gypsum grains. Thickness can amount up to 37 m. Also so-called recent local deposits consisting of silty sand with local clay seams may be present above the gypsiferous rock (Figure 4).

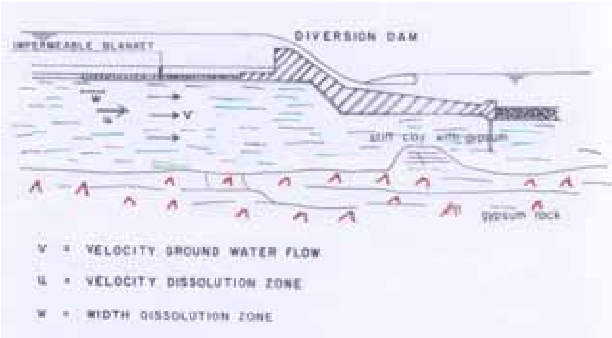


Fig. 13 Design of the diversion weir on stiff clay with impermeable blanket.

The gypsiferous rock series is a sequence of layers of varying lithology. Massive gypsum layers, several metres in thickness, alternate with gypsiferous marls and clays and limestone layers (Figures 7 and 11). At every locality where rock containing gypsum comes into contact with fresh water, dissolution may occur (Photo 9).

As a matter of fact, it may seem rather surprising that the water of the Wabi Shebelle has a low salinity, flowing for such a long distance through gypsiferous rock. This is due to the sealing effect of the clay-rich alluvium that was likely deposited by the river during an earlier phase of relatively high sea level.

Photo 9 Collapse of a slope along the river bank in gypsiferous rock. Dissolution is clearly an element determining the stability of the slopes.

The main problem with foundations of dams or other hydraulic structures on gypsiferous rock is how to prevent excessive dissolution to occur. Gypsum is one of the rock-forming minerals which tend to dissolve in fresh water. Compared with carbonate rock such as limestone, it dissolves twice as fast and more importantly, much more material can be taken into solution (about 1 600 times or more) (James & Kirkpatrick, 1980). The rate of dissolution depends on the composition of the dissolving water, the ambient temperature, the flow velocity of the dissolving water, and the contact area of the gypsum exposed to the dissolving water (Table 1).

Table 1 Comparison of dissolution rates for different soluble rocks, using the model of Figure 14: retreat of a soluble surface beneath flowing water. Solubility and dissolution rate constants from James & Lupton (1978).

Type of rock	Temperature (°C)	Flow rate (m/s)	C_s Solubility (kg/m^3)	K_d Dissolution rate (m/s)	Solution rate of surface (m/year)
Rock Salt (NaCl)	10	0.05	360	0.3×10^{-5}	16.2
Limestone ($CaCO_3$)	10	0.05	0.015	0.4×10^{-5}	7×10^{-4}
Gypsum ($CaSO_4 \cdot 2H_2O$)	10	0.05	2.5	0.2×10^{-5}	0.068
same	23	0.5	2.5	4×10^{-5}	1.36
Gypsum with NaCl dissolved in water	10	0.05	10	0.2×10^{-5}	0.27
same	23	0.5	10	4×10^{-5}	5.44

The dissolution rate increases with increasing temperature, flow rate, sodium chloride content of the dissolving water and with decreasing grain size of the soluble grains (i.e. increasing contact area). James & Lupton (1978) have determined the dependence of dissolution rate on these parameters, which allows us to make estimates of dissolution rates.

With respect to problems of dissolution in the foundation rock or soil of hydraulic structures, it is important to realise that the exact location where dissolution may take place is depending highly on the subsurface geology and especially on the specific flow paths the groundwater follows in the subsurface. Prediction of dissolution rates and quantities therefore depends on the (hydro)-geological model used and thus on the quality of the basic data gathered from the subsurface (boreholes, samples, in-situ test results).

8.1 Analysis of foundation diversion dam

Three possibilities were presented for the dam foundation during the site investigation of diversion dam site II: 1) foundation on gypsiferous rock with a grouting curtain; 2) foundation on gypsiferous rock with a blanket; and 3) foundation on stiff clay with a blanket. It was outlined that a foundation on stiff clay would give the least problems with respect to gypsum dissolution. Fortunately, a site has been found where this situation appears to exist. Figure 13 shows a simplified cross-section, and some of the parameters used in the analysis below are given in this figure. In the design the dam is thought to have a blanket upstream, made of an impermeable synthetic foil. This foil is also present under the structure itself, to protect the structure against chemical attack on the cement and concrete by the sulphate-rich groundwater.

Following James & Lupton (1978), the stiff clay with gypsum grains which forms the foundation soil mass of the dam may be treated as a 'particulate' deposit. Two factors with respect to dissolution are important to assess:

(1) The width of the dissolution zone (the distance over which the ground water is capable of dissolving gypsum);

(2) The migration rate of this dissolution zone towards the structure. James & Lupton (1978) give two formulae to solve this problem:

$$T = \frac{\rho \beta l_0}{K_d C_s \alpha} \qquad (1)$$

And:

$$u = \frac{Q / A}{n \beta l_0^3 \rho / C_s + 1} \qquad (2)$$

where:

T = the characteristic time for complete dissolution (s)

u = migration rate of the dissolution zone (m/s)

ρ = density of gypsum = 2 320 kg/m^3

l_0 = dimension of a gypsum particle (m)

β = volume coefficient of a particle (volume = βl_0^3)

α = area coefficient of a particle (area =αl_0^2)

n = number of particles per unit volume (1/m^3)

K_d = dissolution rate (m/s)

C_s = solubility (saturated concentration) (kg/m^3)

Q = volumetric flow rate (m^3/s)

A = flow area (m^2)

In our case the gypsum is present as grains of sand to gravel size (mostly sand size) and the volume of gypsum in the stiff clay is visually estimated from the boring samples. If we consider the gypsum grains as spheres, the volume and area coefficients become:

$\alpha=\pi$ and $\beta=1/6\pi$ (area=$4\pi r^2=\pi l_0^2$; volume = $4/3\ \pi r^3 = 1/6\pi l_0^3$).

We have now an estimate of gypsum volume. The velocity of groundwater flow under the dam can be approximated from Q/A, the flow velocity, which is the hydraulic conductivity K (m/s) times the hydraulic gradient i:

$$v = \frac{Q}{A} = Ki \qquad (3)$$

This allows us to calculate the migration rate of the dissolution zone towards the dam (Figure 13). Using the volume estimate of gypsum in the clay, equation (2) becomes:

$$u = \frac{v}{\frac{vol\%}{100} \cdot \frac{\rho}{C_s} + 1} \qquad (4)$$

With the help of the characteristic time T (equation 1) we can calculate the width of the dissolution zone (which is the distance fresh water coming in contact with gypsum can travel until it is saturated and cannot dissolve gypsum anymore):

$$W = T.v \qquad (5)$$

The parameters needed to estimate u and W for the stiff clay are volume, size, dissolution rate K_d and solubility C_s of the gypsum particles in the groundwater, flow velocity v (i.e. the hydraulic conductivity K of the clay), and the hydraulic gradient i (which depends on the design of the dam structure).

The parameters K_d and C_s are not exactly known at the diversion dam site. K_d depends on temperature and flow velocity, but also on the sodium chloride content of the groundwater. This dependence has been studied by James & Lupton (1978), and estimates of K_d can be obtained from their study. The solubility C of gypsum of the site is determined on water samples taken of the fresh water of the Wabi Shebelle and of the groundwater in the boreholes at dam site II. The solubility in fresh water was 0.13 g/l of dissolved gypsum ($CaSO_4.2H_2O$) and of water from the boreholes 11.5 g/l water. That much more gypsum can dissolve in the groundwater is explained by the presence of sodium chloride in the groundwater. In the river water the sodium chloride (NaCl) content was 0.06 g/l; in the groundwater it was 10.4 g/l. This data can be compared to the literature values given in Table 1.

Electrical conductivity measurements, which can be done in the field using simple hand held devices, give a good indication of the salt content in water: The Wabi Shebelle water has a conductivity of 0.4 mmohs/cm; the groundwater in the alluvial clay has a value of 10.8 mmohs/cm. Using these values and Figure 10 from James & Lupton (1978) estimates of K_d applicable to the situation at dam site II have been made.

At borehole 5 a permeability of $5.4x10^{-7}$ m/s was obtained at the boundary of stiff clay and gypsiferous marl (at a depth of 8.00 m). The hydraulic gradient of a design with a blanket of 75 m length (Figure 13) leads to a hydraulic gradient of 0.05. It follows that $v=2.7x10^{-8}$ m/s. Say the stiff clay has 40% gypsum grains of 5 mm diameter size, of spherical shape. Then (K_d from James & Lupton (1978): $5x10^{-5}$ m/s; C_s = 10 kg/m^3; ρ_{gypsum} = 2 320 kg/m^3):

$$T = \frac{2320.\frac{\pi}{6}.5x10^{-3}}{5x10^{-5}.10.\pi} = 386.7 \times 10^3 s$$

$$W = T.v = 0.01m$$

$$u = \frac{2.7x10^{-8}}{(0.40).\frac{2320}{10}+1} m/s = 0.009 m/year$$

(using equations 1, 5 and 4).

The width of the dissolution zone is 1 cm and this will migrate with a rate of 1 cm/year towards the structure. These values are very low, but they are depending highly on the flow rate, and thus on the reliability of the values found for the hydraulic conductivity. To show the influence of some parameters the same calculation has been made varying the parameters. If we use for example groundwater with no NaCl, the width of the dissolution zone can become of the order of 1 m. The migration velocity of the dissolution zone remains of the order of cm per year.

Apparently with the prevailing parameters the width of the dissolution zone is acceptably low, and its migration rate towards the dam too. The largest influence on the result has the flow velocity, which is determined by the hydraulic gradient and the permeability (hydraulic conductivity). In the preliminary design used for site investigation purposes at dam site II, a 75 m blanket is provided for. This would correspond to a hydraulic gradient of 0.05. The example calculation shows that the width of the dissolution zone is then 0.01 m, which is migrating with a velocity of 0.01 m/year towards the dam structure. This would mean that after 100 years only a 1 m wide zone would have been affected by dissolution. If we decrease the blanket length, however, also the hydraulic gradient increases. A reduction of the blanket length to 30 m will increase the migration rate (i=0.2, $K=10^{-6}$ m/s; $v=2x10^{-7}$ m/s, 10% gypsum, 5 mm size, spherical; w=0.04 m, u=0.3 m/year). The dam would then be reached after 100 years.

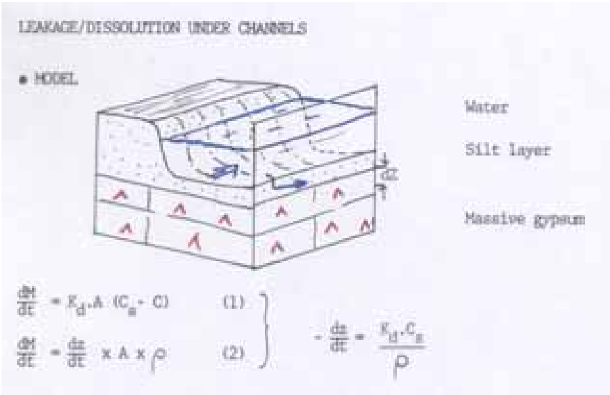


Fig. 14 Concept of dissolution of unlined irrigation canal above gypsum rock.

To fully rely on calculations such as above is unwarranted. Much more should be known about the distribution of hydraulic conductivity in the stiff clay and on the actual dissolution process. A continuous sample of the foundation clay has been obtained from borehole H (Figure 8) and no high permeability layers have been observed.

Another important aspect to consider is how the soil behaves after the gypsum nodules and veins in it have been dissolved. What will be the change in geotechnical properties? Probably the permeability and deformability of the leached zone increases and its shear strength will decrease. Excessive differential settlement may occur, the amount of which is likely to be determined by the (irregular) distribution of gypsum in the clay.

Given these uncertainties a conservative design has been adopted for the dam, with appreciable blanket length, to keep the dissolution zone far away from the dam structure.

8.2 Effects of dissolution below the canals and the irrigation area

During the helicopter survey made in June 1986 it was observed that at many places where gypsum is present near the earth surface (for example near the camp site), sinkholes of varying size (0.1-3 m diameter) are present (Photos 5 and 6).

It may be expected that dissolution along the main canal and primary and secondary canals also could take place giving rise to leakage and surface subsidence.

The situation below a canal can be analysed as follows (Figure 14, from James & Lupton (1978)):

$$-\frac{dM}{dt} = K_d A(C_s - C) \qquad (6)$$

where M is the amount of gypsum dissolved at time t, A is the surface area of gypsum in contact with fresh water and C is the concentration of dissolved gypsum in the water (since this is constantly refreshed this may be taken as zero).

The linear retreat of the gypsum surface below a canal may also be written as -dZ/dt (see Figure 14) and the amount of gypsum dissolved per unit of time:

$$-\frac{dM}{dt} = \frac{dZ}{dt}.A.\rho \qquad (7)$$

Combining (6) and (7), taking C=0:

$$-\frac{dZ}{dt} = \frac{K_d C_s}{\rho} \qquad (8)$$

Again K_d and C_s have to be estimated and depend on flow rate, temperature and concentration of sodium chloride in the groundwater.

Example 1. In the case of a silt layer between massive gypsum and the canal (Figure 14):

Take $K_d=10^{-4}$ m/s and a low hydraulic gradient, then $v \sim 10^{-5}$ m/s. If the silt contains salts, then the groundwater may have dissolved sodium chloride (NaCl) in it and estimates of $K_d=5x10^{-5}$ m/s and $C_s=10$ kg/m^3 may apply. This gives (Equation 8):

$$-\frac{dZ}{dt} = \frac{5 \times 10^{-4}}{2320} = 2.16 \times 10^{-7} = 6.8 \text{m / year}$$

Example 2. If no sodium chloride is present, then $K_d=10^{-5}$ m/s and $C_s=2.5$ kg/m^3 and

$$-\frac{dZ}{dt} = \frac{2.5 \times 10^{-5}}{2320} = 1.08 \times 10^{-8} = 0.34 \text{ m / year}$$

Example 3. If free flowing canal water is flowing directly over a gypsum rock surface, with a velocity of 0.5 m/s, then at 23 °C: $K_d=4x10^{-5}$ m/s (James & Lupton, 1978). Since hardly any sodium chloride is present in Wabi Shebelle water, $C_s=2.5$ kg/m^3.

$$-\frac{dZ}{dt} = \frac{(4 \times 10^{-5}).(2.5)}{2320} = 4.3 \times 10^{-8} = 1.36 \text{m / year}$$

These values are high and show that protection against contact with fresh water is necessary. They also explain why sinkholes may develop rapidly in the wadis of the irrigation area during the rainy season, the only time when water is flowing over these areas. Consider a locality where silt containing some sodium chloride salts is present above gypsiferous rock (the conditions of example 1). Say a localised flow of water during the rainy period occurred for a period of two weeks, then a retreat of the underlying gypsum surface of 6.8 m/year x (2/52) = 0.26 m could occur. This is sufficient for a small sinkhole to develop (Photo 6).

These calculations can be considered as examples. Comparable calculations for different rock types are shown in Table 1. The calculated solution rates are regularly confirmed by observations and reported in literature (James *et.al.*, 1981). Using actual site measurements of flow velocities and electrical conductivity measurements of the water and groundwater, these could be tailored to the real situation.

8.3 Design of monitoring program

Before the actual construction starts, many data can be gathered and monitored that would aid in the refining of the prediction of dissolution behaviour of gypsiferous rocks and soils under hydraulic structures in the irrigation area. Monitoring of piezometric levels in the piezometric stations at the dam site, together with temperature and electrical conductivity measurements will give information on hydraulic conductivity and also whether there is any fluctuation of the salinity of the groundwater. Also the variation in C (solubility) over the site and in the irrigation area can be obtained this way (by similarly measuring the electrical conductivity of the groundwater in other boreholes).

During and after construction it is highly advisable to monitor the groundwater, to be aware of any changes in conditions. After construction, the groundwater should be controlled regularly. Any groundwater coming from the drains of the diversion dam should be saturated with gypsum. A sudden decrease (or change) in salinity (as measured by electrical conductivity) may indicate that somewhere a dissolution zone has reached the dam and a leakage zone (possibly in the blanket) should be found to prevent further damage. Any increase in the salinity of the water flowing in the canals in the irrigation area indicates that somewhere the canal water is in contact with gypsum. The location may be found by tracing increasing conductivity values and repair measures can be taken.

9. Conclusion

The presence of gypsum in the subsurface has serious implications for the engineering of this irrigation project. It should at all times be ensured that no undesired dissolution can take place during the engineering life of the constructions. The engineering solution envisaged at the time of design of this project in 1986 was to place synthetic impermeable foil as a blanket in front of the dam and the protective dykes. Also below the dam foil had to be placed, to protect the concrete from aggressive sulphate groundwater. And last but not least: impermeable foil should be placed at the bottom of the main canal and irrigation canals, everywhere gypsum rock is near the surface. It might also be considered to line the irrigation canals with alluvial clay to prevent contact of gypsum rock with the irrigation water.

Furthermore the recommendation was made to develop a monitoring system, based on the measurement of electrical conductivity of the water in the irrigation system. Any increase in conductivity would signal the dissolution of gypsum.

Epilogue

After Nedeco finished the design of the irrigation works at Gode, I was of course interested to know when the construction work would start. But I did not hear much from Ethiopia. Over the years I used this interesting site investigation case in my lectures and halfway through the nineties I heard from Ethiopian students that this project was still considered to be executed, also by the new government that had taken over from the 'Military Government of Socialist Ethiopia'. But finally I thought that the project had never come off.

When Michiel Maurenbrecher asked me to write a paper on this case for the Newsletter, I started to have a look at our new aid of the past two years: Google Earth. And then I could not believe my eyes! I definitely saw the contours of the canal starting at the location of dam site II and a pattern of irrigation canals in the West Gode region (Figure 15). I was surprised and also a bit puzzled, because things do not look right on this Google image. I do not see any green fields and also the canals do not seem to contain water…

Then I started searching the internet and found bits and pieces of news. The completion of the construction of the dam in December 2001 had taken over 13 years. It appears that digging of the main canal started in 1999 and that IACO (Canada) was called in to assist with re-designing the project. Apparently one of the delaying factors during dam construction was the import of sulphate resistant cement.

Fig. 15 Google Earth image of the West Gode irrigation area near Gode (Bale). Yellow scale bar is 5 km.

I have not found more details on the construction. On the Google image I see no quarry development in the basalt rock, so I assume that it was chosen to build a concrete weir instead of a masonry one. To the southwest of the dam I see something like a sand and gravel pit from which the aggregate could have been derived. But all this is speculation.

Apparently only the West Gode area is irrigated, which is some 7 050 hectares of land. In 2003, 350 hectares of this land were distributed to peasants. Each peasant owns about 1 hectare of land.

Then disaster struck on October 17, 2006. A torrential rain caused massive overflowing of the river. I have seen a map that showed that the complete 12 km width of the alluvial valley was flooded, including the irrigated agricultural area of West Gode. It is reported that high-water indicators that are used to record river levels, have been submerged by the water bursting over muddy banks and flooding more than 17 000 hectares of land. The Wabi Shebelle river doubled in volume within two days. This disaster took the life of more than 20 people and numerous cattle in the Gode area.

The irrigation canal and the dam are reported to have been damaged. Today efforts are underway to repair the damage and reinstall the irrigation works. I will check Google Earth regularly to see evidence of a change...

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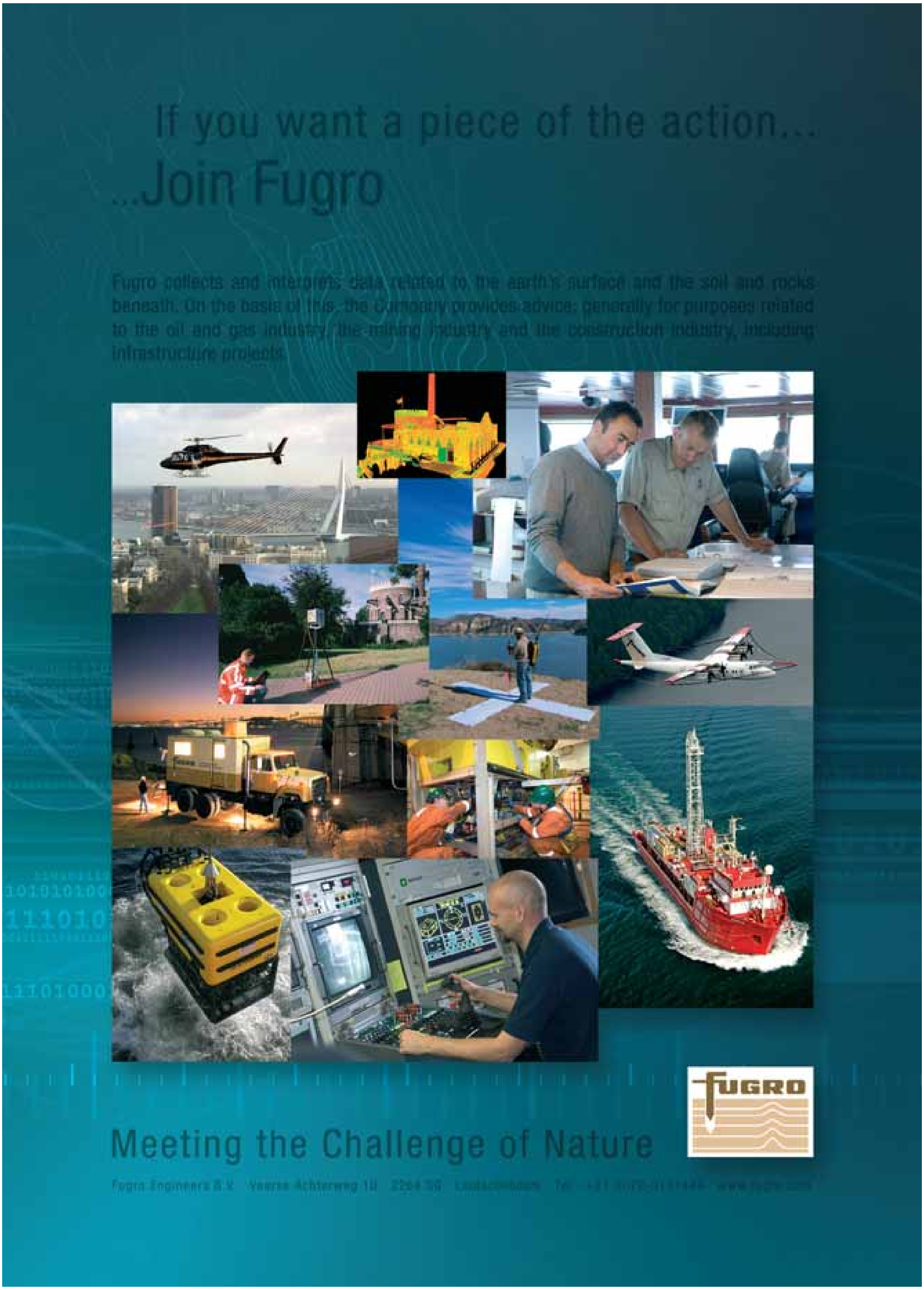
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Meeting the Challenge of Nature

Engineering geologist abroad

Xander van Beusekom, Senior Engineering Geologist, Advanced Geomechanics

From a ship, sailing south along the Northwest Shelf in Australia in some 'mild rough weather', it is my turn to show what it is like to be an engineering geologist abroad. But let me start with the beginning.

Drilling vessel Markab.

Before I ended up here in Australia I worked at Fugro for several years, during which I travelled around the world to lead offshore site investigations. Several of these travels brought me to Australia and I must say that it increased my urge to have a better look at 'Terra Australis' (land down under) formerly known as 'New Holland'. Since business in both the offshore and mining industry is booming in Western Australia it wasn't difficult to get a job and I ended up at a consultancy firm called Advanced Geomechanics. It is a relatively small company (which was a nice change coming from a multinational like Fugro) that is specialised in offshore foundations in carbonate soils.

The first project I took on was one for which I did the preparations for the actual site investigation a year ago and it was my task now to work with these results. The best part was that the report was a typical engineering geological report, in this case on a pipeline shore crossing through calcarenites. A real challenge again for several reasons. First of all, I hadn't really worked with rock since my M.Sc. thesis. Secondly, the material consisted of highly irregular cemented material, which made it almost impossible to assign useful parameters to the different formations.

The other interesting aspect of the work is being a client representative on board during offshore site investigations. And as could be expected, most of the offshore work is still being executed by Fugro which makes it almost feel like home… except I am on the other side of the table now. Having been a site manager for so long is a big plus when working as a client representative. You know how the work is done, it is easy to notice when things are not going according to plan and you know if the optimum has been achieved with the available resources at the time. And it is always good to see some familiar faces. So far, I have been lucky enough to work for BP in West Papua, for Inpex in the Timor Sea and currently for Woodside on the Northwest Shelf.

The offshore geotechnical world is already relatively small, but in Perth it is extremely small. This means that during the day you can have a meeting where the arguments fly back and forth over the table, and in the evening you are having a beer with that same person simply because the person is a former colleague or a good friend. Or you can be at a party having a drink with your direct competitor, while the two major contractors for offshore work are doing exactly the same thing. All in all a very pleasant atmosphere to work in. It is just a matter of separating business and pleasure.

Drilling vessel Mariner.

But of course work is not the only reason I have chosen Perth. I wanted to see something of the country as well. The first thing to realise is that Perth is the most remote city of the world. The closest city is Adelaide, which is a trip of approximately 2 500 km. Even though the amount of people living there is well over a million, it still feels like a village. I have already been able to drive down from Darwin to Perth several years ago, but there were 2 things still high upon my wish list: The Kimberley and Uluru in the Red Centre. By the time I got back from my job in West Papua in January, it was

Great Central Highway.

not really possible to go to The Kimberley because of the wet season. So the choice was easily made to head for the Red Centre.

During this trip, my mate and I travelled for almost 8 000 km (of which 2 000 on dirt road) plus 72 km on a tow truck to get back to civilisation. This allowed us to see the Super Pit (biggest open pit goldmine in the world), camels, emus, Uluru (Ayers Rock), where we towed aboriginals back to their community, have seen sunset and sunrise at Uluru and the sunset at Kata Tjuta. I swam in a water hole in Kings Canyon, camped underground in Coober Pedi (opal capital of the world), tried the 'wichelroede', made a scenic flight above Wilpena Pound in the Flinders Ranges and visited Adelaide. We watched the Admirals Arch and the Remarkable Rocks on Kangaroo Island, where we also got a very close look at Australian sea lions, New Zealand fur seals, possums, wallabies and koalas. We crossed the Nullarbor (a road of 2 000 km with hardly any sidetracks), watched dolphins at Esperance during a boat trip and had a look at Wave Rock on the way back.

All in a 3 weeks journey where the temperature ranged between 20 degrees at the coast up to 45 degrees in the Red Centre. And I can assure you that 20 degrees feels bloody cold if you just got out of 45 degrees!

And apart from all this: the fantastic weather in Perth, the beautiful beaches and the wineries nearby. All in all not a bad place to live.

Uluru (Ayers Rock).

Estimating 'efficiency-permeability' of upstream cut-off membranes for dams

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Introduction

Thirty years ago I claimed the British had made a Dutchman out of me. The last project I was involved with was designing flood defences along the north bank of the Thames downstream from the Thames barrier. The firm of consulting engineers, called Binnie & Partners (they later amalgamated with Black & Veatch, American consultants by which name they are now known), were prominent in the field of water works, sewerage and water storage. A few years back I wrote an article in the Ingeokring Newsletter about the river Thame, the dirtiest river of England, which was another project I was then involved with during my three years with Binnie & Partners.

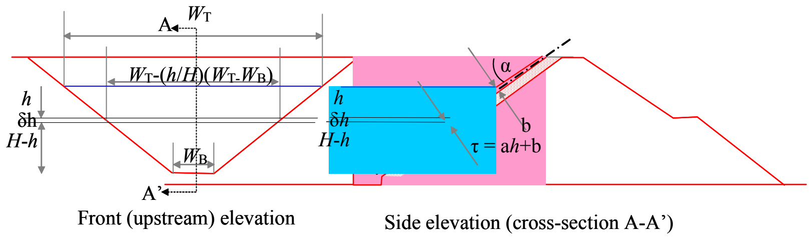
One task I was asked to perform was to look at the effectiveness of upstream membrane cut-offs on earth and rock fill dams as a design study for a proposed dam at Romsley, near Birmingham. Traditionally such dams have an internal clay core. Much of the literature I examined on the subject came from ICOLD (International Commission on Large Dams) congresses. Part of the data that was obtained and is referring to reinforced concrete upstream membranes, is summarised in Table 1 (the original table for the design study contained data on 24 dams and included asphalt as well as reinforced concrete membranes). Data was provided on membrane thicknesses, height of reservoir levels and leakage. To make a comparison with regard to their 'efficiency' there was a problem to reduce this data to a more meaningful comparison. This would be the 'permeability' of the membrane. There were high narrow dams and wide lower dams of similar membrane design giving different leakages. Two dams with comparable membranes were the Salt Springs Dam in California and the Paradela Dam in Portugal. Both dams had similar concrete membranes which were thick and narrow in width at the toe and thinned and widened towards the crest of the dam to, more or less, resist the high hydraulic head at the toe and the low hydraulic head at the crest. The literature stated that the Paradela Dam membrane was based on the design of the Salt Springs Dam. Of interest was the Salt Springs Dam's significant drop in leakage with a partially filled reservoir. The analysis presented below shows this did not result from an increase of overall membrane permeability as it was thought that the higher hydraulic pressures may influence the opening of concrete construction joints.

Analysis: derivation of a leakage equation for a variable membrane

The analysis adapts Darcy's equation for flow through an upstream triangular membrane with a linearly variable thickness. The method assumes that there is a full potential drop in water head across the membrane which is reasonable if the permeability of the filter and rock fill beneath the membrane is of several orders greater than that of the membrane. Hence, to calculate the permeability of the membrane the measured flow at the downstream toe of the dam is assumed only to have passed through the membrane. The literature did not seem to provide a method for solving for 'k'. Width, thickness and hydraulic head could be easily related with a linear equation with height. Hence, by integrating the leakage across a horizontal strip of membrane δh high at depth h having a surface area $\delta A = w\delta h$ where $w=f(h)$ and having a hydraulic gradient h/τ where τ is the thickness of the membrane at depth h, and $\tau=f(h)$. Leakage through a strip δh at depth h is from Darcy's equation $q=k(h\gamma/\tau)w\delta h$. This expression is then integrated over the height and width of the upstream membrane, the solution for this is provided in Figure 1 and the equation box.

This was then used to estimate the 'efficiency-permeabilities' for a number of dams from which remarkably the Paradela Dam (after repairs!) and the two lake levels at Salt Springs Dam gave very similar values. The lower 'efficiency-permeabilities' at the other dams reflect improved membrane design. A web search today reveals both Paradela and Salt Springs dams required repairs to their membranes and that Cethana Dam, also of similar design but an order of permeability less than the first two dams, had performed consistently well.

Figure 1 shows the model used for the analysis to which Equation Box 1 refers. The derivation first assumes that the width at the base of the dam is $W_B=0$. The width at reservoir level is taken as W_T. This results in equation 1 in the equation box. Allowing for a finite width at the base of the dam W_B, results in equation 2 where $r=1-W_B/W_T$. Equations 1 and 2 are then used to determine k based on the leakage and geometrical data of the dams. The results are given in Table 1. Figures 2 and 3 are taken from websites showing how the dams appear today.



$w = W_T[1-(h/H)(1\text{-}W_B/W_T)]=W_T[1-rh/H]$

$r = 1\text{-}W_B/W_T$

Fig. 1 Model used to solve for overall membrane permeability k.

Fig. 2 Paradela Dam, Northeast Portugal (source: cnpgb.inag.pt/gr_barragens/gbingles/ParadelaIng.htm).

Fig. 3 Salt Springs Dam, central California (source: concise.britannica.com/ebc/art-73277/Salt-Springs-Dam-a-rock-dam-is-on-the-North).

Table 1 'Efficiency-permeability' of dams with upstream reinforced concrete membrane cut-offs.

Dam	Height [m]	Crest length [m]	Slope (upstream/ down-	Geology	Fill	Type membrane/ foundation cut-off	Membrane			
							Thickness [m]		Leakage [l/s]	Permeability, k [m/s][1]
							Crest	Base		
Cethana (Tasmania, 1971)[A]	110	213	1.3/1.3	Quartzite	Rock, compacted	Reinforced concrete (RC)/8 m grout curtain	0.3	0.6	35	$2x10^{-8}$
Paradela (Portugal, 1957)[B]	110	525	1.3	Granite, weathered	Rock, sluice dumped	RC/30 m grout curtain	0.3	1.1	3020/315[2]	$1.3x10^{-7}$
Salt Springs (California, 1931)[C]	100	396	1.3/1.4	Granite, sound	Rock, sluice dumped	RC/1.5 m wide 1 to 6 m deep trench & 15 m deep	0.3	1.1	113/556[3]	$2x10^{-7}/3x10^{-7}$
Lower Bear 1	75	293	1.3/1.4	Granite, massive		RC/no information	0.3-0.75	0.6	57	$4.7x10^{-8}$
Lower Bear 2 (California, 1952)[C]	46	264	1.0/1.3	Granite, massive			0.3	0.6		
Vilar (Portugal)[B]	55	210	1.1/1.3	Granite and	Rock, sluice	RC/no information	0.3	0.6	130	$7.4x10^{-8}$
Quioch (Scotland, 1955)[D]	38	320	1.3/1.4	Rock, sound	Rock, rolled sluiced	RC/2 m concrete wall & 12 m grout curtain	0.3		22	$5x10^{-8}$
La Sassiere (France, 1959)[E]	30	300	1.4/1.4	Moraine	Rock, dumped	RC/2m wide, more than 15 m deep grout curtain	0.35		12	$5.4x10^{-8}$

[1] Value for k is back-calculated; [2] Before/after repair; [3] partly filled/full reservoir
[A] Wilkins (1973); [B] Da Palma Carlos, *et al.* (1973); [C] Steele & Cooke (1958, 1969); [D] Roberts (1958); [E] Destenay & Lemay (1961)

Equation Box 1 Derivation of equations used to solve for dam upstream membrane permeability.

$$Darcy's\ equation:\ \delta q = k.i.\delta A$$

$$\delta A = (W_T - \frac{h}{H}(W_T)) \times \delta h\ \operatorname{cosec}\alpha\ ,\ i = \frac{h}{\tau} = \frac{h}{ah+b}$$

where $a = \tan\beta / \sin\alpha$

substituting for i and δA

$$\delta q = \frac{k\,W_T (H-h)\operatorname{cosec}\alpha}{H} . \frac{h}{a\,h+b} \times \delta h$$

$$\delta q = \frac{k\,W_T \operatorname{cosec}\alpha}{H} . \frac{h\,(H-h)\,\delta h}{a\,h+b}$$

$$\int_0^Q \mathrm{d}q = \int_0^H \frac{k\,W_T \operatorname{cosec}\alpha}{H} . \frac{h\,(H-h)}{a\,h+b}\mathrm{d}h$$

put $ah+b=\tau$, $h=(\tau-b)/a$, $a\mathrm{d}h = \mathrm{d}\tau$, $\mathrm{d}h = \frac{\mathrm{d}\tau}{a}$

$$\int_0^Q \mathrm{d}q = \frac{K}{a^3}\int_0^H . \frac{(Ha(\tau-b) - (\tau-b)^2)}{\tau}\mathrm{d}\tau = \frac{K}{a^3}\int_0^H . \left[(Ha+2b) - \tau - \frac{(Hab+b^2)}{\tau}\right]\mathrm{d}\tau$$

$$Q = \frac{K}{a^3}\left[\left((Ha+2b)\tau - \frac{\tau^2}{2} - (Hab+b^2)\ln\tau + C\right)\right]_0^H$$

$$Q = \frac{K}{a^3}\left[\left((Ha+2b)(ah+b) - \frac{(ah+b)^2}{2} - (Hab+b^2)\ln(ah+b) + C\right)\right]_0^H$$

$$Q = \frac{K}{a^3}\left[\left([(Hab + \tfrac{3}{2}b^2 + h(a^2H+ba) - \tfrac{1}{2}a^2h^2 - (abH+b^2)[\ln(aH+b)+C\right)\right]_0^H$$

$$Q = \frac{K}{a^3}\left[(a^2H+ba)h - \frac{a^2}{2}h^2 - (abH+b^2)\ln(ah+b) + C'\right]_0^H \text{ where } C' = C + Hab + \tfrac{3}{2}b^2$$

$$Q = K\left[\left(\frac{H^2}{2a} + \frac{Hb}{a^2}\right) - \left(\frac{bH}{a^2} + \frac{b^2}{a^3}\right)(\ln(aH+b) - \ln(b))\right]$$

$$Q = kW_T\operatorname{cosec}\alpha\left[\frac{H}{2a} + \frac{b}{a^2} - \frac{b}{a^2}\left(1+\frac{b}{Ha}\right)\ln\left(\frac{aH+b}{b}\right)\right] \qquad (1)$$

for a triangular (front elevation) dam

$$Q_r = kW_T\operatorname{cos}ec\alpha\left[\frac{H}{2a} + r\frac{b}{a^2} - \frac{b}{a^2}\left(1+r\frac{b}{Ha}\right)\ln\left(\frac{aH+b}{b}\right)\right] \text{where } r = \left(1-\frac{W_B}{W_T}\right) \qquad (2)$$

W_B = is the width at the base of the dam for a trapezium shape (see figure 1 front elevation).

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Review of the 10th IAEG Congress, September 2006, Nottingham (UK)

Leon van Paassen

From September 6-11, 2006 the 10th Congress of the International Association for Engineering Geology and the environment (IAEG) was organised in Nottingham, United Kingdom. Hosted by the UK branch of the IAEG, the Engineering Group of the Geological Society of London, the conference was attended by more than 400 participants, of which 11 from our Dutch division Ingeokring, and some 800 abstracts were received from all over the world.

Part of the Dutch delegation to Nottingham. From left to right: Robrecht Schmitz, Michiel Maurenbrecher, Niek Rengers, Robert Vuurens and Leon van Paassen.

On the first day of the conference the yearly IAEG council meeting was held, presided by Niek Rengers, honorary member of Ingeokring. During this meeting the IAEG executive committee presented their yearly reports on activities and finances and all the technical committees of IAEG and joint technical committees of IAEG, ISRM, ISSMGE gave an overview of their current activities. The new IAEG president, Fred Baynes, was elected together with 7 new regional vice-presidents.

A proposal by the presidents of IAEG, ISRM and ISSMGE to found a Federation of International Geo-Engineering Societies (FIGS), was supported by a narrow majority of council members. The objective of FIGS is to raise awareness for the Geo-engineering profession and coordinate activities in overlapping interests between the members of the federation. Brian Hawkins, editor of the IAEG bulletin, showed some nice quoting statistics and encouraged all members to quote and write papers for the bulletin. Finally the location of the IAEG conference for 2010 was chosen to be Auckland, New Zealand. For more details on these IAEG issues go to the website www.iaeg.info.

Next day the conference started with an opening lecture by John Burland, giving an overview of the current status of Geotechnical Engineering (as a merge of engineering geology, soil mechanics and rock mechanics) and emphasising the necessity of theoretical models, laboratory experiments and practical case histories to make predictions about future behaviour. The next 4 days many papers were presented in 3 parallel sessions and all technical committees organised their yearly meetings. Meanwhile continuous poster sessions prevented boredom and gave opportunity for discussions. Some of my personal highlights were presentations about using rock models to predict the behaviour of ancient historical buildings in Rome and the design of a new metro in Napoli, TC meetings on building stones and historical monuments and presentation of an upcoming publication on deserts and a youth forum. The main goal of each conference: to gain new ideas, meet old friends and make new ones, was very well established during this one. Especially the conference dinner and final day of field excursions added to the already good atmosphere.

Summit of Mam Tor on the left forms the back scarp of a major landslide (bottom right) containing 3.2 Mm^3 of sliding material, which started sliding 300 to 400 years ago and still continues to slide at an average rate of 100 mm/year.

Road damage on the Mam Tor landslide.

One European field trip was organised to see several tunnelling projects in the Alps and three field trips were made in the UK, one of them visiting the highlights of the Peak district: limestones, shales and sandstones near Castleton and the major landslide on the slopes of Mam Tor, including a climb to the summit. Apart from the nice scenery we got detailed information about the local geology and geo-engineering features. After additional visits to the caves of Nottingham and Sherwood Forest and a drink with a part of the Dutch conference delegation at Nottingham Castle it was time to go back home fully inspired to continue our ordinary jobs.

So continue writing papers for both the Ingeokring Newsletter and IAEG Bulletin, become a member of IAEG and join one of the technical committees and I'll see you in Auckland in 2010.

Piping phenomenon in earth dams: case histories

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M. Alvarez Grima, Project Manager, MTI Holland, Kinderdijk, The Netherlands

Introduction

This paper reviews the main factors that caused piping in five earth dams built in Cuba in the late 1980's. As a common geological feature we can mention that the dams were built in alluvial basins consisting of bedded granular particles. These soils are characterised by a high permeability. In the situation where the continuity of the layers under the dam foundation is extended downstream, the likelihood of occurrence of piping is very high.

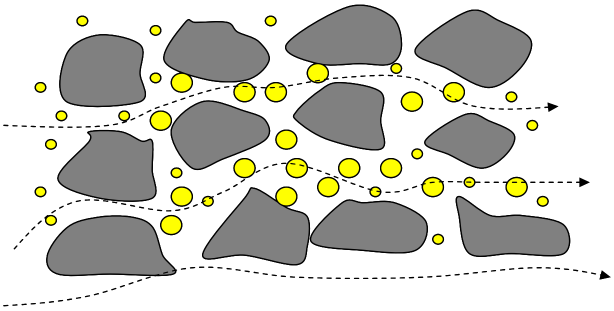

Fig. 1 Piping phenomenon: transport of fine soil (fine sand) amongst coarse particles (gravel).

In the five historical cases analysed, piping took place in the dam foundations and not in the embankments. The principal reason is that dam embankments are made from selected materials which are subjected to rigorous compaction and control tests whereas dam foundations are dictated by Mother Nature.

It was noticed that the position of the dam axis contributed considerably to the occurrence and acceleration of the piping process. In some cases piping might have been avoided by making a better selection of the dam axis location.

The distribution of continuous and permeable layers beneath the dam foundations which connected the reservoir with areas downstream was a common feature as well. In most of the projects the selection of dam axis was based not only on geological and geotechnical considerations, but also on economical aspects such as the transportation distance of suitable borrow materials for the construction of the embankments.

Piping occurred in sandy gravels. It is worth mentioning, however, that piping can also take place in dispersive soils. These soils are characterised by a dissolved sodium content of the pore water which is higher than in ordinary soils. Dispersive soils usually have a high exchangeable sodium content. They rapidly erode, forming tunnels by a process in which the clay particles go into suspension in slow moving water (colloidal erosion) damaging earth dams. This type of piping is often called chemical piping.

Piping mechanics and identification

Piping occurs when water seeps through the soil and tends to transport the soil particles with it. This generally occurs when the pore sizes are larger than the soil particles (Figure 1). This phenomenon occurs in sites composed of soils characterised by a high permeability. As a consequence, a change of water flow rate occurs and the piping process commences, giving rise to an outcrop of water seepage. As a result of this, the finest soil particles are washed away with the water flow. The diameter and the depth of the resulting hole will enlarge in time, bringing into existence the piping through which the reservoir and lower dam slope toe are connected (Figure 2).

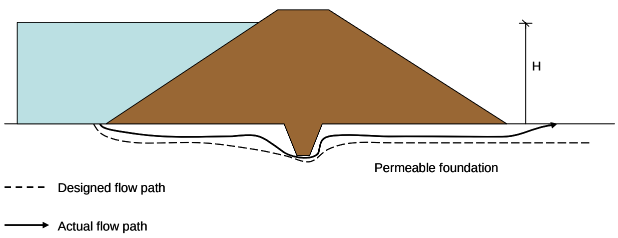

Fig. 2 Schematic representation of seepage under dam foundation causing piping.

Piping can occur along a spillway and other conduits through the embankment. Sinkholes that develop on the embankment are signs that piping has commenced. A whirlpool in the reservoir surface may soon follow and then likely a rapid and complete failure of the dam. Emergency procedures, including downstream evacuation, must be rapidly implemented if this condition is observed.

The failure can be shown as a sudden raising of the foundation at the apron of the embankment. This undermining is generally caused by a gradual increase of piping, and it can lead to the appearance of ruptures such as longitudinal and transversal cracks on the embankment (Figures 3 and 4). The most dangerous ruptures are transversal cracks, in as much as if they appear on the same level with the banked-up water level, a new outlet of reservoir water starts to form, and it could lead to a complete catastrophic failure.

Fig. 3 Longitudinal view of dam axis.

Seepage can cause slope failure by creating an increase in water pressure or by saturating the slope. The pressure of seepage within an embankment is difficult to determine without proper instrumentation. A slope which becomes saturated and develops slides may be showing signs of excessive seepage pressure.

Engineering geological properties of investigated piped soils

Properties of the piped soils in the five dams were analysed. In all cases the soil consisted mainly of non-cohesive granular particles: fine and medium sand (Figure 5). They had a vertical permeability of K=0.5-10 m/day. The horizontal permeability varied between 5-50 m/day. In fact, this is characteristic for Cuban alluvial terraces where the dams were built.

Fig. 4 Transversal crack created on dam axis.

By analysing the geological site conditions of the dams, it was observed that the affected dike stretch was founded on permeable alluvial deposits consisting of heterogeneous soil with continuous horizontal stratification. This caused a natural path beneath the dike between upstream and downstream. Seepage took place as a result of the existing hydraulic gradient (i). Table 1 lists the values of the actual critical gradient of the five dams analysed.

$i=\Delta H/L$

ΔH = difference between reservoir and downstream water levels

L = water running length from reservoir to water outlet lower dam

Table 1 Critical hydraulic gradient.

Earth dam	Hydraulic load (ΔH), [m]	Water running length (L), [m]	Critical gradient (i)
Case A	3.2	45	0.07
Case B	6.0	60	0.10
Case C	15.0	115	0.13
Case D	20.0	150	0.13
Case E	18.0	300	0.06

It can be seen in Table 1 that the piped soils have a critical hydraulic gradient smaller than 0.15, with an average value of about 0.10. Table 2 lists the smallest values found in the literature at that time, these values were used in the design of the dams as an admissible gradient to prevent piping.

Table 2 Admissible hydraulic gradient used in the designs.

Type of soil	Admissible gradient
Clay	1.20
Sandy clay	0.65
Gravelly sand	0.45
Medium sand	0.38
Fine sand	0.29

Observations and analysis

The engineering properties analysed in the piped soils were: grain size distribution, hydraulic conductivity, admissible hydraulic gradient, critical gradient, and geological and engineering/geotechnical conditions of the sites.

The analysis of the results revealed that the piped soils had more than 50% of granular particles and the transported particles consisted mainly of fine and medium sand (Figure 5). Moreover, we could verify that for the piped soils with a low value of hydraulic load (ΔH=3-6 m) and a low hydraulic gradient (i=0.06-0.13) piping can occur; provided that granular layers are laying underneath the dam foundation. The presence of fine particles (0.2-2.0 mm) in large amounts can also be an indicator for the possible occurrence of piping based on the grain size distribution of the piped soils.

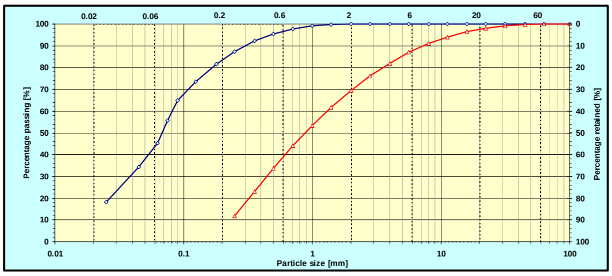

Fig. 5 Grain size distribution range of piped soils.

We could observe that for these soils the assumed design values for admissible gradient were generally ranging between 0.20 and 0.40 and they were based on experience and scarce data available from literature at that time (Table 2).

Despite of the fact that the designers took into consideration some preventive measures to reduce seepage underneath the dams, in practice these measures were not effective at all. Some of them were incomplete or inefficient, resulting in locally high seepage through the foundation. The use of sheet piles did not avoid high seepage, as it was thought.

Table 3 Main factors that caused piping.

Dams	Factors
Case A	Sandy layer cut by steel pile, but laying on fissured limestone
Case B	The impervious cut-off did not cut up the sandy layer at affected zone
Case C	Reinforced concrete structure placed (water in-take) on granular layers, with impervious apron and drainage structure, but design critical gradient 0.25 compared to 0.13 (the actual gradient measured)
Case D	Grout curtain made from cement in granular foundation, but reservoir was filled before grout curtain was finished, seepage was concentrated in affected stretches
Case E	Deep impervious layer was not cut up with steel piles used. However, in the zone where bentonite mix was applied, there was no problem

Also permeable layers beneath the foundations were not cut up totally. In case B piping took place at low water level in the reservoir (about 5 m below the normal water level). Also we could observe clearly the tunnels created by the piping mechanism. The diameter of the tunnels observed varied between 15 cm and 35 cm. Also the presence of flow of fine sand was observed.

Besides the above mentioned aspects, it can be said that some geological and geotechnical conditions contributed to the occurrence of the piping phenomenon too: dam location (axis) in meander zones and location of concrete structures on permeable sandy layers laying parallel to the underground water flow. Also preventive measures were not totally completed before the filling of the reservoir. Table 3 summarises the main factors that caused soil piping.

Considerations about some measures taken against piping

The main considerations are as follows:

- Drainage structure with an inverted filter acts as pore water pressure dissipater. However, it does not avoid seepage, which is considered high in permeable foundations. This measure might be feasible when the dam foundation is less granular and when the soil permeability is not too high.
- An apron or a partial cut-off (or a combination of both) lowers the hydraulic gradient since the water running path is enlarged. It was found that in practice this measure was not totally effective.
- Grout curtains made from cement and bentonite are effective when the soil porosity allows it, but requires finishing 3 to 4 profiles before the reservoir is filled up, avoiding in this way seepage concentration at the middle stretches. This measure requires time and control measures while it is executed.
- Soil-cement walls built by means of a trench or sheet piling together with pouring of blended bentonite-cement is considered a safe measure when permeable layers are cut.

Concluding remarks

- In the five analysed dams we could corroborate that the admissible gradient reported in the literature at that time was not safe.
- Piping took place in permeable foundations consisting of fine and medium sand having a high permeability.

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The activities of the Netherlands' National Committee on Large Dams (NETHCOLD)

Ir. J. van Duivendijk, NETHCOLD Chairman

In this article, a short description is given of the aims of the International Commission on Large Dams (ICOLD) and those of its member representing The Netherlands, the Netherlands' National Committee on Large Dams (NETHCOLD). In the definition of ICOLD all dams having a height of 15 m or more are large dams. However, also 10 m high dams having a crest exceeding a certain length or a reservoir exceeding a certain volume or area are considered to be large dams. It implies that The Netherlands has some 10 large dams. Worldwide, the number of large dams is in the order of 50000. In recent years ICOLD members have broadened their scope to the so-called small dams. Some national committees have even taken the trouble to delete the word 'large' in their committee name. Obviously, our sea defences (sea dikes) and flood embankments (river dikes) have much in common with small dams.

ICOLD, having its secretariat in Paris, has been founded in 1928 in order to create a forum for discussion and exchange of knowledge and experience in the field of dam design and construction. Anybody who, in one way or another, is involved in dam building and water management may become active in ICOLD. In the year 2006, 85 countries were a member of ICOLD through their national committees. In those national committees one usually finds representatives of governmental departments, technical institutes, consulting engineers and contractors; but also financing institutions, environmentalists and insurers may play an active role.

NETHCOLD was founded in 1969. The implementation of the Delta Project was at its peak at that time. In an article published in 'De Ingenieur' (November 22, 1968) the writer stated:

"Also in The Netherlands dams have been built and are being built which fit the ICOLD definition. In this respect one should not only refer to our estuary dams in the delta region of Rhine and Meuse but also to the dams closing off tidal inlets like the Zuyderzee, the Braakman and the Lauwerszee. During construction of (for instance) tidal power plants (France, England, Canada, Russia) conditions are encountered which are comparable to those prevailing in our own coastal and deltaic regions. One could imagine that the experience and know-how, obtained during design, model and field investigations and construction of the Delta Project, after its completion…could be useful for providing expertise and carrying out assignments in other countries. But in that case one must make sure that others are aware of the Dutch skills and technology in this field. One way of reaching this goal is the membership of ICOLD".
Partly as a consequence of this article NETHCOLD (a 'stichting') was founded and, at that time, placed under the supervision of KIVI (some years ago KIVI did not any longer think this supervision to be necessary). Members can be governmental entities as well as commercial enterprises and individuals. At present two departments of Rijkswaterstaat, Delft University of Technology (Fac. CiTG), institutes (Delft Hydraulics, IHE-UNESCO), engineering consultants and contractors are corporate members as well as some individuals. In the Statutes it is laid down that in case the Chairman is somebody employed by Rijkswaterstaat, the Secretary must come from a commercial enterprise and vice-versa.

Within ICOLD, exchange of experience and know-how takes place in the so-called technical committees, during the annual technical symposia and during the congresses which are held every three years. A list of technical committees can be found at ICOLD's web site (www.icold-cigb.org) under the heading ICOLD -> Technical Committees.

ICOLD members meet each other first of all during the said congresses (some 1 500 participants) but also during the committee meetings which are held during the Annual Meetings. During such annual meetings the host country will, apart from a technical symposium, also organise workshops and study tours to dam projects in the host country and neighbouring countries.

A new development in ICOLD is the creation of regional 'clubs'. NETHCOLD is member of the Club for European National Committees. This club also has a number of working groups. They discuss and formulate rules on such topics as floods, risk assessment, dam safety and the European Water Directive but also on seismic criteria for dams. The work in these groups is of great interest for the Netherlands as the basin-wide approach plays a significant role. An example is the analysis of the role of dams in European rivers during flood periods.

NETHCOLD members have always been very active in various technical committees. In most cases the findings and conclusions of such committees are laid down in so-called

Bulletins. During the past 40 years NETHCOLD has had an important input in some 15 Bulletins. Currently there are some 100 Bulletins available (see ICOLD's web site under publications).

The input from the Netherlands covers a wide range of topics such as the use of bitumen in fill dams, hydraulics for dams, sedimentation of reservoirs, dam safety, dams and floods, the role of dams in the development and management of river basins, public awareness and education and, last but not least, environmental issues. Members of NETHCOLD have chaired some committees for shorter or longer periods.

Apart from this committee work, NETHCOLD has also organised study tours to the Netherlands (1983 and 1995) and taken the initiative (in 1993) for a special study group to get ICOLD members, when designing a dam, more focused on a river basin-wide approach and sustainable solutions.

NETHCOLD, in collaboration with KIVI, Waterbouwdispuut and IHE-UNESCO, from time to time organises symposia on matters related to dams (1985, 1990, 1995 and 2001). The symposia have resulted in the publication of two books: 'De Keerzijde van de Dam' (edited by Rijkert Knoppers and Walter van Hulst) (1995) and 'Dams and Dykes in Development' (2002).

Finally, it should be mentioned that NETHCOLD members translated the ICOLD Technical Dictionary on Dams (1990). This translation is now available for the public at VSSD Delft in its series of Lecture Notes.

Reference is made to NETHCOLD's website for more specific information and details: www.nethcold.org.

Dams in the Western USA

Erik Schoute, Royal Boskalis Westminster nv

Introduction

While working in the United States, I spent some time vacationing in the Southwest of the USA in the summer of 2003 and in the Northwest of the USA/Southwest of Canada in the fall of 2004. During these two road trips, on which occasions we clocked 6 000 and 10 000 kilometres on the car's odometer, we came across some very interesting hydropower projects. As I was travelling with a friend who graduated at TU Delft as a Civil Engineer, we certainly could not just drive by without getting a closer look into some of these projects. In this article, some of my own impressions about the dams will be presented together with background information collected while on the road and from the internet.

2003 Road Trip: Glen Canyon Dam and Hoover Dam

During the trip in the summer of 2003 which started and ended in Las Vegas, the route went clockwise through Nevada, Utah, Colorado, New Mexico, Arizona, California and back into Nevada. The magnificent scenery, good roads and road side services make the Southwest of the USA a particularly great region to travel around. In this area of the USA, a vast amount of National Parks can be found (to name just a few that were visited: Zion, Bryce Canyon, Capitol Reef, Glen Canyon, Canyon de Chelly, Grand Canyon, Lake Mead and Death Valley). Two of these National Parks are also home or close to some of the most spectacular hydropower projects in the world: Glen Canyon Dam and Hoover Dam.

Glen Canyon Dam

Glen Canyon Dam is part of the Glen Canyon National Recreation Area, which covers over 5 000 km^2 in Arizona and Utah. The dam is situated in the Colorado River near Page, Arizona.

Glen Canyon Dam is a concrete thick gravity arch dam ('thick' meaning with a base thickness to structural height ratio of 0.3 or greater) with a height of 216 m and a crest length of 475 m. Width of the dam is 7.5 m at the crest and 91 m at the base. See Figure 1 for an impression of Glen Canyon Dam and Bridge.

The remote location in a part of the Colorado River valley called Marble Canyon was selected by a group of Bureau of Reclamation engineers and geologists, who explored the area in a period from 1946 to 1948. The site met several criteria: the area forming the basin could contain an immense amount of water, the canyon walls and bedrock foundation were strong and stable enough to safely support the high dam and a large source of good rock and sand was available at nearby Wahweap Creek.

Fig. 1 Looking south, with Lake Powell at the bottom of the photo, then Glen Canyon Dam and Glen Canyon Bridge (source: en.wikipedia.org/wiki/Glen_Canyon_Dam).

Glen Canyon was carved by differential erosion from the Colorado River over an estimated 5 million years. The Colorado Plateau, through which the canyon cuts, arose some 11 million years ago. Within that plateau, there are layers of rock from over 300 million years old to the relatively recent volcanic activity. Navajo Sandstone, the result of compressed sand dunes, forms the canyon walls at the dam site and throughout most of Lake Powell. The sandstone is remarkably uniform and homogeneous over wide areas and nearly identical samples can be obtained from areas separated by many kilometres. Navajo Sandstone is buff to reddish, fine to medium grained, and soft to moderately hard. It is massive with pronounced cross-bedding and commonly indistinct horizontal bedding. The sandstone is moderately porous and highly absorptive, owing to the high capillarity created by the small size of intergrain pore spaces.

Fig. 2 Glen Canyon Dam and Bridge (note the white band on the Navajo Sandstone, result of the low level of the reservoir behind the dam) (source: author's photo).

Construction of the dam started in 1956 with the creation of water diversion tunnels. By 1959, nearby Glen Canyon Bridge was completed (see Figure 2), allowing trucks to deliver equipment and materials for the dam and the new town of Page. Concrete placement started around the clock on June 17, 1960. The last bucket of concrete was poured on September 13, 1963 (see Figure 3 for typical bucket size). Over 3.7 million m^3 of concrete was used. Construction of the dam, bridge and all other adjacent facilities cost \$314 million (adjusted for inflation) and 18 people's lives.

Next, turbines and generators were installed from 1963 to 1966. The plant consists of eight 155 500 horsepower Francis turbines that generate more than 1 300 MW of electricity with each of the 40 ton steel shafts turning at 150 rpm. With all eight generators operating at full output, over 55 million litres of water will pass through the power plant's penstocks each minute.

Fig. 3 Bucket which was used to pour concrete (source: author's photo).

Upon completion of Glen Canyon Dam the Colorado River and its tributaries began to back up, no longer being diverted through the tunnels. The newly flooded Glen Canyon formed Lake Powell, stretching for over 300 km upstream. As the lake filled over the years, seismic activity in the area increased as the ground shifted beneath the increasing weight of the water. It took 17 years for the lake to rise to the high water mark, which was reached on June 22, 1980. Since then the reservoir level has fluctuated wildly and is currently less than 60% of its total capacity. The reservoir is currently suffering from prolonged drought. The reservoir has been evaporating as much as ever, and the long-submerged side canyons are being revealed. This lower level can be clearly seen as the so-called 'bathtub ring' (white band) in Figure 2.

Fig. 4 Aerial photo of Hoover Dam with the U-shaped power plant in front and Lake Mead in the back (source: photo.net/photo/pcd2882/hoover-dam-aerial-91.4.jpg) .

Hoover Dam

Almost 2 200 km and 5 days later we had arrived at the last stop before leaving from Las Vegas again: the giant Hoover Dam. Hoover Dam is situated in another NPS administered National Recreation Area: Lake Mead, which stretches out over an area of 750 km^2. Hoover Dam is situated in the Black Canyon of the Colorado River valley, on the border between Nevada and Arizona (the actual border runs over the crest of the dam), some 50 km southeast of Las Vegas.

In total, 21 000 people worked on the project which was completed in 1936. Construction costs of the dam were \$676 million (adjusted for inflation) and 114 people's lives were lost over the total project period from 1922 to 1942. Hoover Dam is a similar type dam as Glen Canyon Dam, a concrete thick gravity arch dam, with a height of 220 m and a crest length of 380 m. Width of the dam is 14 m at the crest and 200 m at the base.

The foundation and abutments mainly consist of andesite breccia and welded ash-flow tuff intruded by latites. The first concrete for the dam was placed on June 6, 1933 and until May 29, 1935 3.3 million m^3 of concrete was poured. Hoover Dam is the tallest solid concrete dam in the Western Hemisphere. There is one taller dam in the USA (Oroville Dam), but this is not a solid concrete dam. Worldwide, Hoover Dam ranks number 20 in the list of tallest dams. Lake Mead, which was formed by the construction of Hoover Dam, is the largest reservoir in the USA.

The power plant is located in the U-shaped structure at the base of the dam (see Figure 4). Each power plant wing is 200 m long and there are 17 main turbines. The plant has a total capacity of 2.99 million horsepower and 2 079 MW.

Hoover Dam generates more than 4 billion kWh per year of low-cost hydro-electric power, which is enough to serve 1.3 million people in Nevada, Arizona, and California. Contrary to what many people think, only 4% of Las Vegas' massive energy consumption is generated by the Hoover Dam hydropower plant.

Fig. 5 Spectacularly placed transmission masts in the Black Canyon near Hoover Dam (source: author's photo).

2004 Road Trip: Hungry Horse Dam, Grand Coulee Dam, Chief Joseph Dam and Diablo Dam

In October and November 2004, I made a road trip through the Northwestern states of the USA (Washington, Oregon, Idaho, Wyoming and Montana) and the Western provinces of Canada (Alberta, British Columbia). This area can be called 'dam country' without exaggerating too much, as can be

Fig. 6 Columbia River basin with dams (source: www.nwd-wc.usace.army.mil/report/colmap.htm).

seen in Figure 6 which (by coincidence) is also more or less the area that I visited on this road trip. The picture shows the Columbia River basin, an area with roughly the size of France. The Columbia's heavy flow and its large elevation drop over a relatively short distance give it tremendous potential for hydropower generation. It is the largest hydroelectric power producing river in North America, with 14 dams in the Columbia River alone, in the United States and Canada.

3 of the 4 dams that were visited are in the Columbia River basin: Hungry Horse Dam (Hungry Horse, Montana), Grand Coulee Dam (Grand Coulee, Washington) and Chief Joseph Dam (Bridgeport, Washington). The other dam is the Diablo Dam in the Skagit River, just east of Seattle.

Hungry Horse Dam

On the road from Kalispell to Glacier National Park, I stopped along the road for a view of the Hungry Horse Dam. This dam is a concrete thick arch dam, located in a narrow canyon in the Flathead River, near Columbia Falls in Montana. Construction started in 1948 and at the time of its completion in 1953, the dam was the third largest dam, and the second highest concrete dam, in the world. Currently, it is the largest dam in Montana and nr. 11 in the list of largest concrete dams in the USA.

Fig. 7 View on Hungry Horse Dam from the road between Columbia Falls and Glacier National Park (source: author's photo).

The 172 m high dam is a variable-thickness concrete arch structure with a crest length of 645 m. Crest width is 10 m, base width 98 m. The dam and appurtenant works contain 2.4 million m^3 of concrete. The spillway is the highest morning-glory structure in the world. Water cascading over the spillway rim drops a maximum distance of 149 m. The capacity of the spillway is 1 415 m^3 per second, and the reservoir has a total surface area of 96 km^2.

The dam is used for power generation, using 71 MW generators. The generator capacity was upgraded in the 1990´s to 107 MW each, for a total plant capacity of 428 MW. The dam's foundation consists of Siyeh Limestone in regular beds ranging from a few centimetres to metres thick, made highly insoluble by impurities. Cut by one major and several minor shear zones, and one major bedding-plane slip.

Grand Coulee Dam

After Hungry Horse Dam, I travelled through Canada for two weeks and after that I went south again and visited the beautiful northeastern part of the state of Washington. After crossing the border between the USA and Canada, I went to the giant Grand Coulee Dam. This is a concrete gravity dam, situated in the Columbia River near the towns of Grand Coulee and Electric City. The dam is part of Lake Roosevelt National Recreation Area, administered by the National Park Service. The dam itself is operated by the U.S. Bureau of Reclamation.

Grand Coulee Dam, with its three powerhouses, forms the largest hydro-electric project in the USA and the fourth largest in the world. It is the largest concrete structure in the USA, with 9.16 million m^3 of concrete used. Crest length of the dam is 1 592 m, while its height is 168 m. Base width is 152 m and crest width is 9 m.

Construction of the original project started in December 1933, as part of the Columbia Basin Project for irrigation of desert areas of the Pacific Northwest and for the production of electricity for the then rapidly expanding cities of Spokane, Portland, Seattle and Tacoma. Foundation rocks at the dam site consist of hard, sound, massive granite varying from coarse-grained in the right abutment to fine-grained porphyritic in the left abutment.

Construction finished in 1942, and numerous modifications followed in more recent years. The first power was generated in 1941 and the last of the original 18 units (of the initial two powerhouses, one at each side of the dam's spillway) started production in 1950. Between 1966 and 1974 the dam was expanded to add the Third Powerhouse. The original dam was modified for the Third Powerhouse by a 357 m long, 61 m high forebay dam along the right abutment approximately parallel to the river and at an angle of 64 degrees to the axis of Grand Coulee Dam. This involved demolishing 80 m of the northeast side of the dam. The addition accommodates six new generators. These new turbines and generators, three 600 MW and three 805 MW

Fig. 8 Grand Coulee Dam, with Right and Left Powerplants at both sides of the spillway and Third Powerhouse and Forebay Dam at the left side (source: U.S. Bureau of Reclamation).

units, are some of the largest ever produced. The expansion was completed in the early eighties and made the Grand Coulee Dam one of the largest hydro-electric producers in the world. In total, it has 3 power plants with 33 generators, and a pumping-generating plant, all together generating a total of 6 809 MW of electricity. The lake which formed during construction of the dam is called Franklin D. Roosevelt Lake and stretches over 243 km upstream.

I was able to tour the dam, with a descent into the Third Powerhouse by a ride in a glass elevator on top of the forebay penstocks in the Third Powerhouse (elevator visible at left side in the aerial photo in Figure 8). In the Third Powerhouse, the world's largest indoor cranes are active for maintenance work on the 6 gigantic Francis turbines (see Figure 9).

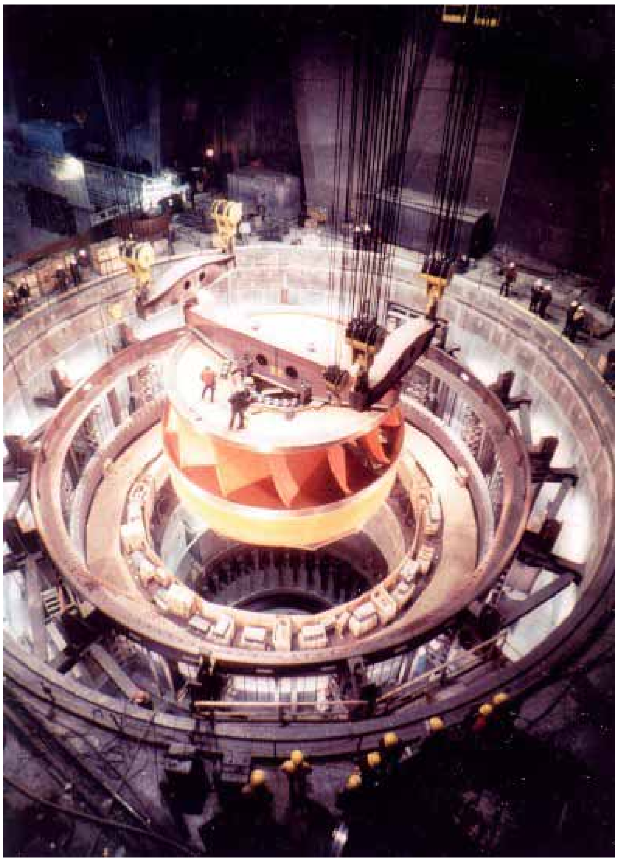

Fig. 9 Francis turbine in Third Powerhouse (source: www.usbr.gov/power/data/sites/grandcou/grandcou.html).

Chief Joseph Dam

A little further down the road from the Grand Coulee Dam I stopped to take a look at Chief Joseph Dam, which is also situated in the Columbia River near the town of Bridgeport in the state of Washington.

It is a concrete gravity dam with a height of 72 m and is designed, constructed and operated by the U.S. Army Corps of

Fig. 10 View on Chief Joseph Dam and Rufus Woods Lake (source: www.nws.usace.army.mil/publicmenu/images/cjdam/chief_joseph_dam_large.jpg).

Engineers. It is also the Corps' largest dam and the second largest hydropower producing dam in the USA (behind Grand Coulee Dam). Its capacity is 2 600 MW, and average annual production is 12 million MWh, which is enough electricity to meet the needs of all homes, businesses and industries in the Seattle metropolitan area. It has the longest straight-line powerhouse in the USA, stretching 621 m.

Construction of the dam started in 1949 and 9 years later, the dam's first 16 turbine generators were in place. Total costs of construction were $145 million. By 1981, 11 more generators had been added and the dam was raised 3 m (cost: $396 million). 1.68 million m^3 of concrete was used in the construction of the dam. The dam is constructed in such a way that a maximum length is exposed to the river to generate as much electricity as possible. The total crest length of the dam is 1 817 m, where the river is only 300 m wide. The dam is therefore constructed in a L-shape, which made it possible to extend the powerhouse and install more generators. Construction of the dam created Rufus Woods Lake, which stretches 82 km upstream to the Grand Coulee Dam.

Fig. 11 View on the spillway of the dam (source: author's photo).

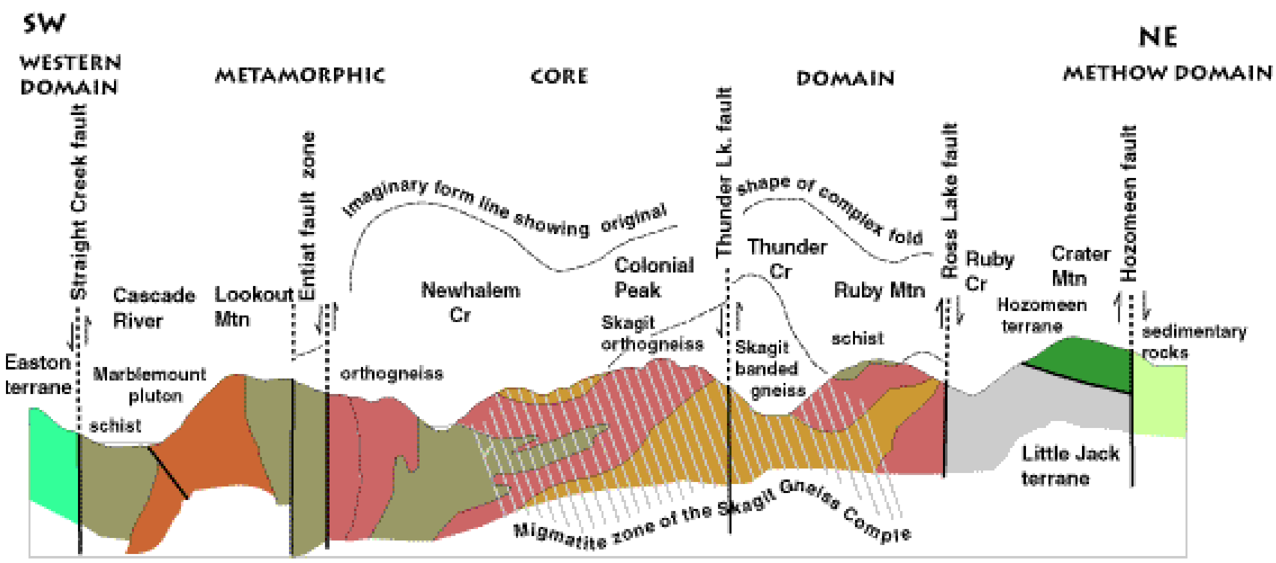

Fig. 12 Cross sectional sketch of giant fold exposing migmatites in the Metamorphic Core Domain (Chelan Mountains terrane) (source: www2.nature.nps.gov/GEOLOGY/usgsnps/noca/nocastc.html#Napeequa).

Diablo Dam

After a nice scenic drive across iced roads in the North Cascades National Park, I stopped briefly for a look at the Diablo Dam. This dam forms part of a series of dams (Ross, Diablo and Gorge) that were built in the Upper Skagit River in the 1930's as part of the Skagit River Hydro-electric Project. The dams were constructed without a single road traversing the North Cascades at that time (only in 1972 a paved road was opened in the then brand new National Park).

Construction of Diablo Dam started in 1927. It is a 116 m high concrete arch dam which was the world's highest of this type when construction was finished in 1930. On October 20, 1936 Diablo Dam started generating hydropower, its current capacity is 169 MW. The lake behind the dam is called Diablo Lake.

Fig. 13 Diablo Dam and Lake (source: www.ci.seattle.wa.us/light/tours/Skagit/Images/DiabloBig.jpg).

Geologically, the wilderness of North Cascades National Park forms part of the Skagit Gneiss Complex. This is derived from the metamorphosed strata of the Chelan Mountains terrane by intrusion of many igneous sills and dikes, which were then metamorphosed. Two types of gneiss are present in the region around Diablo Dam: banded gneiss and orthogneiss. Banded gneiss is made up of layers of granitic orthogneiss alternating with schist. The gneiss layers are mostly deformed igneous sills. Most of the Skagit Gneiss Complex consists of migmatite. Figure 12 gives an impression of the complex geology in this area.

Orthogneiss of at least two different ages makes up much of the Skagit Gneiss Complex. Some magma intruded while rocks of the Metamorphic Core Domain were being squeezed and probably folded. While the magma crystallised or soon thereafter, the squeezing (or flattening) aligned the minerals and the rock became foliated orthogneiss. Twenty million years or so later, much of the squeezing had ceased. When new magma invaded, the newly crystallised granitic rocks developed much less foliation than their predecessors. But because they were subjected to an episode of stretching, these younger granitic rocks exhibit strong lineation. The older magmas solidified about 65-90 million years ago (Cretaceous) and the younger ones about 45 million years ago (Eocene).

In many areas, the complex sequence of invading magma and deformation left a confusing mixture of rocks. Gneiss was cut by light-coloured dikes or sills which were then all squeezed and deformed. This deformation was followed by intrusion of still more dikes and further squeezing and stretching. Even more dikes may have intruded after that.

Fig. 14 Diablo Dam during the Skagit River flood of October 2003 (source: www.nwrfc.noaa.gov/floods/oct_2003/sld023.html).

Literature

- en.wikipedia.org/wiki/ (Wikipedia entries on the various projects covered in this article)
- www.usbr.gov/dataweb/dams (large database of dams managed by the U.S. Bureau of Reclamation)
- www.canyon-country.com/lakepowell/gcdam.htm (Glen Canyon Dam and Lake Powell)
- www.visitmt.com/categories/moreinfo.asp?IDRRecordID=10846&SiteID=1 (Hungry Horse Dam)
- www.nws.usace.army.mil/PublicMenu/Menu.cfm?sitename=cjdam&pagename=hydropower (Chief Joseph Dam)
- www2.nature.nps.gov/GEOLOGY/usgsnps/noca/nocageol6.html#skagit (Skagit Gneiss Complex)
- Maps and guides of the various national parks visited during the road trips

Further reading

- www.lib.utah.edu/spc/photo/crampton/crampton2.htm (beautiful images of Glen Canyon before the flooding)
- www.nps.gov (National Parks Service website)
- www.usbr.gov (U.S. Bureau of Reclamation website)

Fig. 15 View of the Diablo Dam spillways resting on metamorphic rocks of the Skagit Gneiss Complex (source: author's photo).

Review of the 11th ISRM Congress, July 2007, Lisbon (Portugal)

Dr.ir. Robrecht Schmitz

Lisbon, the city where the first conference of the International Society for Rock Mechanics (ISRM) was organised, hosted in July 2007 the 11th ISRM Congress, titled: 'The second half century of rock mechanics'. Especially our most senior ISRM colleagues, of whom there were no few that joined the first ISRM personally, pointed out at several occasions that the founders of the ISRM would have been surprised of the dimensions, standards and world-wide reach that the ISRM attains today.

Photo taken on a stroll through Lisbon. The camera was accidentally set to black and white mode (leaving you puzzled if this photo was taken during the 1st or the 11th congress).

Even after 50 years of intensive academic and industrial research it is interesting to note that:

1) There are still basic aspects in rock engineering that are unsolved, e.g. questions regarding the behaviour of rock mass in block caving (presentation by Prof. E.T. Brown). A very important issue, as the rush for raw materials forces an increase in production rates of underground mining activities.

2) Automation of outcrop mapping still presents challenges. The most sophisticated tools are, at present, only capable of identifying 50% of the discontinuities the trained eye/brain can recognise (presentation by Prof. Z.N. Flynn). For the long slopes in large surface mines there is no automated way to map kilometres of slopes on an automated basis (discussion during the laser scanning session).

3) There is need to consolidate the knowledge gained by half a century of ISRM activity. Examples are the publication of the book 'The complete ISRM suggested methods for rock characterisation, testing and monitoring' presented at the conference by the Turkish national group of the ISRM as well as the set-up of a mining interest group by past president Nielen van der Merwe.

4) The diversity in presentations was very wide. Subjects covered were a.o.: underground- and surface mining, dam construction, tunnelling and slope stabilisation in virtually all rock classes. All subjects were treated very theoretically but also (fortunately) from the practitioner's point of view. The question was raised if this broad spectrum of presentations is good or if one should focus on more detailed aspects during future conferences.

Summarising, one can say the organisation committee did a perfect job. The lectures started on time, changes in the programme were clearly communicated and the help offered to the lecturers to set up the presentation was excellent. Nevertheless, the old tradition of simultaneous translation should be kept in mind for following conferences since the quality of many excellent presentations suffered from a linguistic point of view.

The information gained during the day was discussed with colleagues from all over the world during the coffee breaks and continued during the evening, framed by Lisbon's excellent gastronomic setting.

Professor E.T. Brown who attended the 1st and 11th ISRM conference was honoured this year by receiving the Rocha medal.

Excursions were organised to Portugal's thriving marble quarrying industry and ore mining activity in the Iberian Pyrite Belt. Another excursion showed how Lisbon's metro is extended with all challenges accompanying construction through very inhomogeneous, partly anthropogenic deposits with virtually no overburden.

The conference centre near Bélem next to the river Tagus.

Literature

Ulusay, R. & J.A. Hudson (2007). *The complete ISRM suggested methods for rock characterisation, testing and monitoring: 1974-2006. Suggested Methods prepared by the Commission on Testing Methods, International Society for Rock Mechanics (ISRM)*. ISRM Turkish National Group.

Ribeiro e Sousa, L., C. Olalla & N. Grossmann (eds.) (2007). *Proceedings 11th Congress of the International Society for Rock Mechanics: The Second Half Century of Rock Mechanics*. Taylor & Francis/Balkema, Leiden.

Verification of two-dimensional numerical earthquake site effects on a dam site, Costa Rica

Sigarán-Loría, C. & H.R.G.K. Hack (2007). Verification of two-dimensional numerical earthquake site effects on a dam site, Costa Rica. *Proceedings 11th Congress of the International Society for Rock Mechanics: The Second Half Century of Rock Mechanics.* Taylor & Francis/Balkema, Leiden, pp. 1 203-1 207.

Abstract: The ground response and site effects of a steep valley were verified with a two-dimensional plane strain analysis in the time domain against recorded motions. Weak motions (<0.15 g) have been measured on the site with three accelerographs placed at the base and crests of the slopes. The instrument at the base is on rock and the ones towards the crests on soil. Dynamic models were done with the finite element method (FEM), loading at the base the signals recorded on rock. The peak ground accelerations (PGA) obtained with the FEM fit with the ones measured in situ. The comparison between the main frequencies of the events and the fundamental frequencies of the site show resonance effects. The raised amplifications were demonstrated to be caused by geological site effects, related to the upper geotechnical unit and not by the topography. These results show the importance of subsurface structure in causing resonance effects.

1. Introduction

Numerical methods have recently been applied for back- and parametric analysis of earthquake ground response and site effects (e.g. Athanasopoulos *et al.* (1999), Havenith *et al.* (2002), Lokmer *et al.* (2002), Paolucci (2002), Papalou & Bielak (2004), Bouckovalas & Papadimitriou (2005) and Psarropoulos *et al.* (2007)). There are few attempts to verify measured with modelled data in time and frequency domains accounting for topographic and geologic effects separately (e.g. Semblat *et al.* (2005), Assimaki *et al.* (2005)). Thimus *et al.* (2006) present a theoretical verification of wave propagation with finite differences (FDM). Sincraian & Oliveira (2001) had field measurements but could not find a good fit with the 2D and 3D FEM they used. Geli *et al.* (1988) found that 2D models underestimate field observations, caused by the simplicity of the models. Only the 3D model in the frequency domain showed a good match. They concluded that for peak ground accelerations (PGA) smaller than 0.24 g hills behave approximately linear.

Users of these tools should be aware of the limits of their applicability and benchmark results. Weak motions have been validated with field data with a 3D hybrid approach (indirect boundary elements method) by LeBrun *et al.* (1999). They found a good correlation on the responses for frequencies lower than 1 Hz. On the other hand, 1D analysis methods have been extensively verified as documented by Kramer & Stewart (2004) with nearby rock-soil signals as well as vertical arrays. Strong amplifications have been measured in some cases on hills (Bouchon & Barker, 1996) and related to topographical site effects, without quantifying the effects of the local geology.

This paper presents the comparison of the ground responses of a site, modeled with two-dimensional (2D) linear elastic finite elements (FEM) and field measurements, comparing the site effects of topography and geology. The site is located in a highly seismic region of Costa Rica.

2. Addressed problem

Two-dimensional (2D) plane strain numerical models with FEM were used to verify the ground response and site effects of an instrumented site with in-situ recorded weak motions (PGA<0.15 g) in the time domain.

The location was a dam site in a steep valley, mainly consisting of rock with a transitional weathering profile towards a saprolitic type of material on top. This project is located in a highly seismic region in Costa Rica, at the central Pacific side along the fore-arc region (Figure 1). The main seismic sources are the Meso-American subduction trench (interplate) along the Pacific, as well as local faulting (intra-plate).

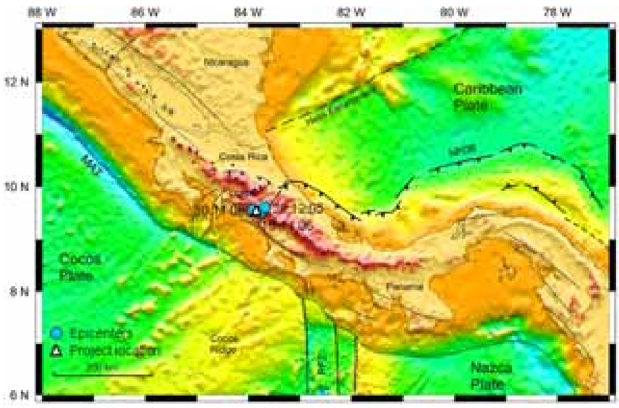

Fig. 1 Geotectonic setting of Costa Rica (modified after Flueh & Von Huene, 2007).

3. Field measurements & models

3.1 Field measurements

Three digital Etna Kinematrics accelerographs have been registering motions on the site since September 2005 (one at the base of the valley on rock, and one at each abutment towards each slope crest, on saprolitic materials (highly weathered rock), at heights of 190 m and 135 m above the river). They were aligned with a north-south (NS) strike, parallel to the dam axis (Figure 2).

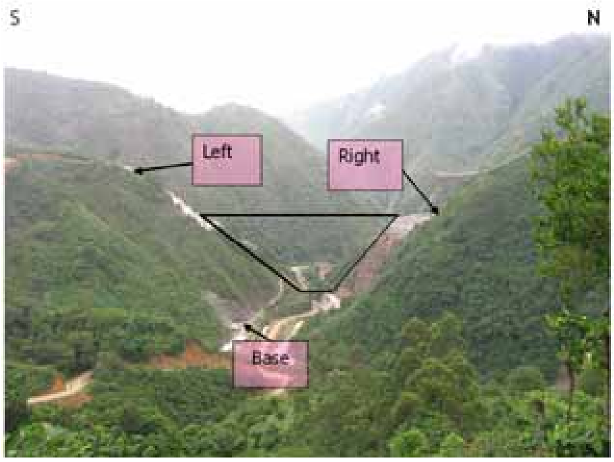

Fig. 2 Dam site looking downstream with sketch of dam and placement of accelerographs.

These instruments have a register capacity of 2 g amplitude, and a frequency range from 0.12 Hz to 45 Hz. Some weak motions have been registered, of which three near-field events were chosen for this analysis (Table 1).

The responses at the measurement points are listed in Table 2, with the main frequencies obtained from smoothed Fourier amplitude spectra. The left slope is not registered during the last two events due to a defect in the accelerograph.

The rock motions ('base' signals) from the NS component were the input for the numerical models. The three events had base-line correction, the same frequency range (0.12-45 Hz) and duration (about 20 s each).

Table 1 Characteristics of the measured earthquakes.

Event date	Magnitude (Mw)	Source of rupture	Hypocentral depth (km)	Distance and direction from dam (km)
28/12/2005	5.1	Subduction	43	24.2 NW
18/11/2006	5.0	Local fault	23.8	6.5 SW
30/11/2006	4.8	Local fault	9	23 W

Table 2 Site responses and Fourier spectra frequencies.

Event/ Station	Peak ground acceleration (PGA)			Fourier spectra [Hz]
	NS	EW	Vertical	
28/12/2005				
Base	10.99	16.82	9.53	1-10
Right crest	129.26	82.91	39.25	4
Left crest	120.18	120.15	58.05	3
18/11/2006				
Base	7.26	16.02	6.42	3-10
Right crest	40.96	40.91	33.79	4-5
30/11/2006				
Base	3.94	8.80	3.66	1-9
Right crest	20.75	18.84	13.91	4-6

3.2 2D FEM models

All the FEM models from the valley had the same dimensions and mesh coarseness. The elements used were 15-noded triangular, fine enough to ensure appropriate wave propagation in accordance with the wavelength and shear wave velocities of the materials. The models were further refined towards the surface.

The distances from the boundaries were approximately 1300 m from the slope crests to the lateral boundaries and from the base of the valley to the bottom boundary 20 m depth. They were chosen after a sensitivity assessment of the site response under different dynamic loads to find the models for which the responses converge, avoiding possible reflections from the boundaries.

The responses in the models were measured at different points to control the amplification patterns but the focus of this paper is on the points that correspond with the accelerograph locations. These are shown in Figure 3. 'L' is the accelerograph at the left slope, corresponding with the slope crest, and 'R' the one at the right slope, located approximately 90 m behind the slope crest.

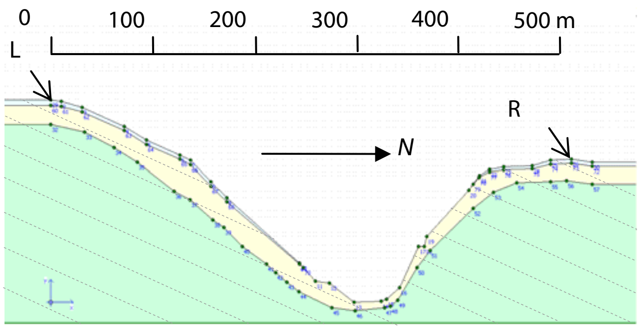

Fig. 3 Measurement points at the slope crests, L: left, R: right (equal horizontal and vertical scales) and apparent dip of the layers.

3.3 Ground properties and constitutive model

The site consists of a discontinuous rock mass of turbidites (sandstones and shales) of the Térraba Formation (Oligo-Miocene). The average bedding is 290°/25° dipping downstream (Figure 3). Geotechnically, it is divided in three units: (a) lower: slightly weathered rock, (b) intermediate: moderately weathered and decompressed rock and (c) upper: completely weathered, transitional to soil (saprolite). The lower unit is found at approximately 23 m depth in the left abutment and 20 m in the right abutment. It transitionally turns into the intermediate unit, which has about 18 and 16.5 m thickness on the left and right abutments respectively. The upper unit (saprolitic material) is considerably weaker and less stiff. Its thickness is 5 m and 4 m on the left and right abutments respectively, at the accelerograph locations. The spatial distribution of the units is sketched in Figure 3, their properties are summarised in Table 3. The geotechnical characterisation was done through extensive in situ and laboratory tests (e.g. plate bearing, SPT, triaxial), geophysical (e.g. microseismicity, seismic refraction) and geological survey (e.g. drillcore, mapping).

Table 3 Geotechnical parameters of the site.

Parameter	Name	Unit	Geotechnical Units		
			Upper	Intermediate	Lower
Material model	*Model*	-	Linear elastic		
Material behaviour	*Type*	-	Drained		
Unit weight	γ	kN/m^3	18	26.5	27.1
Young's modulus	E_{ref}	kPa	$5e^{+4}$	$2.5e^{+6}$	$7e^{+6}$
Poisson's ratio	ν	-	0.3	0.3	0.25
Shear wave velocity	v_s	m/s	102	596	1006
Wavelength for highest frequency	λ	m	2.3	13.2	22.4
Rayleigh damping constants	α $ß$	- -	0.0314 $3.2e^{-4}$	0.0314 $3.2e^{-4}$	0.0314 $3.2e^{-4}$

The left slope has a steepness of 43° and 36° towards the crest. It is gentler than the right slope due to the bedding dip and strike angles towards the river in that side. The opposite happens on the right abutment, giving higher steepness to the slope (57° to 50°).

Due to the scale of this analysis, the complexity of the discontinuities characterisation, nonlinearity, heterogeneity and anisotropy of the rock mass could not be incorporated in the models. In the models the site was considered as a homogeneous, isotropic continuum. The properties were used following Table 3.

As the strains for these types of events are small, deformations range within the elastic range. Therefore, the linear elastic model was suitable for the purpose of this analysis. Different geological-geotechnical scenarios were evaluated to discern between the geological and topographical site effects and to quantify it. This was done with a model with the site topography and without the upper geotechnical unit. This unit was substituted with the intermediate unit, giving this model two units. Another evaluation was made with only the lower unit.

3.4 Dynamic properties and loads

Dynamic viscous absorbent boundaries were used to avoid reflections. The horizontal NS component of the registered acceleration time histories from the base station on rock were prescribed along the bottom boundary. The peak accelerations and main frequencies of the input signals are given in Table 2.

The damping of the system was Raleigh type, chosen for the frequency band of the input signals. The resultant mass (α) and stiffness (ß) proportional constants were estimated for a 5% damping (Table 3).

The natural frequencies of the site were estimated from transfer functions on a one-dimensional equivalent-linear system with different earthquake signals from the world given by the software. The right side displayed a first oscillation mode between 5 and 6 Hz and a second around 9.5 Hz. The left side had the first oscillation mode between 3.5 and 4.4 Hz, and a second around 8.5 Hz. The left slope has a slightly lower fundamental frequency due to the thicker upper unit of the saprolite. The results of the right slope are shown in Figure 4.

4. 2D FEM ground responses

The site responses from the FEM models were measured behind the crests of both slopes and along the slope at the left margin. The output from the points that correspond with the accelerograph locations are listed in Table 4. The strongest registered motion (December 28, 2005) was also modeled with variations in the geology, keeping the site topography to quantify the site effect of geology on the response (Table 4).

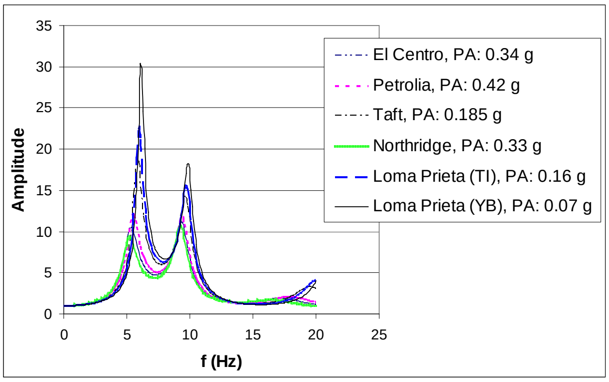

Fig. 4 Oscillation modes at the accelerograph location on the right slope.

4.1 Amplitude

The peak ground accelerations of the site under the three motions are given in Table 4 and Figure 5. A good match in amplitude between the field measurements (referred as 'instr.') and the responses from the numerical models was obtained. Figure 6 illustrates the spatial distribution of the horizontal accelerations on the left slope.

For the left slope, a consistent trend of higher amplification towards the slope crest was obtained coinciding with the position of the accelerograph (Figure 5 and Figure 6). The right slope showed a peak around 100 m from the crest for the stronger event (approximate location of the accelerograph, described in 3.1), but for the other two motions that trend was not clear (Figure 5). More measurement points are needed to make further conclusions.

Table 4 Peak ground accelerations and amplification factors at the right and left crests.

Event	Right crest		Left crest	
	PGA (cm/s^2)	A.f.*	PGA (cm/s^2)	A.f.
28/12/2005	127.6	11.6	118.8	10.8
2 units	36.9	3.4	29.1	2.6
1 unit	28.8	2.6	32.0	2.9
18/11/2006	43.3	5.9	53.3	7.3
30/11/2006	25.8	6.5	28.9	7.3

** A.f.: Amplification factor*

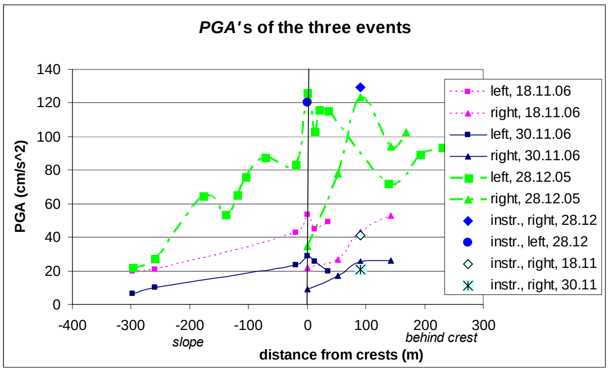

Fig. 5 Peak ground accelerations of the three motions along the left slope and behind the crests of both slopes. The zero on the x-axis represents the crest of each slope. "Instr." are the instrumental field measurements.

When overlapping the time domain outputs from the model with the accelerograph signals, a good correspondence for the stronger event (December 28, 2005) was found but the others showed a time disparity of 2 to 4 s, although the signals kept similar shapes amongst them.

4.2 Site effects on the amplification

Three models with variations in the geotechnical units keeping the site topography the same were assessed to quantify the amplifications associated with the geological site effects.

The two-units model was performed without the upper unit (saprolite). That layer was substituted with the material of the intermediate unit. Similarly, the one-layer model was carried out considering only the response of the lower unit within the site morphology. The peak accelerations and amplification factors of these models are given in Table 4. As expected, the model with one unit gave the lowest amplifications, followed by the two-units one. The difference between the responses of these models with the real situation (three units) is important. When removing the upper unit, the amplification factors drop down 3.4 to 4 times, from 11.6 to 3.4 in the right slope, and from 10.8 to 2.6 in the left (Table 4). The removed layer has important differences in its physical-mechanical properties and lower impedance. The one-unit model displayed similar range and trend in its responses to the two-units one (Table 4).

Locally, below the accelerograph locations, the fundamental frequencies were estimated as 5-6 Hz at the right side and 3.5-4.4 Hz at the left side (Figure 4). The frequency content of the Fourier spectra (Table 2) of the three events showed a similar range between 1 and 10 Hz at the base. The same spectra at the accelerograph locations displayed a main frequency peak between 4-6 Hz at the right slope and 3 Hz at the left. Comparing the Fourier amplitude spectrum of the site with the fundamental natural frequencies, it is seen

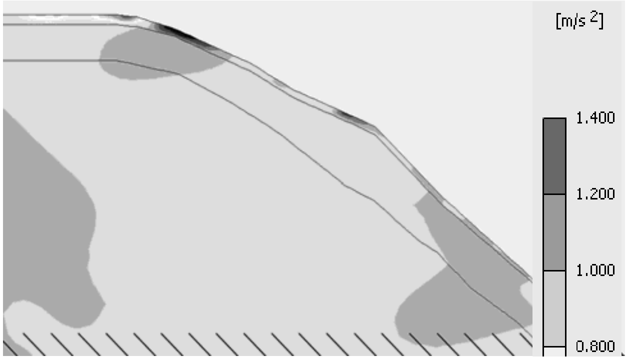

Fig. 6 Peak horizontal accelerations on the left slope (in m/s²). Peak towards the crest. Response under the motion of December 28, 2005.

that under those places resonance develops in the upper geotechnical unit.

5. Conclusions

The numerical ground responses of this plane strain survey provided a good approximation of three field-measured weak earthquakes (<0.15 g) in time domain, applying the linear elastic material model. The magnitudes of the amplitudes from the models gave a good fit to reality, despite the restrictions two dimensions induce on wave propagation for spatial analyses of ground response and site effects, besides the local geotechnical complexities (nonlinear behaviour, heterogeneity, etc.).

The high amplification ratios measured on the site (10 to 12 at the slope crests for the stronger motion), are related to a resonance development in the upper unit due to geological site effects. The fundamental site frequencies and main frequencies of the events coincide, and the geotechnical properties suggest a high impedance difference between the units, being greatly lower for the upper unit. This was clear from the model without the upper unit ('two layers'), below the accelerograph locations. On that model the amplifications decreased 3.4 to 4 times to factors of 2.6 to 3.4.

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Excursie Hubertustunnel, Den Haag

Leon van Paassen en Marcel Remijn

Op een schitterende vrijdagmiddag (13 april 2007) verzamelde een grote groep ingenieursgeologen en belangstellenden zich bij het bezoekerscentrum van de Hubertustunnel in Den Haag. Daar werden zij hartelijk ontvangen door Sallo van der Woude, projectleider bij aannemerscombinatie HTC (Hubertus Tunnel Combinatie).

Nadat iedereen een kopje koffie voor zichzelf had ingeschonken begon Sallo te vertellen over de tunnel: over de problemen die ze tot nu toe bij het boren waren tegengekomen, maar vooral ook over de snelle voortgang van het project.

Door de toenemende verkeersdrukte wordt de bereikbaarheid van Den Haag en met name Scheveningen steeds slechter. Door de ligging aan zee heeft Den Haag geen mogelijkheid het verkeer rondom de stad te leiden, verkeer uit de regio moet door de stad heen. Woonwijken worden bovendien ernstig belast door verkeer dat een andere weg zoekt om files en vertragingen te vermijden. Een groot deel van deze problemen wordt opgelost door de aanleg van de

Hubertustunnel, die 1 600 meter lang wordt. Het tracé van de tunnel loopt van het terrein van de voormalige Kleine Alexanderkazerne naar het Hubertusviaduct. De tunnel zal bestaan uit twee tunnelbuizen, waarvan de eerste bij het bezoek al was geboord. Het boren van de tweede buis was kort tevoren gestart. In vier boorploegen wordt er 24 uur per dag, 7 dagen per week gewerkt, zodat de tweede tunnelbuis begin juni 2007 klaar zou zijn. In de zomer van 2008 moet de tunnel open gaan voor verkeer.

Tijdens het boren van de eerste buis ontstonden er problemen met de pasvorm van de tunnelsegmenten waaruit de tunnelbuis wordt opgebouwd. Omdat de vorm van sommige segmenten meer afweek dan de tolerantie toeliet traden er piekspanningen op in de hoeken. Dit leidde tot beschadigingen aan de segmenten tijdens, of kort na de plaatsing. Het repareren en/of vervangen van deze segmenten zorgde voor extra reparaties en tijdsverlies. Nadat de gietmallen van de segmenten waren aangepast was dit probleem grotendeels verholpen.

De Hubertustunnel is een van de eerste tunnels in Nederland die onder bestaande bebouwing doorgaat. Het monitoren van zettingen is daarom van groot belang. In samenwerking met Fugro is een uitgebreid en volledig geautomatiseerd monitoringssysteem opgezet. Dit systeem houdt 24 uur per dag de zettingen van de grond en gebouwen boven en rond de Hubertustunnel in de gaten. Indien de vervormingen groter zijn dan verwacht kan er besloten worden om het boorproces aan te passen, door bijvoorbeeld de steundruk van het boorfront te verhogen of te verlagen.

Na de presentatie van Sallo werden de laarzen aangetrokken en de helmen opgezet en verplaatste de hele groep zich naar het bouwterrein. Onder begeleiding van Sallo gingen we de eerste tunnelbuis in. Hier was men bezig met de aanleg van de dwarsverbinding naar de tweede tunnelbuis. Dit wordt gedaan door de grond tussen de twee buizen te bevriezen en dan grond te verwijderen en de dwarsverbinding aan te leggen.

Hierbij willen we Sallo bedanken voor de goede ontvangst en het organiseren van deze gezellige en leerzame excursie.

Water at the desert fringe

The Marib dams and irrigation schemes in the Republic of Yemen. A personal memory and a historical view

Charles Dufour, Project co-manager Water Resources Assessment Yemen Arab Republic project (WRAY) 1985-1989

In memory of Koos Uil (1946-2003), hydro-geologist WRAY project 1985-1989

Location, population and economic activities

The Marib dam and the surrounding irrigation area are located within the longitudes 45°15' and 45°40' east and latitudes 15°20' and 15°45' north at the northeastern fringe of the mountainous area of the Yemen highlands. The site of the Marib dam is located where the Wadi Dhana (also known as Wadi Adhanah) enters the plain and is called the Wadi As Sudd at that spot. Towards the east of Marib the Ramlat extends as Sab'atayan, towards the north of Marib the desert extends as Rub Al Khali, translated in English as The Empty Quarter. The area under cultivation around Marib is 10 000 hectares, spread over an area of 20x25 km (500 km^2).

The population density in the Marib area was estimated to be 49 persons per km^2, giving an estimate of 24 500 persons by 1986. The main activities are agriculture and trade of goods from Saudi Arabia to the highlands of Yemen. Although from 1985 onwards the successful exploration and exploitation of oil and gas is the most important activity in the area in terms of macro economy, the local population is hardly involved in these activities.

Meteorology and hydrology

For the Marib area, the principal source of water is the catchment of the Wadi Dhana. Its runoff-generating area covers 8200 km^2. Average rainfall in this area is estimated to be 140 mm per year. Rainfall in the Marib area itself is estimated to be less than 100 mm per year. Most of the rain falls only a few days a year. The highest daily rainfall recorded prior to 1989 was 65.8 mm at the southern boundary of the Wadi Dhana, the highest daily rainfall registered in the runoff-generating zone of the catchment area in the period 1985-1989 was 39 mm. The mean annual runoff of the Wadi Dhana was 104 million m^3 (calculation based on the observations in the period April 1, 1986-April 26, 1989).

In Marib, the temperature is high with an annual average of 26.4 °C, the relative humidity is low with an annual average of 36%. The reference crop evapotranspiration in Marib is 3180 mm (calculated according the modified Penman method).

The ancient Great Marib Dam

Historical background

Marib was the capital over which the legendary Queen of Sheba (or Sabah), known as Bilqis to the Arabs, ruled around 950 B.C. The Queen of Sheba is mentioned in the Koran as well in the Old Testament for her famous visit to King Solomon in Jerusalem (I Kings, chapter 10).

Fig. 1 Looking from south sluice gate of ancient Marib dam to north sluice gate.

The Kingdom of Sheba was prosperous and its power was based on trade, both by sea (to and from India and the Persian Gulf) and land routes (the frankincense roads in the Arabian peninsula). Camels were first domesticated for freight in the second millennium B.C. and were soon used to transport the highly valuable incense along a network of oasis trading stations. Marib, capital city of the kingdom, was the focal point of this trade route. The Sabaeans built the Marib dam to protect the area against flash floods and to develop an irrigated agriculture on the land around the city. The dam was considered one of the wonders of the ancient world and played an important role in the economic strength of the Sabaen rulers. The Kingdom of Sheba is described as a fertile land. The land was cultivated by controlling floodwater in the 'wadis', and had fertile gardens and fields with fruits and spices.

The ancient dam was not intended to create a reservoir that could be drawn all over the year round. The main object was flood control, i.e. to catch and divert the unpredictable floods resulting from rainfall in the mountainous catchment area and secondly to raise the water to the level of the agricultural land at both sides of the wadi known as the two gardens. Raising this water level made it possible to distribute the water over distant fields, so extending the irrigated agricultural area.

Dam construction and irrigation scheme

Prior to the construction of the ancient Great Marib Dam, a smaller diversion dam existed in the Wadi As Sudd. The remnants are located approx. 300 m to the east of the ancient Great Marib Dam in the centre of the valley. It is estimated that this dam was built 1 000-2 000 years before the Great Marib Dam. It was 55 m long and 30 m wide, built with stones as big as 2 m which were cemented with lead. However, the attempts to maintain this structure for irrigation were not successful due to the accumulation of sediment and deposits.

The initial Great Marib Dam was built around 750 B.C. The dam was a simple earth structure, 580 m in length and is estimated to have been 4 m high. It ran straight across the wadi between high rocks on the southern side to a rock shelf on the northern bank belonging to the Jabal al Balaq mountains. The dam was built slightly downstream of the narrowest point in the Wadi Dhana to allow space for a natural spillway and sluices between the northern end of the dam and the high rocky cliff. The initial storage capacity was about 55 million m^3.

Around 500 B.C. the dam was heightened. The second structure was a 7 m high earth dam. The cross-section was triangular with both faces sloping at 45 degrees. The water face (i.e. the upstream slope) was covered with stones set in mortar to make the dam watertight and to resist the erosive effects of waves. The final form of the Marib dam was reached after the end of the rule of the Kingdom of Sheba. From 115 B.C. onwards the ruling people in Southern Arabia were the Himyarites and the next major reconstruction appears to be a Himyarite work. This reconstruction led to a new 14 m dam with elaborate water works at both ends.

Like any irrigation scheme, Marib faced an ever increasing threat from silt. The seasonal floods brought each year about 2.5 million m^3 of silt which was either held behind the dam or spread over the irrigated land. As a result, the level of the irrigated land rose about 1 cm a year or about 1 m each century. The great dam functioned for about a thousand years, during which time the level of the two gardens must have risen by about 10 metres. Upstream of the dam a mass of sediment piled up as well. A considerable amount is nowadays still found at the northern sluice system. The Sabaeans and their successors raised the levels of the sluice gates, north and south, they built up the overflow weir on the north sluice and they heightened the main wall, but it was a struggle they could not win. It is estimated that the fields could be flooded to a depth of 60 cm and the total area irrigated amounted to around 9600 hectares. The total population was probably between 30 000 and 50 000. The fields were situated at a short distance from the two sluices. At the north side the fields extended at a distance of 3 km from the wadi. The pattern of those fields was still clearly visible on the aerial photographs of the area taken in 1974.

Southern sluice system

The southern sluice system was known as 'Marbat el-Dimm'. It is a smaller and less complicated structure than the north but was in much better condition in 1987. The spillway was an overfall located about 7 metres below the top of the dam, the spillway width was 3.5 m.

Northern sluice system

The northern system of sluices included an overfall spillway and a channel outlet. The channel outlet was located between the spillway and the earth dam, and it contained one great wall over 140 m long, about 9 m thick and from 5 to 9 m high. This system conducted water from the northern end of the dam through a single 1 000 m long and 12 m wide canal to a rectangular structure that divided the water entering into twelve different streams.

At the dam, the discharge flowing into the canal was controlled by two gates. The overfall gates were 2.5 m wide, and the maximum gate openings were 1 m and 4 m below the dam crest. In the initial section of the canal, the canal walls widened to form a basin of about 23 m wide and 65 m long (essentially the length of the northern wall) which acted as a settling basin for heavier material carried by the wadi. The 1000 m long canal had earth walls that were covered with a cemented stone lining on the inside (i.e. water side). The cross-sections of the canal and of the settling basin were approximately rectangular.

The spillway was a 40 m wide overfall with a broad unlined rock crest. The crest elevation of the overflow spillway was located 3 m below the height of the dam and the spillway was not gated.

End of the ancient Marib dam

Silt, as already mentioned above, presented an ever-growing threat to the functioning of the entire irrigation scheme. The dam suffered numerous breaches caused by overtopping and maintenance works were substantial. At the same time economic and political factors weakened society's ability to respond to the growing natural challenges.

Fig. 2 Ancient Marib dam from south sluice system flow channel.

After the wind patterns in the Indian Ocean and in the Red Sea became better known and understood, transport over sea became a preferred alternative for the route over land in the first century A.D. The economic as well as political power ebbed from Marib. With the diminishing power of the local leaders the necessary ability for a proper management was absent. Mobilisation of sufficient work force and capital to maintain or repair the system became impossible. In 575 A.D. the dam was overtopped and never repaired. Some of the people remained but most of them emigrated either north into the Hejaz or eastwards to Oman and the Arabian Gulf. The final destruction of the dam was a milestone in the history of the Arabic peninsula. The fame of the Marib dam was such that its final destruction is recorded in the Koran (Sura 34 15-17).

For hundreds of years following the destruction of the dam till the 1970's, spate irrigation was applied along the Wadi As Sudd downstream of the ancient dam. Based on aerial photo interpretation it was estimated that the irrigated area was approximately 2 500 hectares. In the 1970's groundwater pumps driven by diesel engines were introduced in the area. This, in combination with a rapidly increasing use of drilling rigs, resulted in a considerable increase in drilled wells and hence in groundwater exploitation in the area.

The new Marib dam

Introduction

The new dam was funded by Sheikh Zayed, ruler of Abu Dhabi and president of the United Arab Emirates and by the Abu Dhabi Fund for Arab Economic Development. Sheikh Zayed's ancestors are believed to have lived in Marib and to have migrated to the Gulf when the ancient dam finally collapsed. The dam was built by the Turkish contractor Dogus Construction and Trading from Istanbul. The design, engineering and supervision were executed by the Swiss company Electrowatt Engineering Services Ltd., Zurich. Contrary to the ancient dam the new dam is intended to create a reservoir that can be drawn all over the year round.

The new dam is situated at a distance of 3 km upstream from the location of the ancient dam. The foundation stone of the new dam was laid in October 1984, construction was achieved according to plan. The dam was inaugurated in December 1986. It was remarkable that during the construction there was no flood of any importance, but within months after closing of the dam a flash flood of 135 million m^3 filled the reservoir for one third of the maximum capacity in April 1987.

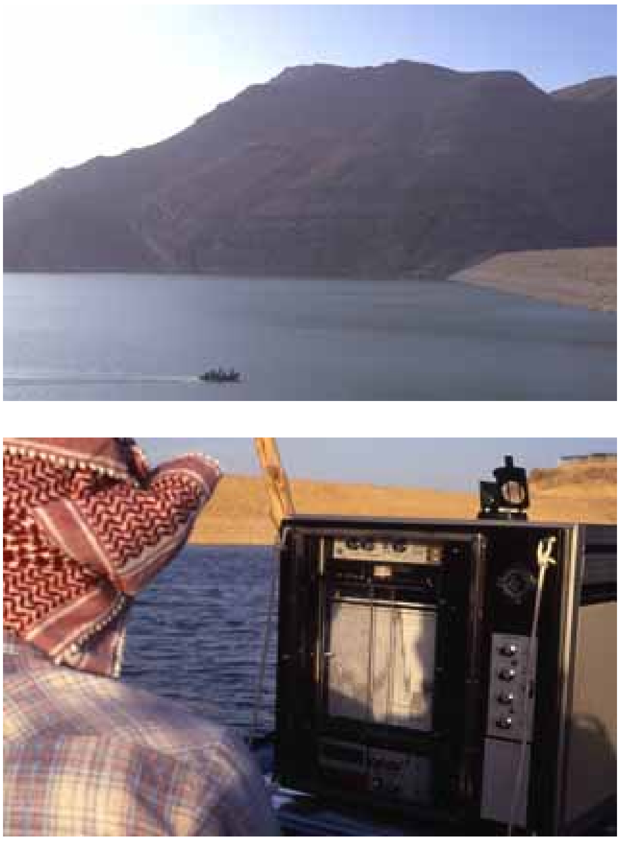

Fig. 3 Marib Dam and reservoir with survey boat (top), seismic reflection sediment sub-bottom profiler (bottom).

Dam construction, irrigation scheme and groundwater abstraction

The new dam is basically a small concrete core that reaches the bedrock, surrounded by a larger clay core and 3 million m^3 of earth fill and rocks. It is 38 m high, 763 m long and 227 m wide at the base and 6 m wide at the crest. The reservoir has a capacity of 396 million m^3. The dam was designed to cope with an average annual inflow of 200 million m^3 and it was estimated that 150 million m^3 will be released per annum once the irrigation scheme is fully in operation. Outflow into the Wadi As Sudd downstream of the dam is controlled by means of a radial gate construction at the end of a tunnel in the dam. The maximum capacity of the gate is only 35 m^3/s. Approximately 10 and 14.5 km downstream of the dam, intake structures have been built in the wadi bed to divert the released water into the main canals of the irrigation system. The object of the irrigation project was to irrigate an overall area of about 6 900 hectare by means of integrated development of surface water and ground water.

When the scheme is in operation, water will be released from the lake and enter the old wadi bed (Wadi As Sudd) downstream of the dam. It is anticipated that half of the water released from the lake will infiltrate into the wadi bed and recharge the underlying aquifer, the other half will be diverted at the two mentioned intake structures. From these intake structures a network of primary canals was constructed in the period 1986-1989. Two main canals reach approx. 18 km from the second structure towards the north-east.

By recharging the aquifer it remains possible to continue the traditional well irrigation. This well irrigation was the main source of water during the centuries between the collapse of the ancient dam and the closing of the new dam in 1985.

Implementing a water resources management plan in the Marib area turned out to be extremely difficult due to the special social circumstances. The relations between the different tribal communities in the area and between the tribes and governmental authorities were and are still subject to tension and conflict. Although the construction of canals of the primary and secondary irrigation network was completed at the end of 1989, there was no sign of operation of the scheme at that time. The inhabitants not only continued well irrigation but increased groundwater abstraction. The amount of groundwater abstraction was estimated to be 29 million m^3 per year in 1977-1978. Abstraction rate was estimated to have increased to 136 million m^3 per year in 1987. In the framework of the WRAY project a further average increase of extraction of 22 million m^3 per year was observed in the period 1985-1989. This significant increase in the extraction of groundwater resulted in a considerable fall in groundwater level in the area, especially in the period 1985-1989, the period between the closing of the new dam and the replenishment of groundwater by releasing water from the reservoir. The irrigated area by groundwater extraction had increased to approximately 10 000 hectares in 1989.

Fig. 4 Geo-electric survey of aquifer in Marib Region.

Fig. 5 Exploratory drilling of aquifers in Marib region.

Reliable recent information concerning the period 1989-2007 could not be obtained, but by studying modern satellite images it may be concluded that the total irrigated area has extended further. It may be assumed as well that the main source of water is still the extraction of groundwater through private wells of farmers.

Observations on the reservoir in the period 1986-1989

As already mentioned, the mean annual inflow in the lake was 104 million m^3 in the observed period. The Marib Dam and irrigation project was designed by Electrowatt for an average recharge into the reservoir of 196 million m^3 per year. This estimated amount exceeded the annual flow observed in the period 1985-1990. The year to year variation of stream flow was large, varying from 135 million m^3 to 87 million m^3 a year. The mean annual reservoir evaporation was 2 093 mm, i.e. 34.9 million m^3. This is a considerable and concerning loss.

In the period observed (April 1986-May 1989) only 122.5 million m^3 was released to replenish the aquifer. This is only 27% of the designed release capacity.

The WRAY project paid special attention to the sedimentation rate in the Marib reservoir. This in view of the fact that evaporation of this shallow reservoir will increase with increasing surface of the lake caused not only by inflow of water, but by influx of sediment as well. Thickness and location of recent sedimentation was mapped by application of a shallow sub-bottom profiler. Data obtained on the lake in February/March 1989 revealed that the total volume of sediment accumulated in the lake was 4.5 million m^3 over an estimated survey area of 8.2 km^2. This means that the storage of the reservoir was reduced by 1.12% in its first three years of impounding and that the average sediment transport/supply into the reservoir is approximately 1.5 million m^3/year. Based on these observations storage capacity reduction would not be an immediate problem.

Fig. 6 Levelling at run-off measuring station, Wadi Adhanah.

However, tremendous amounts of sediment deposited in the valleys of the Wadi Dhana catchment area may be funnelled into the reservoir should a really high flash flood occur. The observation period was characterised by floods which were mainly low, except for the flow of April 1987 which was moderate. Consequently, based on the observed rainfall/runoff amounts for the observed period, the reduction of the storage capacity of the Marib reservoir would only be 15% after 40 years, which should not cause immediate concern. However, for a more realistic prediction of the reduction in storage capacity, medium and high floods (for which sediment loads of 2% and 5% to 14% respectively are assumed) should be included. One high flood period could reduce the reservoir capacity by 6 to 17 million m^3, assuming sediment loads of 5% to 14%. Different storage reduction models based on medium and high flood sequences predict a storage reduction between 15% and 35% within 30 years.

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Excursie GeoCentrifuge (GeoDelft), Delft

Leon van Paassen

Op 7 juni 2007 organiseerde de Ingeokring een excursie naar de GeoCentrifuge bij GeoDelft. De GeoCentrifuge van GeoDelft is een van 's werelds krachtigste centrifuges voor het uitvoeren van onderzoek voor ontwerp en uitvoering van uitdagende civieltechnische werken op slappe grond. Deze werken zijn vaak uitdagend door hun schaal en/of complexiteit. Bij de kenmerkende onderzoeksvragen zijn grond en constructie beide van belang. Proeven zijn uitgevoerd voor grootschalige bouwwerken zoals bruggen, tunnels, kadeconstructies, dijklichamen onder statische en/of dynamische belastingen, etc.

Kenmerkend voor de centrifuge is dat het onderzoek wordt uitgevoerd onder een kunstmatig verhoogd zwaartekrachtveld op kleiner verschaalde modellen van de werkelijkheid. Deze 'kunstmatige verhoging' is nodig om de fysische processen die de interactie tussen grond, grondwater en de constructie bepalen, waarheidsgetrouw te modelleren. De krachten die hierbij worden opgewekt zijn enorm. Zo kan een model met een massa van maximaal 2 000 kg onder een kunstmatig krachtveld van 300 keer groter dan de zwaartekracht worden onderzocht. Het zal duidelijk zijn dat dit veel eist van de onderzoeksfaciliteit, het model en de modelleur.

Aan de excursie was een prijsvraag voor studenten verbonden. Zij kregen de uitdaging om te komen met een onderzoeksvoorstel voor een (kleinschalige) proef in de GeoCentrifuge. Het beste voorstel zou door de studenten zelf, met hulp en begeleiding van (onder meer) GeoDelft, daadwerkelijk worden uitgevoerd. Op het moment van dit schrijven staat deze prijsvraag nog steeds open.

Om een idee te krijgen wat uitvoering van een dergelijke proef inhoudt kregen we een presentatie over de centrifuge en een voorbeeld project door Paul Schaminee en Vera van Beek van GeoDelft, gevolgd door een rondleiding langs de centrifuge en bijbehorende testfaciliteiten.

Onder water storten van klei - Voorbeeld van een winnend prijsvraag geocentrifuge-onderzoek

Vera van Beek

Het onderwater storten van kleibrokken wordt in de praktijk uitgevoerd ten behoeve van de vorming van een waterdichte laag. Het wordt vaak toegepast in het voorland van dijken of voor het aanbrengen van een onderlaag voor een stortgebied.

Het is echter nooit onderzocht in hoeverre de gestorte kleibrokken daadwerkelijk een ondoorlatende laag vormen. Als vuistregel wordt aangehouden om kleibrokken van circa 15 cm te storten, met een totale dikte van circa 2 m. Er wordt aangenomen dat de kleibrokken onder invloed van water en/of hun eigen gewicht tot een ondoorlatende laag samenvoegen. Ook wordt in sommige situaties getracht om de kleibrokken samen te drukken met behulp van een kraanbak of slee.

Fig. 1 De opstelling gereedgemaakt voor de eerste proevenserie, klei in (blauw gekleurd) water, lucht en (rood gekleurde) olie.

Om de vraag te kunnen beantwoorden of kleibrokken een ondoorlatende laag vormen, is gekeken naar maatgevende mechanismen die een rol spelen bij de vorming van de laag; de verweking van klei ten gevolge van de interactie met water, zoals slaking en dispersie, en het samendrukken van de kleibrokken onder hun eigen gewicht. Het is onbekend in welke mate deze processen een rol spelen. Ook is onduidelijk of het belasten van klei de doorlatendheid verlaagt. Deze variabelen zijn onderzocht in een korte kijkproef in de geocentrifuge.

In de praktijk duren de genoemde processen enkele maanden. In de geocentrifuge worden de processen versneld door de vergrote zwaartekracht. Hierdoor kan binnen enkele uren getest worden wat de verandering in doorlatendheid is.

De opstelling bestaat uit een drietal kolommen (Fig. 1) waarin op schaal de situatie van gestorte kleibrokken is nagebootst. In de eerste testserie zijn kleibrokken gestort in respectievelijk water, lucht en olie, om de processen die onder invloed van water plaatsvinden (slaking, dispersie) te kunnen onderscheiden van de processen die niet beïnvloed worden door de aanwezigheid van water (deformatie). De doorlatendheid is vervolgens vergeleken door middel van een doorstroomtest. De kleibollen in water bleken de meest ondoorlatende laag te vormen. De intrede van het water in de kleibollen zorgt voor het uiteenvallen van de klei. In de kolommen met olie en lucht kon geen water intreden, waardoor er geen slaking van de klei optrad, alleen samendrukking. De kleilagen in deze kolommen waren dan ook aanzienlijk meer doorlatend.

In de eerste test was Speswhite klei (geproduceerd uit kleipoeder) gebruikt, welke in dispersiegedrag niet geheel overeenkomt met natuurlijke klei. In de tweede test is daarom gekozen om natuurlijke kleien met verschillende eigenschappen te gebruiken. Hoewel het deformatie-, slaking- en dispersiegedrag van de verschillende kleisoorten verschillend bleek te zijn, is er geen verschil in doorlatendheid waargenomen tussen de drie kolommen. Geen van de kleien resulteerde in een slecht doorlatende laag, zoals waargenomen in de eerste test.

De invloed van dynamische belasting, zoals die in de praktijk wordt toegepast, is in de derde proef gesimuleerd door het plaatsen van een statische belasting op de klei (natuurlijke klei). De klei in de drie kolommen werd belast met 0, 10 en 30 kPa. Uit de doorlatendheidstest is gebleken dat belasting van de klei de doorlatendheid sterk beïnvloedt. Terwijl de resultaten bij de onbelaste klei identiek waren aan de resultaten in de vorige proef, was de met 10 kPa belaste klei een stuk minder doorlatend. De met 30 kPa belaste klei liet zelfs helemaal geen water meer door.

Als conclusie kan gesteld worden dat de doorlatendheid bewerkstelligd wordt door een combinatie van deformatie en slaking. Het belasten van klei om de doorlatendheid te verkleinen is doeltreffend.

Fig. 2 De onderkant van de kleilaag na uitvoeren van de proef. Het is duidelijk te zien dat het blauwe water in de klei gedrongen is. Bij de klei in lucht en olie is er nog ruimte aanwezig tussen de kleibrokken.

Book review:

Carbonate Sediments and Rocks. A Manual for Earth Scientists and Engineers,

Colin Braithwaite

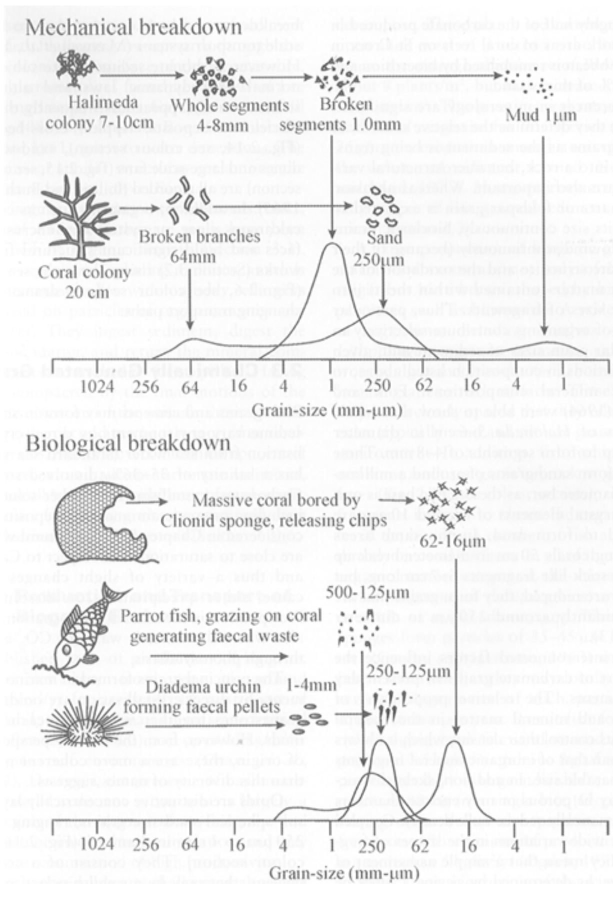
Peter Verhoef, Royal Boskalis Westminster nv

This book is written to fulfil the need felt for an undergraduate text on carbonate rocks for geologists and earth scientists working within the field of geotechnical and applied engineering. The author believes that it might also be useful for engineers and non-specialists working in the petroleum and minerals industries. However, I think that most engineers or non-specialists would find it hard to work through this book since a lot of basic and also more advanced knowledge of geology is assumed. Yet, for geologists or engineering geologists this book of only 164 pages contains a lot of information.

The subjects treated can best be indicated by listing the chapters of the book:

- The Mineralogy and Compositions of Carbonate Rocks and Sediments
- Characteristics of Carbonate Sediments
- Marine Carbonate Environments
- Evaporites Associated with Carbonates
- Continental Carbonate Environments
- Classification of Carbonate sediments and Rocks
- Carbonate Diagenesis: from Sediment to Rock
- Dolomites
- Calcrete
- Limestones, Dolomites and Karst
- Karst Hydrogeology
- Engineering Properties of Carbonate Sediments and Rocks
- Methods of Extraction of Carbonate Sediments and Rocks
- Engineering Case Histories: The Hazards of Karst
- Hydrocarbons, Mineral Deposits and Carbonates
- Carbonates and Conservation

To learn something from this book, you do have to read the complete text of these chapters. It is not possible to divert to the colour plates in the middle of the book or to the graphs, tables or illustrations, scan some text, and grasp what is going on. But not to worry, the chapters are short and the writing is concise and economic. The author succeeds in sharing his decade's long experience of working and researching carbonate rocks. For me, the main importance of this book lies in the chapters outlining carbonate geology. The applied chapters, from 13 onwards, are less illuminating for practicing engineering geologists.



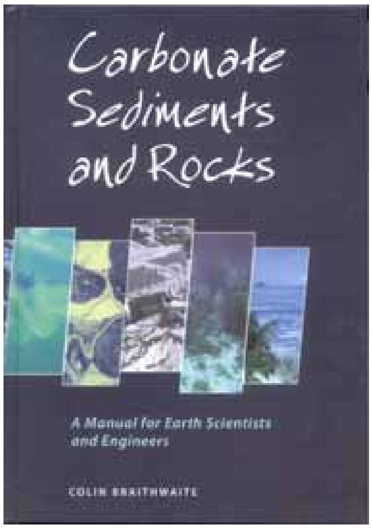
Grain size fractions resulting from mechanical breakdown and biological breakdown (bio-erosion) in reef systems (Figure 2.13 of the book).

The main difference with other sedimentary materials is that most carbonate grains originate within the basin of deposition and many show little influence of transport. In the book, the Bahama Banks are chosen to illustrate the development of a present-day marine carbonate environment. A clear picture of the development of beaches, tidal margins, restricted shelves and lagoons, open shelves, oolite shoals, grapestone and hardgrounds and reef systems is given. Attention is given to the origin of the carbonate. Carbonate mud may, for example, originate from direct precipitation in the form of aragonite needles, or be biogenic. Shallow platforms, such as the Bahama Banks, act as carbonate factories, generating an excess of sediment that is swept off during storms to be deposited in deeper water. The established rates of production in some reef environments are up to 13 $kg/m^2/yr$. It is striking that most of the sand grains are not generated by erosion due to wave action, but by bio-erosion. Biological breakdown occurs through the action of sponge, sea-urchins and fish grazing on coral. Parrot fish produce 0.5 $kg/m^2/yr$ of particles ranging from mud to fine sand. A large individual can take as much as 82 mg coral in a single bite. Boring echinoderm produce sand-size fragments with production estimates of more than 5 $kg/m^2/yr$. Bio-erosion generates, thus, a significant proportion of the sediment and is far more important than conventional erosion by physical processes. Roughly half of the carbonate produced in framework areas of coral reefs on St. Croix in the Caribbean is remobilised by bio-erosion and 75% of this is sand.

One of the guiding geochemical principles to understand the origin of carbonate sediments that Braithwaite reiterates throughout the book, is the role of CO_2. The solubility of calcium carbonate in de-ionised water is comparable to quartz; increased solubility in natural waters is due to the presence of dissolved CO_2. This has to do with the forming of carbonic acid $H^+ + HCO_3^-$ in the water. The well known anomalous behaviour of calcium carbonate to dissolve in colder water has to do with the ability to dissolve more CO_2 in colder water, thereby generating more carbonic acid. What struck me is that this principle also seems to work at micro-scale. The growth of tufa (travertine) at the site where underground streams emerge from cave systems in karst areas, for example, is related to the presence of plants, algae, cyanobacteria and bacteria. The extraction of CO_2 from the water during photosynthesis causes a localised increase in alkalinity and results in precipitation of calcium carbonate. I used to think that only the exposure to the oxygen in the air atmosphere, when the saturated waters emerge from the underground are enough to cause precipitation. But apparently in many cases more is needed. Also supersaturated marine waters often need a local trigger to cause precipitation. I learned that each environment needs a closer examination to understand history and process of cementation.

Throughout the book, carbonate environments from all over the world are discussed. Many of them are relevant for the field I work in, which is dredging and coastal engineering. Much of the information that is new for me comes from the results of petrographic study of the soils and rocks. Nowadays there is a much better understanding of the processes of nucleation and crystallisation, calcitisation (transformation to crystalline calcite involving a liquid phase at the grain boundaries) and recrystallisation (solid state transformation) that are involved. Braithwaite effortlessly succeeds to bring this information across in the various carbonate environments discussed. The author illustrates with well chosen examples how he thinks the rocks have formed. I find that from these descriptions often a good picture of the engineering situation of a particular carbonate type can be obtained. It is also good to know that some of these, and particularly reef areas, have a complex and often unpredictable distribution of massive, cemented and uncemented parts. Better understanding of the way beach rocks, aeolianites, calcrete and all those other forms of carbonate rocks are formed indeed will help us to better model and foresee their engineering behaviour.

I do welcome this book.



164 pages with colour plates in the middle of the book

Whittles Publishing, Caithness, Scotland; 2005

ISBN 1-870325-39-7

£ 40,-

GeoGolf

Robert Hack

On August 30, 2007 a new episode of GeoGolf was played. Nine geotechnical engineers and engineering geologists did their utmost best to show that they do not only control the ground, but also the behaviour of a stupid ball on the ground. The match was played on the Rijswijkse Golfclub. The weather was good except for the last ten minutes when it started to rain and playing in the last flight, the author got soaked. The competition was fierce and after 18 holes Martin Kroezen showed the best control over the bouncing golf ball and won. Runner-up was Bas Hemmen. An extensive dinner gave the opportunity to discuss the peculiarities of the game of golf and the profession in which ground can also behave seemingly quite erratic. The next episode of GeoGolf will likely be played on the Hooge Bergsche near Rotterdam. Any golf-playing geotechnical engineer, engineering geologist or those otherwise involved with geotechnics are very welcome to join and should contact Bas Hemmen at GeoDelft (b.r.hemmen@geodelft.nl) or Robert Hack at ITC (hack@itc.nl).

Dirk Luger as match referee handing over the first price to Martin Kroezen with an approving Floris Schokking.

GeoGolf dinner. Clockwise from bottom left: Michiel Maurenbrecher, Bas Hemmen, Martin Kroezen, Robert Hack, Jos Maccabiani, Dirk Luger, Anke Rusch and Joost Wentink.

Thesis abstracts

Plaxis Soft Soil Creep: de toepassing van een isotroop kruipmodel op de anisotrope ondergrond

Rens Servais

In met name West-Nederland bestaat de ondergrond voornamelijk uit slappe klei- en veenlagen. Bij het aanbrengen van een maaiveldbelasting zullen er in deze slappe grondsoorten verticale en horizontale vervormingen optreden. Voor het berekenen van de vervormingen zijn verschillende modellen beschikbaar. Het 3-dimensionale SSC model is echter het eerste materiaalmodel in Plaxis waarin kruip is geïntroduceerd. Dit model is gebaseerd op het 1-dimensionale a,b,c-Isotachenmodel. Uit praktijkervaring blijkt dat bij de aanbevolen omrekening van de a,b,c-Isotachen in SSC samendrukkingsparameters de zetting van het a,b,c-Isotachenmodel niet in overeenstemming is met de zetting volgend uit een 1D SSC berekening. Dit is de belangrijkste reden dat dit onderzoek is opgesteld. Er is onderzoek gedaan naar de oorzaak van dit verschil, wat vervolgens is gevalideerd aan de hand van een samendrukkingsproef en een case. Verder is het SSC model beschreven en gevalideerd aan 2D vervormingen. Het algemene doel van dit onderzoek is het verbeteren van de praktische toepasbaarheid van het SSC model.

Uit modeltechnisch onderzoek blijkt dat de zetting van het SSC- en a,b,c-Isotachenmodel bij een 1D berekening exact met elkaar overeen moeten komen onder de voorwaarde dat de verhouding horizontale/verticale spanning constant is. Bij een niet constante verhouding ontstaan er verschillen die normaliter te verwaarlozen zijn. De simulatie van een samendrukkingsproef bevestigt dit, de No-Recess case echter niet. De oorzaak hiervan is het op een andere wijze in rekening brengen van onderwater zakken (bij de samendrukkingsproef speelt onderwater zakken geen rol). In Plaxis (SSC) is onderwater zakken correct geïmplementeerd, MSettle (a,b,c-Isotachenmodel) benaderd onderwater zakken. De verschillen tussen de modellen ontstaan dus t.g.v. de implementatie in verschillende rekenprogramma's.

Bij het beschrijven van het SSC model voor 2D vervormingen blijkt de K_0^{nc}-afhankelijke M parameter een belangrijke modelparameter. Aandacht is daarom besteed aan de bepaling van deze parameter. Een K_0-C.R.S. proef geeft realistischer waarden voor $M(K_0^{nc})$ dan de vuistregel van Jaky. Verder heeft de geometriekeuze (axisymmetrisch of plane strain) en de bepaling van de doorlatendheid significante invloed op de resultaten. Bij de No-Recess case is de anisotrope ondergrond gemodelleerd met het isotrope SSC model. De gevonden horizontale vervormingen sluiten bij een juiste parameter- (doorlatendheid en M) en geometriekeuze (axisymmetrisch of plane strain) goed aan bij de gemeten vervormingen. Voor de zettingen geldt dit echter niet. Het model overschat de zettingen vooral in het begin van de ophoogfase. Niettemin lijkt het model de horizontale en verticale kruip én horizontale/verticale vervormingsverhouding goed in te schatten. Bij de gekozen parameters is de anisotrope ondergrond daarmee goed met het isotrope SSC model te simuleren.

Evaluation of parameters influencing pressure arching over shallow room-and-pillar mines

Elles Bader

Pressure arching is a well known phenomenon in underground excavations around the world. However, most information on arching is empirical and little has been published on the subject.

In this study, a numerical analysis using FLAC was performed to investigate the arching effect over room-and-pillar mines. The effectiveness of pressure arches was investigated by varying parameters such as total mine span, thickness of roof layers, thickness of soil overburden, stiffness and strength of pillars, and roof rock properties. The material properties used in this analysis are based on experimental data obtained from the underground limestone quarries in South Limburg, the Netherlands.

From these numerical simulations, an overview of how various parameters affect the arching phenomenon was derived. By assessing these parameters, relationships were found which make it possible to predict the amount of arching that occurs and subsequently assess the large scale stability of the mine system. The width of the excavation and thickness of the rock overburden are most important for this assessment.

The results of this numerical analysis increase the knowledge on arching, which is an important step towards a better understanding of the stability of all underground structures.

Massive flank collapse at La Palma: numerical slope stability models of the Cumbre Vieja volcano

Janneke van Berlo

In 1999, Steven Ward and Simon Day launched a scientific article which discussed possible tsunami effects in the USA as a result of a massive flank collapse of the Cumbre Vieja volcano at La Palma. One problem of their hypothesis is that the slope instability has not been investigated in a quantitative way. The research question therefore aims to partially solve this problem: *"Under what boundary conditions could the Western flank of the Cumbre Vieja volcano start sliding and at what recurrent times would such mass movement occur?"* Methodology concentrates on integrally modelling three model attributes (geometry, material and processes) in a FEM computer model. The table below shows the modelling results: calculated Factors of Safety for various worst case scenarios. 'PCS' stands for Post Collapse Sequence: a plane of weakness below the volcano. Magmastatic dyke pressure at the back of the possible slide volume with magma infill and excess pore pressures due to the heating of pore water resulted in vertical failure mechanisms rather than lateral flank collapses. This is true if the pore pressures are limited to the rift zone of the volcano.

In conclusion, two mechanisms, dyke intrusion and future growth, bring the volcano dangerously close to a failure situation. However, it is important to note that this only occurs under the input of very conservative parameters and in a 2D model situation where friction of the sidewalls of the landslide mass is not taken into account. Additional mechanisms, like pore pressures occurring inside the PCS, may further enhance instability and should be investigated in further research.

Calculated Factors of Safety for various scenarios in a most dangerous volcanic cross section.

Circumstances	Dip PCS average (8)		Parameters standard	
Processes	Parameters standard	Parameters low	Dip PCS low (5)	Dip PCS high (9)
Gravity	1.70	1.44	1.95	1.67
Dyke elastic infill	1.63	1.34	NA	NA
Dyke 'magma' infill	see text	see text	-	-
Dyke no infill	1.27	1.07	-	-
Pore pressures riftzone	see text	see text	-	-
Future Growth + 200m	1.64	1.40	-	-
Future Growth + 400m	1.55	1.32	-	-
Future Growth + 600m	1.46	1.23	-	-
Future Growth + 800m	1.41	1.19	-	-
Future Growth + 1000m	1.33	1.12	-	-

Disregarding the uninvestigated mechanisms in the volcano, the results indicate that overall flank failure at least requires a combination of thoroughly weakened rock- and soil mass *and* steeper flanks than in the current configuration. This supplements the conclusion: to reach substantial growth over the full width of the landslide mass necessary to trigger failure, a time span in the order of 10 000 years will be required.

Comparing discontinuity surface roughness derived from 3D terrestrial laser scan data with traditional field-based methods

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Discontinuity surface roughness estimation is very important in determining the hydro-mechanical properties of rock masses. The normal ways of roughness estimation have been made traditionally by visual and simple instrumental field observations. These methods may require physical access to the exposed rock faces. This may expose field personnel to hazardous situations because the measurements have to be carried out below steep rock faces, in a quarry or tunnel. The higher parts of a steep exposed rock face are often difficult to reach. These traditional methods are also time consuming and subjective. This thesis describes a new method by which 3D point cloud data obtained by laser scanning can be used to model and determine discontinuity surface roughness in an automated way with high detail, high accuracy, and no human bias. The laser scan technique is also much faster and safer than the traditional field-based methods, since no direct physical access is needed to the rock face.

In order to determine the surface roughness from the 3D laser scan point cloud data, two point cloud data sets with an average spatial resolution of 1 cm and 6 cm are used. These point data sets are interpolated as a 3D virtual surface model using a scattered data interpolation technique. To extract the 3D surface roughness from the 3D virtual surface model, the mean orientation of circular windows with diameters of 5, 10, 20 and 40 cm were computed. The large-scale directional roughness is also extracted by cropping and fitting narrow strips using curve fitting techniques.

The results revealed that laser scan data can be used to model and estimate rock surface roughness. The extracted directional large-scale roughness profiles are comparable and can be used in a rock mass index classification system. These can also be used to estimate roughness using the standard reference roughness determination profiles. However, it is difficult to compare the field-based disc orientations with the circular windows mean orientation of the laser scan data. This is due to the low resolution point cloud data set and possible shifting of the exact location of the sample grid. The field-based instrument error and the dead zone due to inclination of undulations with respect to the scanner position have also appreciable influence on the possible relation of these methods.

Risks in the production of geothermal energy in clayey soils as existing in the Netherlands

Rene van Hinthum

Geothermal energy is energy derived from the natural heat of the earth. It is a sustainable form of energy and it is being used more and more all over the world. In the Netherlands there is still no electricity generation from geothermal energy. But if we look at the several successful pilot plants that are running at the moment in countries surrounding us like Germany and France, it is very likely that it will be only a matter of time until the Netherlands will start to generate electricity from the heat of the earth. The scope of this research is to investigate the problems that can arise during the production of geothermal energy in clayey soil such as in the Netherlands. It is a known fact that by heating clay volume changes can occur and excess pore pressures can develop. This could be very damaging to structures built on this heated clay, if no remedial measures are taken in advance.

In this thesis, first the backgrounds of geothermal energy are given and the possibilities for the use of this energy in the Netherlands are described. Then some background on heat transfer in soils is described. Next, possible changes in soil parameters due to heating will be reviewed. In the next chapter calculations of the driving force of all changes in the soil, the heat flow from the hot water conductors, are made. After that, we take a look at the expansion behaviour of clay and the risks and hazards that this could give. For this purpose, the calculations of the heat flow are used as input for the expansion calculations. At the end of the thesis the author gives some useful recommendations for further research on this topic. One of the recommendations is to make a commercially usable program to be able to assess the risks in advance and to be in time to take remedial measures, which can save the producing company a lot of money and troubles.

The investigation on the formulation of a new design code for MV-piles

Roeland van Hof

This thesis investigates whether the current design code for MV-piles is still valid or needs to be updated, as these days piles used are much larger than when the current code was formulated. In order to check if the current code is still valid, the α_t-factor in the formula below was calculated for 47 piles, using the bearing capacity derived from the pile load-tests.

$$F_{Pile} = q_{c_{av}}.L.O.\alpha_t$$

In which:

F_{Pile} = bearing capacity

q_{cav} = average cone resistance over the length where shaft resistance is mobilised

L = active length of pile

O = circumference of pile

α_t = coefficient for shaft friction = 1.4%

From the analysis of the dataset it followed that the current design code does not fit this dataset, since no constant value of 1.4% for the α_t-factor was found. Instead of a constant value for the α_t-factor, a negative relation between the circumference and the α_t-factor was found. This leads to an overestimation for the piles with large circumferences and thus to dangerous situations. Therefore, various modifications of the current design code were investigated in order to give a better prediction for the larger piles as well.

The following method gave the best predictions for the dataset:

$$F_{Pile} = q_{c_{av}}.CV.\alpha_t = q_{c_{av}}.L.(2.W + 0.38.H).(0.025)$$

This formula differs from the old one on the following two aspects:

- CV = calculation value for the circumference of the pile (W = width of the pile, H = height of the pile)
- α_t = 2.5%

The final conclusion of this thesis is to use the lowest value for the bearing capacity that is predicted with either the formula above or the existing formula. The differences in correctly predicted bearing capacities between both methods were too small to replace the current formula by the new one. But since the new formula does not overpredict the bearing capacity of the larger piles, this will lead to safer design of MV-piles.

Influence of spatial correlation length on predicted settlements of a road embankment

Spyridon Kalamatas

In the common geotechnical engineering practice the shallow subsurface is often modelled in several layers by assigning average material properties to each of these layers. However, the reality is quite different. Soil properties are characterised by spatial variability within each homogeneous lithological unit. This variability can be taken into account by incorporating the value of the spatial correlation length on the simulation results.

The spatial correlation length describes the distance over which a random variable tends to be correlated in the underlying Gaussian field. A large value of spatial correlation length describes a smoothly varying field, while a small value implies a ragged field. The objectives of this graduation project are to extract the value of the spatial correlation length for a specific case study and to investigate its influence in the settlement calculations of a road embankment. The study area deals with the A2 highway connecting Amsterdam and Utrecht. A representative section of 8.5 km was analysed and data was gathered from 248 Cone Penetration Tests, 35 boreholes and a geophysical survey of 2.5 km length (Consoli Test). The project consists of 4 major sections.

Firstly, the methodology used for processing of the dataset is given (CPT, borelogs and geophysical outcomes). Further, a literature review is performed and three different techniques for the quantification of the spatial variability are presented (Moving Average Window technique, Autocorrelation function, and Semivariogram function). Based on the evaluation of these techniques the Semivariogram technique was selected for the extraction of the spatial correlation length. By implementing the Semivariogram function in every lithological unit, 50 different 3D lithological maps of the Holocene deposits were made. By importing that lithological information in the settlement calculations (Finite Element Method), 300 simulations were performed along and across the road axis for three representative locations. The methodology of creating a lithological random field is followed. Furthermore, the outcomes of the realisation technique are compared with the outcomes of the classical approach which is used in the common civil/geotechnical engineering practice.

The Intrinsic Compression Line of Tertiary clays

Yun Yao Hu

Geotechnical information on Tertiary clays is limited in the Netherlands despite their widespread distribution in the subsurface. This is because the clays have limited access due to depth and due to outcrops at only a few locations where they can be sampled at an even further limited number of clay pits. Hence most of the information on these clays is based on more extensive investigation in Belgium (Boom and Ieper Clays) and the United Kingdom (London Clay, Barton Clay). Outcrops of Tertiary clays occur at Winterswijk, South Limburg and Zeeuws-Vlaanderen. Clay pits suitable for sampling only exist in Winterswijk and South Limburg. The strategy of the research is based on these 'sampling' constraints and examines basic Winterswijk Rupel Clay geotechnical properties for comparison with that of Mol, other locations in Belgium and from the Westerschelde in Zeeland (from the Westerschelde tunnel). Use is made of a testing/investigative approach set out by Burland (1990) in his Rankine lecture to the British Geotechnical Association to obtain relevant consolidation parameters for comparison. The first step is to obtain, to quote Burland, "the compressibility and strength characteristics of reconstituted clays so as to obtain a basic frame of reference for interpreting the corresponding characteristics of natural sedimentary clays. The properties of reconstituted clays are termed 'intrinsic' properties since they are inherent to the soil and independent of the natural state." The concept is simple but involves a significant amount of time, as is the case with most consolidation tests on clays. The method requires determination of the ICL, 'Intrinsic Compression Line'. The ICL is based on reconstituted clay at a moisture content of between that of its liquid limit and 1.5 times the liquid limit, followed by one-dimensional consolidation. The plot of void ratio to log pressure represents the ICL. The ICL is then compared with compression curves of undisturbed samples. This research reviews the geology of Tertiary clays in the Netherlands and provides the results of the ICL determination with comparisons from tests done on undisturbed samples, showing (according to Burland) that the clays are all over-consolidated, the degree of consolidation being greater in Winterswijk. This suggests that processes such as glaciation, desiccation and tectonics must have had a significant influence on the Winterswijk clay as the Belgian/Westerschelde clays would have been subject to greater sedimentary loading.

Weblinks

General information on dams:

www.pbs.org/wgbh/buildingbig/dam/index.html

www.nationalgeographic.com/resources/ngo/education/geoguide/dams/index.html

www.hydropower-dams.com/

www.ronterpening.com/extras/tropic_ex.htm

www.guinnessworldrecords.com/records/science_and_technology/structures/highest_concrete_dam.aspx

npdp.stanford.edu/damhigh.html

www.usbr.gov/lc/hooverdam/History/essays/biggest.html

en.structurae.de/structures/data/index.cfm?ID=s0003701

Information on specific dams/subjects:

Hoover Dam under construction: **www.eng.auburn.edu/users/zechwes/pictures.html**

Diablo Dam under construction: **nwda-db.wsulibs.wsu.edu/findaid/ark:/80444/xv97136**

Diablo Dam geology: **www2.nature.nps.gov/GEOLOGY/usgsnps/noca/nocaft4.html**

Grand Coulee Dam: **users.owt.com/chubbard/gcdam/**

Teton Dam failure: **web.umr.edu/~rogersda/teton_dam/**

USACE website about Risk Analysis for Dam Safety: **www.wes.army.mil/ITL/damsafe/**

Ice dams in Chile: **www.engineering360.nl/index.php?option=com_content&task=view&id=5696&Itemid=30**

Tailing dams: **www.tailings.info/stava.htm**

Organisations:

ICOLD: **www.icold-cigb.net**

NETHCOLD: **www.nethcold.org**

IAEG: **www.iaeg.info**

ISSMGE: **www.issmge.org**

ISRM: **www.isrm.net**

KNGMG: **www.kngmg.nl**

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Geoportals:

dinolks01.nitg.tno.nl/dinoLks/DINOLoket.jsp

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